THURBER ENGINEERING LTD.

DRAFT
FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF HIGHWAY 401 UNDERPASS AT BAINSVILLE ROAD
TOWNSHIP OF LANCASTER
SITE 31-241, G.W.P. 4027-14-00
ASSIGNMENT NUMBER: 4014–E–0014

GEOCRES NUMBER: -

SUBMITTED TO
MMM GROUP LIMITED

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual data obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the replacement of the Highway 401 underpass structure at Bainsville Road located within the Township of Lancaster. Thurber carried out the investigation as a subconsultant to MMM Group Limited (MMM), under Agreement No. 4014-E-0014.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on this data, provide a borehole location plan, record of boreholes, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions.

2 SITE DESCRIPTION

Site 31-241 is located on Highway 401, approximately 36 km east of Cornwall, Ontario. The location of the structure is shown on the inset Key Plan on Drawing No. 1 in Appendix A.

Based on the historical contract documents, the six-span structure is an AASHTO girder structure, approximately 121.5 m long, and 10.5 m wide that carries two lanes of Bainsville Road traffic over Highway 401 and County Road 2. It is noted that for project orientation purposes, Highway 401 and County Road 2 (located to the north of Highway 401), will be assumed to run east-west and Bainsville Road to be oriented north-south.

Highway 401 at this location has two through lanes in each direction with paved shoulders. The eastbound and westbound lanes are generally separated by a wide, vegetated median ditch, however, a flat gravel surfaced area is present in the immediate vicinity of the bridge. There are steel beam guide rails located along both the median and outside lanes in both directions. The median guiderails terminate at the overpass structure.

Bainsville Road at this location has one lane in each direction. Concrete curbs or barrier walls are present at the edge of pavement on the bridge deck and approach slabs. A steel beam guide rail is present on both sides of the roadway along both the north and south approach embankments. County Road 2 within the project limits also has one lane in each direction with a rural cross-section and gravel shoulders.

The site is located within a physiographic region known as the Lancaster Flats which are characterized as lowlands in which the till plain has been buried under water-laid deposits of clay to very fine sand (Chapman and Putnam, 1984).

The lands surrounding the project limits are typically agricultural with some residential properties. Storm water drainage in the area is to existing ditches and culverts. Site photographs showing the structure and approach embankments are presented in Appendix D.

The approach embankments are up to 6.1 m high and include a 16.7 m wide stability berm sloped at 20H:1V (Horizontal:Vertical) at the sides of both the north and south embankment and in front of the north abutment; the upper and lower slopes are at 2H:1V. The upper and lower slopes of the embankments are sloped are 2H:1V. The embankment slopes are vegetated with long grasses, trees, and occasional shrubs. No evidence of slope instability was noted during the site reconnaissance, however, evidence of settlement of the approach embankment was noted at the north abutment and the location is posted with a “bump” sign.

Historical contract drawings indicated that the clay was removed from beneath the south abutment and replaced with granular fill.

3 SITE INVESTIGATION

3.1 Previous Investigations

A GEOCRES report is available for this site (Report 31G00-151, 1961). It includes 9 boreholes drilled in support of the original bridge design and construction. A copy of the Borehole Location Plan and Borehole Logs for the previous investigation is provided in Appendix B

The stratigraphy in the area of the bridge is generally described as upper sand deposit over medium strength clay over glacial till over limestone bedrock. The thickness of the clay was identified as ranging from approximately 1.0 m at the south end of the alignment to approximately 12 m beneath the north approach. Only two of the nine boreholes, both north of County Road 2, extended to bedrock at depths ranging from approximately 14 to 16 m below original grades.

3.2 Field Investigation

The field investigation plan was finalized after discussion with the MTO Foundations Section. Approximate locations of boreholes are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A. The field investigation for this site included advancing ten boreholes drilled between November 9, 2015 and November 30, 2015. The locations and elevations of the boreholes are shown on Drawing No. 1 are summarized in Table 3-1.

Table 3-1: Borehole Summary

Borehole	Location	Latitude (degrees)	Longitude (degrees)	Ground Surface Elevation (m)	Depth (m)
201	South Abutment	45.17606	-74.40870	55.8	19.1
202	South Abutment	45.17604	-74.40875	55.8	23.3
203	Existing Pier 5	45.17627	-74.40878	48.9	16.4
204	Existing Pier 4	45.17646	-74.40896	49.0	16.8
205	Existing Pier 3	45.17666	-74.40913	48.6	16.2
206	Existing Pier 3	45.17656	-74.40923	48.3	13.0
207	Existing Pier 2	45.17682	-74.40921	49.2	32.4
208	Existing Pier 2	45.17674	-74.40943	49.1	12.7

Borehole	Location	Latitude (degrees)	Longitude (degrees)	Ground Surface Elevation (m)	Depth (m)
209	North Abutment	45.17714	-74.40959	54.6	24.0
210	North Abutment	45.17712	-74.40960	54.6	20.0

As a component of our standard procedures and due diligence, Thurber contacted Ontario One Call to provide utility locate clearances for the intended borehole locations.

The boreholes were advanced with CME truck and track mounted drill rigs equipped with NW and HW size casing. The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. Split spoon samples were collected at regular depth intervals in the boreholes via the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586-11. In-situ shear vane testing was carried out within the soft to firm cohesive strata. Thin-walled tube samples of the cohesive deposits were also collected at selected locations. All soil samples recovered from the boreholes were placed in moisture-proof containers and the samples were transported to Thurber’s Ottawa geotechnical laboratory for further examination and testing. Bedrock was cored in Boreholes 202 to 205 and 209 with NQ size coring equipment and Borehole 207 with HQ size coring equipment following ASTM Standard D6032-08. Bedrock core samples were stored in core boxes for transport.

Borehole 207 was advanced to a termination depth of 32.4 m in order to conduct downhole seismic testing and analysis in order to measure the in-situ shear wave velocity profile at the site. Thurber engaged Geophysics GPR International Inc. (GPR) to carry out downhole seismic testing and analysis. The downhole survey allowed the measurement of the shear wave profile of the overburden and the bedrock to determine the average shear wave velocity, V_{s30}. A copy of the shear wave velocity profile for this site is provided in Appendix B.

A 25 mm inside diameter PVC piezometer was installed in Borehole 208 to allow for measurement of the groundwater level at the site. Piezometer construction details are illustrated on the Record of Borehole sheet for Borehole 208, provided in Appendix B.

The boreholes without monitoring wells installations were backfilled with a low-permeability combination of auger cuttings and bentonite pellets in general accordance with the intent of Ontario MOE Regulation 903. Boreholes advanced within paved areas were capped with 300 mm of cold patch asphalt.

The as-drilled locations of the boreholes and ground surface elevations at the borehole locations were surveyed by Thurber on November 16, 2015. The vertical datum used was Benchmark 830066 (BM) located in the north face of Pier 2, near Borehole 205. The BM had a geodetic elevation of 49.474 m as indicated on the drawings provided by MMM. The location of the BM is indicated on Drawing No. 1 in Appendix A.

3.3 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all soil samples in accordance with the current MTO standards. Grain size distribution analyses, Atterberg Limits testing and consolidation testing were also carried out on selected samples to MTO and ASTM standards.

The laboratory test results are presented on the Record of Borehole sheets in Appendix B and are illustrated on the figures in Appendix C.

4 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Overview / General

Reference is made to the Record of Borehole sheets in Appendix B for details of the soil stratigraphy encountered in the boreholes. A stratigraphic profile for the site is presented on the Drawing No. 1 in Appendix A for illustrative purposes. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the Record of Boreholes governs any interpretation of the site conditions.

For reference, the stratigraphy encountered in the boreholes advanced at Site 31-241 differs between those advanced north of Pier 5 to those advanced south of Pier 5 which has been attributed to the past clay removal undertaken at the site.

In general the stratigraphy in the area of the boreholes advanced through the south embankment is generally characterized by an asphaltic surface, overlying sand with silt and gravel fill, overlying a sand with gravel fill with varying amounts of silt and clay, overlying sand with silt gravel, overlying a granular glacial till, underlain by a limestone bedrock. It should be noted that the clay layer was not encountered in the south abutment boreholes; however, was encountered in the historical approach Borehole BH6, located approximately 40 m south of the south abutment.

The stratigraphy in the area of the boreholes advanced at the north embankments is generally characterized by characterized by an asphaltic surface, overlying sand with silt and gravel fill, overlying a sand with gravel fill with varying amounts of silt and clay, overlying silt, overlying a weathered clay crust, over soft to stiff clay, overlying a granular glacial till, and underlain by a limestone bedrock.

More detailed descriptions of the individual strata are presented below.

4.2 Topsoil

A topsoil layer with a thickness ranging from 125 mm to 225 mm was encountered in Boreholes 203 to 208, expect Borehole 207.

4.3 Granular Fill

Boreholes 201, 202, 209 and 210 were advanced through Bainsville Road. An asphaltic surface layer with a thickness of 125 mm was encountered in both the north and south abutment boreholes. No boreholes were advanced through the pavement structure of either County Road 2 or Highway 401.

A granular fill layer consisting predominantly of sand and gravel with varying amounts of silt was encountered below the asphalt in the embankment boreholes. The top of this layer ranges from Elevation 55.7 m to Elevation 54.5 m and has a thickness ranging from 400 mm to 1.4 m. The SPT ‘N’ values ranged from 33 to greater than 100 blows per 0.3 m of penetration; indicating a dense to very dense condition.

The moisture content of the samples tested ranged from 2% and 3%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-1 and are illustrated on Figure 1 in Appendix C.

Table 4-1: Gradation Results for Pavement Structure Fill

Soil Particles	%
Gravel	27 to 35
Sand	63 to 54
Silt and Clay	3 to 11

4.4 Embankment Fill

A granular fill layer consisting predominantly of sand with varying amounts of silt and gravel was encountered beneath the granular fill. Occasional cobbles were noted in this layer. The top of this layer ranges from Elevation 55.3 m to Elevation 53.0 m and has a thickness ranging from 6.3 m to 8.6 m. The SPT ‘N’ values ranged from 3 to greater than 100 blows per 0.3 m of penetration; indicating a loose to very dense condition; but typically compact to dense.

The moisture content of the samples tested ranged from 2% to 25%. The results of grain size analysis conducted on samples of this fill material are summarized in Table 4-2 and are illustrated on Figures 2 and 3 in Appendix C.

Table 4-2: Gradation Results for Embankment Fill

Soil Particles	%
Gravel	0 to 40
Sand	19 to 91
Silt and Clay	8 to 81

4.5 Sand Fill

A granular fill layer consisting predominantly of sand with varying amounts of silt and gravel was encountered beneath the topsoil layer in Boreholes 203, 204 and 208 and at the ground surface in Borehole 207. A strong hydrocarbon odour was noted in Borehole 203, in this layer at a depth ranging from 0.76 m to 1.4 m.

The top of this layer ranges from Elevation 49.2 m to Elevation 48.8 m and has a thickness ranging from 0.8 m and 3.3 m. The SPT ‘N’ values ranged from 3 to 17 blows per 0.3 m of penetration; indicating a loose to compact condition.

The moisture content of the samples tested ranged from 4% to 20%. The results of grain size analysis conducted on samples of this fill material are summarized in Table 4-3 and are illustrated on Figure 4 in Appendix C.

Table 4-3: Gradation Results for Sand Fill

Soil Particles	%
Gravel	7 to 42
Sand	48 to 85
Silt and Clay	6 to 37

4.6 Silt Fill

A fill layer consisting predominantly of silt with varying amounts of sand and trace gravel was encountered beneath the topsoil layer in Boreholes 205 and 206 and beneath the sand fill material in Boreholes 202, 205 and 207. The top of this layer ranges from Elevation 48.5 m to Elevation 48.1 m and has a thickness ranging from 1.3 m and 1.6 m. The SPT ‘N’ values ranged from 3 to 16 blows per 0.3 m of penetration; indicating a very loose to compact condition.

The moisture content of the samples tested ranged from 4% to 28%. The results of grain size analysis testing conducted on samples of this fill material are summarized in Table 4-4 and are illustrated on Figure 5 in Appendix C.

Table 4-4: Gradation Results for Silt Fill

Soil Particles	%
Gravel	0 to 18
Sand	13 to 31
Silt	51 to 83
Clay	4 to 6

Based on the results of Atterberg Limits testing the material is a non-plastic silt.

4.7 Silt (ML)

A silt with varying amounts of sand and clay was encountered beneath the fill materials in Boreholes 207 to 210. The top of this layer ranges from Elevation 47.6 m to Elevation 46.8 m and has a thickness ranging from 700 mm and 2.9 m. The SPT ‘N’ values ranged from 7 to 23 blows per 0.3 m of penetration; indicating a loose to compact condition; but typically compact.

A hydrocarbon odour was noted in Borehole 210 at depths from 7.6 m to 8.4 m.

The moisture content of the samples tested ranged from 19% to 28%. The results of grain size analysis testing conducted on samples of this material are summarized in Table 4-5 and are illustrated on Figure 6 in Appendix C.

Table 4-5: Gradation Results for Silt

Soil Particles	%
Gravel	0 to 3
Sand	9 to 21
Silt	75 to 81
Clay	5 to 13

Based on the results of Atterberg Limits testing the material is a non-plastic silt.

4.8 Clay (CH)

A grey clay deposit with silt and trace sand was encountered beneath the fill materials in Boreholes 204 to 206 and beneath the silt stratum in Borehole 207 to 210.

It should be noted that the clay layer was not encountered in the south abutment Boreholes 201 and 202 which has been attributed to the past clay removal undertaken at the site. However, the clay layer with an approximately thickness of 3.1 m was encountered in the historical approach Borehole BH6, located approximately 30 m south of Boreholes 201 and 202. The location of BH6 is illustrated on the Plan View on Drawing No. 1 in Appendix. A copy of the historical Borehole Log for BH6 is provided in Appendix B.

The top of this layer ranges from Elevation 47.5 m to Elevation 43.9 m and has a thickness ranging from 1.5 m at Borehole 204 to 9.2 m at Borehole 210. In-situ shear vane test results indicated undrained shear strengths ranging from 15 kPa to 70 kPa; indicating a soft to stiff consistency; typically soft to firm. The moisture content of the samples tested ranged from 42% to 86%. The results of grain size analysis testing conducted on samples of this material are summarized in Table 4-6 and are illustrated on Figure 7 in Appendix C.

Table 4-6: Gradation Results for Clay

Soil Particles	%
Gravel	0 to 5
Sand	0 to 16
Silt	16 to 57
Clay	23 to 84

The results of Atterberg Limits testing completed on samples of this material are summarized in Table 4-7 and are illustrated on Figures 8 and 9 in Appendix C. All but one of the results indicated a clay of high plasticity.

Table 4-7: Atterberg Limits Test Results

Liquid Limit	30 to 72
Plastic Limit	19 to 24
Plasticity Index	11 to 48

The results of oedometer (one-dimensional consolidation) tests carried out on an undisturbed clay sample are summarized in Table 4-8. The results of the testing indicate that the clay is slightly over-consolidated.

Table 4-8: Consolidation Test Results

Parameter	Value
Borehole	208
Sample	TW7
Depth / Elevation (m) (mid-sample)	4.3 / 44.8
Moisture Content, (%)	85
Unit Weight, (γ) (kN/m^3)	15.2
Specific Gravity (G_s)	2.78
Initial Void Ratio (e_o)	2.31
Pre-consolidation Pressure, (kPa)	62
Compression Index (C_c)	1.28
Recompression Index (C_r)	0.08

4.9 Sand (SP–SM)

A sand layer with varying amounts of silt and gravel was encountered beneath the fill materials in Boreholes 202 to 204 and beneath clay stratum in Boreholes 205 and 206. The top of this layer ranges from Elevation 46.6 m to Elevation 41.7 m and has a thickness ranging from 700 mm and 4.2 m. The SPT ‘N’ values ranged from 6 to 42 blows per 0.3 m of penetration; indicating a loose to compact condition; but typically compact.

The moisture content of the samples tested ranged from 9% to 22%. The results of grain size analysis conducted on samples of this material are summarized in Table 4-9 and are illustrated on Figure 10 in Appendix C.

Table 4-9: Gradation Results for Sand

Soil Particles	%
Gravel	6 to 35
Sand	30 to 84
Silt and Clay	8 to 50

4.10 Glacial Till

A stratum of glacial till consisting predominantly of sand with silt and gravel was encountered in all boreholes except Borehole 209. The top of this layer ranges from Elevation 46.6 m to Elevation 35.5 m and has a thickness where completely penetrated ranging from 1.0 m in Borehole 210 to 10 m in Borehole 201. The SPT ‘N’ values ranged from 12 to greater than 100 blows per 0.3 m of penetration; indicating a loose to very dense condition; but typically compact to dense. Occasional cobbles and boulders were noted in this stratum.

The moisture contents of the samples tested were 3% and 21%. The results of a grain size analysis testing conducted on samples of this material are summarized in Table 4-10 and are illustrated on Figures 11 and 12 in Appendix C.

Table 4-10: Gradation Results for Glacial Till

Soil Particles	%
Gravel	17 to 43
Sand	35 to 56
Silt and Clay	10 to 37

Based on the results of Atterberg Limits testing the fines content is classified as non-plastic.

4.11 Bedrock

Limestone bedrock was encountered beneath the glacial till in Boreholes 202, 203, 204, 205 and 209; as proven by NQ and Borehole 207 as proven by HQ coring. The bedrock surface ranged from Elevation 36.6 m to Elevation 35.1 m. Photographs of the bedrock core are provided in Appendix B.

A stratum of slightly to moderately weathered bedrock was encountered at the bedrock surface in Boreholes 203, 205, 207 and 209 with a thickness ranging from 1.0 m in Borehole 205 to 3.2 m in Borehole 209. Within the weathered layer the total core recovery (TCR) ranged from 43% to 100%, the solid core recovery (SCR) ranged from 8% to 70% and the Rock Quality Designation (RQD) ranged from 0% to 57%. Based on the RQD value the weathered bedrock is classified as very poor to fair quality.

Below the weathered layer the TCR ranged from 73% to 100%, the SCR ranged from 48% to 100%, the RQD ranged from 31% to 75%. Based on the RQD value the weathered bedrock is classified as poor to good quality.

4.12 Groundwater Conditions

The groundwater level in the piezometer installed in Borehole 208 was recorded on December 7, 2015 at a depth of 1.9 m; corresponding Elevation 47.2 m.

This observation is considered a short-term reading and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

5 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber surveyed the borehole locations, and determined the stationing, offsets and ground surface elevations based on contract drawings provided by MMM Group Limited. Downing George Estate Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, and in-situ testing. Geophysique GPR International Inc. of Longueuil, Quebec carried out the downhole seismic testing and analysis. The drilling, and sampling operations in the field were supervised on a full time basis by Mr. Simon Paxton and Justin Grey of Thurber. Laboratory testing was carried out by Thurber in its MTO-approved laboratory in Ottawa.

Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents the interpretation of the factual data obtained from a foundation investigation conducted by Thurber for the15 replacement of the Highway 401 underpass structure at Bainsville Road, along with a geotechnical assessment and geotechnical recommendations for the foundations and approach embankments. The geotechnical assessment and recommendations have been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-14.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The design-build contractor must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

6.1 Historical Performance of Existing Structure and Embankments

Based on the historical contract documents, the six-span structure is an AASHTO girder structure, approximately 121.5 m long, and 10.5 m wide that carries two lanes of Bainsville Road traffic over Highway 401 and County Road 2. The piers and abutments are supported on steel H-piles driven to bedrock. The abutment and piers were designed to be supported on Steel H piles driven to bedrock, with pile lengths ranging from 15.2 m to 18.2 m and design load was 50 tons per pile (500 kN per pile).

Additional discussion within the GEOCRES file indicated the following:

- A letter dated November 25, 1960, presented the results of settlement analysis for the approach embankments. It predicted consolidation settlement ranging from 2.0 ft. to 7.8 ft. (0.6 m to 2.4 m).
- It was also recommended that the thin layer of clay beneath the foreslope at the south abutment be removed and replaced with granular fill in order to avoid the need for a stability berm at this location and therefore allow for a shorter structure. A historical schematic drawing illustrating the location and area of the clay removal is provided in Appendix G. The location is also noted on Drawing No. 1 in Appendix A.

The existing bridge abutments are perched within the approach embankments. The approach embankments are approximately 6.7 m high and include a 16.7 m wide stability berm sloped at 20H:1V (Horizontal:Vertical) at the sides of both the north and south embankment and in front of the north abutment; the upper and lower slopes are at 2H:1V. The embankment slopes are vegetated with long grasses, trees, and occasional shrubs.

No evidence of slope instability was noted during the site reconnaissance, however recent site observations by Thurber staff indicate that settlement of the embankment and tilting of the approach slab is evident at the north approach and the location is posted with a “bump” sign.

The performance of the existing structure was discussed in a technical paper prepared by Ministry staff for presentation at the 20th Canadian Soil Mechanics Convention (Stermac, Devata, and Selby, 1967). The paper titled “Unusual Abutment Movements at Underpass Structures on the Macdonald-Cartier Freeway” describes the conditions encountered at the site and indicates:

> *“Settlements of the approach fill at the site have been considerable. Settlement records are shown in Figure 20 for the south side; they are not available for the north side.”*
>
> *“The north abutment seems to have moved away from the bridge more than an inch, as shown in Figure 21. However, no corrective action has so far been necessary.”*

A copy of Figure 20 is provided in Appendix G which indicates that between 0.2 and 0.3 feet (60 to 90 mm) of settlement occurred at the south approach (within the width of the roadway) during the first 400 to 500 days post construction and that the settlement slightly exceeded 0.3 ft. (90 mm) by the time of the last reading between 700 and 800 days post construction. As noted above, settlement data for the north approach is not available, however, it is reasonable to expect that the settlement was significantly greater than at the south side since the underlying clay is at least three times as thick as the south side and was large enough to cause movement of the abutment away from the bridge.

6.2 Proposed Structures and Embankments

Based on information provided by MMM, it is understood that replacement of the bridge structure will be on the existing alignment with a full road closure and detour. The following are structural design consideration for the structure replacement:

- Long 3-Span Alternative was adopted by MTO as the technically preferred alternative
- Bridge deck will have an approximate width of 10.1 m to accommodate two lanes, shoulders and parapet walls.
- The structure has been designed with a 60 km/hr design speed for Bainsville Road

Based on discussions with the design team it is understood that proposed bridge design may include the reuse of existing pier foundations.

Based on the preliminary span configuration, Highway 401 clearance requirements and the proposed design speed for Bainsville Road, the vertical profile for Bainsville Road will be raised approximately 0.8 m and 0.9 m at the north and south abutments respectively. The existing elevations, grade raises and proposed elevations after raising the embankment grade are outlined in Table 6-1.

Table 6-1: Proposed Profile grades

Abutment	Existing Top of Pavement (m)	Approximate Grade Raise (m)	Proposed Top of Pavement (m)
North	55.0	0.8	55.8
South	56.2	0.9	57.1

The following sections address the foundation aspects of the replacement of the existing underpass structure. The discussions and recommendations presented in this report are based on the information provided by MMM Group and on the factual data obtained during the course of this investigation.

6.3 Geotechnical Assessment

The design of the bridge structure foundations and approach embankments are governed by the presence of a soft to firm compressible clay deposit throughout the site. Based on the results of the field and laboratory investigation and the information provided by MMM of the proposed project requirements, geotechnical foundation design considerations include:

- The soft to firm clay layer will not offer sufficient support to support bridge piers and abutments on shallow foundations; deep foundations will be required.
- The soft to firm clay layer is highly compressible. Any additional load applied to the underlying clay layer will result in new settlement of the approach embankments. The design will need to incorporate mitigation measures to ensure that embankment settlement due to the proposed grade raise meet the MTO embankment settlement criteria.
- Stability of the approach embankments will also need to be verified, including stability under the seismic conditions included in the current CHBDC.
- From a geotechnical perspective, the ground conditions at the site are generally suitable for integral abutments.

Further discussion regarding these design considerations, evaluation of design options and foundation recommendations are provided in the sections that follow.

7 STRUCTURE CLASSIFICATION

In accordance with CHBDC CSA S6-14, the analysis and design of structures takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case in the Ministry of Transportation, Ontario (MTO).

Is understood that MTO has designated this structure as follows:

Table 7-2: Bridge Structure Classification

Criteria	Classification	CHBDC Section
Importance Category	Major Route Bridge	4.4.2
Consequence Classification	Typical Consequence	6.5.1

Based on the above, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances.

If the consequence classification changes, the geotechnical assessment and recommendations will need to be reviewed and revised.

8 SEISMIC CONSIDERATIONS

8.1 Seismic Site Class – Soil Profile

The results of the in-situ shear wave velocity testing indicate that the Vs_{30} for the project site is 286 m/s (a copy of the results is provided in Appendix E), which typically indicates a Site Class D.

However, in accordance with Section 4.1 of the CHBDC a site is assigned a Site Class E regardless of the shear wave profile if *“Any profile with more than 3 m of soil with the following characteristics”*

- Plasticity index: $PI > 20$
- Moisture content: $w \geq 40\%$ and
- Undrained shear strength: $S_u < 25$ kPa

The Boreholes 206, 207 and 208 encountered a clay deposit that meets the soil profile outlined above and therefore the site must be assigned a Site Class E.

8.2 Seismic Hazard - Spectral and Peak Acceleration Values

The seismic hazard data for the CHBDC is based on the fifth generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC’s seismic hazard calculator. The data includes peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values (Sa(T)) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix G.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA).

8.3 Seismic Liquefaction

Based on the combination of the grain size distribution, and the relative the density values of the glacial till, and the highly plastic nature of the native clay the overburden soils at this site are classified as “not susceptible” to liquefaction during the design earthquake event.

9 APPROACH EMBANKMENTS

The proposed profile and bridge spans require a maximum grade raise of 0.75 m and 0.90 m at the north and south approach embankments respectively. The proposed grade raise would also result in a widening of the approach embankments in order to maintain the platform width at the top and the existing embankment side slope geometry (2H:1V). It is understood that existing stability berms are to remain as part of the proposed embankments.

9.1 Assessment of Settlement

An assessment of the time dependent settlement that would result from construction of the proposed grade raise using conventional granular fill with 2H:1V side slopes was carried out using Rocscience's Settle^{3D} modelling software. The design pre-consolidation pressure profile has been derived from the oedometer tests, both current and historical, as well as correlations with the undrained shear strength and plasticity. Compression characteristics have been modelled using C_c, C_r, C_v and C_{vr} values from the current and historical oedometer test results.

The following design geotechnical parameters have been used in the analysis:

- $e_o = 2.311$
- $C_c = 1.28$
- $C_r = 0.08$
- $C_v = 0.064$ cm^2/min / 3.4 m^2/year
- $C_{vr} = 0.563$ cm^2/min / 29.6 m^2/year

It is noted that the stresses associated with a grade raise constructed with conventional granular would exceed the pre-consolidation pressure for a portion of the depth profile at both abutments.

The results of the analysis are summarized in the Table 9-1.

Table 9-1: Time Dependant Settlement – Grade Raise Constructed with Granular Fill

Location	Grade Raise (m)	Settlement Beneath Centreline After 20 Years (mm)		
		0 to 20 m from abutment[1]	+20 to +50 m from abutment	+50 m from abutment
North Approach	0.75	65	20	0 0 m grade raise beyond +25 m of the north abutment
South Approach	0.90	30	55	20

Note 1: The predicted settlement values provided at the south approach in Table 9-1 are for the approach embankment underlain by clay. The settlement due to the grade raise directly beneath the abutment is expected to be less than 5 mm due to the clay removal carried out during the original construction.

The predicted settlement values reflect both the maximum embankment height after the grade raise as well as the aerial distribution of fill and fill height.

The estimated settlement of the approach embankments at the abutments is in excess of the MTO Guidelines for post construction settlement over a period of 20 years after paving outlined below:

- 25 mm within 20 m behind bridge abutment
- 50 mm from 20 to 50 m from the bridge abutment
- 100 mm for greater than 50 m from the bridge abutment

The time rate of settlement has also been assessed and it is estimated that it would require a preload of several years to meet the MTO settlement guidelines for the north approach fill.

9.2 Assessment of Global Stability

The global stability for the proposed grade raise constructed using conventional granular fill with 2H:1V side slopes was evaluated using GeoStudio 2012 Slope/W software for limit equilibrium analysis. Input parameters for undrained analysis are based on the in-situ shear vane test results. The values of cohesion and internal friction angle used in the drained analysis are based on empirical correlations developed for the Champlain Sea clay deposits present in the area (Tavenas and Leroueil, 1981).

The following additional parameters were used in the analysis:

- A traffic surcharge load as per Section 6.12.5 of the CHBDC
- A seismic horizontal loading of 0.19, equal to ½ of the PGA value (0.381g) was used for seismic analysis
- Existing embankment side slope geometry (2H:1V) and maintaining the existing stability berms

Table 9-2: Global Stability Analysis Results – Grade Raise Constructed with Granular Fill

Location	Factory of Safety		
	Static Conditions		Seismic Conditions
	Undrained	Drained	
North Abutment	1.5	1.5	1.0
South Abutment	1.3	1.4	1.0

The factor of safety does meet the target value of 1.3 and 1.0 under static and seismic conditions respectively.

9.3 Evaluation of Embankment Design Options

Based on the initial assessment of the embankment constructed using conventional granular fill, additional embankment design options to address both settlement and global stability were developed and assessed.

The embankment design options considered include:

1. Conventional granular fill embankment

2. Lightweight fill embankments

3. Ground improvement techniques
4. Accelerated settlement (surcharging either with or without wick drains)

Options 3 and 4 were ruled out since the proposed profiles are being constructed as grade raises to the existing embankments. As the zone to be treated (clay layer) is buried beneath existing embankments all treatments would have to be done through the existing embankment material. Also, a drainage layer could not be constructed under the existing embankment to work in conjunction with any wick drains installations therefore consolidation of the clay layer would take longer to accomplish causing significant delays to the construction schedule.

A summary of the advantages and disadvantages of the remaining options is provided in Table F-1 in Appendix F.

Several lightweight fill options, including slag, tire-derived aggregate, foamed concrete and expanded polystyrene (EPS) were considered. The unit weight of the EPS fill is significantly lower than all of the other lightweight fill options and was selected as the preferred type of lightweight fill as it is the only option that would allow for appropriate control of the anticipated settlement without excessive sub-excavation and replacement of native subgrade soil. EPS is also an MTO approved lightweight fill.

9.4 Recommendations for Embankment Grade Raise Design and Construction

It is recommended that the embankment grade raise be constructed using EPS lightweight fill *(Option 2).* This option addressed the settlement concerns, and does not result in significant time delays to the project. It is noted that since the grade raise is generally less than 1.0 m, the volume of lightweight fill is anticipated to be relatively small. The EPS lightweight fill option is the preferred option from both a technical and risk management perspective and should be implemented at both the north and south abutments.

The preliminary limits of the EPS fill considered were as follows:

- Where the proposed grade raise will result in settlement in excess of the limits outlined in the MTO embankment settlement guidelines, expanded polystyrene (EPS) fill should be placed within the core of the embankment with a minimum thickness equal to the height of the proposed grade raise in order to limit settlement to within acceptable limits.

The MTO embankment settlement guidelines indicate acceptable limits for post construction settlement over a period of 20 years after paving as follows:

- 25 mm within 20 m behind bridge abutment
- 50 mm from 20 to 50 m from the bridge abutment
- 100 mm for greater than 50 m from the bridge abutment

Based on settlement analysis, EPS fill will be required where the proposed grade raise:

- is greater than 300 mm within 20 m of the bridge abutments
- is greater than 400 mm within the zone 20 m to 50 m from the bridge abutments

To limit differential settlement, the thickness of the EPS should be stepped down in the longitudinal direction in increments no greater than 0.5 m and no steeper than 4H:1V.

For preliminary design of the EPS limits, the width of the EPS should be:

- Centred along the roadway centerline.
- Where the thickness of the EPS is 1.0 m or less the width of the EPS layer should be the greater of 10 m or the width of the roadway platform including shoulders and curbs.

Table 9-3 outlines the preliminary minimum EPS thicknesses required at each abutment. The thicknesses provided in Table 9-3 are based on the above criteria and the profile tie-in for the Long 3 Span Arrangement provided by MMM. It should be noted that the final thicknesses and limits may vary based on standard EPS block geometry and the design vertical profile

Table 9-3: Preliminary Minimum EPS Thicknesses required for each Approach Embankment

Location	Distance from Abutment (m)	Minimum EPS Thickness (m)
South Approach	0 to 20 20 to 40 >40	1.0 0.5 0
North Approach	0 to 10 10 to 12 > 12	1.0 0.5 0

Implementation of the EPS design option will limit stress increases due to the proposed grade raises at the abutments. Since a limited stress increase is developed little additional load is applied to the underlying clay layer which will result in little settlement of the approach embankments.

The results of the global stability analysis using EPS to construct the grade raise at both the north and south approach embankments are summarized in Table 9.4. The predicted settlement values reflect both the maximum embankment height after the grade raise as well as the aerial distribution of fill and fill height.

Table 9-4: Global Stability Analysis Results – Grade Raise Constructed with EPS

Location	EPS Layer Thickness (m)	Factory of Safety		
		Static Conditions		Seismic Conditions
		Undrained	Drained	
North Approach	1.0	1.3	1.4	1.0
South Approach	1.0	1.3	1.4	1.0

An assessment of the time dependent settlement and global stability that would result from construction of the proposed grade raise using EPS lightweight fill with 2H:1V side slopes and maintaining the existing stability berms was carried out using Rocscience's Settle^{3D} modelling software.

The stress increase beneath the EPS layer and resulting total settlement from 0 to 20 years is outlined in Table 9-5.

Table 9-5: Settlement Analysis Results – Grade Raise Constructed with EPS Fill

Location	Grade Raise (m)	Settlement Beneath Centreline After 20 Years (mm)		
		0 to 20 m from abutment[1]	+20 to +50 m from abutment	+50 m from abutment
North Abutment	0.75	< 10	< 5	0 0 m grade raise beyond +25 m of the north abutment
South Abutment	0.90	< 10	< 15	< 5

Guidelines for the design of EPS embankments can be found in NCHRP Report 529. The contract must include an NSSP for the EPS embankment materials and construction. Selection of the EPS grade will depend upon surcharge loading including traffic loading, and the combined dead weight of the pavement structure, earth cover and concrete slab for the EPS blocks. A draft version of suggested NSSP wording is provided in Appendix H.

<u>General EPS Installation Notes</u>:

- The embankment design will need to take into consideration the potential for conflict between the EPS fill and foundations for signs, guiderails, utilities or other structures.
- A granular levelling pad consisting of a 300 mm of compacted OPSS Granular A should be provided beneath the EPS. It is recommended that a non-woven geotextile be placed horizontally beneath the granular levelling pad as a separation layer between the leveling pad and the existing embankment fill materials.
- The top surface of the EPS beneath the roadway platform should be covered with a concrete slab. The top of the concrete slab should be at the underside of the pavement subbase layer.

10 STRUCTURE FOUNDATIONS

10.1 Foundation Type

The results of the field and laboratory investigation and historical data indicate that the site soil stratigraphy is underlain by a thick clay deposit, underlain by a glacial till deposit, underlain by limestone bedrock.

Key elevations are as follows:

Existing ground surface at the piers	49.2 to 49.0 m
Existing ground surface at the abutments	55.8 to 55.6 m
Top of glacial till deposit	46.6 to 35.5 m
Top of bedrock	36.6 to 35.1 m

The clay can generally be characterized as moderately sensitive with high plasticity. The clay is generally soft to firm within the upper portion with strength increasing gradually with depth. The clay deposit offers low bearing resistance and is susceptible to settlement under even moderate loads. The clay deposit has insufficient strength to support the foundation loads associated with the proposed abutments and piers.

The glacial till deposit generally consisted of silty sand with gravel and occasional to frequent cobbles and boulders.

Based on the soil stratigraphy and anticipated loading, deep foundations are therefore required at this site.

The following deep foundation alternatives were considered:

1. Steel pipe piles
2. Steel H-piles
3. Caissons (drilled shaft piles)

A comparison of the technical advantages and disadvantages of alternative foundation schemes is presented in Table F-2 in Appendix F. Based on this comparison, steel H-piles are the recommended foundation support option from a geotechnical perspective.

Design recommendations for driven steel H-piles are provided in the sections that follow.

10.2 Deep Foundations – Steel Piles

Based on the depth to bedrock it is recommended that the design use steel HP section piles driven to refusal on or in the limestone bedrock. It has been assumed that HP 310 x 110 piles sections will be used to support both the piers and abutment foundations.

Steel piles (Grade 350W steel) end-bearing on the bedrock at this site may be designed on the basis of the following factored vertical geotechnical resistances at ULS:

- 2,000 kN per HP310x110 pile

The SLS condition will not govern for piles end-bearing in or on the bedrock.

The factored geotechnical resistances include the following factors:

- Consequence factor (Ψ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2):
  - ϕ_{gu} = 0.4 (static analysis; typical degree of understanding)
  - ϕ_{gs} = 0.8 (static analysis; typical degree of understanding)

The estimated pile tip elevations based on piles end bearing on the bedrock are summarized in Table 10-1.

Table 10-1: Estimated Pile tip Elevations

Foundation Element	Approximate Underside of Pile Cap Elevation (m)	Estimated Pile Tip Elevation (m)
North Abutment	50.5	34.5
Pier 1	48.1	35.1
Pier 2	48.1	35.0
South Abutment	50.0	35.8

10.2.1 Pile Lateral Resistance

A soil-structure interaction analysis to assess the response of a pile under lateral loading was carried out using Ensoft Inc.'s LPile software. A copy of the results in the form of load-deflection curves (p-y curves) and lateral load vs maximum bending moment are provided in Appendix G.

The resistance to lateral deflection should include the following factors:

- Consequence factor (Ψ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2):
    - ϕ_{gs} = 0.8; typical degree of understanding

Pile spacing and group effects will need to be considered in assessing the overall lateral resistance of the piles at each foundation unit. The group efficiency factors should be in accordance with Figures C6.11.3(r), C6.11.3(s), and C6.11.3(t) in Section C6.11.3.4 of the Commentary to the CHBDC.

10.2.2 Integral Abutment

The subsurface conditions at this site are considered suitable for integral, semi-integral or conventional type abutment design. If an integral abutment design is considered, the structure will need to be supported on steel H-piles. The H-pile length below the abutment should be a minimum of 5.0 m.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. To provide the required flexibility, the upper 3 m of the piles should be surrounded by a 600 mm diameter column of loose sand as specified by the integral abutment design requirements. A 600 mm diameter CSP may be used to contain the sand. An NSSP should be included in the contract documents specifying the gradation of the sand according to Table 10-2.

Table 10-2: Integral Abutment Sand Backfill Grading

MTO Sieve Designation	Percent Passing (%)
#10	100
#30	80 – 100
#40	40 – 80
#60	5 – 25
#100	0 – 6

10.2.3 Pile Installation

Driven piles must be installed in accordance with OPSS 903. The potential for conflict with the existing steel piles must be checked.

As the piles are anticipated to be driven to bedrock, the pile tips of new piles driven at the site should be protected from damage during driving with pile tip protection from an approved manufacturer such as Titus Steel (standard H-Point) or approved equivalent.

Pile driving should be controlled in accordance with Standard Drawing SS 103-11 (Hiley Formula) and an ultimate pile resistance should be specified by the designer. The Hiley formula need not be used until the piles are within 2.0 m of the design pile tip elevation. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile”. The value of “R” should have a minimum value of twice the design load at ULS as calculated by the Structural Engineer.

10.2.4 Downdrag

Should the grade raise be constructed using EPS backfill as outline in Section 9.4 little to no stress increase is anticipated. Since no stress increase is to be applied to the underlying clay layer no consolidation settlement is anticipated and therefore little downdrag loads will develop along the piles.

Should the proposed grade raise be constructed using conventional granular materials an analysis of the downdrag loads must be undertaken. Consideration of downdrag loads must then be included in the pile design.

10.2.5 Frost Protection

The frost penetration depth at this site is 1.8 m as per OPSD 3090.101. Accordingly, a minimum of 1.8 m of earth cover, or equivalent insulation, must be provided above the base of the pile caps to serve as frost protection.

11 EARTH RETAINING STRUCTURES

Due to the settlement concerns associated with the grade raise of the embankments, the backfill behind the abutments will consist primarily of EPS material. A mechanism for drainage behind the abutment should be provided.

11.1 Static Lateral Earth Pressure Coefficients

The backfill pressures acting on the back of the abutment should consider both:

- The gravity loads of the EPS backfill and overlying pavement structure pressing directly against the wall; and
- The active earth pressure from the soil behind the EPS backfill.

The methodology for assessing the pressures on the back of an abutment wall is described in Section 6 of NCHRP Report 529. The vertical load of EPS blocks will result in negligible active horizontal loading of the abutment wall. The horizontal pressure generated by the vertical stress imposed by the overlying pavement structure can be assumed to be equal to 0.1 times the vertical stress.

The recommended lateral earth pressure parameters for the soil behind the EPS backfill for use in the design for a horizontal back-slope are provided in Table 11-1.

Table 11-1: Static Lateral Earth Pressure Coefficients

Parameter	OPSS Granular A & OPSS Granular B Type II	Existing Fill	OPSS Granular B Type I
Soil Unit Weight, kN/m^3, γ	21	20	20
Angle of Internal Friction, ϕ	35°	33°	32°
Interface Friction Angle, Soil to EPS, δ	35°	33°	32°
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.43	0.46	0.47
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall)	0.27	0.29	0.31

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

For static analysis, passive earth resistance in front of the abutments should be ignored, and therefore has not been provided. A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Section 6.12.3 of the CHBDC.

11.2 Combined Static and Seismic Lateral Earth Pressure Parameters

The following recommendations are per Section C4.6.5 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with:

- k_h = ½ F(PGA)•PGA for structures that allow lateral yielding, and
- k_h = F(PGA)•PGA for non-yielding walls

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls. The ratio of wall movement to wall height required to mobilize the active condition would be approximately 0.002.

The recommended seismic lateral earth pressure parameters for use in the design that are provided in Table 11-2 assume the following:

- Horizontal back-slope behind the wall
- Seismic Site Class of E, and a PGA with a 2% probability of exceedance in 50 years of 0.381 g; as outlined in Section 8.1

Table 11-2: Lateral Earth Pressure (Under Seismic Loads)

Parameter	OPSS Granular A & OPSS Granular B Type II	Existing Fill	OPSS Granular B Type I
Soil Unit Weight, kN/m3, γ	21	20	20
Angle of Internal Friction, ϕ	35°	33°	32°
Interface Friction Angle, Soil to EPS, δ	35°	33°	32°
Yielding Wall			
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.38	0.41	0.42
Non-Yielding Wall			
Dynamic Active Earth Pressure Coefficient, K_{AE}	0.53	0.57	0.59

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soil profile:

$$\sigma_h = K_a \gamma d + (K_{AE} - K_a)\,\gamma\,(H - d)$$

where:

σ_h = lateral earth pressure at depth, d (kPa)
d = depth below the top of the wall (m)
K_a = static active earth pressure coefficient
γ = unit weight of the backfill soil (kN/m^3)
K_{AE} = combined static and seismic earth pressure coefficient
H = total height of the wall (m)

The horizontal coefficient of subgrade reaction of the EPS fill should be calculated based on the following equation:

$$K'_{EPS} = 0.14 * E_{EPS} / \{H * (1 - \nu^2_{EPS})\},$$

where:

K'_{EPS} = horizontal coefficient of subgrade reaction (kN/m^3)
E_{EPS} = Young's Modulus of EPS Blocks (kN/m^2)
ν_{EPS} = Poisson's Ratio of EPS Blocks ($\nu_{EPS} = 0.10$)
H = Thickness (vertical) of EPS behind wall (m)

The horizontal pressure applied by the wall to the EPS fill must be smaller than the Elastic Limit Stress of the EPS. A compressible geofoam inclusion may be considered where required to ensure flexibility of the integral abutment system.

11.3 Backfill Drainage

The parameters provided in Table 11-1 and 11-2 are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in the design.

12 CEMENT TYPE AND CORROSION POTENTIAL

Three samples of the native soils were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, resistivity and conductivity. The analysis was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in the Tables 12-1.

Table 12-1: Results of Chemical Analysis

Borehole	Sample	Depth (m)	Sulphate (µg/g)	pH	Resistivity (Ohm-m)	Chloride (µg/g)	RedOx Potential (mV)
204	SS4	2.2	145	7.4	12.8	274	391
205	SS3	1.8	38	7.6	16.3	323	-
207	SS8	4.2	174	8.0	17.5	65	-

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in the Table 12-1 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

13 CONSTRUCTION CONSIDERATIONS

13.1 EXCAVATION

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills at the site should be classified as Type 3 in accordance with OHSA.

Subgrade preparation and placement of the EPS backfill and pile caps must be carried out in the dry.

Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor.

13.2 DEWATERING

All excavations for foundations must be dewatered prior to the placement of concrete, as per OPSS 902.

The Contractor must be prepared to control the groundwater and surface water flow at the site to permit construction in a dry and stable excavation. Water from either surface flow and/or groundwater must be diverted away from the excavation at all times. Groundwater perched within the embankment fill and, surface runoff will tend to seep into, and accumulate in proposed excavations.

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

13.3 Erosion Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. Normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surficial erosion in general accordance with OPSS 804. The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site as per OPSS 805.

13.4 CONSTRUCTION CONCERNS

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

- Confirmation that the granular backfill is adequately placed and compacted to specifications.
- Confirmation that the EPS backfill is appropriately placed to specifications
- The Contractor's selection of construction equipment and methodology should include assessment of the capability of the subgrade soils to support the proposed construction equipment and any temporary structures or fill (i.e. as a pad for crane support). Site conditions may limit the type of equipment suitable for use. The design and safety of any temporary works is the responsibility of the Contractor. Recommended wording for an NSSP addressing this issue is provided in Appendix H

The successful performance of the embankments will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations by the QVE will be required during construction to confirm that the foundation recommendations are correctly implemented and material specifications are met.

14 CLOSURE

Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.

Kenton C. Power, P.Eng.
Geotechnical Engineer

Paul Carnaffan

Paul Carnaffan, P.Eng.
Associate, Senior Geotechnical Engineer

PKChatterji

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Review Principal, Designated MTO Contact

APPENDIX A

BOREHOLE LOCATIONS AND SOIL STRATA DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

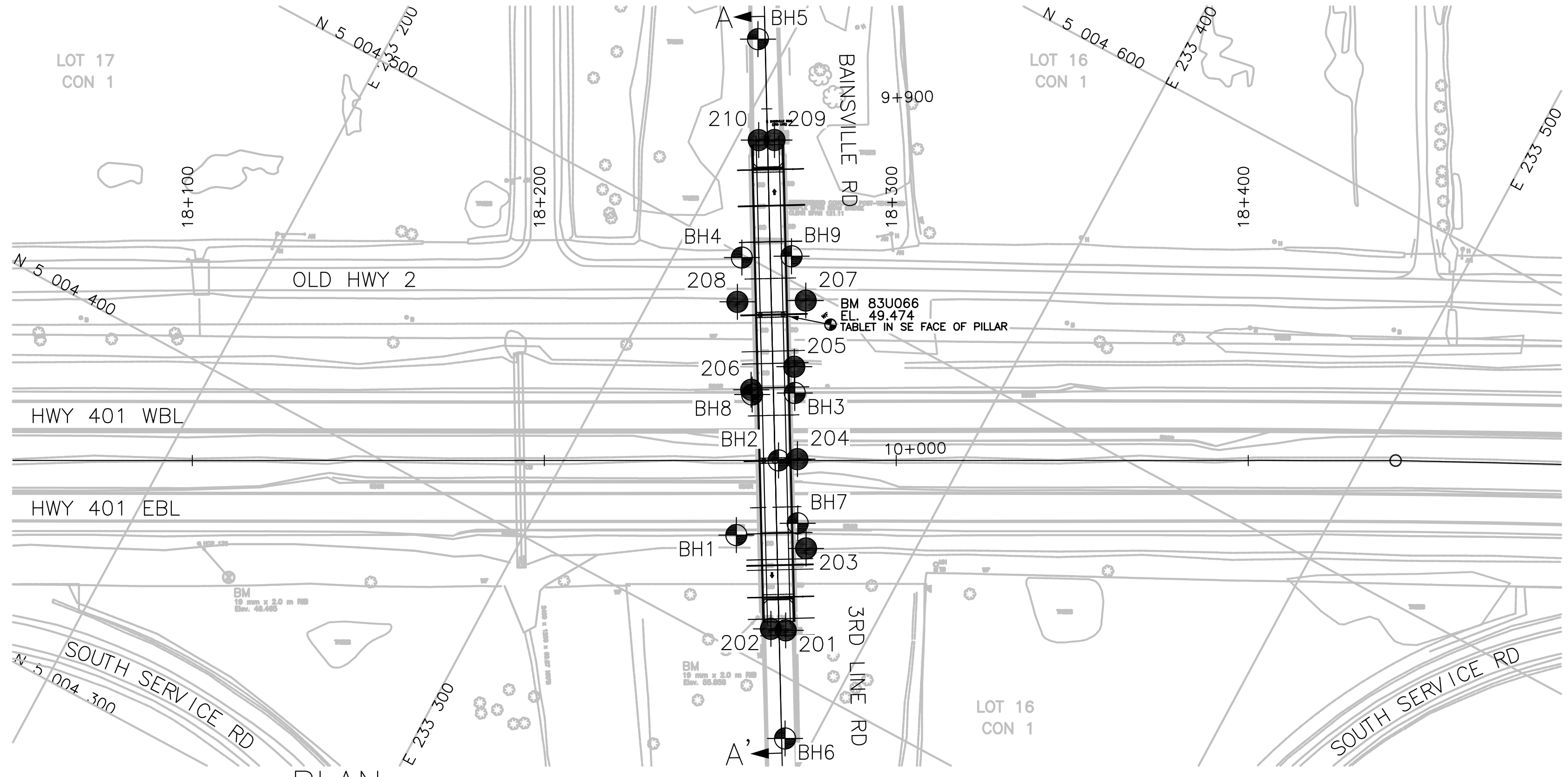
CONT No
WP No 4088-13-01

HIGHWAY 401
BAINSVILLE ROAD UNDERPASS
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

MMM GROUP

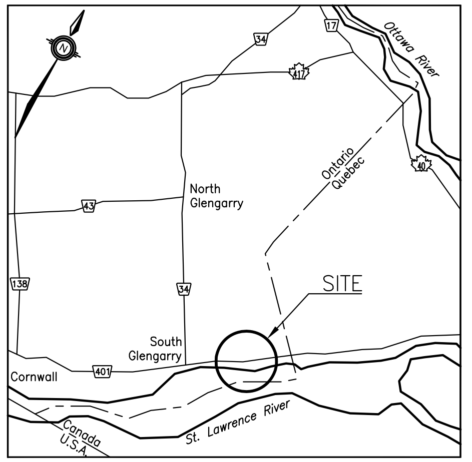
THURBER ENGINEERING LTD.



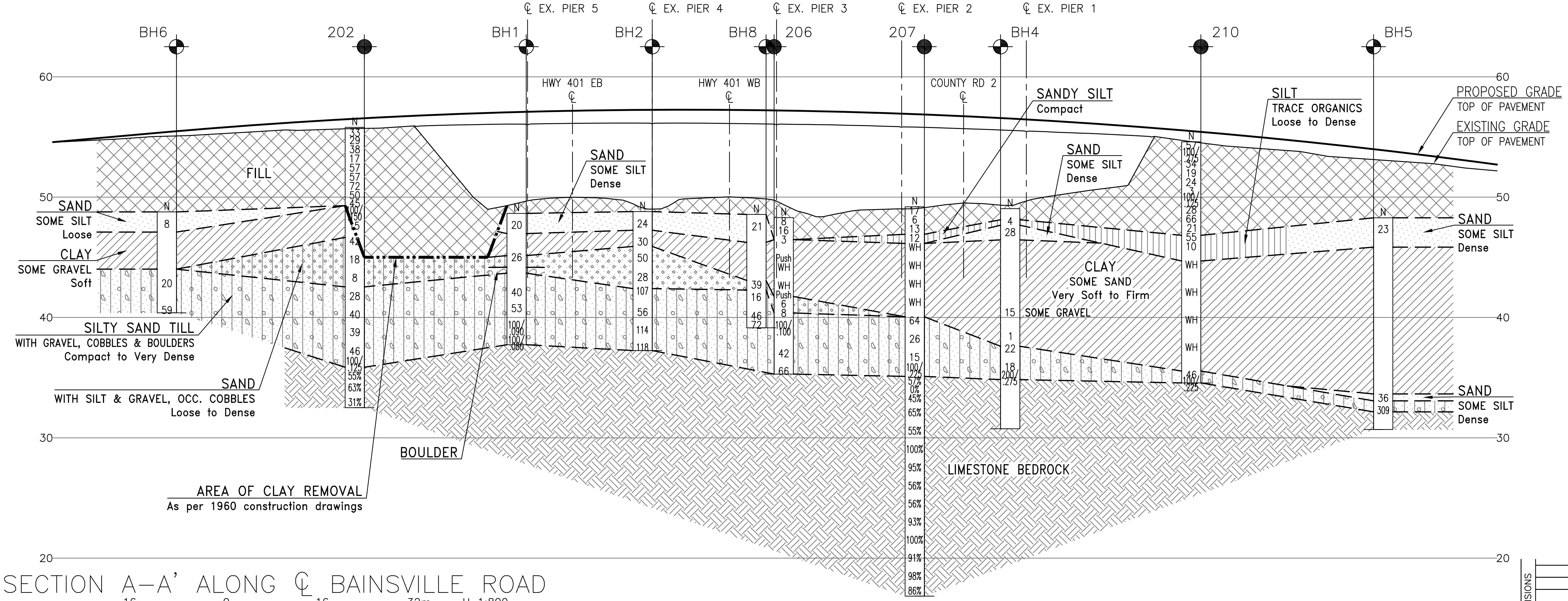
PLAN

40 0 40 80m
SCALE 1:2000

HISTORICAL BOREHOLES			
NO	ELEVATION	NORTHING	EASTING
BH1	48.6	5 004 428.9	233 353.2
BH2	48.8	5 004 453.3	233 353.8
BH3	48.8	5 004 472.4	233 348.8
BH4	49.0	5 004 499.2	233 317.3
BH5	48.3	5 004 556.1	233 292.0
BH6	48.8	5 004 384.5	233 392.8
BH7	48.3	5 004 440.1	233 367.0
BH8	48.6	5 004 466.3	233 338.3
BH9	49.2	5 004 506.2	233 329.6



KEYPLAN



SECTION A–A' ALONG ℄ BAINSVILLE ROAD

16 0 16 32m H 1:800

8 0 8 16m V 1:400

L E G E N D

⌖	Borehole (Present Investigation)
⌖	Borehole (Previous Investigation–1960)
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
201	55.8	5 004 411.7	233 378.4
202	55.8	5 004 410.1	233 374.5
203	48.9	5 004 435.0	233 372.5
204	49.0	5 004 456.2	233 358.3
205	48.6	5 004 478.9	233 345.1
206	48.3	5 004 467.3	233 337.4
207	49.2	5 004 497.0	233 339.1
208	49.1	5 004 487.5	233 322.1
209	54.6	5 004 533.0	233 309.7
210	54.6	5 004 530.9	233 305.7

-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.
3) Borehole locations are shown in MTM Zone 8 coordinates.

GEOCRES No.

REVISIONS			
DATE	BY		DESCRIPTION

DESIGN KP	CHK –	CODE	LOAD	DATE MAR 2016	
DRAWN MFA	CHK KP	SITE 31–233	STRUCT	DWG 1	

FILENAME: H:\Drafting\19\5161\263\ted1263-BainsvilleRoadUnderpass.dwg
PLOTDATE: 3/29/2016 2:10 PM

APPENDIX B

RECORD OF BOREHOLE SHEETS
BEDROCK CORE PHOTOGRAPHS
HISTORICAL PLAN OF BOREHOLE LOCATIONS (1960 FIELD INVESTIGATION)
HISTORICAL BOREHOLE LOGS (1960 FIELD INVESTIGATION)
SHEAR WAVE VELOCITY PROFILE

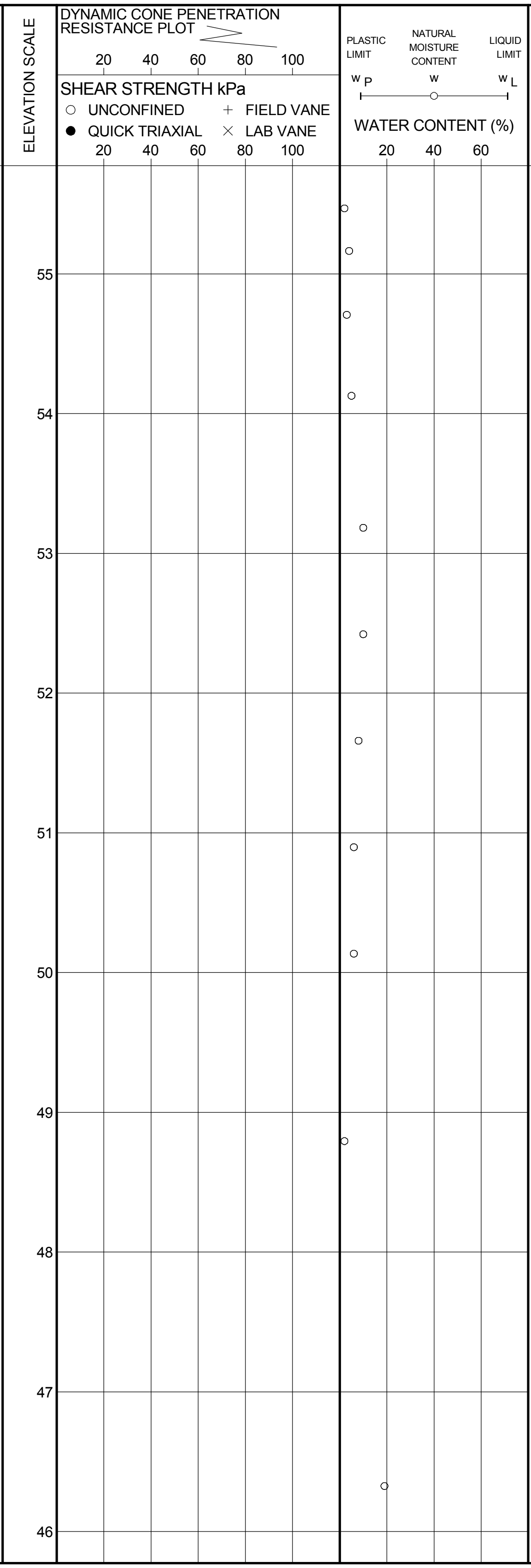
RECORD OF BOREHOLE No 201

1 OF 2

METRIC

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 411.7 E 233 378.4	ORIGINATED BY	JAG
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / NQ Coring	COMPILED BY	KCP
DATUM	Geodetic	DATE	2015.11.17 - 2015.11.17	CHECKED BY	KCP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
55.8										
0.0	125 mm **ASPHALT**									
0.1	Silty sand with gravel Dense Grey **FILL**	1	SS	40						
55.3 0.5	Silty sand Dense Brown **FILL**	2	SS	34			0	88	12 (SI+CL)	
	Some gravel below 1.2 m	3	SS	100/ 225mm						
53.7 2.1	Silty sand with gravel - frequent cobbles Compact to very dense Brown **FILL**	4	SS	11						
		5	SS	22			23	47	30 (SI+CL)	
		6	SS	44						
		7	SS	50						
		8	SS	64						
	- frequent cobbles from 6.1 m to 7.5 m	9	SS	100/ 75mm						
		10	SS	100/ 275mm						
		11	SS	11						
46.6 9.1	**SILTY SAND (SM)** with gravel **TILL** - frequent cobbles and boulders Compact to very dense Grey	12	SS	21			32	50	18 (SI+CL)	



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 201

2 OF 2 **METRIC**

WP# 4088-13-01
LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 411.7 E 233 378.4
ORIGINATED BY JAG
HWY 401
BOREHOLE TYPE Hollow Stem Auger / NQ Coring
COMPILED BY KCP
DATUM Geodetic
DATE 2015.11.17 - 2015.11.17
CHECKED BY KCP

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m^3	GR	SA	SI	CL
	Continued From Previous Page											
						45						
		13	SS	32								
						44						
		14	SS	51								
						43						
						42						
		15	SS	33			○		27	56	17 (SI+CL)	
						41						
		16	SS	29			○					
						40						
						39						
		17	SS	100/ 275mm			○					
						38						
		18	SS	100/ 175mm			○					
						37						
36.7 19.1	End of borehole on inferred bedrock											

+ 3, × 3: Numbers refer to Sensitivity

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

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Ontario

THURBER

RECORD OF BOREHOLE No 202

1 OF 3

METRIC

WP# 4088-13-01
LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 410.1 E 233 374.5
ORIGINATED BY SMP
HWY 401
BOREHOLE TYPE Hollow Stem Auger / HQ Coring
COMPILED BY KCP
DATUM Geodetic
DATE 2015.11.16 - 2015.11.17
CHECKED BY KCP

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
55.8 0.0	125 mm **ASPHALT**
0.1	Silty sand with gravel Dense Grey **FILL**
55.3 0.5	Silty sand Compact Brown **FILL**
54.2 1.5	Silty sand with gravel Compact to very dense Brown **FILL** - grey from 4.5 m to 5.3 m
48.1 7.6	Sandy silt trace clay Compact Brown **FILL**
46.6 9.1	**SAND SP-SM** with silt some gravel Loose to dense Brown

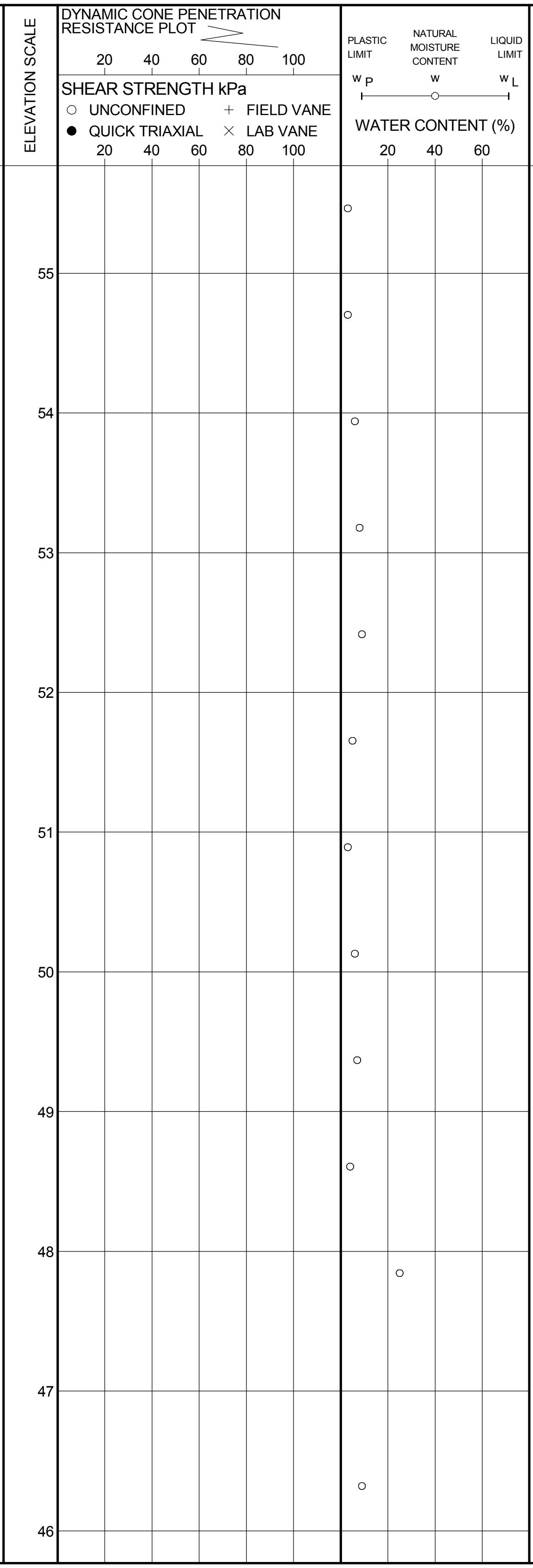
SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	33	35	54	11 (SI+CL)	
2	SS	29				
3	SS	38				
4	SS	17	32	44	24 (SI+CL)	
5	SS	57				
6	SS	57				
7	SS	72				
8	SS	50				
9	SS	45				
10	SS	100/ 150mm				
11	SS	15	0	19	77	4
12	SS	42				

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m 3

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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+ 3, × 3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 202

2 OF 3 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 410.1 E 233 374.5	ORIGINATED BY	SMP
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / HQ Coring	COMPILED BY	KCP
DATUM	Geodetic	DATE	2015.11.16 - 2015.11.17	CHECKED BY	KCP

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	Continued From Previous Page								
	SAND SP-SM with silt some gravel Loose to dense Brown								
					45				
		13	SS	18		13	78	9 (SI+CL)	
					44				
		14	SS	8					
					43				
42.5 13.3	**SILTY SAND (SM)** with gravel **TILL** Compact to very dense - occasional cobbles and boulders Grey								
					42				
		15	SS	28					
					41				
		16	SS	40					
					40				
					39				
		17	SS	39					
					38				
		18	SS	46		30	47	18	5
					37				
					36				
35.8		19	SS	100/					

Continued Next Page

+ 3, × 3 : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 202

3 OF 3

METRIC

WP# 4088-13-01
LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 410.1 E 233 374.5
ORIGINATED BY SMP
HWY 401
BOREHOLE TYPE Hollow Stem Auger / HQ Coring
COMPILED BY KCP
DATUM Geodetic
DATE 2015.11.16 - 2015.11.17
CHECKED BY KCP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
19.9	**LIMESTONE BEDROCK** Fresh Medium to thickly bedded Good to excellent quality		1	NQ	125mm						RUN #1 TCR=100% SCR=85% RQD=55%
							35				
			2	NQ							RUN #2 TCR=73% SCR=73% RQD=63%
							34				
			3	NQ			33				RUN #3 TCR=100% SCR=97% RQD=31%
32.5 23.3	End of borehole										

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+ 3 , × 3 : Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 203

1 OF 2

METRIC

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 435.0 E 233 372.5	ORIGINATED BY	JAG
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / NQ Coring	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.18 - 2015.11.18	CHECKED BY	KCP

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
48.9										
0.0	125 mm **TOPSOIL**									
0.1	Sand with silt and gravel Loose Brown **FILL**	1	SS	9			33	60	7 (SI+CL)	
48.2 0.8	Sand, trace silt, trace gravel Compact to very loose Brown Wet **FILL** **Strong hydrocarbon odour noted from 0.76 m to 1.4 m**	2	SS	10		48				
		3	SS	8		47				
		4	SS	3			9	85	6 (SI+CL)	
45.6 3.4	**SAND (SP-SM)** with silt and gravel - occasional cobbles Compact to dense Brown to grey Wet	5	SS	17		46				
		6	SS	41		45	35	54	11 (SI+CL)	
		7	SS	36		44				
43.6 5.3	**SILTY SAND (SM)** with gravel **TILL** - ocassional cobbles Compact to very dense Grey	8	SS	41		43				
		9	SS	110		42				
		10	SS	35			41	49	10 (SI+CL)	
		11	SS	29		41				
						40				
		12	SS	36		39	20	55	21	4

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 203 2 OF 2 METRIC

WP# 4088-13-01	LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 435.0 E 233 372.5	ORIGINATED BY JAG
HWY 401	BOREHOLE TYPE Hollow Stem Auger / NQ Coring	COMPILED BY CAM
DATUM Geodetic	DATE 2015.11.18 - 2015.11.18	CHECKED BY KCP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	Continued From Previous Page						
	SILTY SAND (SM) with gravel **TILL** - ocassional cobbles Compact to very dense Grey - cobbles from 9.0 m to 12.3 m	13	SS	100/ 175mm		38	
						37	
36.6 12.3		14	SS	100/ 150mm			
	LIMESTONE BEDROCK Moderately weathered Very thinly bedded Very poor to poor quality	1	NQ			36	RUN #1 TCR=79% SCR=43% RQD=17%
		2	NQ				RUN #2 TCR=94% SCR=63% RQD=25%
35.0 13.9	**LIMESTONE BEDROCK** Fresh Thinly to medium bedded Poor to fair quality	3	NQ			35	RUN #3 TCR=82% SCR=61% RQD=45%
						34	
		4	NQ			33	RUN #4 TCR=100% SCR=98% RQD=52%
32.6 16.4	End of borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

GR SA SI CL

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

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

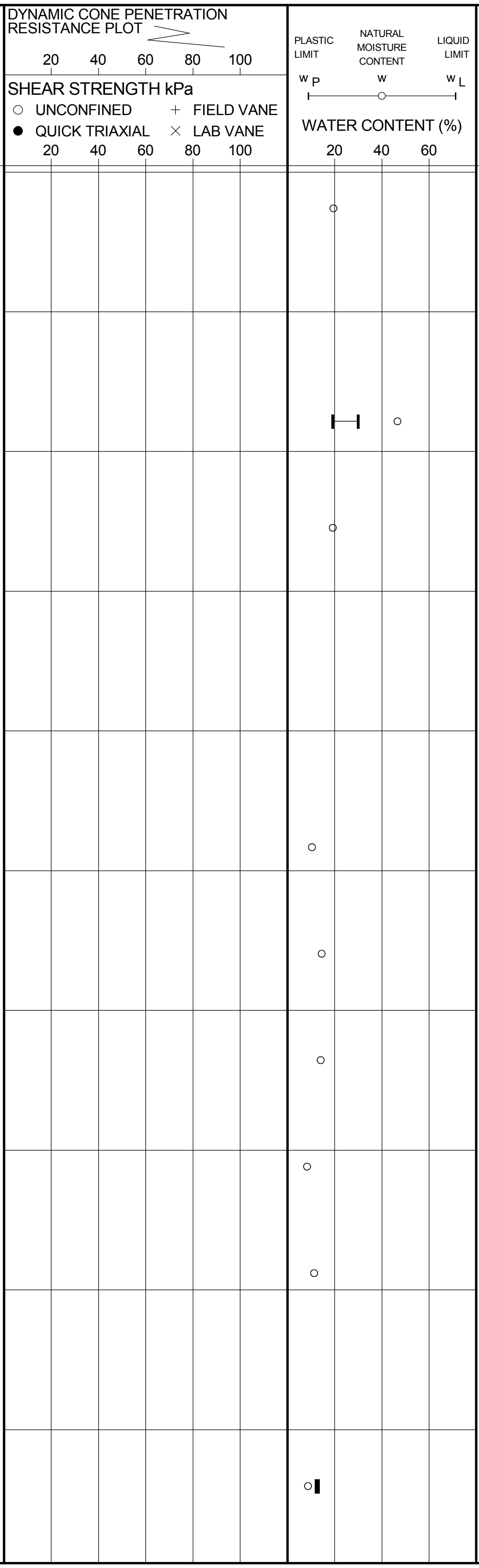
Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 204

1 OF 2 **METRIC**

WP# 4088-13-01	LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 456.2 E 233 358.3	ORIGINATED BY JAG
HWY 401	BOREHOLE TYPE Hollow Stem Auger / NQ Coring	COMPILED BY CAM
DATUM Geodetic	DATE 2015.11.30 - 2015.11.30	CHECKED BY KCP



ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
49.0									
0.0	125 mm **TOPSOIL**						49		
0.2	Silty sand trace gravel Loose Brown **FILL**		1	SS	6				7 56 37 (SI+CL)
			2	SS	6		48		
47.5 1.5	**CLAY (CL)** Stiff Brown		3	SS	3				5 15 57 23
							47		
			4	SS	11				
46.0 3.0	**Sand (SP-SM)** with silt and gravel Compact Brown to Grey						46		
			5	SS	29				
			6	SS	18		45		
			7	SS	19				31 59 10 (SI+CL)
							44		
			8	SS	13				
							43		
42.6 6.4	**SILTY SAND (SM)** with gravel **TILL** Compact to very dense Grey		9	SS	29				
			10	SS	19		42		
			11	SS	67		41		
							40		
			12	SS	53				20 43 29 8

Continued Next Page

+ 3, × 3 : Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 204

2 OF 2 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 456.2 E 233 358.3	ORIGINATED BY	JAG
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / NQ Coring	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.30 - 2015.11.30	CHECKED BY	KCP

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND (SM) with gravel **TILL** Compact to very dense Grey					39				
		13	SS	49		38		○		
						37				
		14	SS	39				○		
35.6						36				
13.5	**LIMESTONE BEDROCK** Thinly bedded Slightly weathered Poor to fair quality	1	NQ							
		2	NQ			35				RUN #2 TCR=78% SCR=48% RQD=33%
						34				
		3	NQ			33				RUN #3 TCR=97% SCR=93% RQD=67%
32.3 16.8	End of borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 205

1 OF 2 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 478.9 E 233 345.1	ORIGINATED BY	JAG
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / NQ Coring	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.16 - 2015.11.17	CHECKED BY	KCP

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
48.6	
0.0	200 mm **TOPSOIL**
0.2	Silt some sand trace clay Loose to compact Brown **FILL**
47.0 / 1.5	**CLAY (CL)** Soft to firm Grey -Thin gravel layer at 3.9 m
44.0 / 4.6	**SILTY SAND (SP-SM)** trace gravel Compact Grey
43.2 / 5.3	**SILTY SAND (SM)** with gravel **TILL** Compact to very dense Grey - cobbles and boulders from 7.6 m to 12.8 m

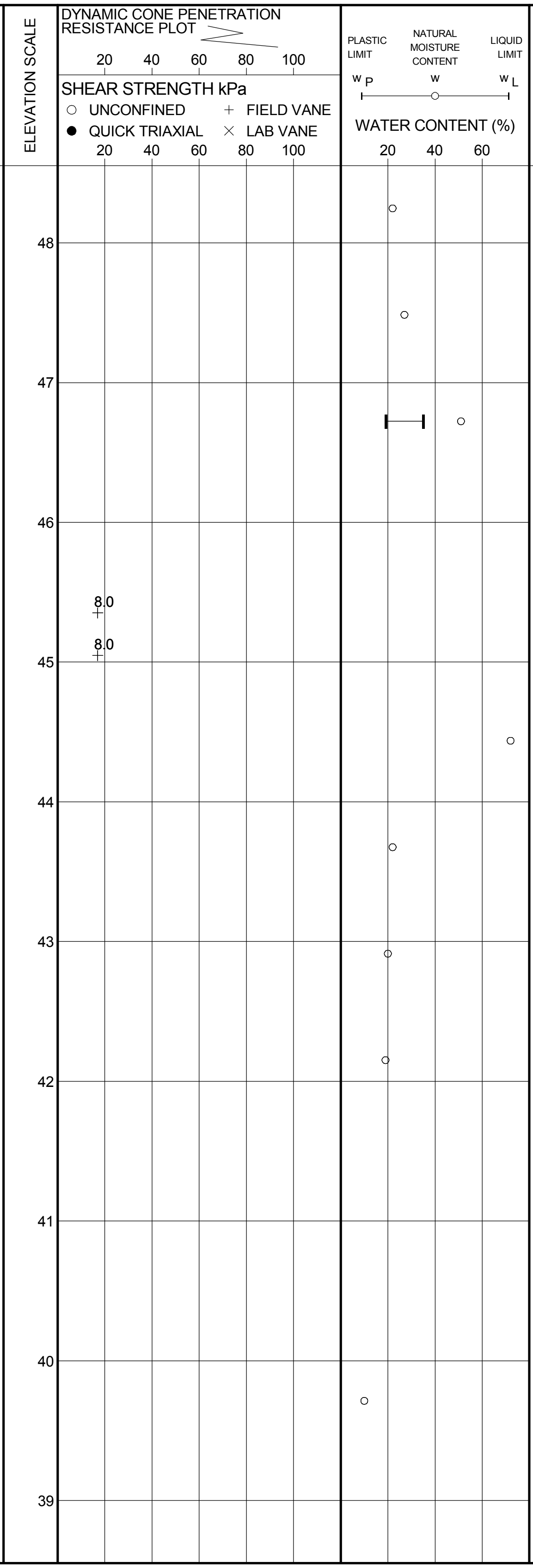
SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	5				
2	SS	11	0	13	83	4
3	SS	3	2	16	45	37
4	SS	WH				
5	SS	19				
6	SS	11	6	84	10 (SI+CL)	
7	SS	11	17	48	26	9
8	SS	23				
9	SS	24				
10	SS	12				
11	SS	42				

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m3

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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+3, ×3: Numbers refer to Sensitivity

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

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THURBER

RECORD OF BOREHOLE No 205

2 OF 2 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 478.9 E 233 345.1	ORIGINATED BY	JAG
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / NQ Coring	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.16 - 2015.11.17	CHECKED BY	KCP

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION (Continued From Previous Page)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND (SM) with gravel **TILL** Compact to very dense Grey	12	SS	20		38	
		13	SS	100/ 225mm		37	21 42 30 7
35.7 12.8	**LIMESTONE BEDROCK** Slightly to moderately weathered Very thinly bedded Poor quality	1	NQ			36 35	RUN #1 TCR=93% SCR=68% RQD=35%
34.7 13.8	**LIMESTONE BEDROCK** Fresh Thinly to medium bedded Fair quality	2	NQ			34	RUN #2 TCR=95% SCR=84% RQD=56%
		3	NQ			33	RUN #3 TCR=100% SCR=97% RQD=72%
32.3 16.2	End of borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

+ 3, × 3: Numbers refer to Sensitivity

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

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Ministry of Transportation
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THURBER

RECORD OF BOREHOLE No 206

1 OF 2

METRIC

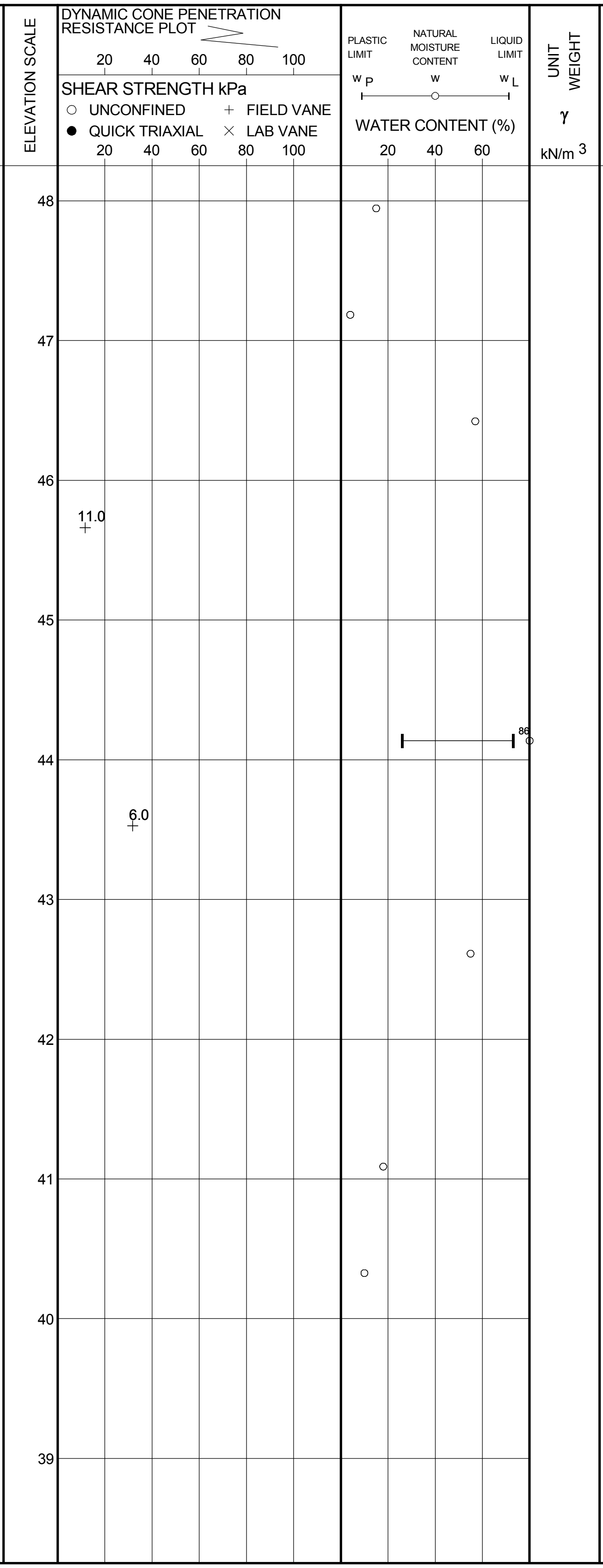
WP# 4088-13-01
LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 467.3 E 233 337.4
ORIGINATED BY JAG
HWY 401
BOREHOLE TYPE Hollow Stem Auger / Casing
COMPILED BY CAM
DATUM Geodetic
DATE 2015.11.17 - 2015.11.17
CHECKED BY KCP

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
48.3	
0.0	150 mm **TOPSOIL**
0.2	Silt with sand trace gravel Loose to compact Brown **FILL**
46.4 / 1.8	**CLAY (CH)** Soft to firm Grey Wet -Thin sand layer at 5.0 m
41.7 / 6.6	**SILTY SAND (SP-SM)** with gravel Loose Grey Wet
40.3 / 7.9	**SILTY SAND (SM)** with gravel **TILL** - occasional cobbles and boulders Dense to very dense Grey

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	8	8	16	70	6
2	SS	16				
3	SS	3				
4	TW	Push				
5	SS	WH	0	0	27	73
6	SS	WH				
7	TW	Push				
8	SS	6	20	30	42	8
9	SS	8				
10	SS	100/ 100mm				



REMARKS & GRAIN SIZE DISTRIBUTION (%)

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+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 206

2 OF 2 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 467.3 E 233 337.4	ORIGINATED BY	JAG
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / Casing	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.17 - 2015.11.17	CHECKED BY	KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SILTY SAND (SM) with gravel **TILL** Dense to very dense Grey						38				
			11	SS	42		37				
							36				
			12	SS	66				○		42 47 11 (SI+CL)
35.2 13.0	End of borehole on inferred bedrock										

+ 3, × 3 : Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

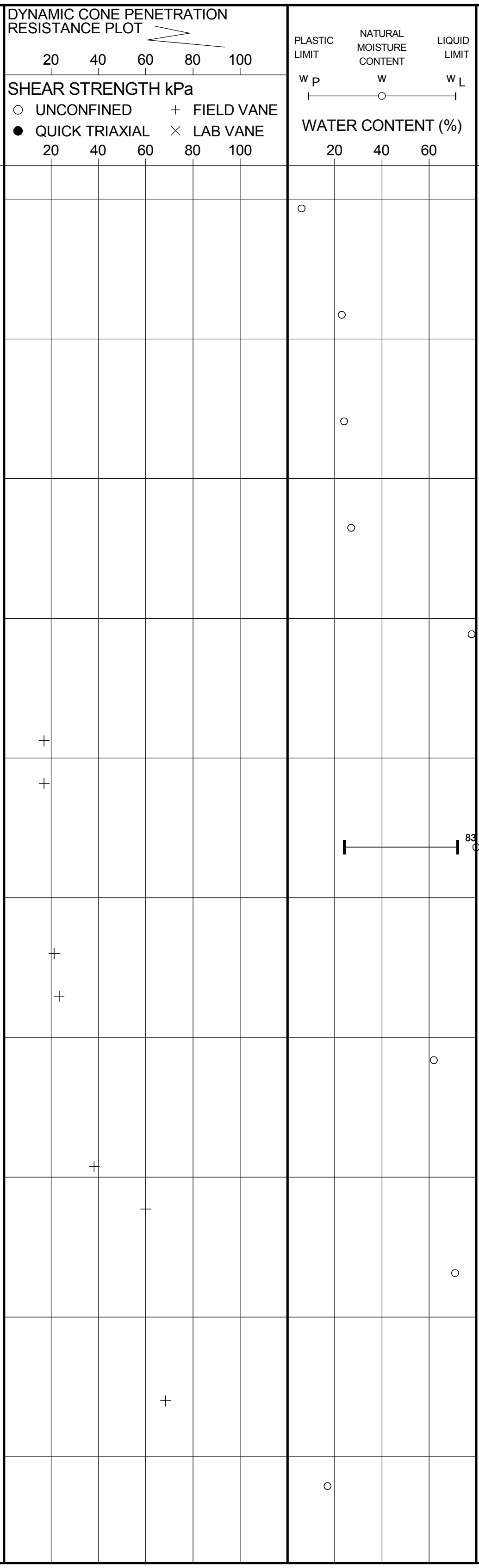
THURBER

RECORD OF BOREHOLE No 207

1 OF 4 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 497.0 E 233 339.1	ORIGINATED BY	SMP
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / HQ Coring / Downhole Seismic Testing	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.09 - 2015.11.10	CHECKED BY	KCP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
49.2									
0.0	Sand with silt and gravel Compact Grey **FILL**	1	SS	17		42	48	10 (SI+CL)	
48.5 0.8	Sandy silt with gravel Loose to compact Brown **FILL**	2	SS	6		18	31	51 (SI+CL)	
		3	SS	13					
47.0 2.3	**SANDY SILT (ML)** Compact Grey	4	SS	12		0	13	87 (SI+CL)	
46.2 3.0	**CLAY (CH)** Soft to stiff Grey	5	SS	WH					
		8	SS	WH		0	0	31	69
		11	SS	WH					
	-Gravely region within clay from 7.6 m to 8.8 m	14	SS	WH					
40.1 9.1	**SILTY SAND (SM)** with gravel **TILL** - occasional cobbles Compact to very dense Grey	16	SS	64					



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

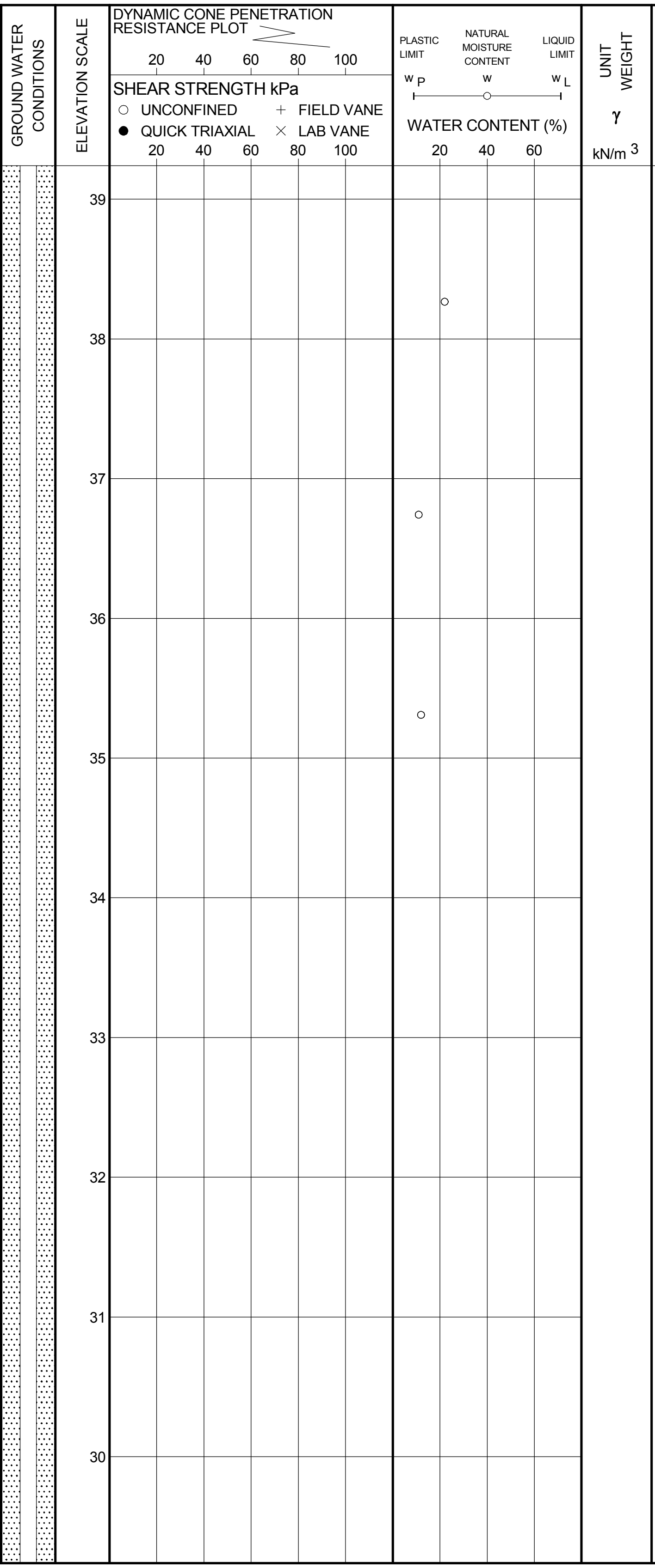
RECORD OF BOREHOLE No 207

2 OF 4 **METRIC**

WP# 4088-13-01
LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 497.0 E 233 339.1
ORIGINATED BY SMP
HWY 401
BOREHOLE TYPE Hollow Stem Auger / HQ Coring / Downhole Seismic Testing
COMPILED BY CAM
DATUM Geodetic
DATE 2015.11.09 - 2015.11.10
CHECKED BY KCP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND (SM) with gravel **TILL** - occasional cobbles Compact to Very Dense Grey	17	SS	26	
		18	SS	15	35 48 17 (SI+CL)
35.1		19	SS	100/ 225mm	
14.2	**LIMESTONE BEDROCK** Slightly to moderately weathered Very thinly to thinly bedded Very poor to fair quality	1	HQ		RUN #1 TCR=88% SCR=70% RQD=57%
33.7		2	HQ		RUN #2 TCR=100% SCR=8% RQD=0%
15.5	**LIMESTONE BEDROCK** Fresh Medium to thickly bedded Poor to excellent quality	3	HQ		RUN #3 TCR=67% SCR=60% RQD=45%
		4	HQ		RUN #4 TCR=100% SCR=98% RQD=65%
		5	HQ		RUN #5 TCR=88% SCR=88% RQD=55%



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 207

3 OF 4

METRIC

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 497.0 E 233 339.1	ORIGINATED BY	SMP
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / HQ Coring / Downhole Seismic Testing	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.09 - 2015.11.10	CHECKED BY	KCP

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

Continued From Previous Page

LIMESTONE BEDROCK
Fresh
Medium to thickly bedded
Fair to excellent quality

NUMBER	TYPE	REMARKS
6	HQ	RUN #6 TCR=100% SCR=100% RQD=100%
7	HQ	RUN #7 TCR=100% SCR=100% RQD=95%
8	HQ	RUN #8 TCR=97% SCR=66% RQD=56%
9	HQ	RUN #9 TCR=98% SCR=83% RQD=56%
10	HQ	RUN #10 TCR=100% SCR=100% RQD=93%
11	HQ	RUN #11 TCR=100% SCR=100% RQD=100%
12	HQ	RUN #12 TCR=100% SCR=100% RQD=91%

ELEVATION SCALE: 29, 28, 27, 26, 25, 24, 23, 22, 21, 20

Continued Next Page

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

$15 \oplus 5$ (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 207

4 OF 4 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 497.0 E 233 339.1	ORIGINATED BY	SMP
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / HQ Coring / Downhole Seismic Testing	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.09 - 2015.11.10	CHECKED BY	KCP

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	LIMESTONE BEDROCK Fresh Medium to thickly bedded Fair to excellent quality						19				
			13	HQ							RUN #13 TCR=98% SCR=98% RQD=98%
							18				
			14	HQ							RUN #14 TCR=100% SCR=91% RQD=86%
16.9							17				
32.4	End of borehole Downhole seismic testing conducted on January 20, 2016 Well construction details: 0 - 32.4 m - 50 mm inside diameter PVC pipe PVC pipe was encased in grout along entire length as per ASTM D7400										

+ 3, × 3: Numbers refer to Sensitivity

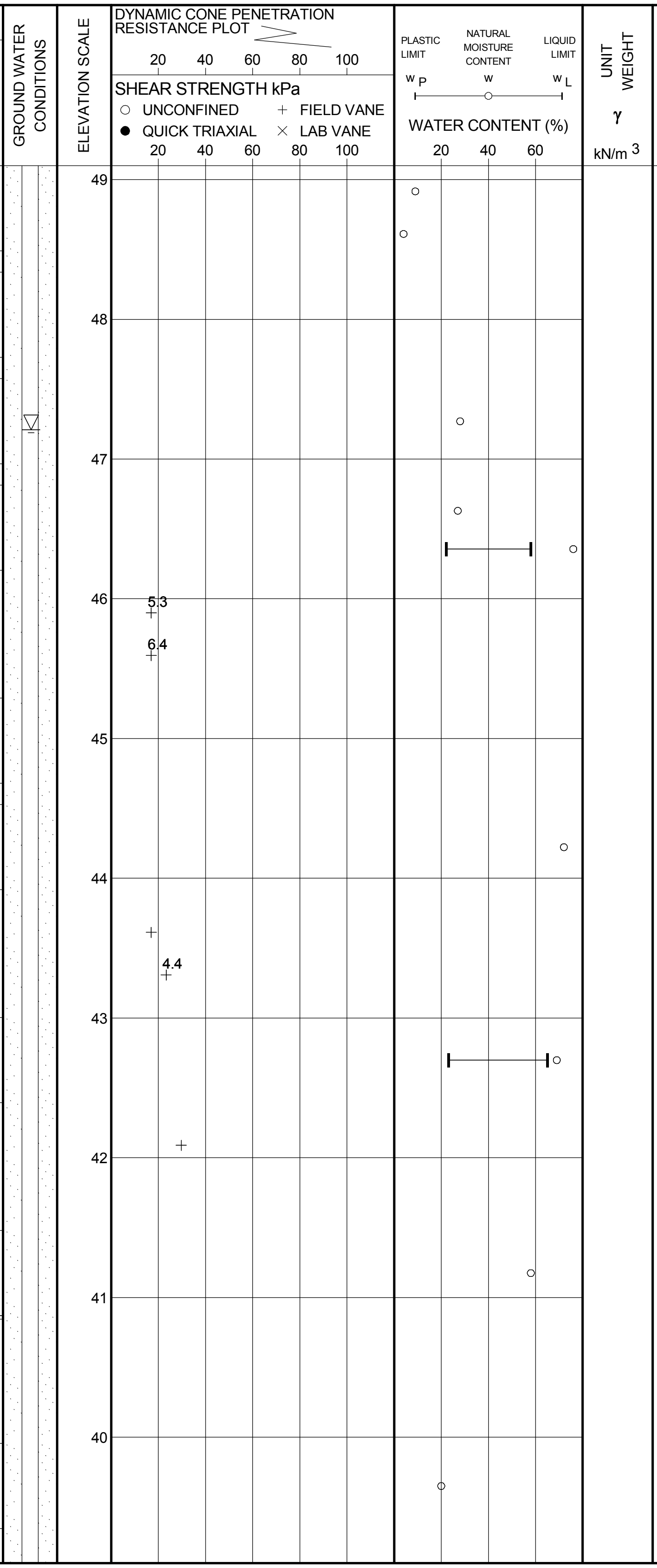
20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 208

1 OF 2 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 487.5 E 233 322.1	ORIGINATED BY	SMP
HWY	401	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.11 - 2015.11.11	CHECKED BY	KCP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
49.1								
0.0	225 mm **TOPSOIL**							
0.2	Silty sand with gravel Loose to compact Brown **FILL**	1	SS	12	32	53	15 (SI+CL)	
		2	SS	5				
47.6 1.5	**SILT (ML)** with sand Compact Grey	3	SS	15	0	21	79 (SI+CL)	
46.7 2.4	**SILTY CLAY** Soft to firm Grey	4	SS	3	0	1	39	60
		7	ST	Push				
		8	SS	WH				
		11	SS	WH	0	4	31	65
	-Gravelly region within clay from 7.6 m to 8.2 m	14	SS	2				
40.8 8.3	**SILTY SAND (SM)** with gravel **TILL** - occasional cobbles Compact to very dense Grey	15	SS	100/ 25mm				
		16	SS	12				



Continued Next Page

+3, ×3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 208

2 OF 2 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 487.5 E 233 322.1	ORIGINATED BY	SMP
HWY	401	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.11 - 2015.11.11	CHECKED BY	KCP

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	SILTY SAND (SM) with gravel **TILL** - occasional cobbles Compact to very dense Grey						39				
			17	SS	34		38		○		
							37				
36.4			18	SS	51				○		26 56 18 (SI+CL)
12.7	End of borehole on inferred bedrock Groundwater measured at 1.9 m BGS on December 7, 2015										

+^{3}, ×^{3}: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

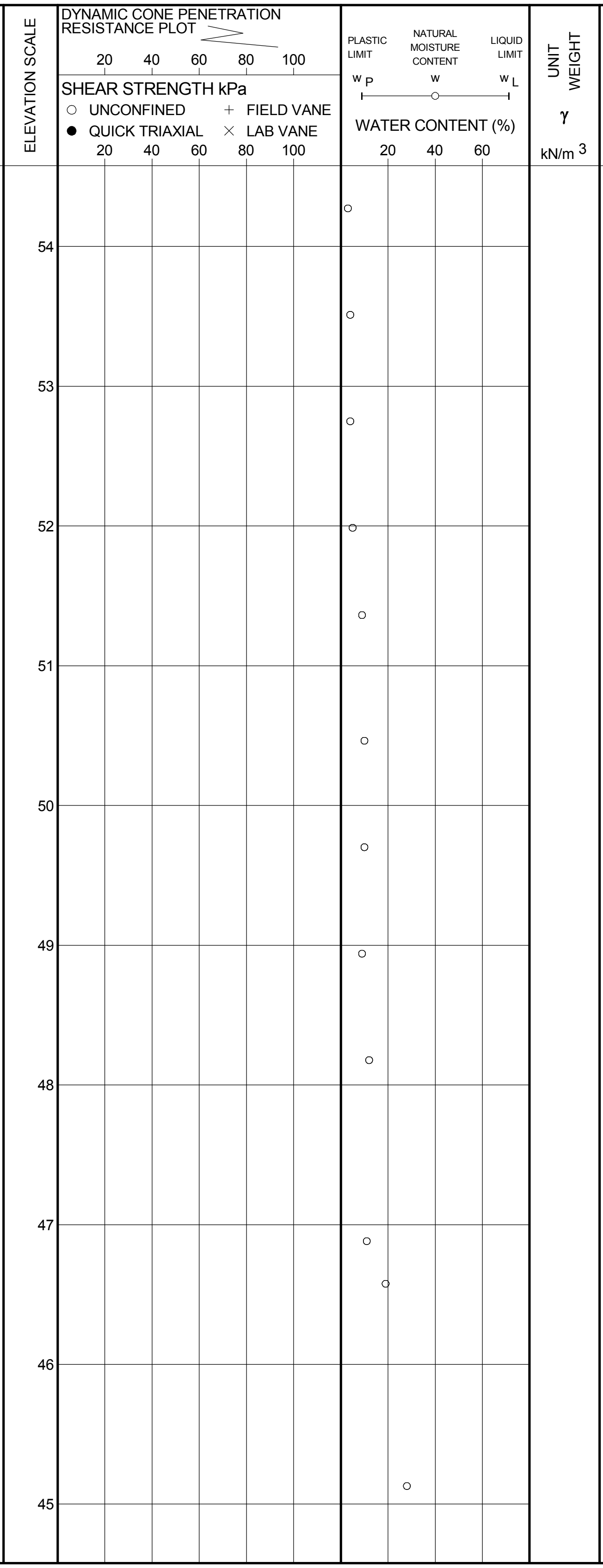
THURBER

RECORD OF BOREHOLE No 209

1 OF 3 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 533.0 E 233 309.7	ORIGINATED BY	SMP
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / NQ Coring	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.12 - 2015.11.13	CHECKED BY	KCP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
54.6								
0.0	125 mm **ASPHALT**							
0.1	Sand with gravel trace silt Dense Brown Moist **FILL**	1	SS	49	33	64	3 (SI+CL)	
		2	SS	100/ 300mm				
53.4 1.2	Sand with silt trace gravel Compact Brown **FILL**	3	SS	30	1	91	8 (SI+CL)	
		4	SS	12				
51.2 3.4	Silty sand with gravel Very loose to very dense Brown **FILL**	5	SS	18				
		6	SS	3	30	46	24 (SI+CL)	
		7	SS	59				
49.2 5.3	Silty gravel with sand - occasional cobbles Dense to very dense Grey **FILL**	8	SS	33	40	39	21 (SI+CL)	
		9	SS	55				
		10	SS	100/ 75mm				
46.8 7.8	**SILT (ML)**, trace organics Compact Greyish Green	11	SS	23	3	9	75	13
		12	SS	7	0	16	79	5



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+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

ONTMT4S BAINSVILLE.GPJ 2012TEMPLATE(MTO).GDT 1/6/16

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 209

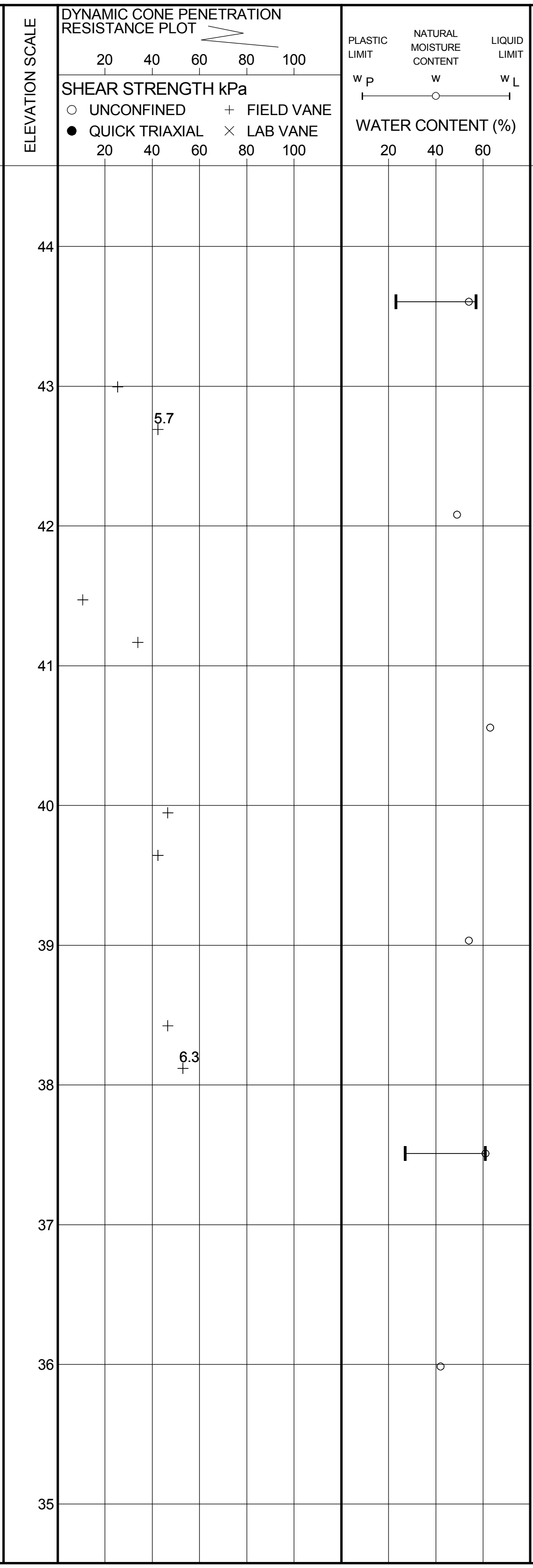
2 OF 3

METRIC

WP# 4088-13-01
LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 533.0 E 233 309.7
ORIGINATED BY SMP
HWY 401
BOREHOLE TYPE Hollow Stem Auger / NQ Coring
COMPILED BY CAM
DATUM Geodetic
DATE 2015.11.12 - 2015.11.13
CHECKED BY KCP

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	SILT (ML), trace organics Compact Greyish Green							
43.9 10.7	**CLAY (CH)** Soft to firm Grey	13	SS	WH	0	1	35	64
		16	SS	WH				
		19	SS	WH				
		22	SS	WH				
		25	SS	WH	0	0	16	84
	- with gravel from 18.1 m to 19.2 m							
		27	SS	WH				
35.4		28	SS	100/0mm				
19.2	**LIMESTONE BEDROCK** - occasional shale seams Moderately weathered Laminated to thinly bedded Poor quality	1	NQ					



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 209

3 OF 3

METRIC

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 533.0 E 233 309.7	ORIGINATED BY	SMP
HWY	401	BOREHOLE TYPE	Hollow Stem Auger / NQ Coring	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.12 - 2015.11.13	CHECKED BY	KCP

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m ^{3}	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	LIMESTONE BEDROCK - occasional shale seams Moderately weathered Laminated to thinly bedded Poor quality		2	NQ							RUN #2 TCR=43% SCR=32% RQD=20%
							34				
			3	NQ			33				RUN #3 TCR=80% SCR=55% RQD=28%
32.2 22.4	**LIMESTONE BEDROCK** Fresh Medium to thickly bedded Good quality						32				
			4	NQ							RUN #4 TCR=95% SCR=95% RQD=75%
							31				
30.6 24.0	End of borehole										

ONTMT4S BAINSVILLE.GPJ 2012TEMPLATE(MTO).GDT 1/6/16

+ ^{3}, × ^{3} : Numbers refer to Sensitivity

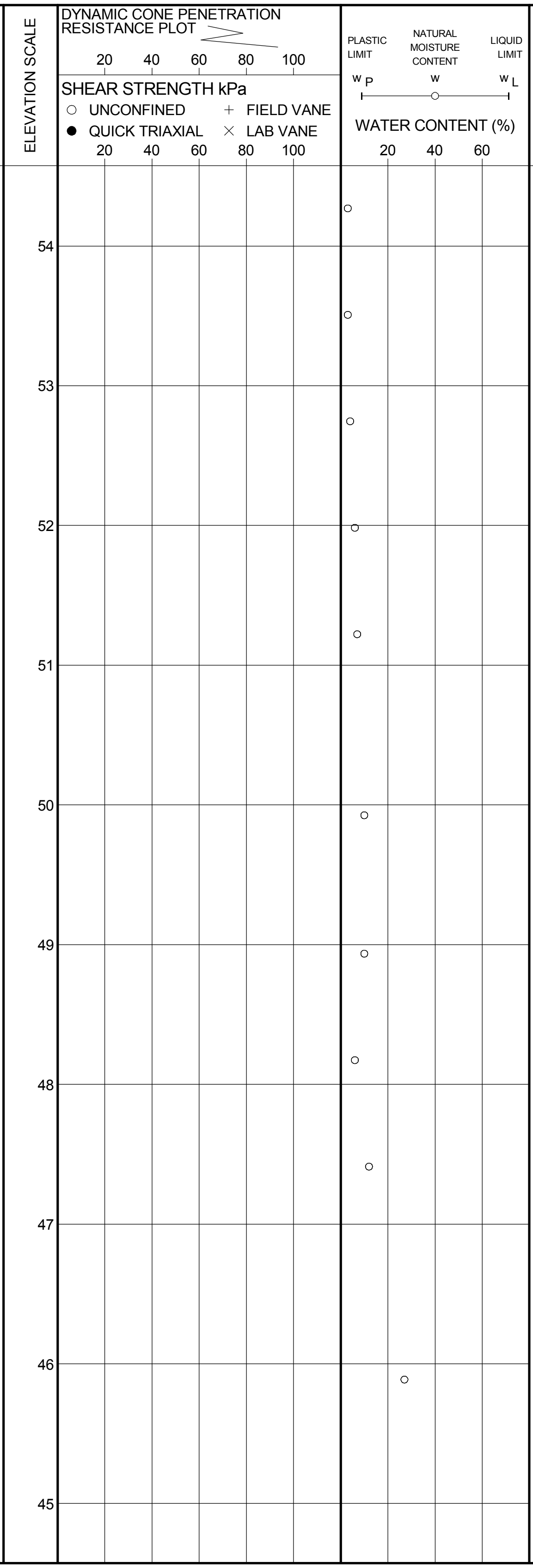
20
15 5
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 210

1 OF 3 **METRIC**

WP#	4088-13-01	LOCATION	Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 530.9 E 233 305.7	ORIGINATED BY	SMP
HWY	401	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	CAM
DATUM	Geodetic	DATE	2015.11.13 - 2015.11.13	CHECKED BY	KCP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
54.6											
0.0	125 mm **ASPHALT**										
0.1	Silty sand with gravel Very dense Grey **FILL**		1	SS	57						
			2	SS	100/ 275mm			27	63	10 (SI+CL)	
53.0 1.5	Sand with silt Compact to dense Brown **FILL**		3	SS	34			0	89	11 (SI+CL)	
			4	SS	19						
			5	SS	24						
50.9 3.7	Silty sand with gravel - occasional cobbles Very loose to very dense Brown to grey Moist **FILL**		6	SS	3			32	44	24 (SI+CL)	
			7	SS	100/ 125mm						
	- Grey from 5.3 m		8	SS	28						
			9	SS	66						
			10	SS	21			34	47	19 (SI+CL)	
46.8 7.8	**SILT (ML)**, trace organics Compact to dense Greyish green Wet **- Hydrocarbon odour noted from 7.8 m to 8.4 m**		11	SS	55						
			12	SS	10			0	12	81	7
44.7											



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+ 3, × 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

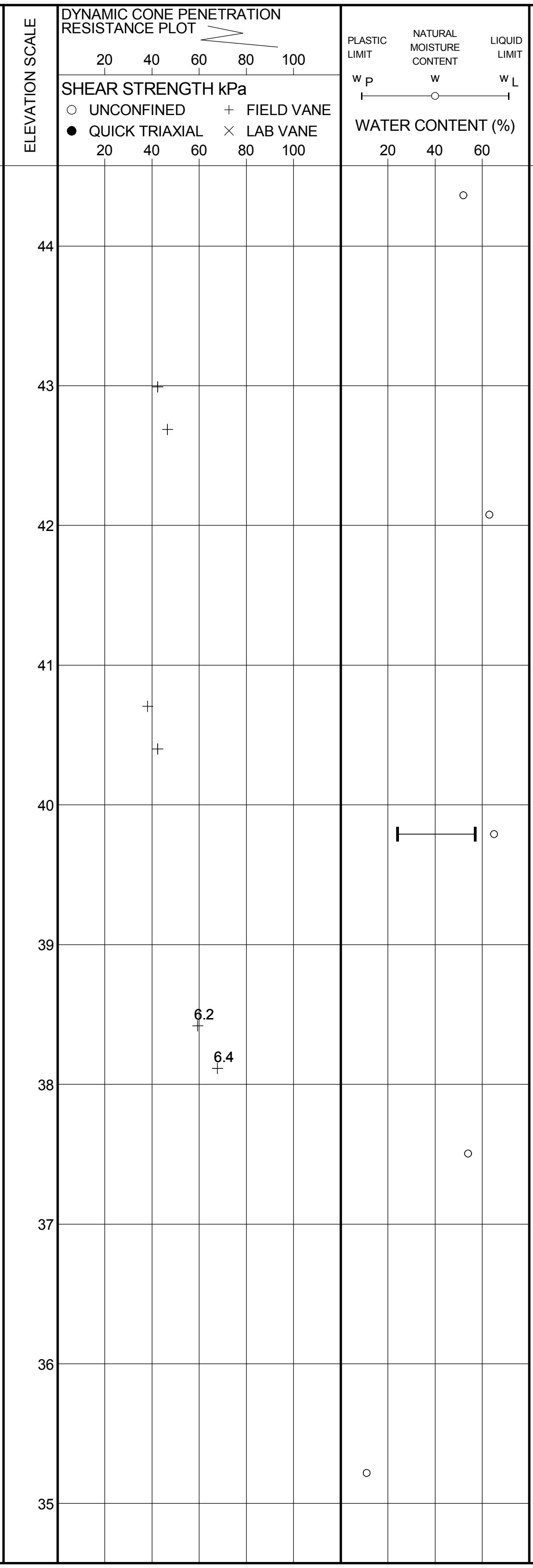
RECORD OF BOREHOLE No 210

2 OF 3

METRIC

WP# 4088-13-01
LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 530.9 E 233 305.7
ORIGINATED BY SMP
HWY 401
BOREHOLE TYPE Hollow Stem Auger
COMPILED BY CAM
DATUM Geodetic
DATE 2015.11.13 - 2015.11.13
CHECKED BY KCP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	Continued From Previous Page										
9.9	**CLAY (CH)** Firm Grey		13	SS	WH						
			16	SS	WH						
			19	SS	WH			0	2	18	80
			22	SS	WH						
	- with gravel from 18.1 m to 19.2 m										
35.5 19.1	**SILTY SAND (SM)** with gravel **TILL** - frequent cobbles Dense Grey		25	SS	46			43	35	16	6
			26	SS	100/						



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+ 3, × 3 : Numbers refer to Sensitivity

20
15 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 210

3 OF 3

METRIC

WP# 4088-13-01
LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8: N 5 004 530.9 E 233 305.7
ORIGINATED BY SMP
HWY 401
BOREHOLE TYPE Hollow Stem Auger
COMPILED BY CAM
DATUM Geodetic
DATE 2015.11.13 - 2015.11.13
CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
	Continued From Previous Page													
34.5 20.0	End of borehole on inferred bedrock				225mm									

+ 3 , × 3 : Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 205A

1 OF 1

METRIC

WP# 4088-13-01
LOCATION Highway 401 Underpass at Bainsville Rd., MTM Zone 8:
ORIGINATED BY JAG
HWY 401
BOREHOLE TYPE Hollow Stem Auger
COMPILED BY CAM
DATUM Geodetic
DATE 2015.11.17 - 2015.11.17
CHECKED BY KCP

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
48.6 0.0	**TOPSOIL (200mm)**									
0.2	**SILTY SAND** Loose to Compact Brown Moist					48				
47.0 1.5	**SILTY CLAY** Soft to Very Soft Grey Wet					47				
		1	TW			46				
		2	TW			45				
44.9 3.7	End of Borehole at 3.7 m									

ONTMT4S BAINSVILLE.GPJ 2012TEMPLATE(MTO).GDT 1/6/16

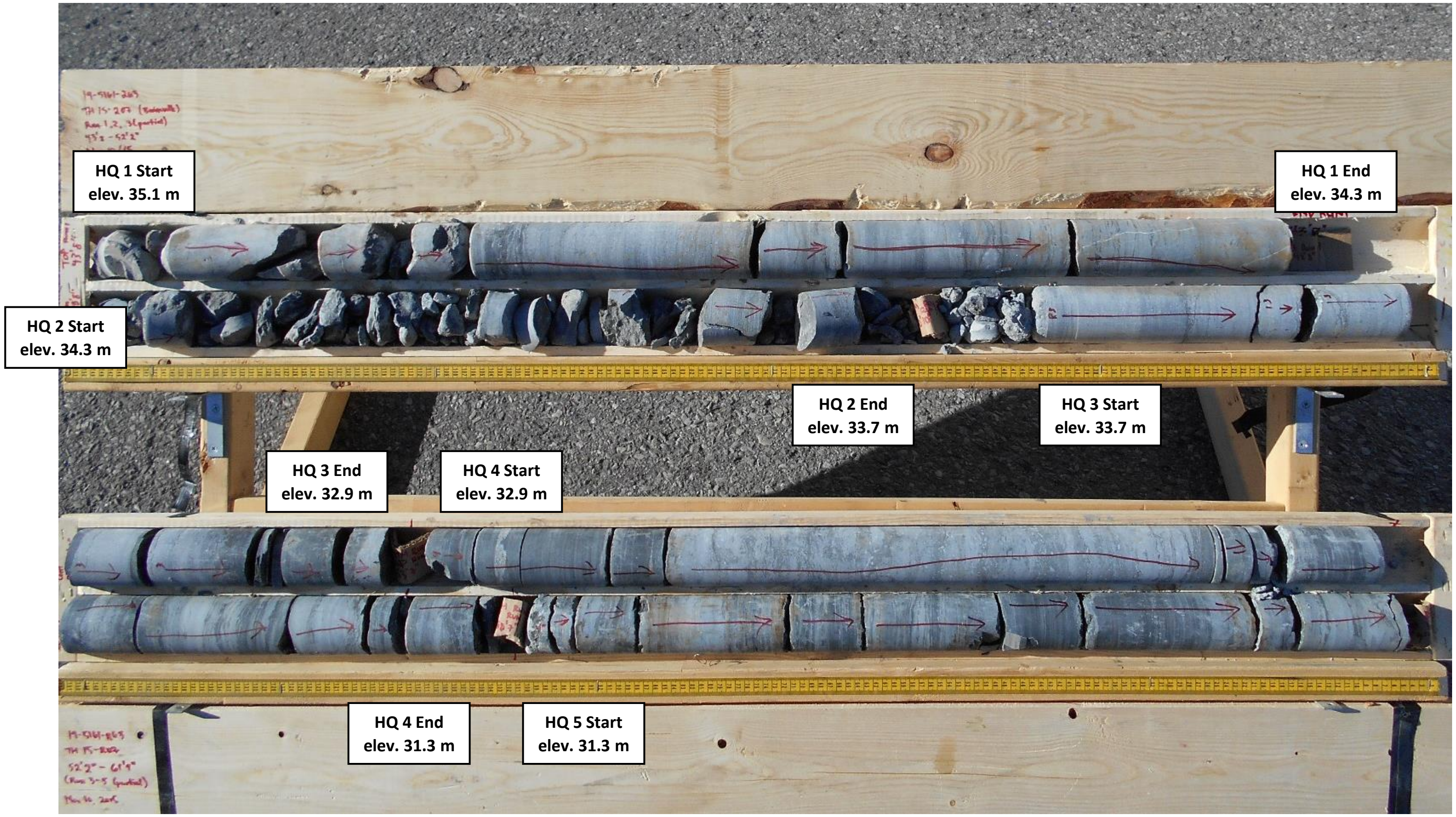
+ ³, × ³: Numbers refer to Sensitivity

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

Borehole 207

HQ 1 to 5 (of 14)

Elevation 35.1 m to 31.3 m



THURBER ENGINEERING LTD.

Foundation Investigation
Highway 401 – Bainsville Underpass
Site 31-241
Township of Lancaster, Ontario

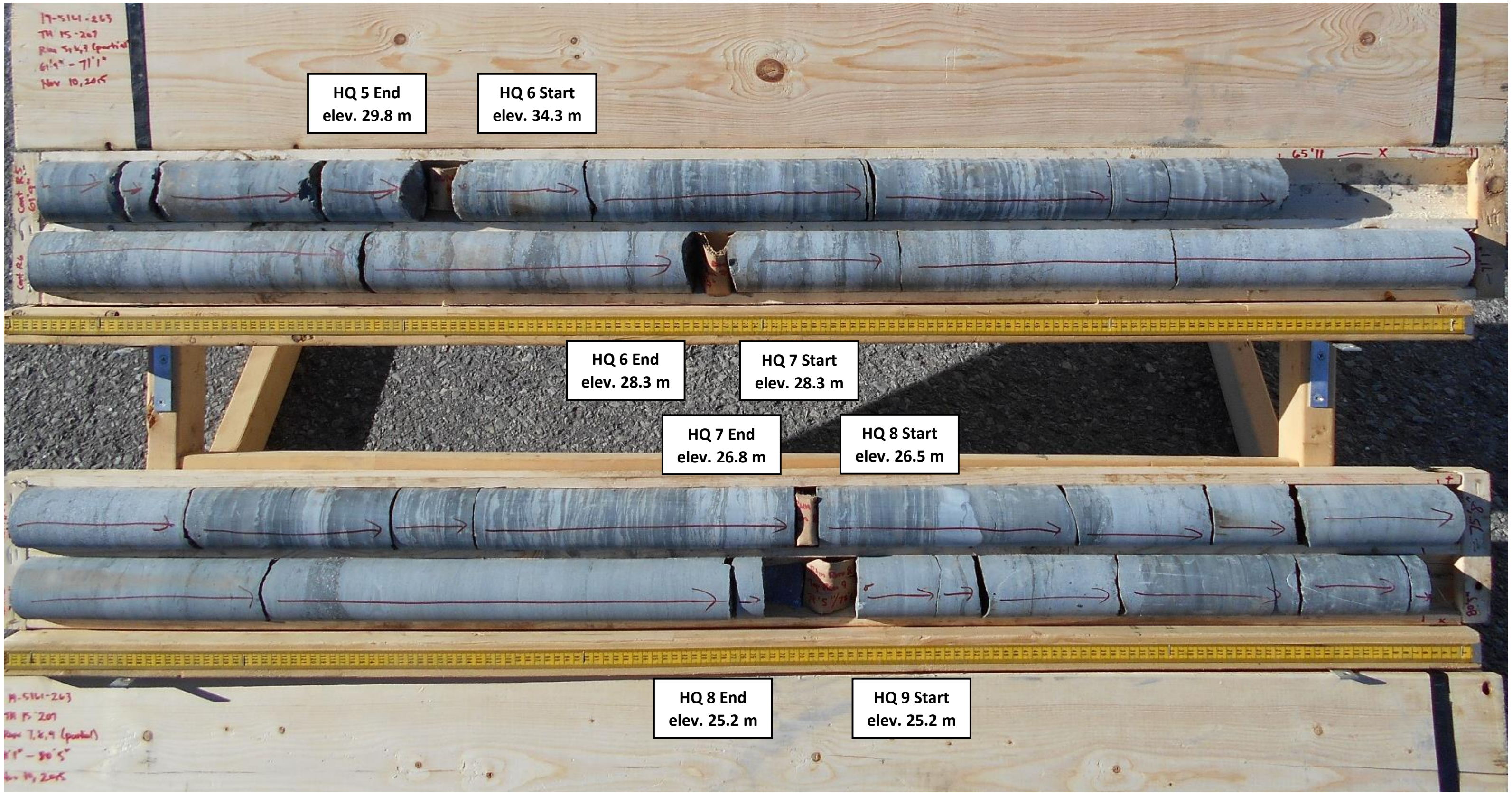
GWP: 4027-14-00

Project No.: 19-5161-263

Borehole 207

HQ 5 to 9 (of 14)

Elevation 29.8 m to 25.2 m



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Foundation Investigation
Highway 401 – Bainsville Underpass
Site 31-241
Township of Lancaster, Ontario

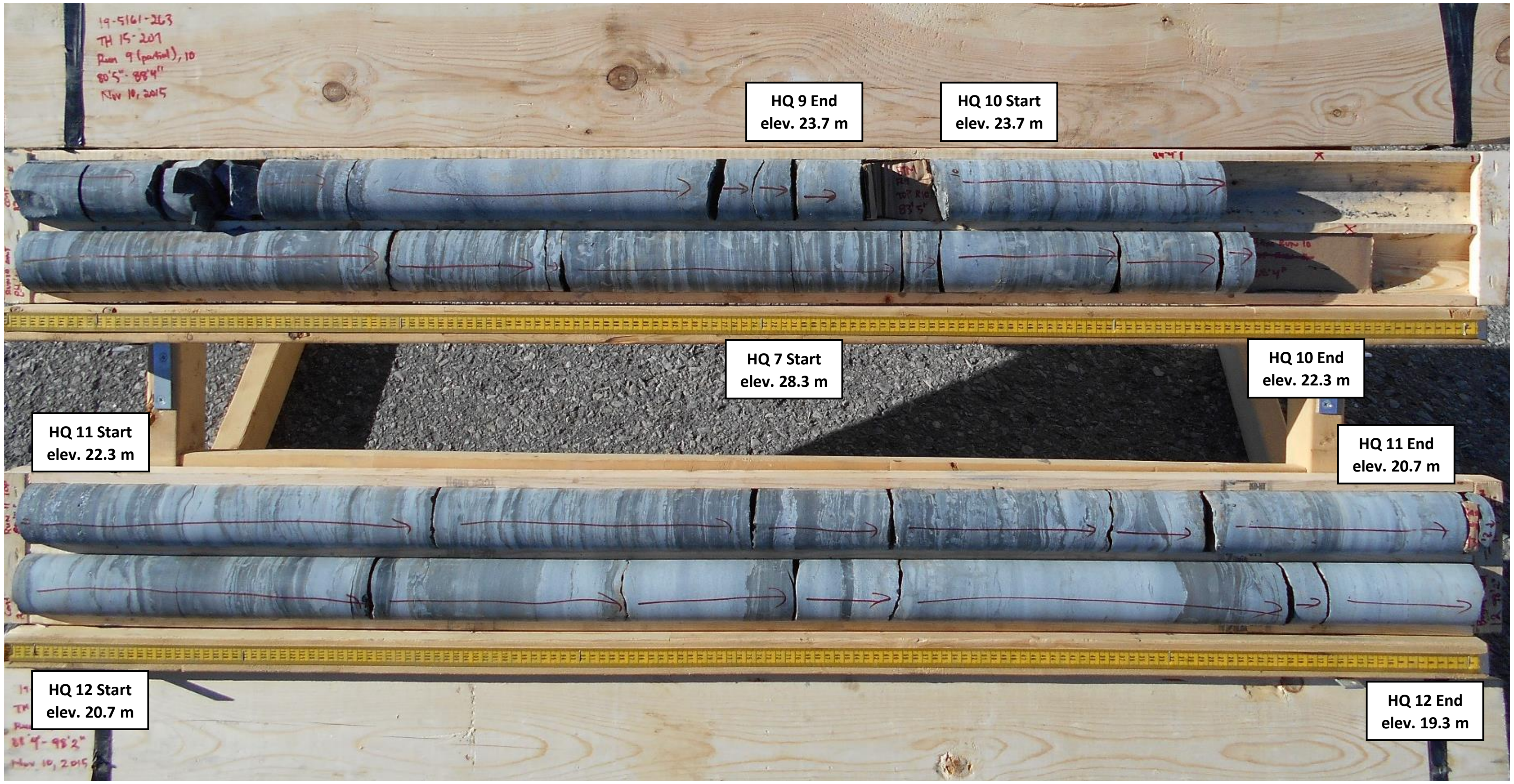
GWP: 4027-14-00

Project No.: 19-5161-263

Borehole 207

HQ 9 to 12 (of 14)

Elevation 23.7 m to 19.3 m



THURBER ENGINEERING LTD.

Foundation Investigation
Highway 401 – Bainsville Underpass
Site 31-241
Township of Lancaster, Ontario

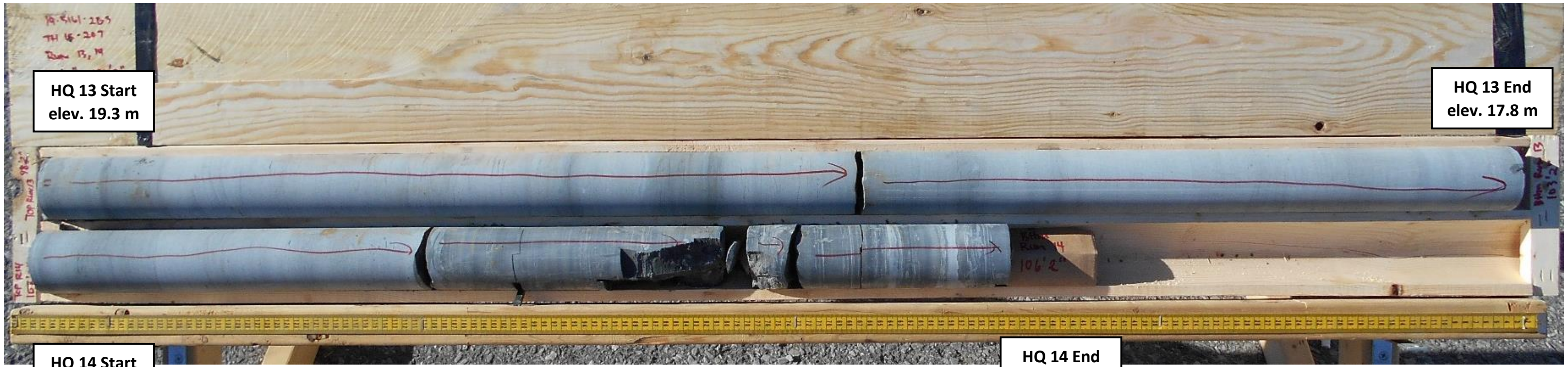
GWP: 4027-14-00

Project No.: 19-5161-263

Borehole 207

HQ 13 to 14 (of 14)

Elevation 19.3 m to 16.9 m



HQ 13 Start
elev. 19.3 m

HQ 13 End
elev. 17.8 m

HQ 14 Start
elev. 17.8 m

HQ 14 End
elev. 16.9 m

THURBER ENGINEERING LTD.

Foundation Investigation
Highway 401 – Bainsville Underpass
Site 31-241
Township of Lancaster, Ontario

GWP: 4027-14-00

Project No.: 19-5161-263

CLIENT DEPARTMENT OF HIGHWAYS-ONT.
JOB NO. 6017 LOCATION TWP OF LANCASTER
PROJECT WP - 176-60
DATE FIELD INVESTIGATION JUNE, 1960
DATE REPORT ______ BY ______ CHKD. ______

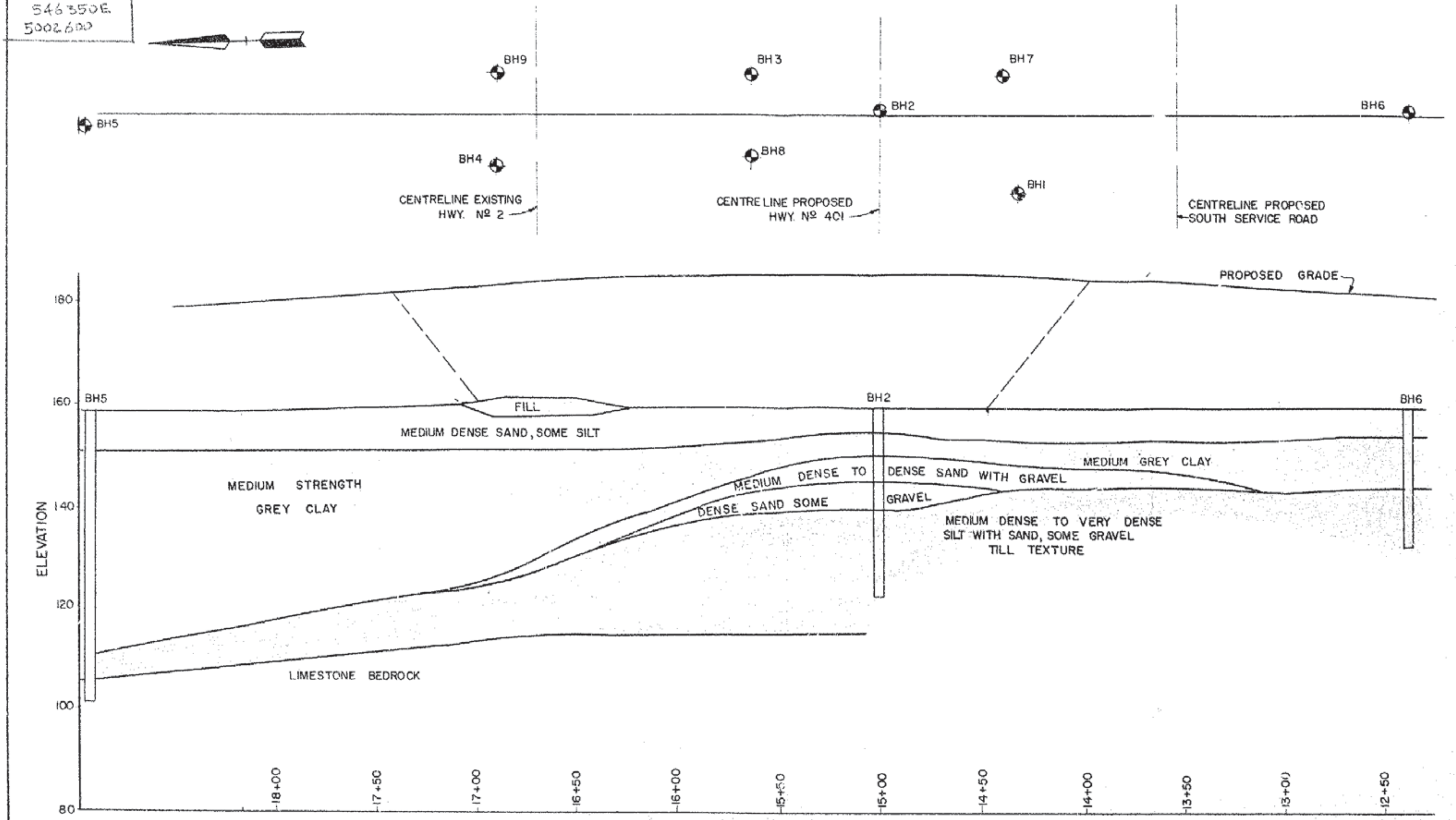
LEGEND
⊕ BOREHOLE

SCALES
HORIZONTAL 1" = 50'
VERTICAL 1" = 20'

ASSOCIATED GEOTECHNICAL SERVICES
Limited

PLAN OF BOREHOLE LOCATIONS AND CENTRELINE PROFILE

31 G1W
18
546350E
5002600



PROBABLE SOILS PROFILE ALONG CENTRELINE COUNTY ROAD TO BAINSVILLE
LOOKING EAST

I - 1

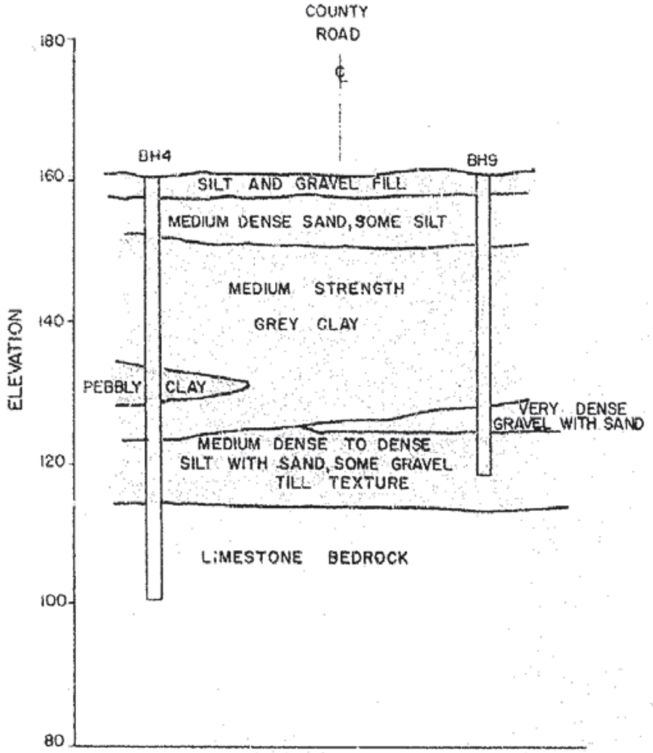
CLIENT DEPARTMENT OF HIGHWAYS - ONT.
JOB NO. 6017 LOCATION TWP. OF LANCASTER
PROJECT WP - 176-60
DATE FIELD INVESTIGATION JUNE, 1960
DATE REPORT ______ BY ______ CHKD. ______

LEGEND

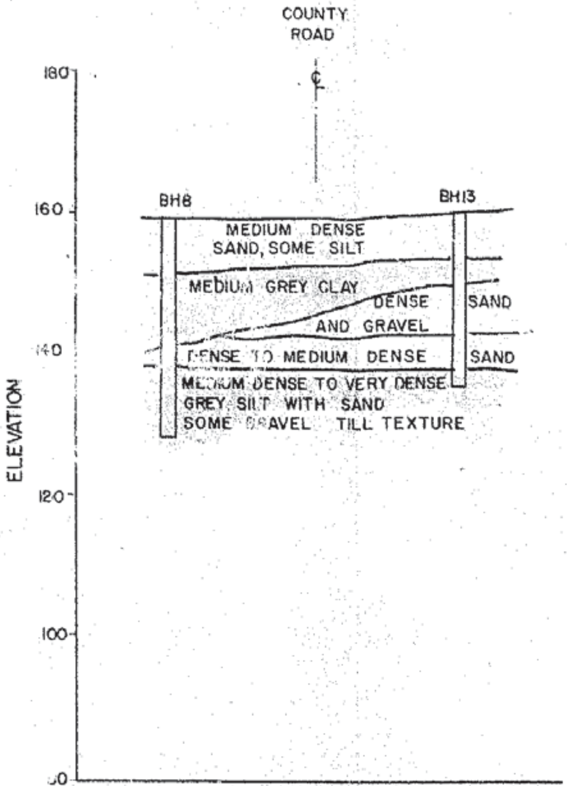
SCALES
HORIZONTAL 1" = 20'
VERTICAL 1" = 20'

ASSOCIATED GEOTECHNICAL SERVICES
Limited

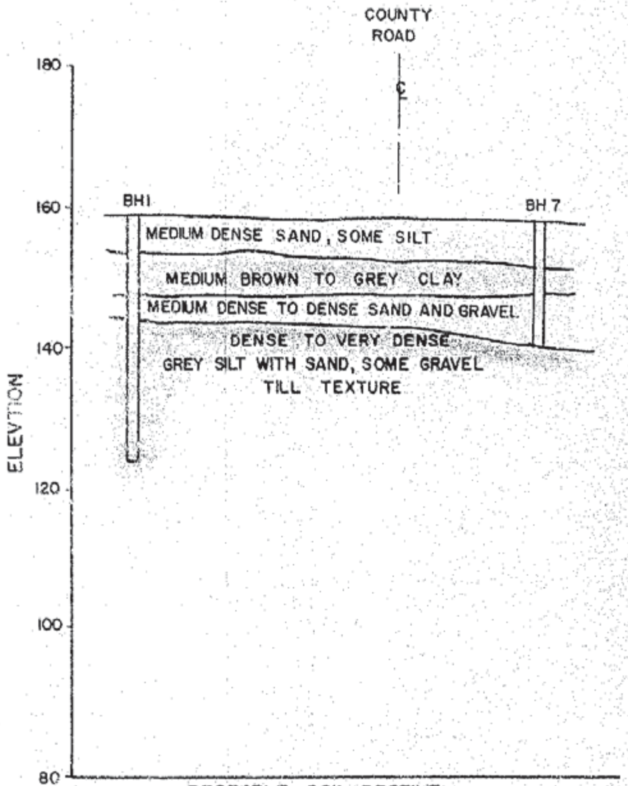
TRANSVERSE SOIL PROFILES



PROBABLE SOIL PROFILE
THROUGH BOREHOLES 4 AND 9
LOOKING NORTH



PROBABLE SOIL PROFILE
THROUGH BOREHOLES 8 AND 3
LOOKING NORTH



PROBABLE SOIL PROFILE
THROUGH BOREHOLES 1 AND 7
LOOKING NORTH

CLIENT Department of Highways of Ontario
JOB NO. 6017 LOCATION Hwy. #401 at Bainsville Rd
CO-ORDINATES Sta: 270 + 81 - 68' RT.
ELEVATION (SURFACE) 159.6 (COLLAR) ______ DATUM DHO
DATE (STARTED) 30/5/60 (FINISHED) 2/6/60 (COMPILED) JK
RIG. NO. 1 TYPE Boyles FIELD SUP. D. S. Oaks

SYMBOLS
▥ SILT ▨ GRAVEL ▲ - VANE SHEAR (NATURAL)
▧ CLAY ≋ PEAT ○ - VANE SHEAR (REMOLDED)
░ SAND ▩ FILL ● - STANDARD PENETRATION
+ - Triaxial, undrained

ABBREVIATIONS
☒ UNDISTURBED
▥ DISTURBED BUT REPRESENTATIVE
▤ FAIR
■ LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
TWP. - THIN WALLED PISTON
DB - DIAMOND BIT

C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

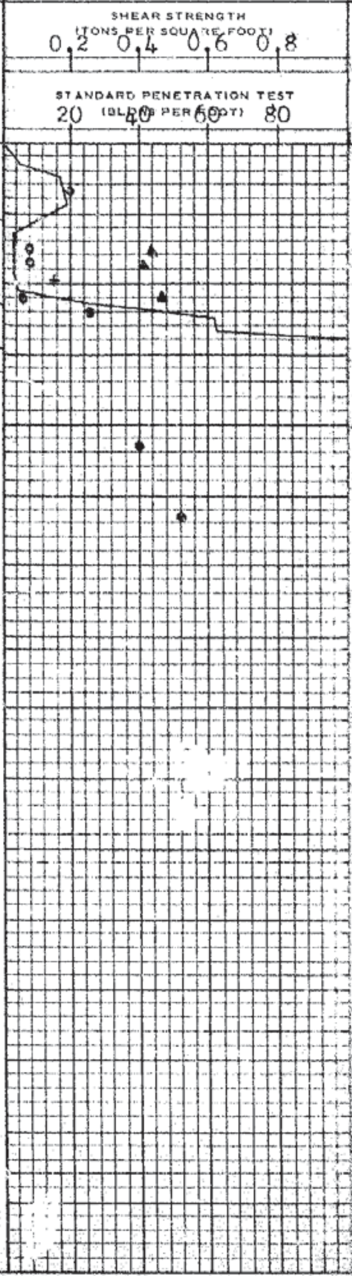
ASSOCIATED GEOTECHNICAL SERVICES
Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 1

BORING LOG

SCALE FEET	DEPTH FEET	ELEV. FEET	WATER OBSERVATION	LOG	DESCRIPTION
	0.8	158.8		░	Medium dense greyish brown fine sand some silt
5					
	5.5	154.1	GWT	▧	Medium brown to grey clay
10					
	11.5	148.1			Grey fine sand some silt & gravel
15	14.7	144.9			Shale boulder
	16.3	143.3			Dense to very dense grey silt, with sand, some gravel occasional cobble (till texture)
20					
25					
30					
35	35.8	123.8			End of borehole
40					

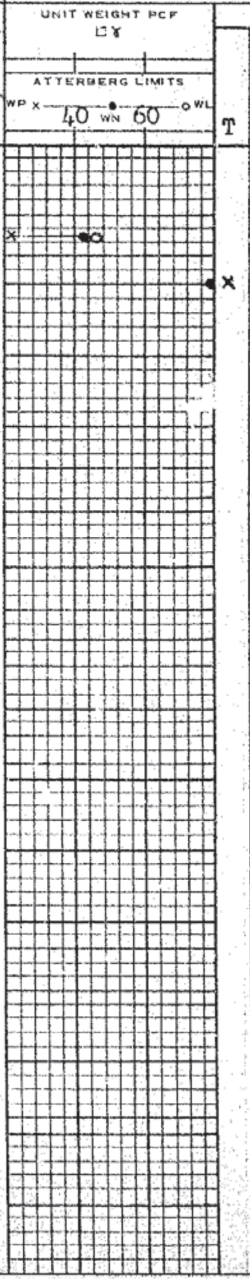
FIELD TESTS



SAMPLING

PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	CONDITION	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.	REMARKS
20	1	▥	2.4	3.9	SS	12/18	
	2	☒	5.5	6.9	ST	13/16	WN = 43.2%
	3	☒	9.0	10.0	ST	22/12	WN = 79.3%
26	4	▥	11.2	12.7	SS	12/18	
	5	☒	14.5	16.3	DB	22/22	
		☒	16.3	20.9	DB	5/55	
40	6	■	20.8	22.2	SS	0/18	Sample recovered with spring trap
53	7	▤	24.9	26.8	SS	1/18	
100/9	8	▥	30.9	31.6	SS	7/9	
100/8	9	▥	35.0	35.7	SS	6/8	

LABORATORY TESTS



I-3

ASSOCIATED GEOTECHNICAL SERVICES
Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 2

CLIENT Department of Highways of Ontario
JOB NO. 6017 LOCATION Hwy. #401 at Bainsville R
CO-ORDINATES Sta: 271 + 22 on centreline
ELEVATION (SURFACE) 160.1 (COLLAR) ______ DATUM DHO
DATE (STARTED) 2/6/60 (FINISHED) 6/6/60 COMPILED WN
RIG NO. 1 TYPE Boyles FIELD SUP. D. S. Oaks

SYMBOLS

▥	SILT	▩	GRAVEL
▨	CLAY	≈	PEAT
░	SAND	▓	FILL

▲ - VANE SHEAR (NATURAL
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION

ABBREVIATIONS

☒	UNDISTURBED
▥	DISTURBED BUT REPRESENTATIVE
☰	FAIR
■	LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
TWP. - THIN WALLED PISTON
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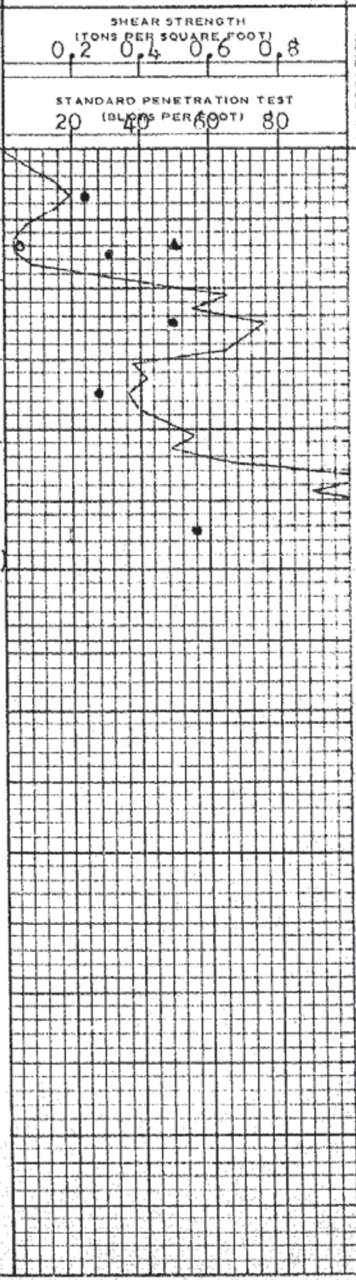
C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

BORING LOG

DEPTH FEET	ELEV. FEET	WATER OBSERVATION	LOG	DESCRIPTION
0.5	159.6		▥	Medium dense greyish brown fine sand, some silt
5.1	155.0	GWT	▨	Medium brown to grey clay & some gravel
9.5	150.6		▩	Dense sand with gravel
15.0	145.1		▩	Dense sand some gravel
21.0	139.1		▥	Very dense grey silt with sand, some gravel, occasional cobble (till texture)
37.9	122.2			End of borehole

SCALE FEET: 5, 10, 15, 20, 25, 30, 35, 40

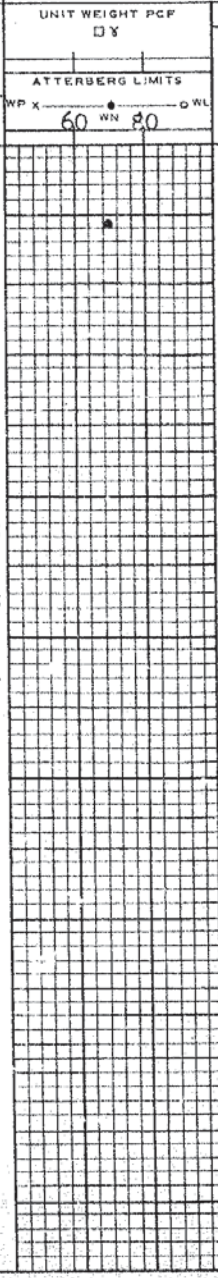
FIELD TESTS



SAMPLING

PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	CONDITION	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.
24	1	▥	2.5	4.0	SS	10/18
	2	☒	4.9	6.2	ST	16/16
30	3	▥	7.5	8.9	SS	9/18
50	4	▥	11.9	13.3	SS	10/18
28	5	▥	17.1	18.6	SS	10/18
107	6	☰	20.9	22.3	SS	5/18
56	7	▥	26.7	28.1	SS	10/18
114	8	☰	31.5	33.0	SS	3/18
118	9	▥	36.1	37.9	SS	15/21

LABORATORY TESTS



REMARKS

WN = 69.0

Unable to drill - casing binding - moved hole 2 ft south

CLIENT Department of Highways of Ontario
JOB NO. 6017 LOCATION Hwy.401 at Bainsville Rd
CO-ORDINATES Sta: 271 + 40 - 62.5' LT.
ELEVATION (SURFACE) 160.1 (COLLAR) DATUM DHO
DATE (STARTED) 8/60/60 (FINISHED) 8/6/60 COMPILED WN
RIG. NO. 1 TYPE Boyles FIELD SUP. D. S. Oaks

SYMBOLS

▥	SILT	⁘	GRAVEL
▨	CLAY	≈	PEAT
░	SAND	▩	FILL

▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION

ABBREVIATIONS

☒	UNDISTURBED
▥	DISTURBED BUT REPRESENTATIVE
☰	FAIR
■	LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
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DB - DIAMOND BIT

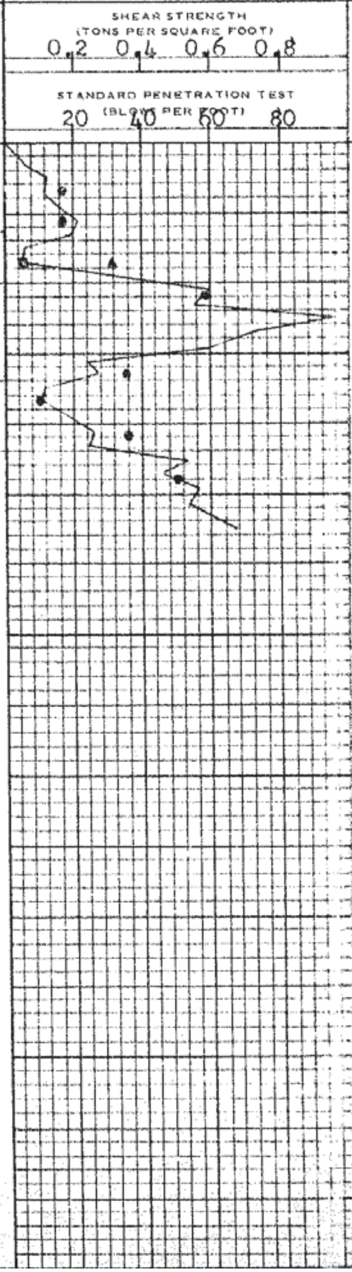
C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

ASSOCIATED GEOTECHNICAL SERVICES
Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 3

BORING LOG — FIELD TESTS — SAMPLING — LABORATORY TESTS

SCALE FEET	DEPTH FEET	ELEV. FEET	WATER OBSERVATION	LOG	DESCRIPTION	PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	CONDITION	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.	REMARKS
	3.0	157.1		░	Medium dense greyish brown fine sand, some silt	17	1	▥	2.3	3.9	SS	14/18	
5	6.2	153.9				17	2	▥	4.5	6.0	SS	14/18	
				▨	Medium grey clay		3	☒	6.5	7.9	ST	12/16	WN = 65.9%
10	9.8	150.3	GWT										
				⁘	Dense sand and gravel	59	4	▥	9.9	11.4	SP	16/18	
15													
	17.0	143.1				35	5	■	14.8	16.8	SS	1/24	Sampler redriven with spring trap - no recovery
				░	Medium dense grey sand, some silt	11	6	■	17.5	18.9	SS	0/18	
20						36	7	■	20.1	21.6	SS	0/18	
	22.0	138.0		⁘	Dense silt with sand some gravel, (till texture)	50	8	■	22.7	24.5	SS	0/22	S.S. lost in hole and unable to recover - hole terminated
25	24.5	135.6			End of borehole								



UNIT WEIGHT PCF □ γ
ATTERBERG LIMITS
WP × —— ● WN —— ○ WL
60 80

I-5

CLIENT Department of Highways of Ontario
JOB NO. 6017 LOCATION Hwy.401 at Bainsville Rd
CO-ORDINATES Sta: 270 + 94.5 - 190' LT
ELEVATION (SURFACE) 160.9 (COLLAR) ______ DATUM DHO
DATE (STARTED) 8/6/60 (FINISHED) 9/6/60 (COMPILED) WN
RIG. NO. 1 TYPE Boyles FIELD SUP. D. S. Oaks

SYMBOLS

▥ SILT	⁘ GRAVEL
▨ CLAY	≋ PEAT
░ SAND	▩ FILL

▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION

ABBREVIATIONS

☒ UNDISTURBED
▥ DISTURBED BUT REPRESENTATIVE
☰ FAIR
■ LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
TWP. - THIN WALLED PISTON
DB - DIAMOND BIT

C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

ASSOCIATED GEOTECHNICAL SERVICES Limited

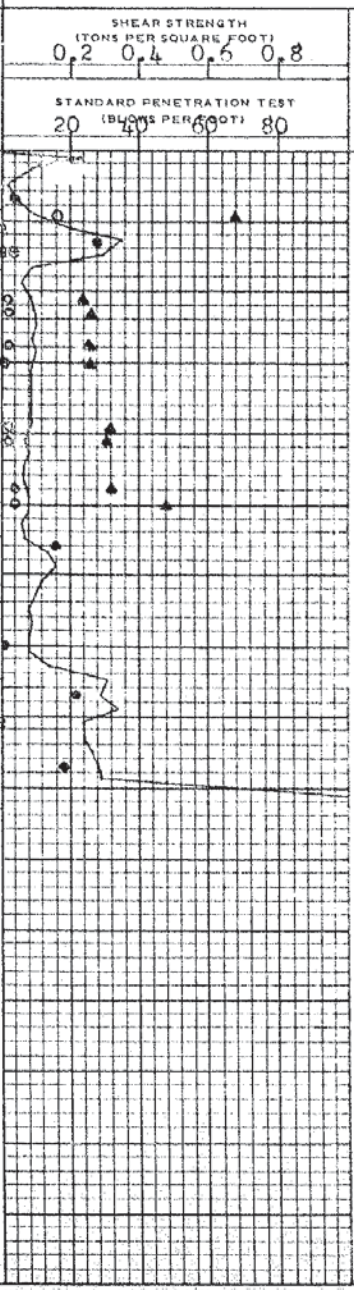
OFFICE BOREHOLE LOG
BOREHOLE NO. 4

BORING LOG

DEPTH FEET	ELEV. FEET	WATER OBSERVATION	DESCRIPTION
3.0	157.9		Black to brown silt with gravel (fill)
4.5	156.4		Loose brown silt
8.5	152.4		Medium dense greyish brown fine sand, some silt
		GWT	
27.5	133.4		Medium grey clay
32.0	128.9		Grey clay, some gravel
37.5	123.4		Soft grey clay
46.6	114.3		Medium dense grey silt with sand, some gravel, occasional cobble (till texture)
60.0	100.9		Limestone bedrock

End of borehole

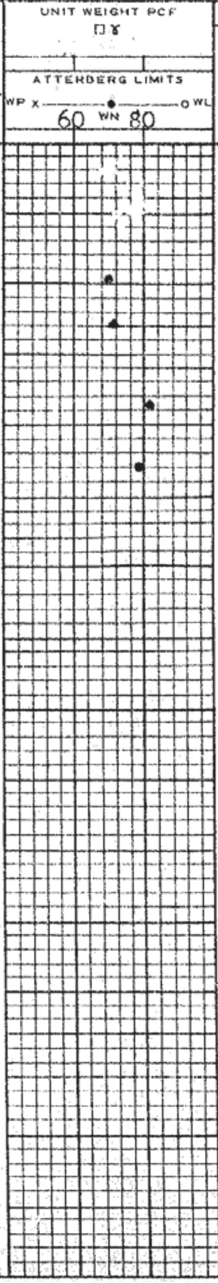
FIELD TESTS



SAMPLING

PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	CONDITION	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.	REMARKS
4	1	☰	2.7	4.1	SS	2/18	
28	2	▥	5.7	7.1	SS	18/18	
	3	☒	8.6	9.9	ST	16/16	WN = 70.3
	4	☒	12.0	13.4	ST	21/16	WN = 71.2
	5	☒	17.5	18.9	ST	16/16	WN = 81.4
	6	☒	22.0	23.3	ST	16/16	WN = 79.8
	7	■	27.0	27.5	ST	0/6	
15	7b	☰	27.5	29.0	SS	3/18	
1	8	☰	34.0	35.5	SS	18/18	
22	9	▥	37.2	39.0	SS	12/21	
18	10	■	42.2	44.0	SS	0/18	
200/11	11	☰	45.7	46.6	SS	4/11	
	12	☒	46.6	50.8	DB	25/50	
	13	☒	50.8	55.9	DB	61/61	
	14	☒	55.9	60.0	DB	50/50	

LABORATORY TESTS



REMARKS

I-6

CLIENT Department of Highways of Ontario
JOB NO. 6017 LOCATION Hwy. 401 at Bainsville Rd
CO-ORDINATES Sta: 271 + 14 - 397.5 LT
ELEVATION (SURFACE) 158.4 (COLLAR) ______ DATUM DHO
DATE (STARTED) 10/6/60 (FINISHED) 14/6/60 (COMPILED) WN
RIG. NO. 1 TYPE Boyles FIELD SUP. D. S. Oaks

SYMBOLS
▥ SILT
▨ CLAY
░ SAND
GRAVEL
PEAT
▩ FILL

▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION

ABBREVIATIONS
☒ UNDISTURBED
▥ DISTURBED BUT REPRESENTATIVE
☰ FAIR
■ LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
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C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
Y - TRIAXIAL COMPRESSION
K - PERMEABILITY
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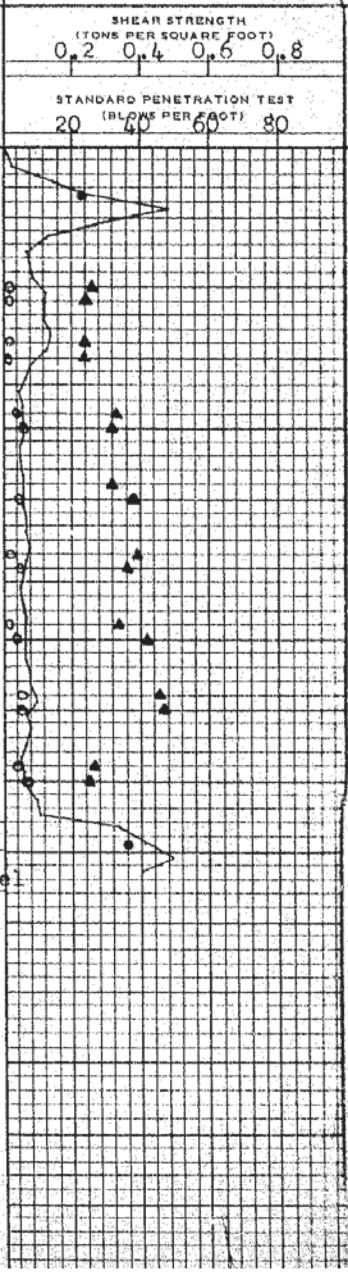
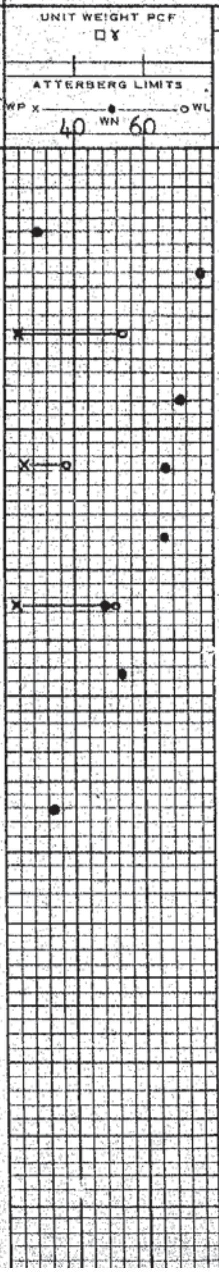
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 5





BORING LOG

SCALE FEET	DEPTH FEET	ELEV. FEET	WATER OBSERVATION	LOG	DESCRIPTION
5	4.5	153.9		░	Medium dense greyish brown fine sand, some silt
10	8.0	150.4	GWT	▨	Soft to medium grey clay
15					
20					
25					
30					
35					
40					
45					
50	48.0	110.4			Dense grey fine sand some silt
	49.9	108.5			Very dense grey silt with sand, some gravel (till texture)
55	52.9	105.5			Limestone bedrock
60	57.7				End of borehole

SAMPLING

PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	CONDITION	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.	TESTS	REMARKS
23	1	▥	2.3	3.8	SS	18/18		
	2	☒	6.1	7.0	ST	10/10		WN = 28.9
	3	☒	8.0	9.4	ST	23/16		WN = 76.0
	5	☒	12.0	13.4	ST	19/16	C	WN = 82.4, W_p = 23.7 W_L = 53.7
	6	☒	17.0	18.3	ST	23/16		WN = 69.8
	7	☒	22.0	23.3	ST	23/16		WN = 65.6, Wp = 26.3, W_L = 37.8
	8	☒	27.0	28.0	ST	20/12		WN = 65.5
	9	☒	32.0	33.0	ST	20/12	C	WN = 48.4, Wp = 23.0, W_L = 49.0
	10	☒	37.0	38.0	ST	16/12		WN = 53.6
	11	■	42.0	43.0	ST	0/12		
	12	☒	46.0	47.4	ST	23/16		WN = 33.3, Wp = 16.4, W_L = 20.5
36	13	▥	48.3	49.9	SS	15/18		
309	14	☰	52.0	52.9	SS	5/10		
		☒	52.9	55.6	DB	75%		
		☒	55.7	57.7	DB	100%		

CLIENT Department of Highways of Ontario
JOB NO. 6017 LOCATION Hwy.401 at Bainsville Rd
CO-ORDINATES Sta: 271 + 22 - 263.5 RT
ELEVATION (SURFACE) 160.0 (COLLAR) DATUM DHO
DATE (STARTED) 13/6/60 (FINISHED) 14/6/60 COMPILED WN
RIG. NO. 2 TYPE LY. FIELD SUP. D. S. Oaks

SYMBOLS

▥	SILT	⁘	GRAVEL
▨	CLAY	≈	PEAT
░	SAND	▦	FILL

▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION

ABBREVIATIONS

☒	UNDISTURBED
▥	DISTURBED BUT REPRESENTATIVE
▤	FAIR
■	LOST

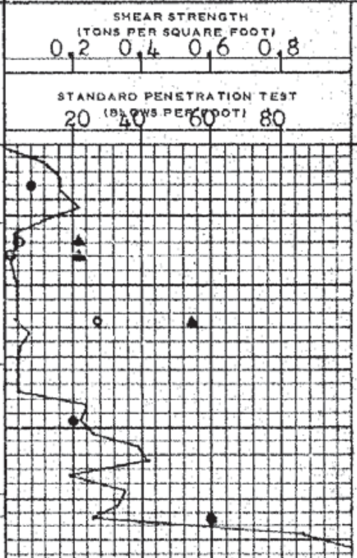
SS - SPLIT SPOON
ST - SHELBY TUBE
TWP. - THIN WALLED PISTON
DB - DIAMOND BIT

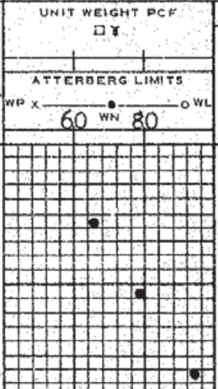
C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

ASSOCIATED GEOTECHNICAL SERVICES
Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 6

BORING LOG				FIELD TESTS		SAMPLING					REMARKS
DEPTH FEET	ELEV. FEET	WATER OBSERVATION	DESCRIPTION	PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	CONDITION	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC. DIST. DRIV.	
3.5	156.5		Loose greyish brown fine sand, some silt	8	1	▥	2.5	4.0	SP	16/18	
5.5	154.5		Soft to medium grey clay, becoming gravelly		2	☒	5.0	6.0	ST	16/18	WN = 66.8
		GWT			3	☒	9.9	11.3	ST	14/18	WN = 78.4
					4	☒	14.9	15.6	ST	7/9	WN = 93.2
15.6	144.4		Medium dense gravel and sand, some silt (till texture)	20	5	▤	18.9	20.0	SS	2/18	
					6						— 22.8 ft: SS bouncing on boulder
27.5	132.5		Boulder	59	7	▥	26.0	27.6	SS	8/18	
			End of borehole								





LABORATORY TESTS

CLIENT Department of Highways of Ontario
JOB NO. 6017 LOCATION Hwy.401 at Bainsville Rd
CO-ORDINATES Sta: 271 + 40 - 60.6' RT
ELEVATION (SURFACE) 158.5 (COLLAR) ______ DATUM DHO
DATE (STARTED) 14/6/60 (FINISHED) 15/6/60 (COMPILED) WN
RIG. NO. 2 TYPE L.Y. FIELD SUP. D. S. Oaks

SYMBOLS

▥ SILT
▨ CLAY
⁘ SAND
⁂ GRAVEL
〰 PEAT
▩ FILL

△ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION
+ - Triaxial, undrained

ABBREVIATIONS

☒ UNDISTURBED
▥ DISTURBED BUT REPRESENTATIVE
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■ LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
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K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

ASSOCIATED GEOTECHNICAL SERVICES Limited

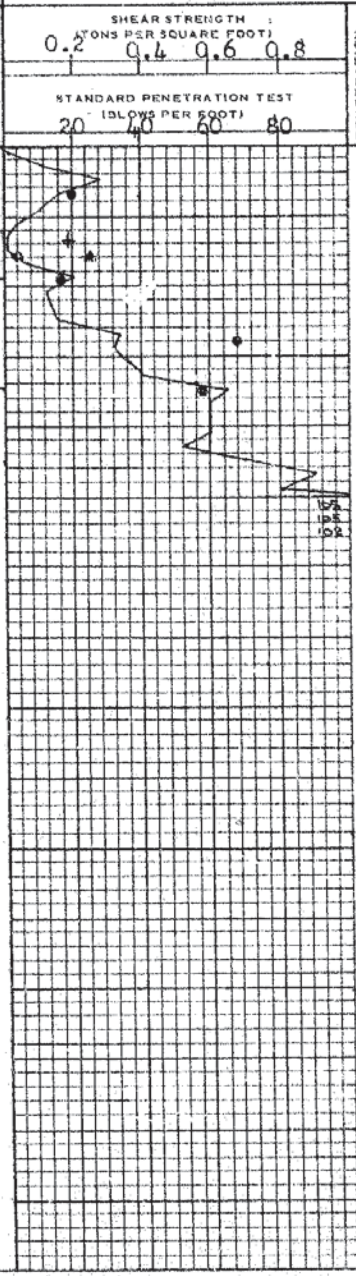
OFFICE BOREHOLE LOG

BOREHOLE NO. 7

BORING LOG

SCALE FEET	DEPTH FEET	ELEV. FEET	WATER OBSERVATION	LOG	DESCRIPTION
5	4.5	154.0			Medium dense greyish brown fine sand some silt
	6.0	152.5			Soft grey clay
10	9.6	148.9	GWT		Medium dense to very dense gravel and sand
15					
	17.6	140.9			
20	18.0	140.5			Very dense gravel and sand, some silt, (till texture)
25					End of borehole
30					

FIELD TESTS



SAMPLING / LABORATORY TESTS

PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	CONDITION	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.	UNIT WEIGHT PCF □ γ	ATTERBERG LIMITS WP × — ● WN — ○ WL (60, 80)	T	REMARKS
19	1	▥	2.5	4.0	SS	18/18				
	2	☒	6.0	7.3	ST	19/16		●	×	WN = 82.2
17	3	▥	8.3	9.9	SS	14/18				
68	4	▥	13.0	14.5	SS	9/18				
58	5	▥	16.5	18.0	SS	15/18				

I-9

CLIENT Department of Highways of Ontario.
JOB NO. 6017 LOCATION Hwy.401 at Bainsville Rd
CO-ORDINATES Sta: 271 + 00 - 62.5' Lt
ELEVATION (SURFACE) 159.3 (COLLAR) DATUM DHO
DATE (STARTED) 15/6/60 (FINISHED) 16/6/60 (COMPILED) WN
RIG. NO. 1 TYPE BOYLES FIELD SUP. D. S. Oaks

SYMBOLS
▥ SILT ▨ CLAY ░ SAND
⁘ GRAVEL ≋ PEAT ▩ FILL
▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION

ABBREVIATIONS
☒ UNDISTURBED
▥ DISTURBED BUT REPRESENTATIVE
☰ FAIR
■ LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
TWP. - THIN WALLED PISTON
DB - DIAMOND BIT

C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 8

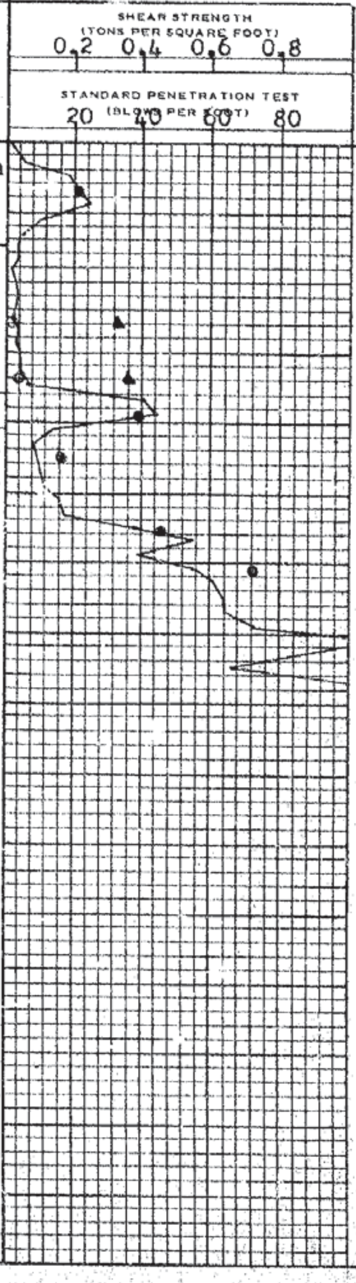
BORING LOG

SCALE FEET	DEPTH FEET	ELEV. FEET	WATER OBSERVATION	LOG	DESCRIPTION
5	4.5	154.8	GWT	░	Medium dense greyish brown fine sand, some silt
10	7.6	151.7		▨	Medium grey clay
15, 20	18.0	141.3		░	Dense grey sand
25, 30	20.6	138.7		⁘	Medium dense to very dense grey silt with sand, some gravel (till texture)
35	30.9	128.4			End of borehole

FIELD TESTS

SHEAR STRENGTH (TONS PER SQUARE FOOT)

STANDARD PENETRATION TEST (BLOWS PER FOOT)



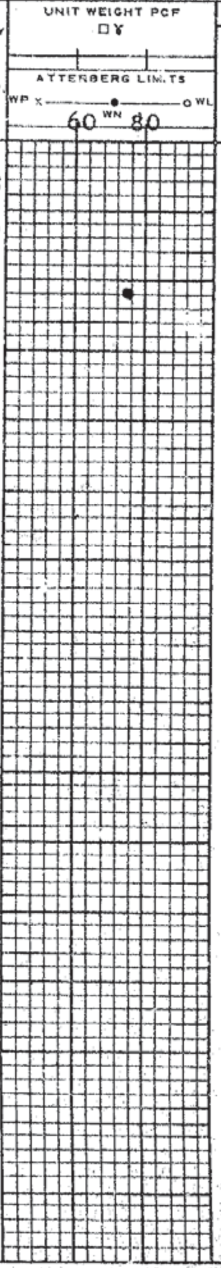
SAMPLING

PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	CONDITION	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.	REMARKS
21	1	▥	2.5	4.0	SS	18/18	
	2	☒	5.9	7.1	ST	16/16	WN = 34.0
	3	☒	10.0	11.3	ST	16/16	WN = 75.8
	4	☒	15.0	16.4	ST	16/16	WN = 39.2
39	5	▥	18.4	19.9	SS	18/18	
16	6	▥	21.8	23.2	SS	8/18	
46	7	☰	26.9	28.4	SS	3/18	
72	8	■	29.3	30.9	SS	0/18	

LABORATORY TESTS

UNIT WEIGHT PCF DY

ATTERBERG LIMITS



ASSOCIATED GEOTECHNICAL SERVICES
Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 9

CLIENT Department of Highways of Ontario
JOB NO. 6017 LOCATION Hwy.401at Bainsville R
CO-ORDINATES Sta: 271 + 45 - 190' LT
ELEVATION (SURFACE) 161.3 (COLLAR) ______ DATUM DHO
DATE (STARTED) 14/6/60 (FINISHED) 15/6/60 (COMPILED) WN
RIG. NO. 1 TYPE BOYLES FIELD SUP. D. S. Oaks

SYMBOLS

▥	SILT	⁘	GRAVEL
▨	CLAY	≈	PEAT
░	SAND	▩	FILL

▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION
+ - Triaxial, undrained

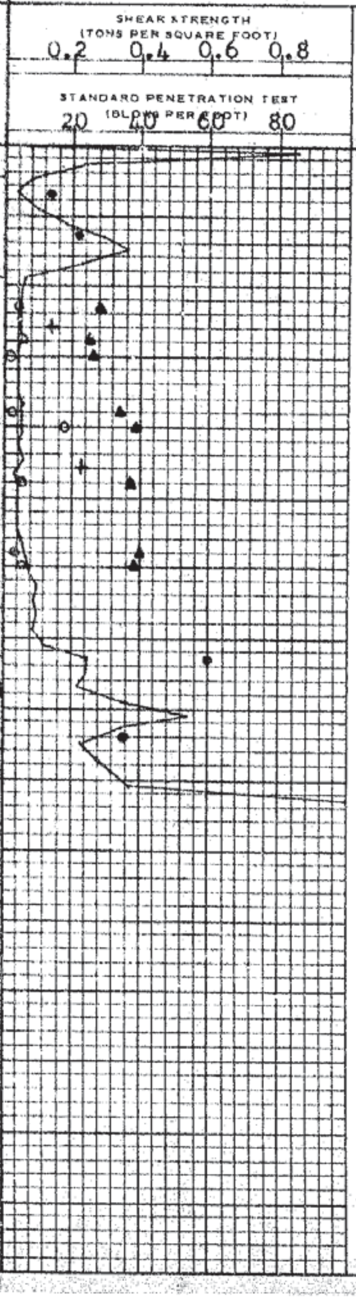
ABBREVIATIONS

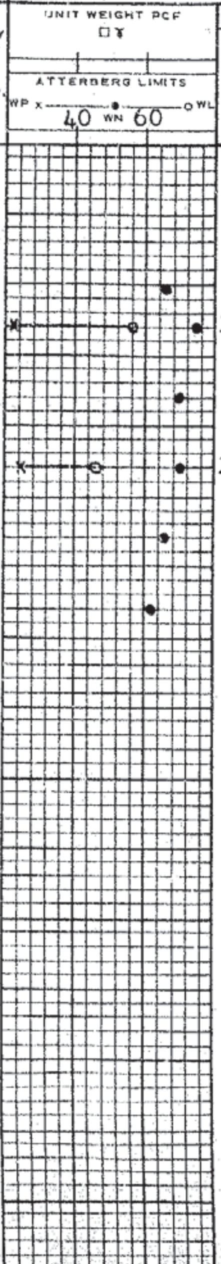
☒ UNDISTURBED
▥ DISTURBED BUT REPRESENTATIVE
▤ FAIR
■ LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
TWP. - THIN WALLED PISTON
DB - DIAMOND BIT

C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

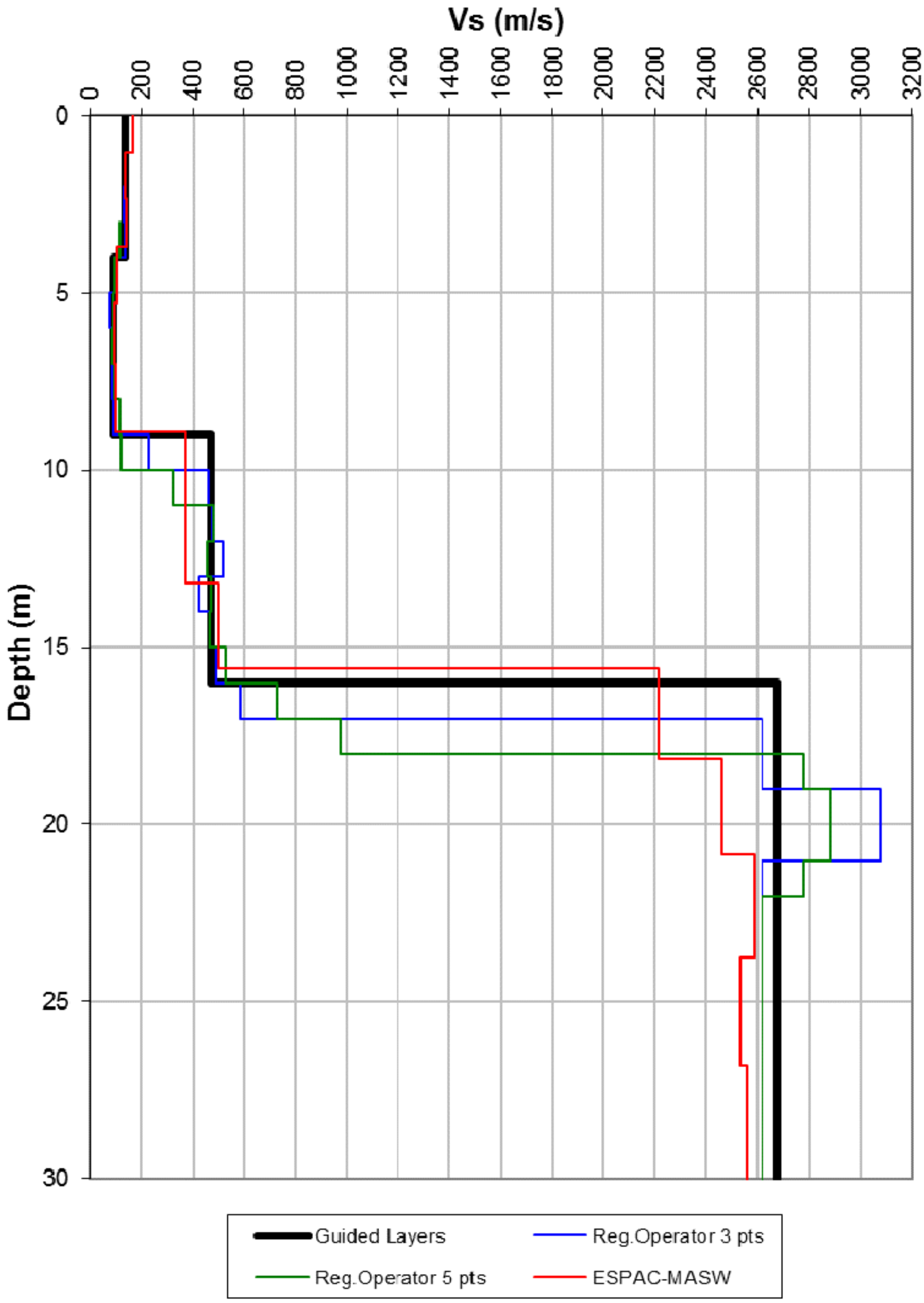
BORING LOG				FIELD TESTS		SAMPLING					LABORATORY TESTS	REMARKS
DEPTH FEET	ELEV. FEET	WATER OBSERVATION	DESCRIPTION	PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	CONDITION	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.	T	
2.6	158.7		Black to brown silt & gravel (fill)									
4.5	156.8		Medium dense greyish brown fine sand, some silt	14	1	▤	2.5	4.0	SS	2/18		
				22	2	▥	5.5	7.0	SS	18/18		
9.9	151.4				3	☒	9.2	10.6	ST	16/16		WN = 66.5
		GWT	Medium grey clay		4	☒	12.0	13.4	ST		X	WN = 83.1 & 77.3 Wp = 22.2 W_L = 56.8 PI = 34.2
					5	☒	17.0	18.4	ST	18/16		WN = 70.0
					6	☒	22.0	23.4	ST	17/16	X	WN = 75.4 & 70.2 Wp = 24.2 W_L = 46.9 PI = 22.7
					8	☒	27.0	28.4	ST	18/16		WN = 66.1
33.0	128.3				9	☒	32.0	33.4	ST	18/16		WN = 62.9
36.5	124.8		Very dense gravel with sand	60	10	▥	35.4	36.9	SS	18/18		
43.0	118.3		Dense grey fine sand some silt (till texture)	35	12	■	41.1	42.5	SS	0/18		— Flap valve used and sampler driven to 43 ft (sample recovered)
			End of borehole									





SCALE FEET: 5 10 15 20 25 30 35 40 45 50 55 60

Shear Wave Velocity Profile
Bainsville Underpass Structure



APPENDIX C

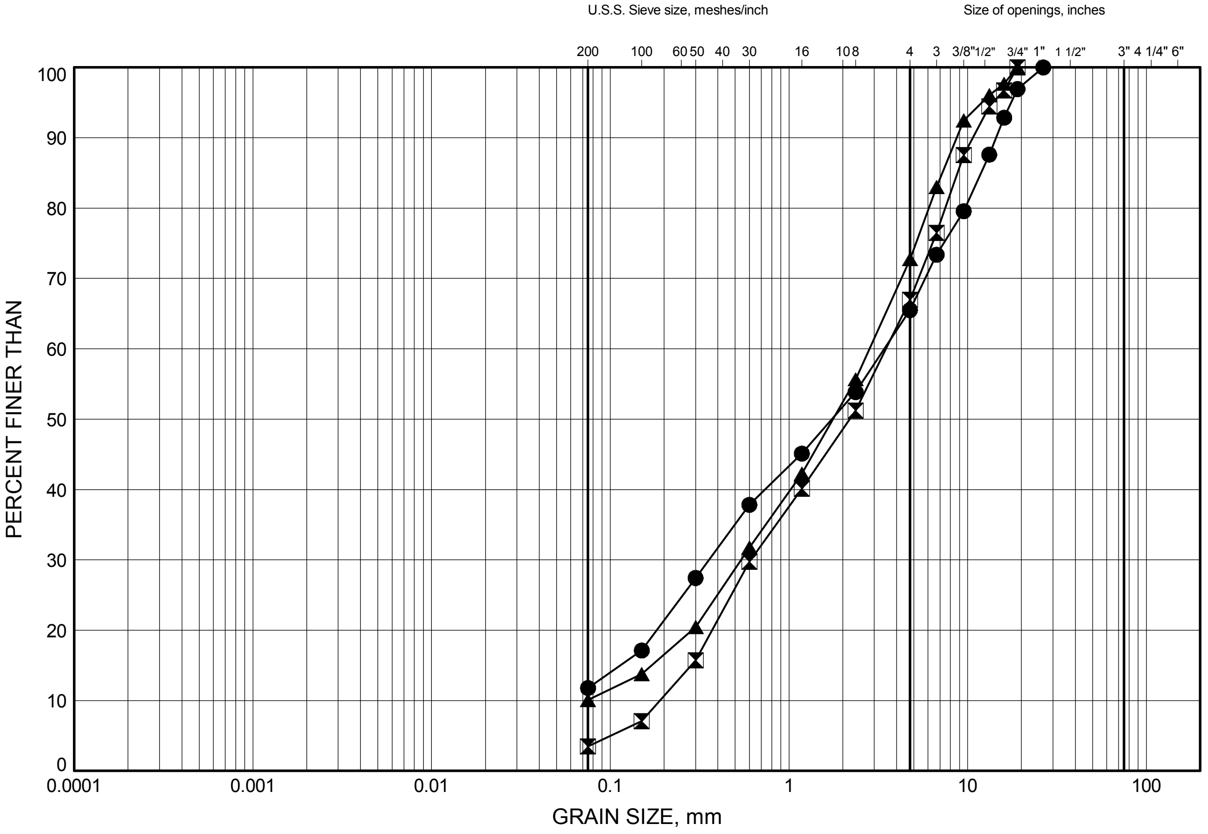
LABORATORY TEST RESULTS

Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 1

Granular Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	202	0.30	55.46
⧗	209	0.30	54.27
▲	210	1.07	53.51

GRAIN SIZE DISTRIBUTION - THURBER BAINSVILLE.GPJ 1/6/16

Date June 2016

WP# 4088-13-01

THURBER

Prep'd KCP

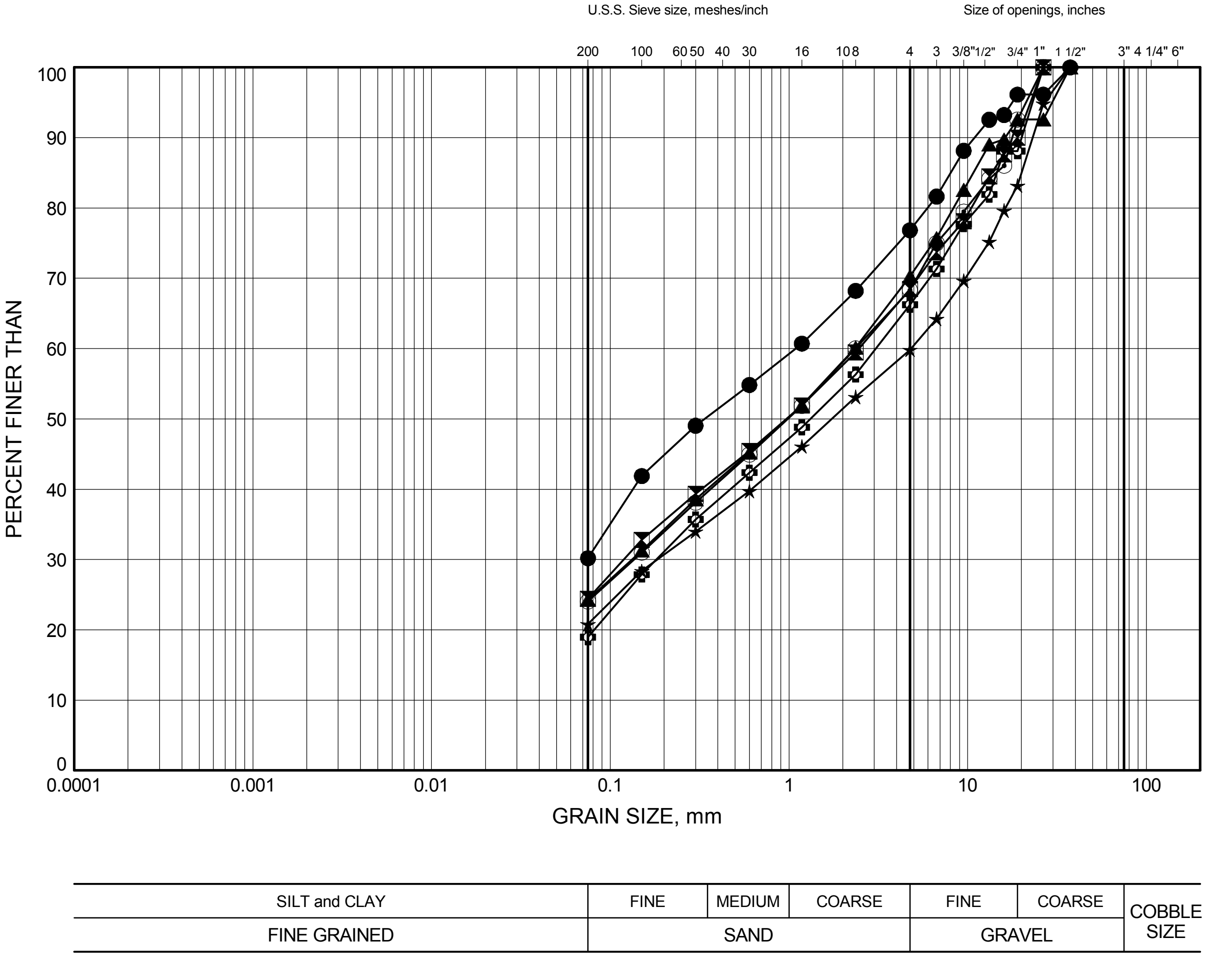
Chkd. PC

Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 2

Embankment Fill



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	201	3.35	52.42
⧗	202	2.59	53.18
▲	209	4.11	50.46
★	209	5.64	48.94
☉	210	4.11	50.46
✿	210	7.16	47.41

Date June 2016

WP# 4088-13-01

Prep'd KCP

Chkd. PC

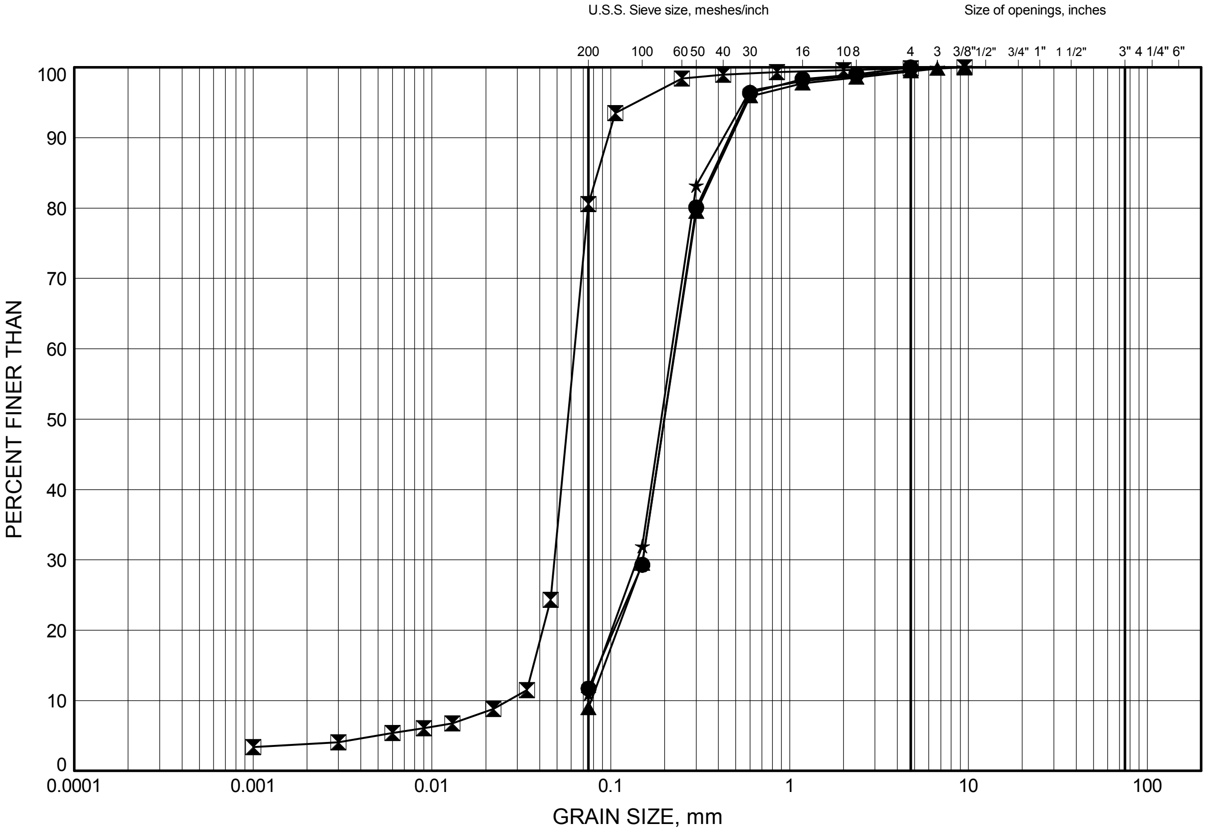
GRAIN SIZE DISTRIBUTION - THURBER BAINSVILLE.GPJ 1/6/16

Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 3

Embankment Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	201	1.07	54.71
⧗	202	7.92	47.84
▲	209	1.83	52.75
★	210	1.83	52.75

GRAIN SIZE DISTRIBUTION - THURBER BAINSVILLE.GPJ 1/6/16

Date June 2016

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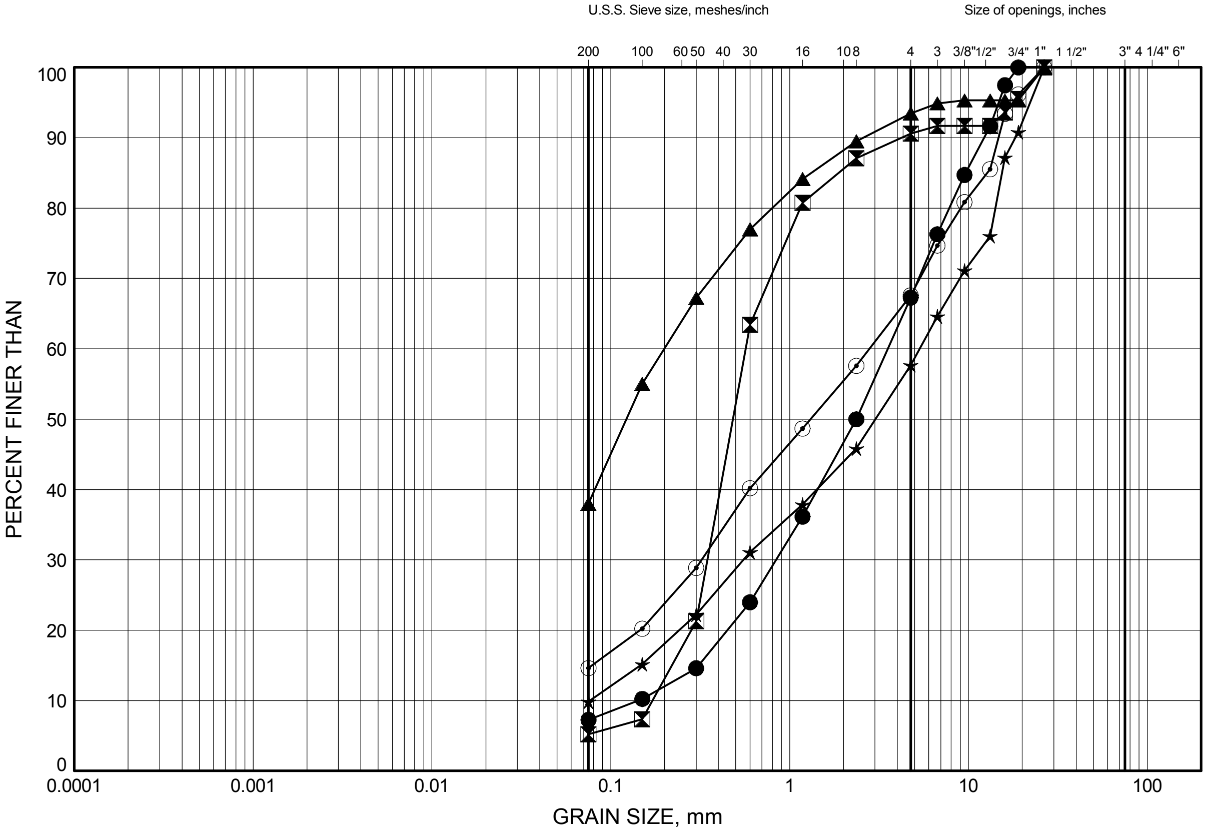
Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 4

Sand Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	203	0.30	48.64
⧗	203	2.59	46.36
▲	204	0.30	48.74
★	207	0.30	48.93
⊙	208	0.49	48.61

Date June 2016

WP# 4088-13-01

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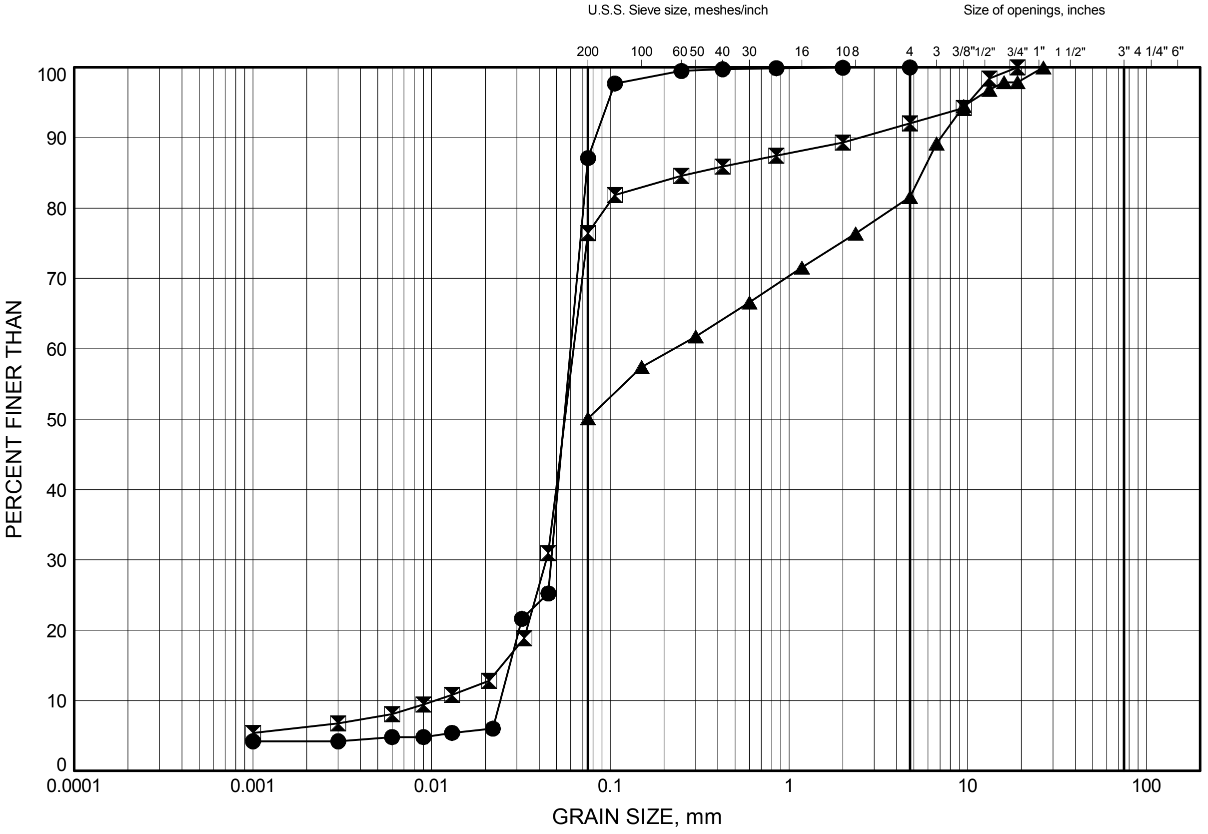
Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 5

Silt Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	205	1.07	47.49
⧗	206	0.30	47.95
▲	207	1.07	48.17

GRAIN SIZE DISTRIBUTION - THURBER BAINSVILLE.GPJ 1/6/16

Date June 2016

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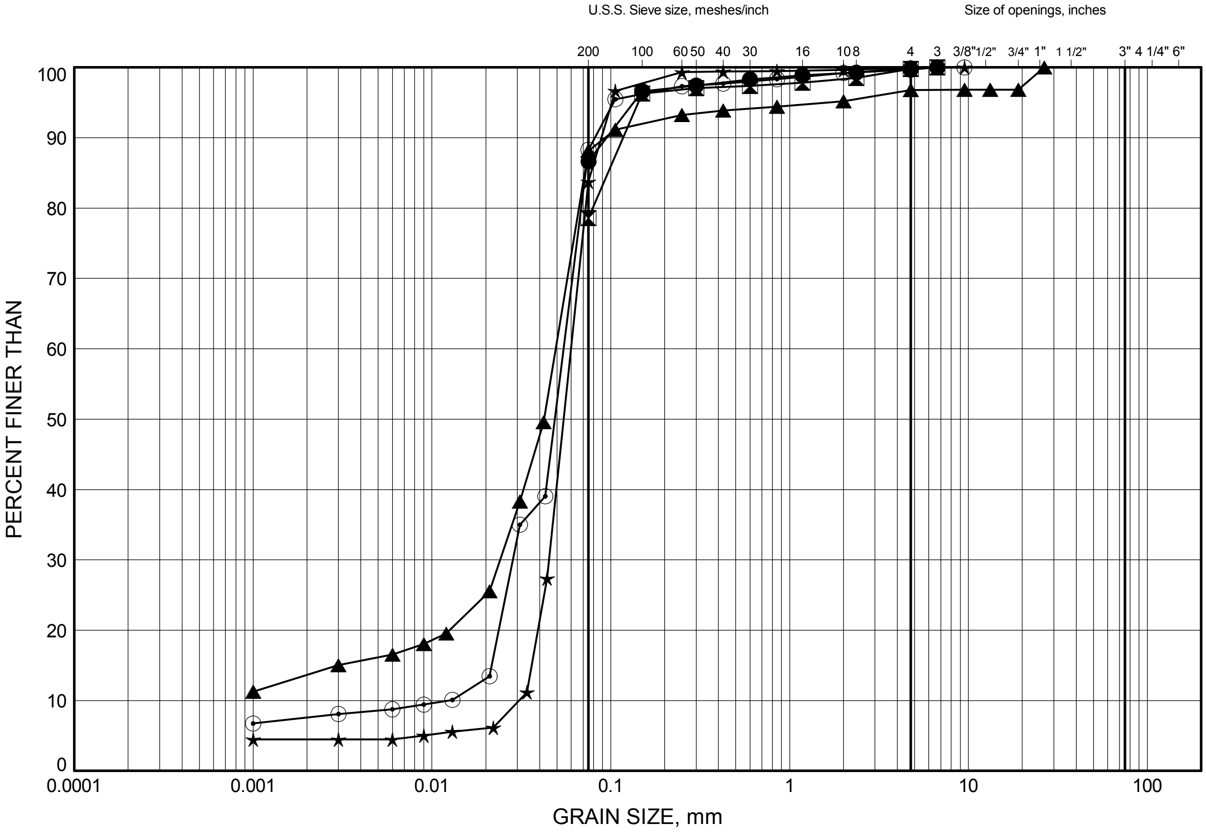
Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 6

Silt



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	207	2.59	46.65
☒	208	1.83	47.27
▲	209	8.00	46.58
★	209	9.45	45.13
⊙	210	8.69	45.89

GRAIN SIZE DISTRIBUTION - THURBER BAINSVILLE.GPJ 1/6/16

Date June 2016

WP# 4088-13-01

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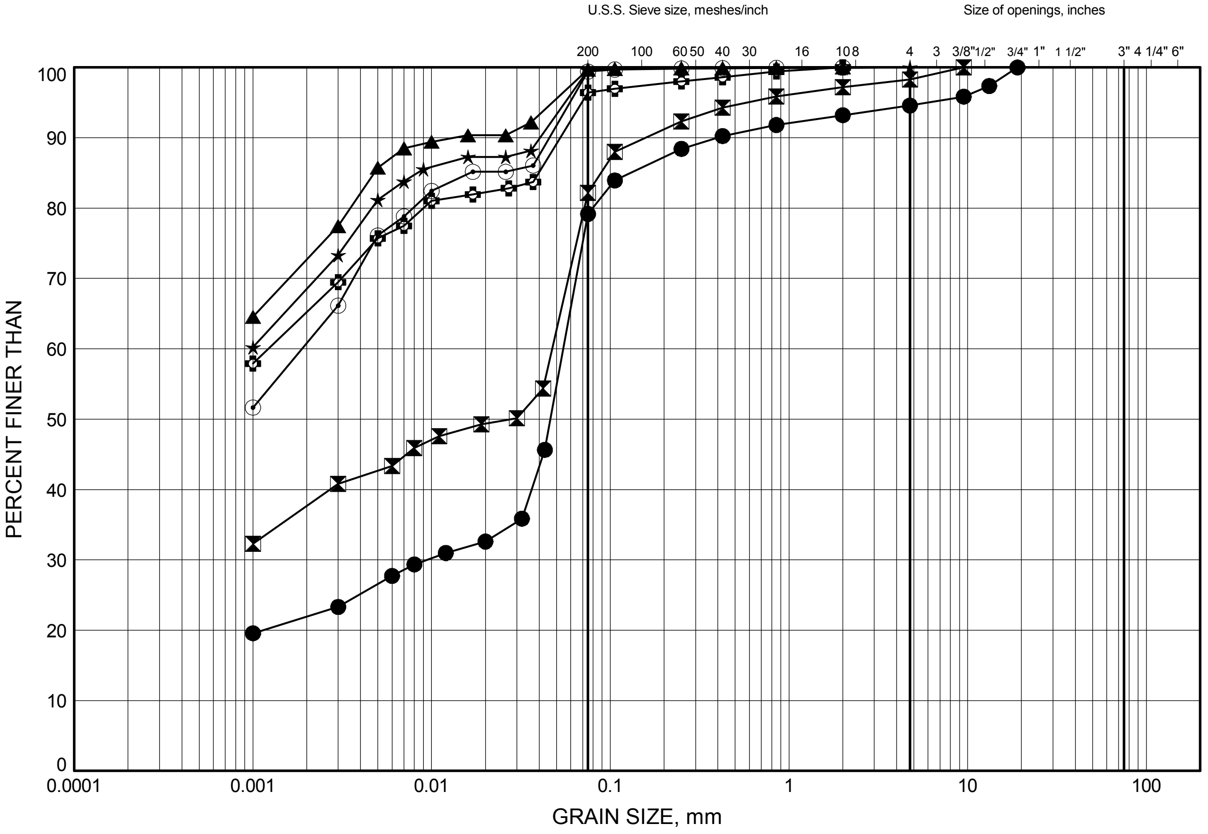
Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 7

Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	204	1.83	47.22
⧗	205	1.83	46.72
▲	206	4.11	44.14
★	207	4.88	44.36
⊙	208	2.74	46.36
✤	208	6.40	42.70

GRAIN SIZE DISTRIBUTION - THURBER BAINSVILLE.GPJ 1/6/16

Date June 2016

WP# 4088-13-01

THURBER

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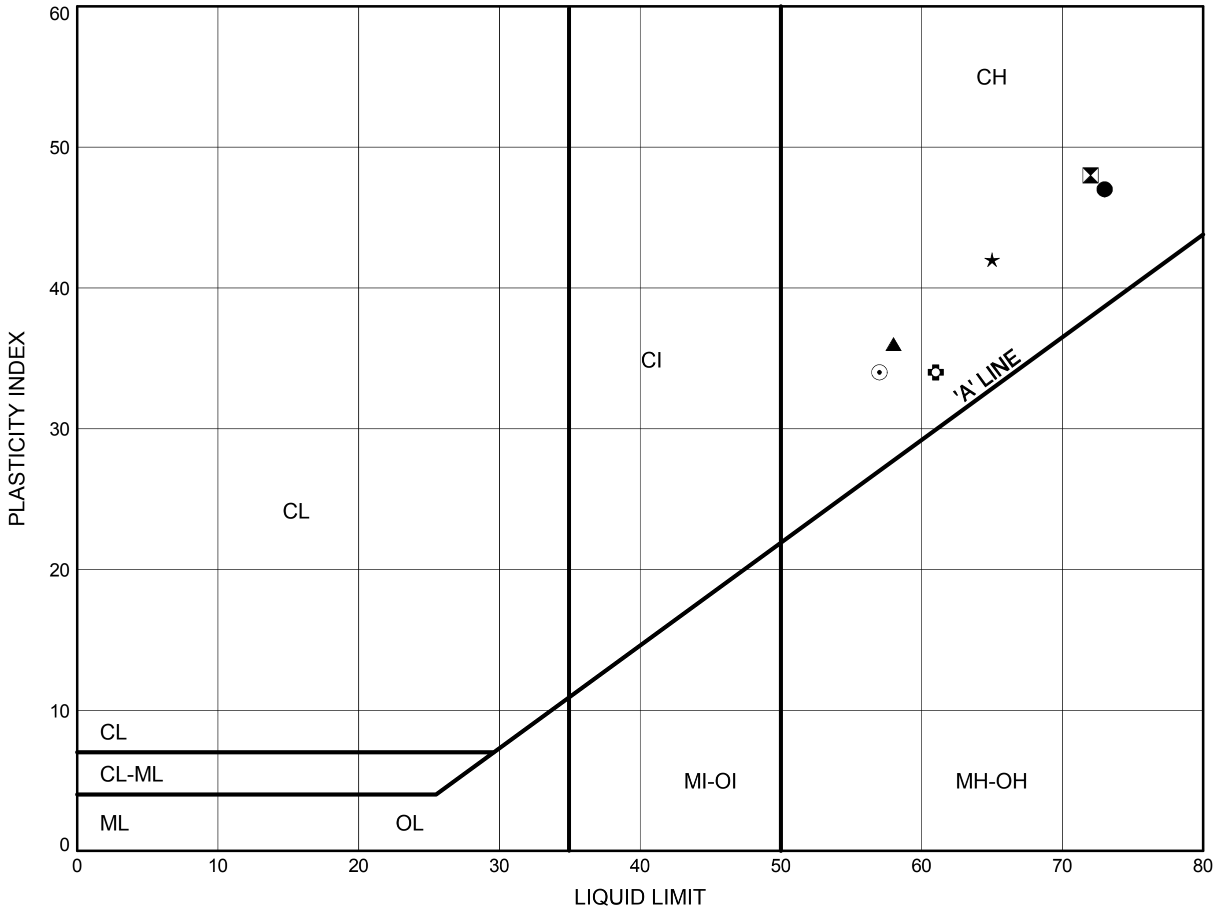
Chkd. PC

Site 31-241 - Highway 401 Underpass at Bainsville Rd.

ATTERBERG LIMITS TEST RESULTS

FIGURE 8

Clay



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	206	4.11	44.14
⧗	207	4.88	44.36
▲	208	2.74	46.36
★	208	6.40	42.70
⊙	209	10.97	43.61
✿	209	17.07	37.51

THURBALT BAINSVILLE.GPJ 1/6/16

Date June 2016

WP# 4088-13-01

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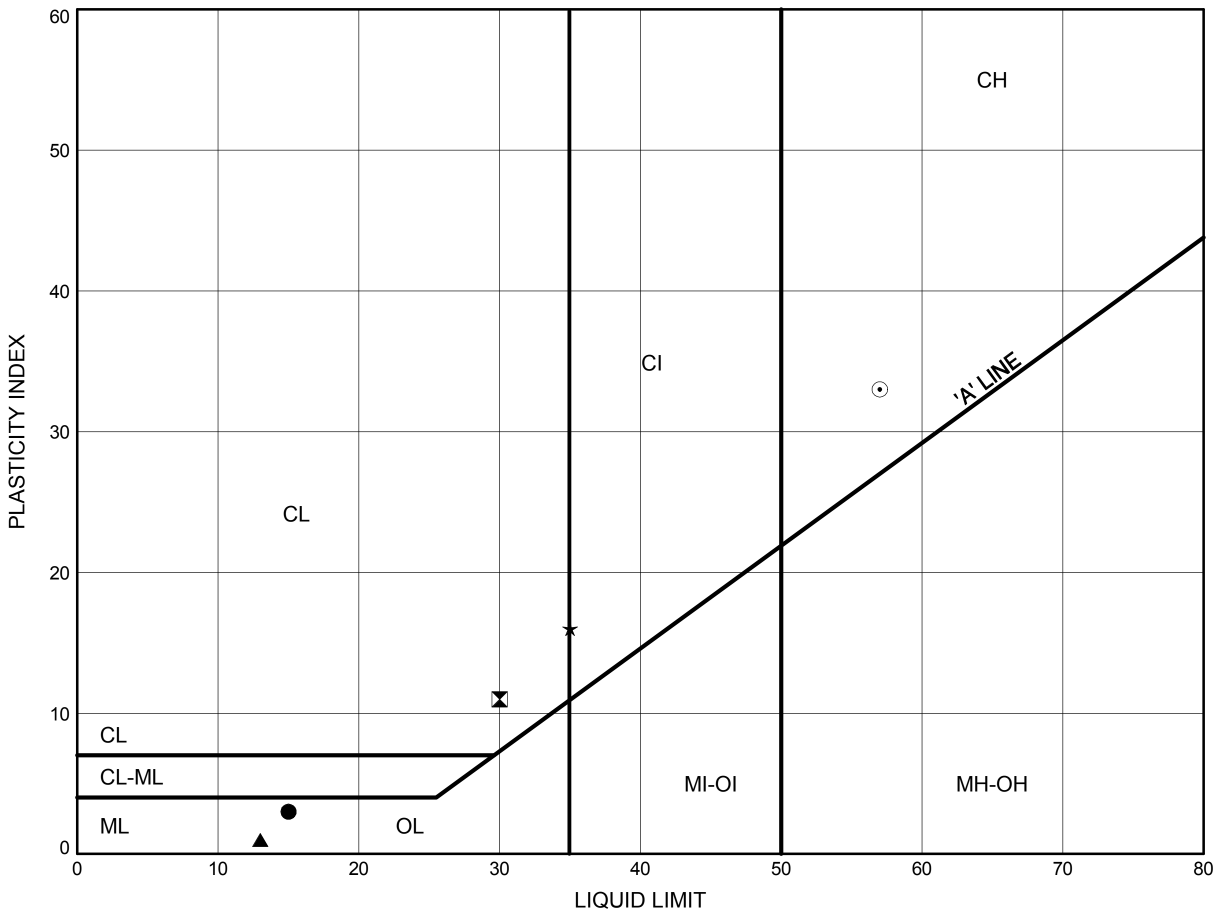
Site 31-241 - Highway 401 Underpass at Bainsville Rd.

ATTERBERG LIMITS TEST RESULTS

FIGURE 9



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	202	18.59	37.18
⧗	204	1.83	47.22
▲	204	9.45	39.60
★	205	1.83	46.72
☉	210	14.78	39.79

Date June 2016

WP# 4088-13-01

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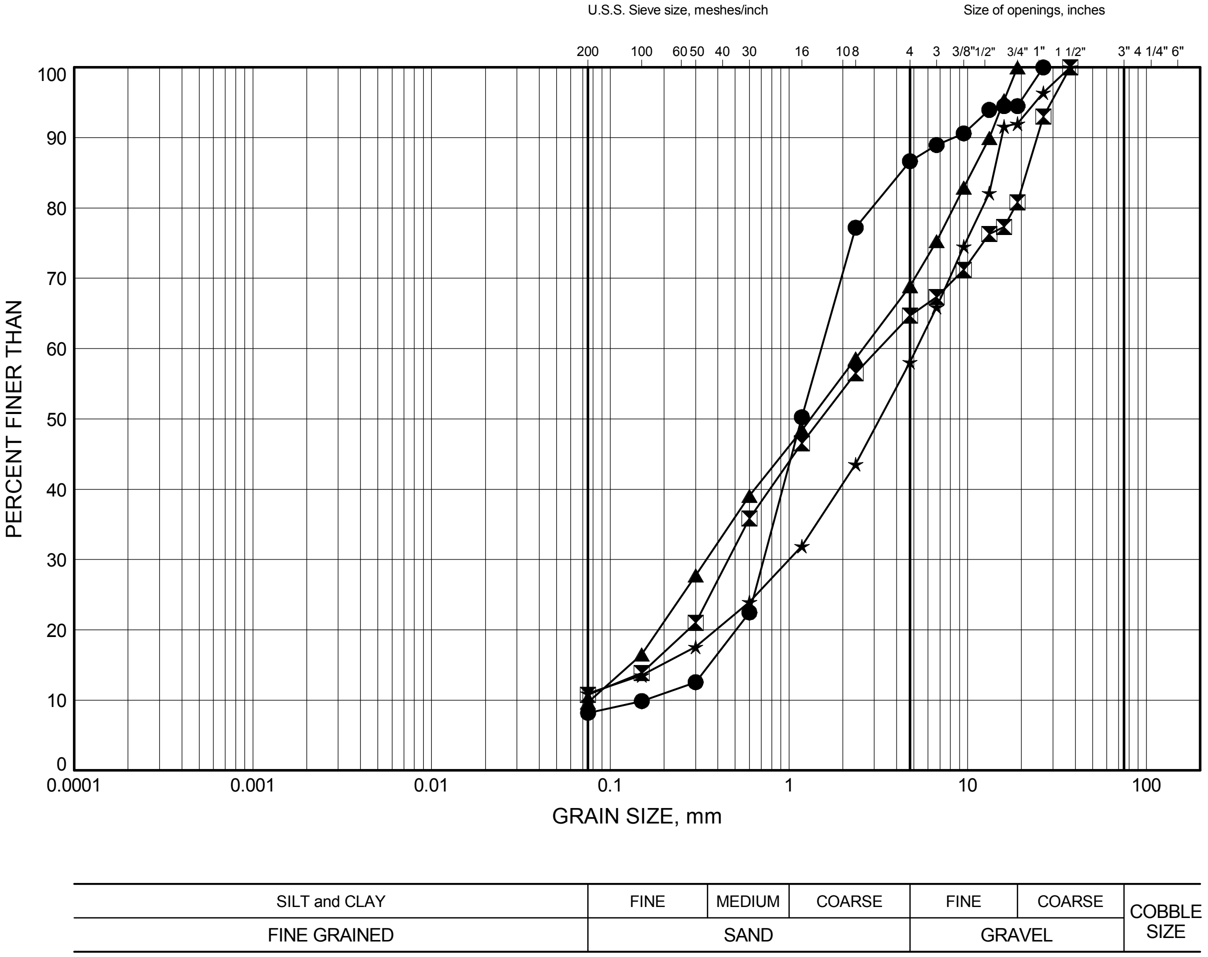
Chkd. PC

Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 10

Sand with silt



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	202	10.97	44.80
⧗	203	4.11	44.83
▲	204	4.88	44.17
★	206	12.73	35.53

Date June 2016

WP# 4088-13-01

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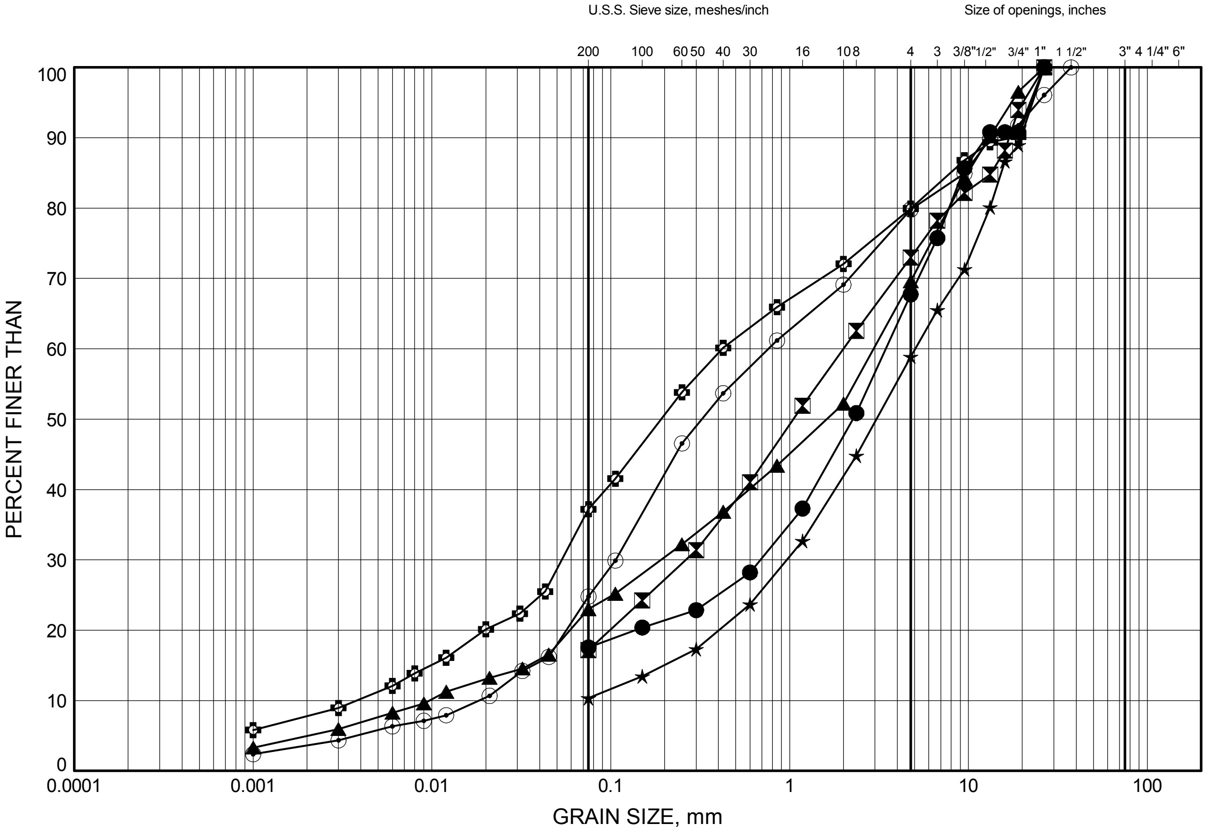
Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 11

Glacial Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	201	9.45	46.33
⧗	201	14.02	41.75
▲	202	18.59	37.18
★	203	7.16	41.78
⊙	203	9.45	39.50
✿	204	9.45	39.60

Date June 2016

WP# 4088-13-01

THURBER

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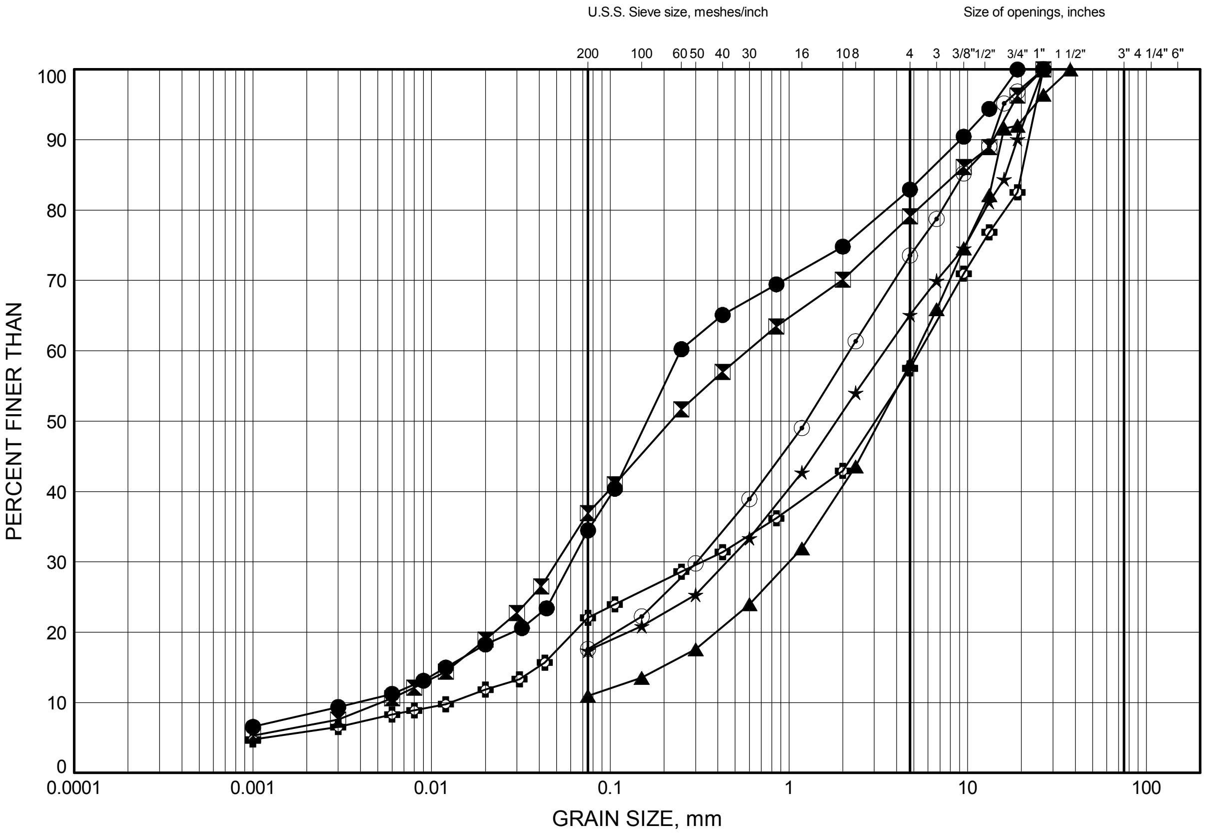
Chkd. PC

Site 31-241 - Highway 401 Underpass at Bainsville Rd.

GRAIN SIZE DISTRIBUTION

FIGURE 12

Glacial Till



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	205	5.64	42.91
⧗	205	11.89	36.66
▲	206	12.73	35.53
★	207	12.50	36.74
⊙	208	12.50	36.60
✤	210	19.35	35.22

Date June 2016

WP# 4088-13-01

THURBER

Prep'd KCP

Chkd. PC

GRAIN SIZE DISTRIBUTION - THURBER BAINSVILLE.GPJ 1/6/16

Stantec

Stantec Consulting Ltd.
400 - 1331 Clyde Avenue
Ottawa ON K2C 3G4
Tel: (613) 722-4420
Fax: (613) 722-2799

February 16, 2016
File: 122410864

Attention: Kenton Power
Thurber Engineering Ltd.
104 – 2460 Lancaster Road
Ottawa, Ontario, Canada, K1B 4S5
Tel: 613-274-2121
e-mail: kpower@thurber.ca

Dear Mr. Power,

Reference: Consolidation Test Results for Mega 3 Bainsville
Thurber File# (19-5161-263): Sample TW208 ST7 sampled on November 30, 2015

This letter presents the results of a one-dimensional consolidation test carried out on the above referenced sample in accordance with ASTM D2435. The test results are provided in the attached table and figure.

This letter provides test results only and does not constitute any interpretation or engineering recommendations with respect to material suitability or specification compliance.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Regards,

STANTEC CONSULTING LTD.

Raymond Hache, M.Sc., P.Eng.
Principal Geotechnical Engineer
Phone: (613) 738-6055
Fax: (613) 722-2799
Raymond.Hache@stantec.com

Attachment: Consolidation test results (1 table + 1 Figure)

Stantec

Consolidation Test Results

Project	Thurber Engineering, File#, 19-5161-263	Project No.	122410864
Sample No.	TW 208, ST7, Mega 5 Bainsville	Sample Depth (m)	4.267

Sample Data

Initial Ht. of soil, H_i	19.03 mm	Wet unit weight	15.16	kN/m^3	Odometer	B
Initial sample volume, V_i	38.66 cm^3	Dry unit weight	8.23	kN/m^3	ASTM Method	A
Specific gravity, G_s	2.780 Tested	Initial height of voids, H	1.328	cm	Load Duration	24 hours
Initial Water Content	85.0 %	Ht. of solids, H_s	0.575	cm	Start Date	30-Nov-15
Wet mass of soil	59.76 g	Initial Void Ratio, e_o	2.31		End Date	16-Dec-15
Dry mass of soil	32.46 g	Degree of Saturation	100.0	%		

Stage	Test Type	Stress Increment (kPa)	End of Load Deformation (cm)	Strain $\varepsilon = \Delta H/H_i$	$\Delta e = \Delta H/H_s$	Void Ratio e	End of Load Height (cm)	Corrected deformation ΔH50 (cm)	Specimen height H50 (cm)	Time t_{50} (min)	Coefficient of Consolidation c_v (cm^2/min)	Time t_{90} (min)	Coefficient of Consolidation c_v (cm^2/min)
Seating	Seating	0.00	0			2.311							
1	Consolidation	4.88	0.0014	0.00074	0.002	2.309	1.902						
2	Consolidation	10.59	0.0050	0.00263	0.009	2.302	1.898	0.0022	1.9008			0.90	0.8510
3	Consolidation	20.44	0.0122	0.00641	0.021	2.290	1.891	0.0068	1.8962			1.30	0.5863
4	Consolidation	41.28	0.0290	0.01524	0.050	2.261	1.874	0.0161	1.8869			1.40	0.5391
5	Consolidation	81.11	0.1898	0.09974	0.330	1.981	1.713	0.0740	1.8290			10.90	0.0651
6	Consolidation	119.34	0.3742	0.19664	0.651	1.660	1.529	0.2621	1.6409			12.40	0.0460
7	Consolidation	160.75	0.4688	0.24635	0.816	1.495	1.434	0.4091	1.4939			12.20	0.0388
8	Consolidation	320.05	0.6220	0.32685	1.082	1.229	1.281	0.5250	1.3780			4.70	0.0856
9	Consolidation	639.79	0.7360	0.38676	1.281	1.031	1.167	0.6634	1.2396			3.90	0.0835
10	Rebound	160.75	0.7112	0.37373	1.237	1.074	1.192						
11	Rebound	41.28	0.6790	0.35681	1.181	1.130	1.224						
12	Rebound	10.59	0.6542	0.34377	1.138	1.173	1.249						
13	Rebound	4.88	0.6420	0.33736	1.117	1.194	1.261						

Notes: Test Method A loading
Specimen from 304.8 - 330.2 mm from top of tube

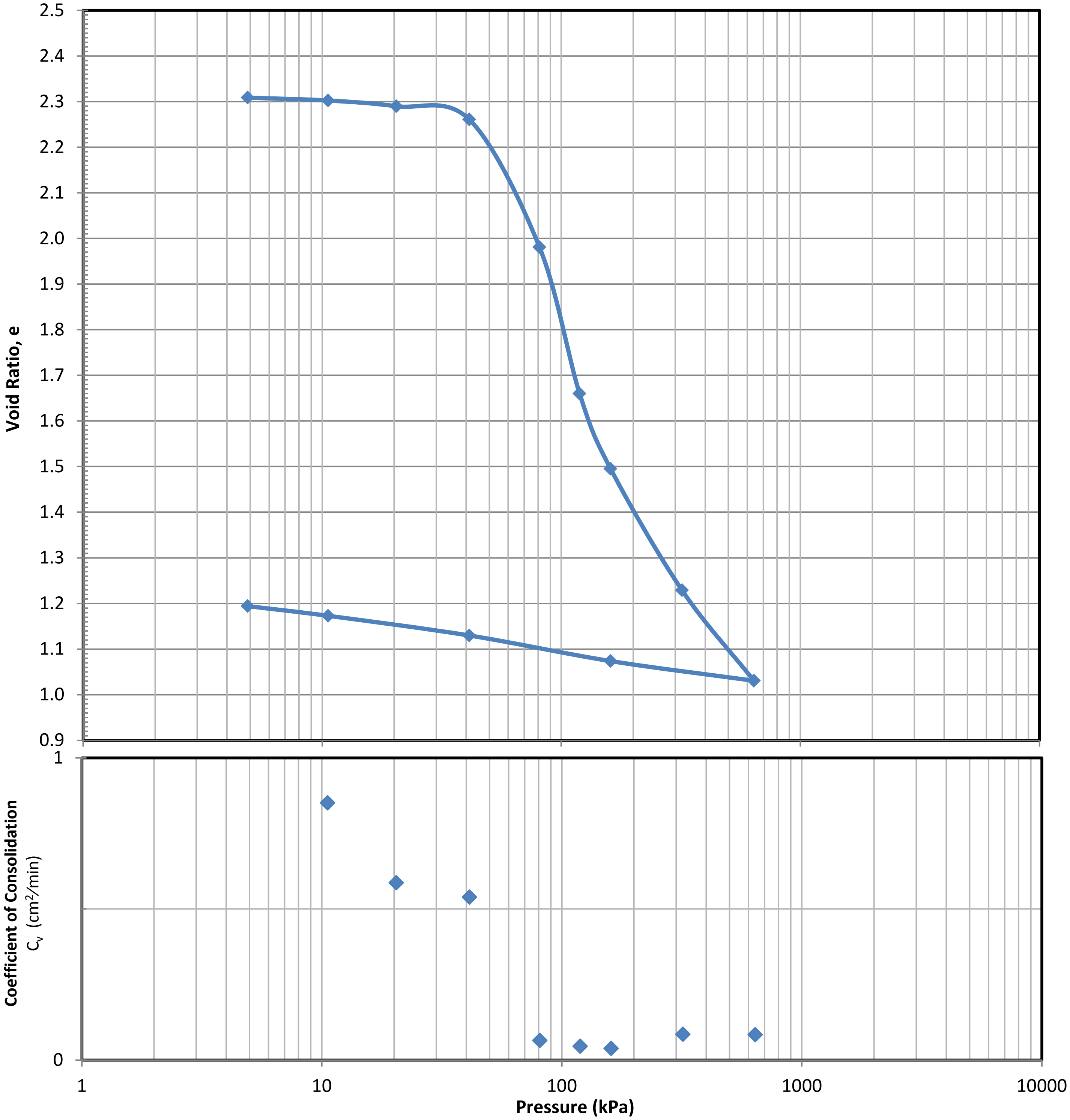
Conducted by: DB

Checked by: AN

One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D2435

Project	**Thurber Engineering, File#, 19-5161-263**
Project No.	**122410864**
Sample No.	**TW 208, ST7, Mega 5 Bainsville**
Sample Depth (m)	**4.267**



APPENDIX D

SELECTED PHOTOGRAPHS OF THE UNDERPASS LOCATION

Figure 1: Highway 401 looking eastbound from Bainsville Road

Figure 2: Highway 401 looking eastbound from Bainsville Road

Figure 3: Highway 401 underpass at Bainsville Road looking north

Figure 4: County Road 2 underpass at Bainsville Road looking west

Figure 5: Northwest embankment stability berm

Figure 6: North abutment foreslope slope pavers

Figure 7: South approach embankment looking south along Bainsville Road towards Boreholes 201 and 202

Figure 8: South abutment embankment foreslope looking east

Figure 9: Southeast embankment stability berm looking south

Figure 10: South abutment foreslope slope pavers

Figure 11: North approach embankment looking north towards Boreholes 209 and 210

APPENDIX E

DOWNHOLE SHEAR WAVE VELOCITY REPORT

GEOPHYSICS GPR INTERNATIONAL INC.

100 – 2545 Delorimier Street
Longueuil (Québec)
Canada J4K 3P7

Tel. : (450) 679-2400
Fax : (514) 521-4128
info@gprmtl.com
www.geophysicsgpr.com

February 4th, 2016

Transmitted by email: kpower@thurber.ca
Our Ref.: M-15171-A

Kenton C. Power, M.A.Sc., P.Eng.
Geotechnical Engineering
Thurber Engineering Ltd.
104-2460 Landcaster Road
Ottawa (ON) K1B 4S5

Subject: **Downhole Shear wave Velocity Survey, Bainsville**
[WP No.: 4113-01-01]

Dear Mr. Power,

Geophysics GPR International Inc. was requested by Thurber Engineering Ltd. to carry out a downhole shear wave velocity sounding under the Highway 401 Bainsville's overpass, to obtain the $\overline{V}_{S30}$ value for the site (seismic) classification according with the National Building Code.

The borehole was located beside the south shoulder of County Road 2 (cf. Figure 1). The surveys were carried out on December 10th, 2015 by Mr. Charles Trottier, M.A.Sc. phys. and Mr. Maxime Boudreault, and January 27th 2016 by Mr. Nicolas Beaulieu, Eng. and Mr. Patrick Therrien, E.I.T. Figure 1 illustrates the location of the borehole.

The following paragraphs briefly describe the survey design, the principles of the test method, the methodology for interpreting the data and finally, the results.

Downhole Survey

Prior to the seismic measurements, a 31 meters deep bore-hole was realized by Thurber Engineering Ltd. (BH 207). A 2 inches diameter PVC pipe was also installed and grouted. For the seismic measurements, a probe (BHG-2) including 3 orthogonal axis geophones (15 Hz resonance frequency) was used. The seismic data were recorded with a Terraloc Mark 6 seismograph (from ABEM Instruments).

The downhole survey was conducted using source points located 1.0 meter laterally apart from the borehole center. For every measurement, 3 different surface impacts were recorded using a 18 pounds sledgehammer:

- One vertical strike on a steel plate, recorded every meter of depth;
- Two reversed transversal strikes on a soil coupled steel H-beam, recorded every meter of depth.

The seismic records were realized with 4096 data sampled at 50 µs, with a pre-trig delay of 10 ms.

An electrical mechanism (BHGC-1) allowed the seismic probe to be adequately coupled with the PVC pipe at each depth of measurement, thus allowing the adequate seismic wave transmission from the surface to the geophones. Figure 2 schematically illustrates the general principle of this type of seismic survey.

A small scale MASW survey with 1 meter geophones spacing was also carried out on the site. These data sets would be used in case the seismic shear wave (S) arrivals near the ground surface would not be identifiable due to the compressional (P) wave-train interference.

More detailed descriptions of the methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.

Results

The seismic data were of moderate quality, most likely due to a guide wave through the grout. Figure 3 shows the reconstructed polarized seismogram for a horizontal component. To ease the shear wave identification for the overburden, the ESPC-MASW calculations results were used (Figure 4). The compressional seismic velocities (V_P) measured for the rock were also used to guide the deeper shear wave recognition.

The picks of the shear wave's arrival times, according to depth, are shown in Figure 5. Linear regressions were calculated on the picked data for segments showing linear trends (guided regressions). Figure 6 presents the results of the guided linear regressions, the sliding linear regression operators for 3 and 5 consecutive picks, and the ESPAC-MASW modelling results. The downhole guided regressions model consists of four velocity layers: 137 m/s from the surface to 4 meters deep; 90 m/s from 4 to 9 meters; 468 m/s from 9 to 16 meters; and 2677 m/s for the rock.

The $\overline{V}_{S30}$ value is based on the harmonic mean of the shear wave velocities, from the surface to 30 meters deep. It is calculated by dividing the total depth of interest (e.g. 30 meters) by the sum of the time spent in each velocity layer from the surface up to that depth. This harmonic mean value reflects an equivalent single layer response.

The calculated $\overline{V}_{S30}$ value is 285.8 m/s (Class "D"). Details of the $\overline{V}_{S30}$ calculation are presented in Table 1. Low seismic shear wave velocities were measured and calculated from the surface to 9 meters deep, especially from 4 to 9 meters.

Conclusion

A seismic site classification survey was realized by Geophysics GPR International inc. using the seismic downhole and ESPAC-MASW methods at the Highway 401 Bainsville's overpass. The borehole (BH 207), the PVC pipe installation and its grouting were provided by Thurber Engineering Ltd.

The downhole survey allowed measuring the shear wave velocities of the overburden and the rock. ESPAC-MASW results complemented the shallow portion, for the overburden materials. Based on this value (determined through the downhole and the MASW/ESPAC methods), Table 4.1.8.4.A of the NBC, and the Building Code, O. Reg. 332/12, the investigated site presented a calculated $\overline{V}_{S30}$ value of 286 m/s, corresponding to Site Class "D" ($180 < \overline{V}_{S30} \leq 360$ m/s).

Some low seismic shear wave velocities were measured and calculated for the overburden materials, from the surface to 9 meters deep. A geotechnical assessment could have to be addressed to the corresponding materials, regarding at least, the potential of liquefaction and the clay sensitivity.

It must be noted that other geotechnical information gleaned onsite; including the presence of liquefiable soils, soft clays, high moisture content etc. can supersede the site classification provided in this report based on the $\overline{V}_{S30}$ value.

The V_S values calculated are representative of the in situ materials, and were not corrected for the total and effective stresses.

This report has been written by Jean-Luc Arsenault, M.A.Sc, P.Eng.

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Project Manager

Figure 1: Bore Hole Location
(Source: Google Earth™)

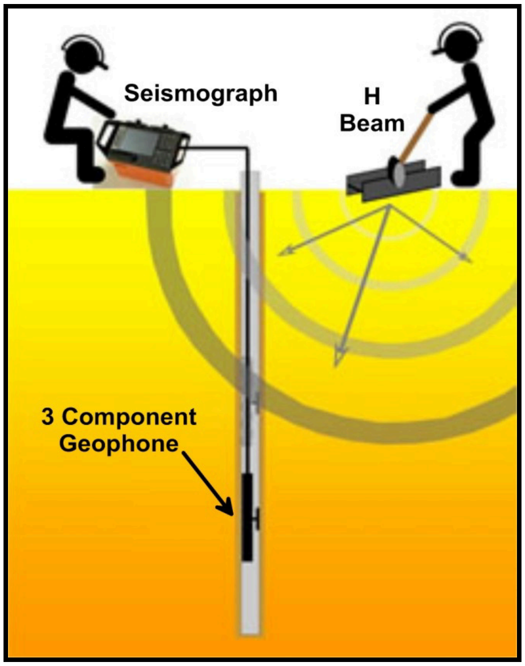


Figure 2: Schematic of a Downhole Seismic Survey

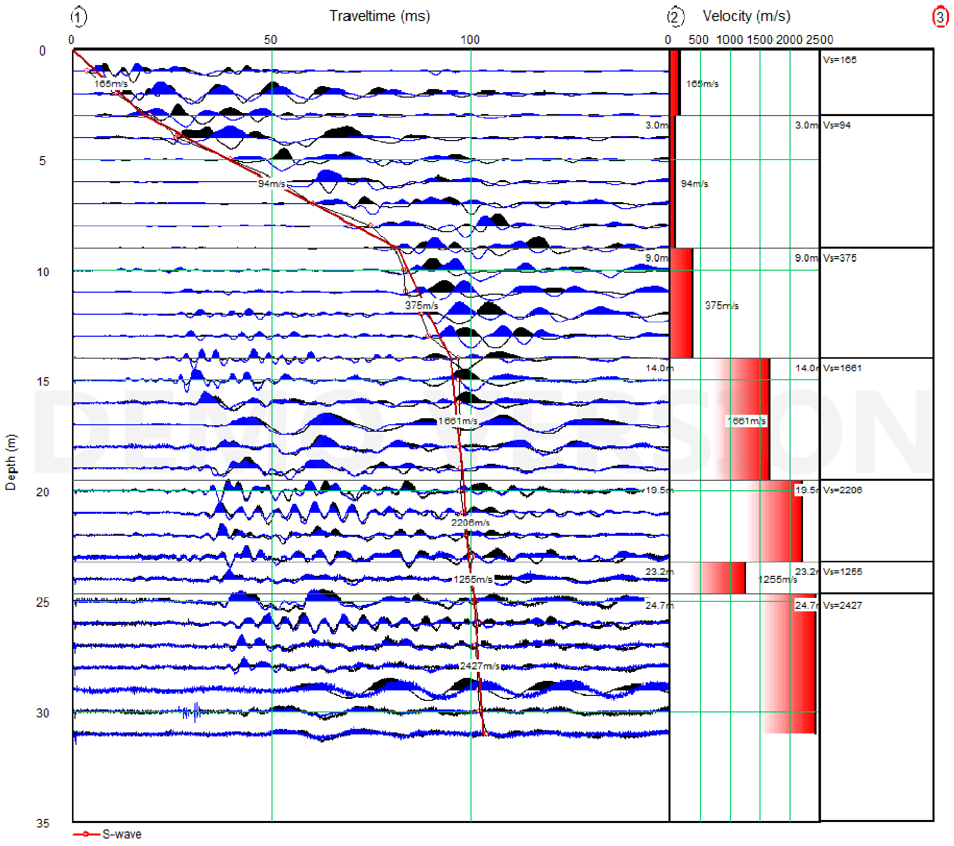
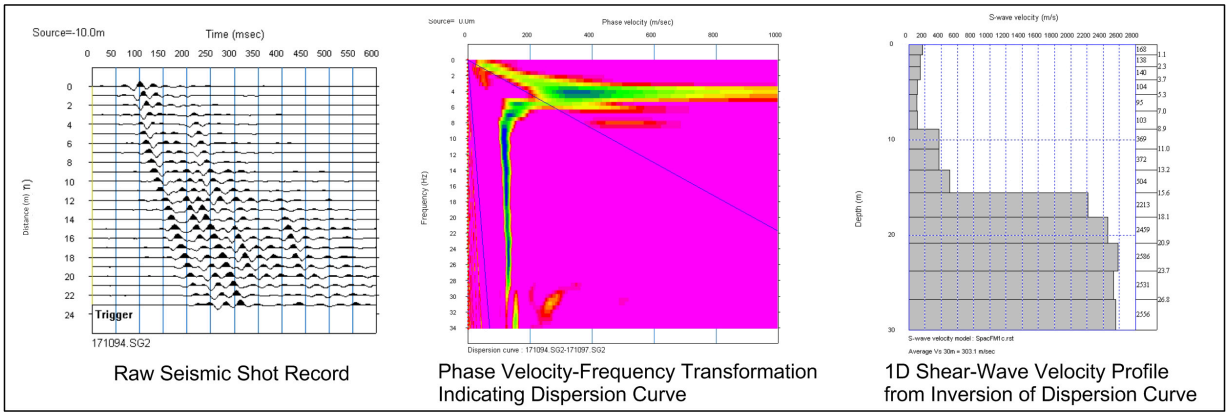


Figure 3: Downhole Polarized Seismogram (Horizontal Axis)



Raw Seismic Shot Record

Phase Velocity-Frequency Transformation Indicating Dispersion Curve

1D Shear-Wave Velocity Profile from Inversion of Dispersion Curve

Figure 4: MASW-ESPAC Procedure Steps

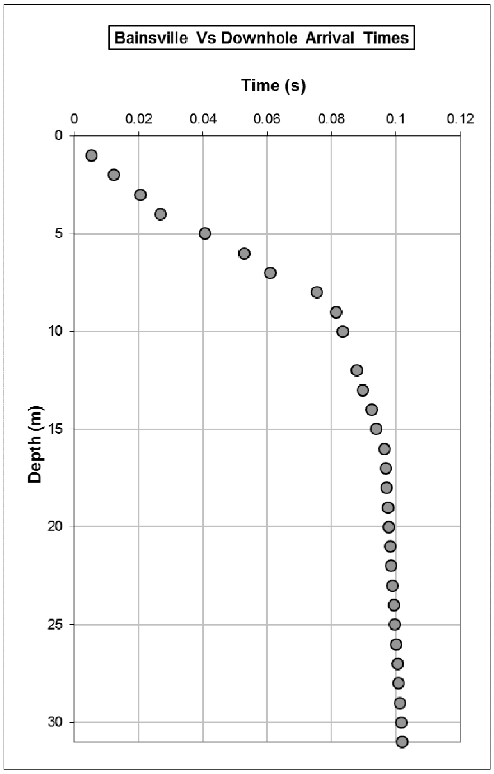


Figure 5: Shear Wave's Arrivals Times Picks

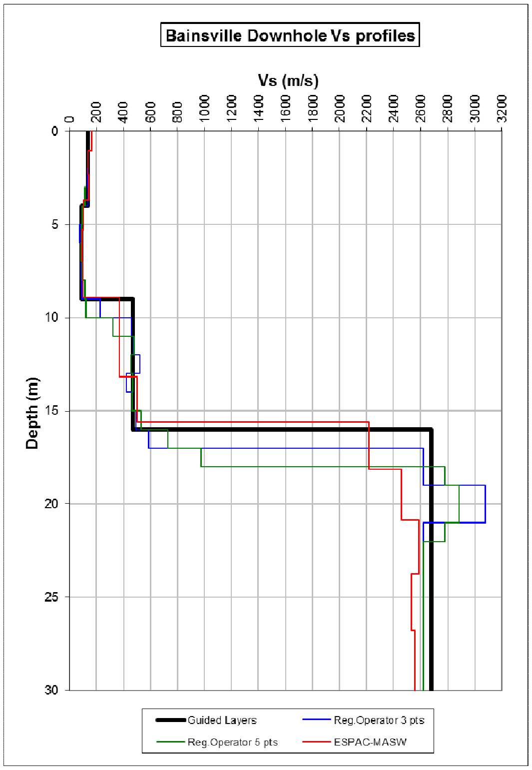


Figure 6: Downhole Survey Results (with ESPAC-MASW)

Table 1: Vs_{30} Calculation from Downhole and MASW Surveys Results

Depth	Vs	Thickness	Delay	Cumulated Delay	Cumulated Thickness	Vs (Z)
(m)	(m/s)	(m)	(s)	(s)	(m)	(m/s)
0	***136.8***					
4	***89.8***	4	0.029246	0.029246	4	136.8
9	468.1	5	0.055658	0.084903	9	106.0
16	2677.3	7	0.014955	0.099859	16	160.2
30		14	0.005229	0.105088	30	285.5

Vs30 =	285.5
Site Class:	D *

*: subject to geotechnical assessment of the unconsolidated materials from surface to 9 m deep.

APPENDIX F

TABLE F-1: FOUNDATION ALTERNATIVES COMPARISONS
TABLE F-2: COMPARISON OF FOUNDATION OPTIONS

F- 1: Evaluation of Embankment Design Options

Option	Description	Advantages	Disadvantages	Risks / Consequences	Relative Cost	Comments
1	**Granular Embankment** Construction of embankment fills at 2H:1V using conventional construction techniques.	Conventional construction Low cost	Pre-loading period of several years required to achieve sufficient degree of consolidation	Settlement is slower than expected and pre-load period needs to be extended / further delays to project schedule	Low	**Not Recommended**
3	**Lightweight Fill** Use of lightweight material for embankment fill in order to limit stress increase. Can achieve zero stress increase by excavating and replacing some material beneath the embankment. Lightweight fill options include slag based aggregate, tire derived aggregate, expanded polystyrene and cellular concrete.	Relatively fast construction Addresses both settlement and stability concerns	Specialized construction techniques required therefore a contractor with experience in the design and constructing embankments with light fill will be required		Medium	**Recommended**
2	**Ground Improvement** Treatment of the ground to make it less compressible through methods such as deep soil mixing.	Relatively fast construction Addresses both settlement and stability concerns	The sensitivity of the clay and thickness of the clay deposit means that very few ground improvement techniques are feasible and also increases the cost. Zone to be treated is buried beneath existing embankment		Medium to High	**Not Recommended**
4	**Accelerated Settlement** Acceleration of the settlement process by surcharging the site. Settlement could be further accelerated by inclusion of wick drains.	Settlement timing can be controlled by wick drain spacing	Zone to be treated is buried beneath existing embankment A drainage layer cannot be constructed under the existing embankment to work in conjunction with the wick drains therefore consolidation of the clay layer will take longer to accomplish Difficulty advancing wick drains through existing embankment Significant delay to construction schedule	Settlement is slower than expected and surcharge period needs to be extended / further delays to project schedule	Low to Medium	**Not Recommended**

F- 2: Comparison of Deep Foundation Alternatives

Steel Pipe Piles	Steel H-Piles	Caissons
Advantages: Quick installation procedure Low cost	***Advantages:*** Quick installation procedure Low cost	***Advantages:*** High axial and lateral resistance
Disadvantages: Generally lower resistance than H-piles Increased risk of damage during driving through glacial till deposit.	***Disadvantages:*** N/A	***Disadvantages:*** High cost Constructability concerns due boulders within glacial till and existing piles
FEASIBLE	**RECOMMENDED**	**NOT RECOMMENDED**

APPENDIX G

HISTORICAL SETTLEMENT DATA
HISTORICAL CLAY REMOVAL SCHEMATIC DRAWING
GSC SEISMIC HAZARD CALCULATION
SLOPE STABILITY ANALYSIS
L-PILE ANALYSIS FOR HP 310X110 STEEL PILES

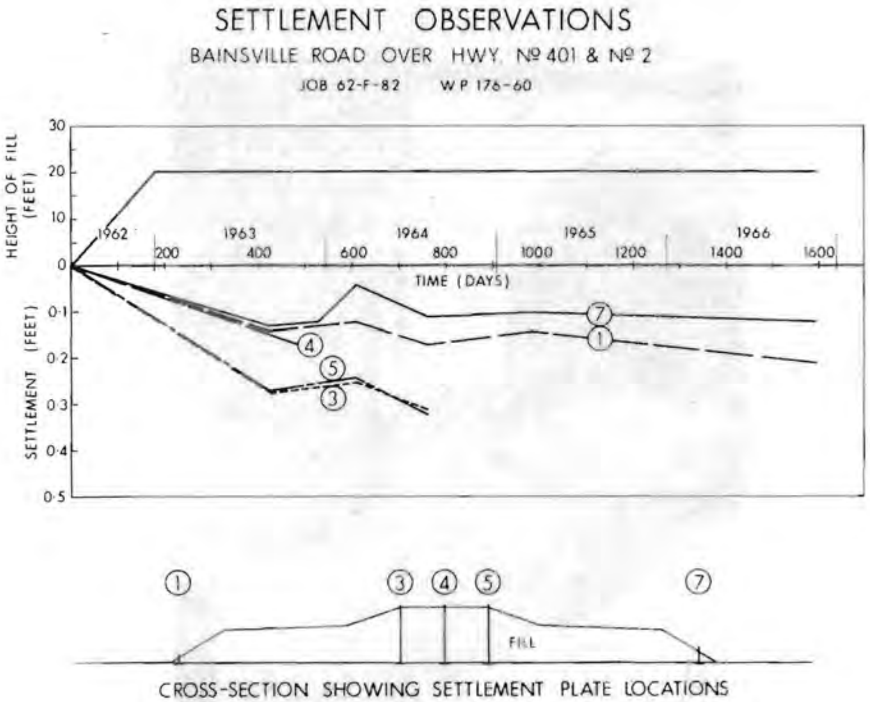
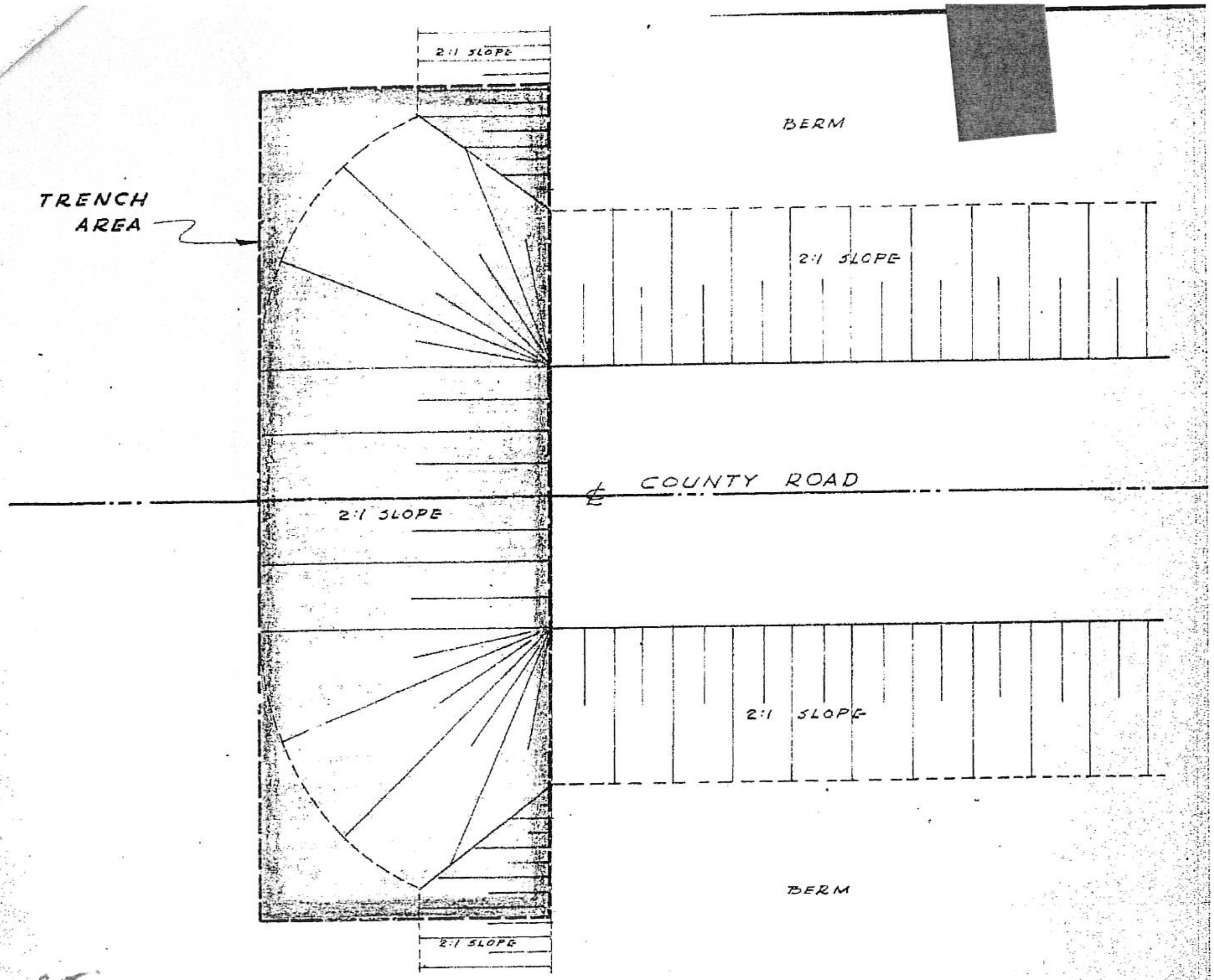


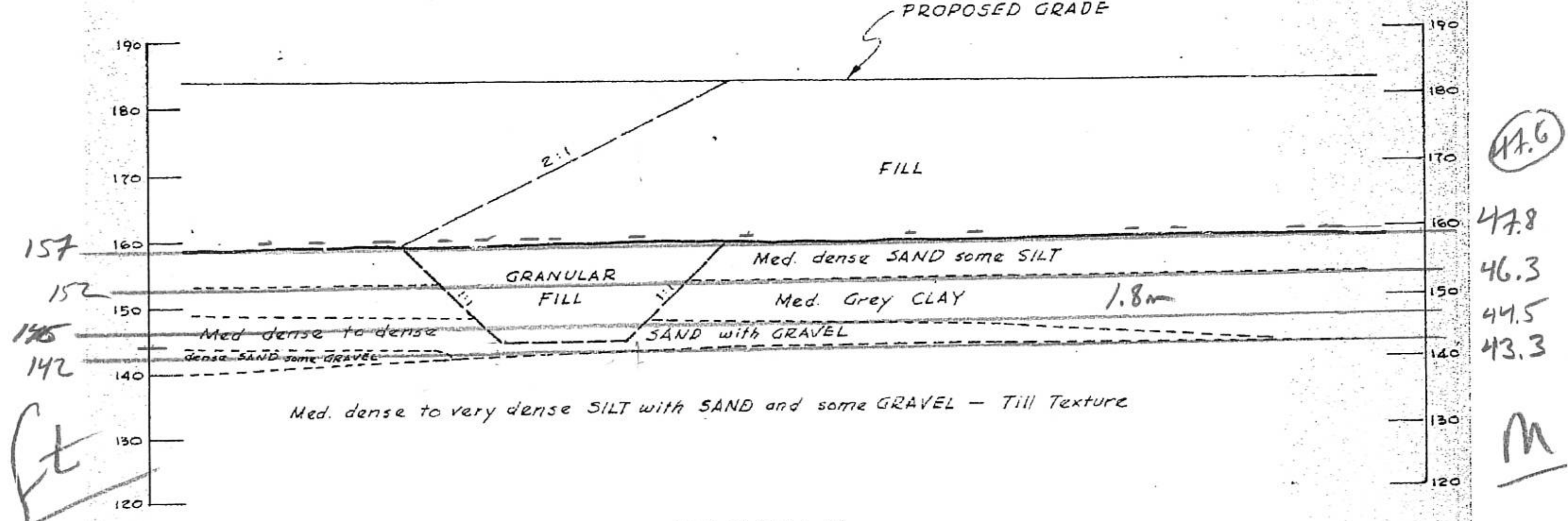
FIGURE 20, SETTLEMENT RECORDS, HIGHWAY 401, HIGHWAY 2 AND BAINSVILLE ROAD UNDERPASS (SOUTH SIDE)



210
~~209~~
9.2m
8.5m

PLAN

Scale :- 1 inch = 20 feet



PROFILE

Scale :- 1 inch = 20 feet

W.P. 176-6

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

February 25, 2016

Site: 45.1765 N, 74.4092 W User File Reference: 31-241 Bainsville Road Underpass

Requested by: , Thurber Engineering Ltd.

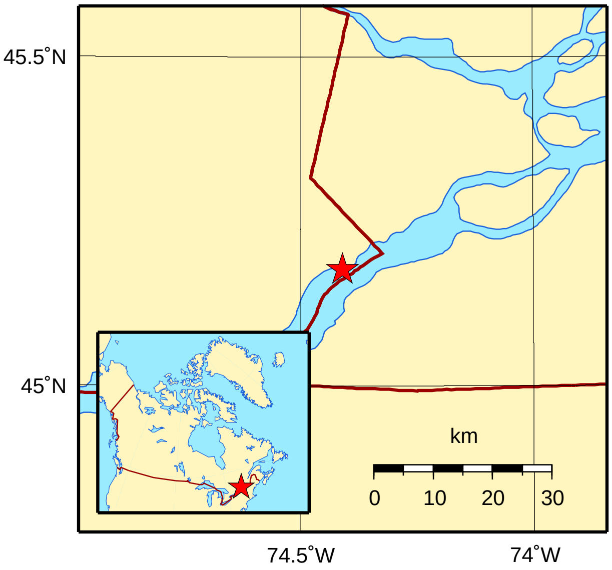
National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.634	0.725	**0.599**	0.450	**0.315**	**0.151**	**0.069**	**0.018**	**0.0062**	**0.381**	**0.260**

Notes. Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. ***These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.057	0.211	0.354
Sa(0.1)	0.077	0.257	0.418
Sa(0.2)	0.067	0.216	0.348
Sa(0.3)	0.052	0.163	0.261
Sa(0.5)	0.036	0.112	0.181
Sa(1.0)	0.018	0.053	0.086
Sa(2.0)	0.0070	0.024	0.039
Sa(5.0)	0.0014	0.0055	0.0096
Sa(10.0)	0.0007	0.0021	0.0035
PGA	0.041	0.138	0.224
PGV	0.025	0.086	0.144



References

National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français

Title: Highway 401 Underpass at Bainsville Road - North Abutment
Comments: Embankment Stability Assessment
Name: 1 - Existing Embankment Drained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-11.933333, 67.2) w/ Radius: 15.8 m
FoS Contours: 1.2 to 2.2, ++0.1

FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (ESA)	17 kN/m³	4 kPa	27 °
Silt	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

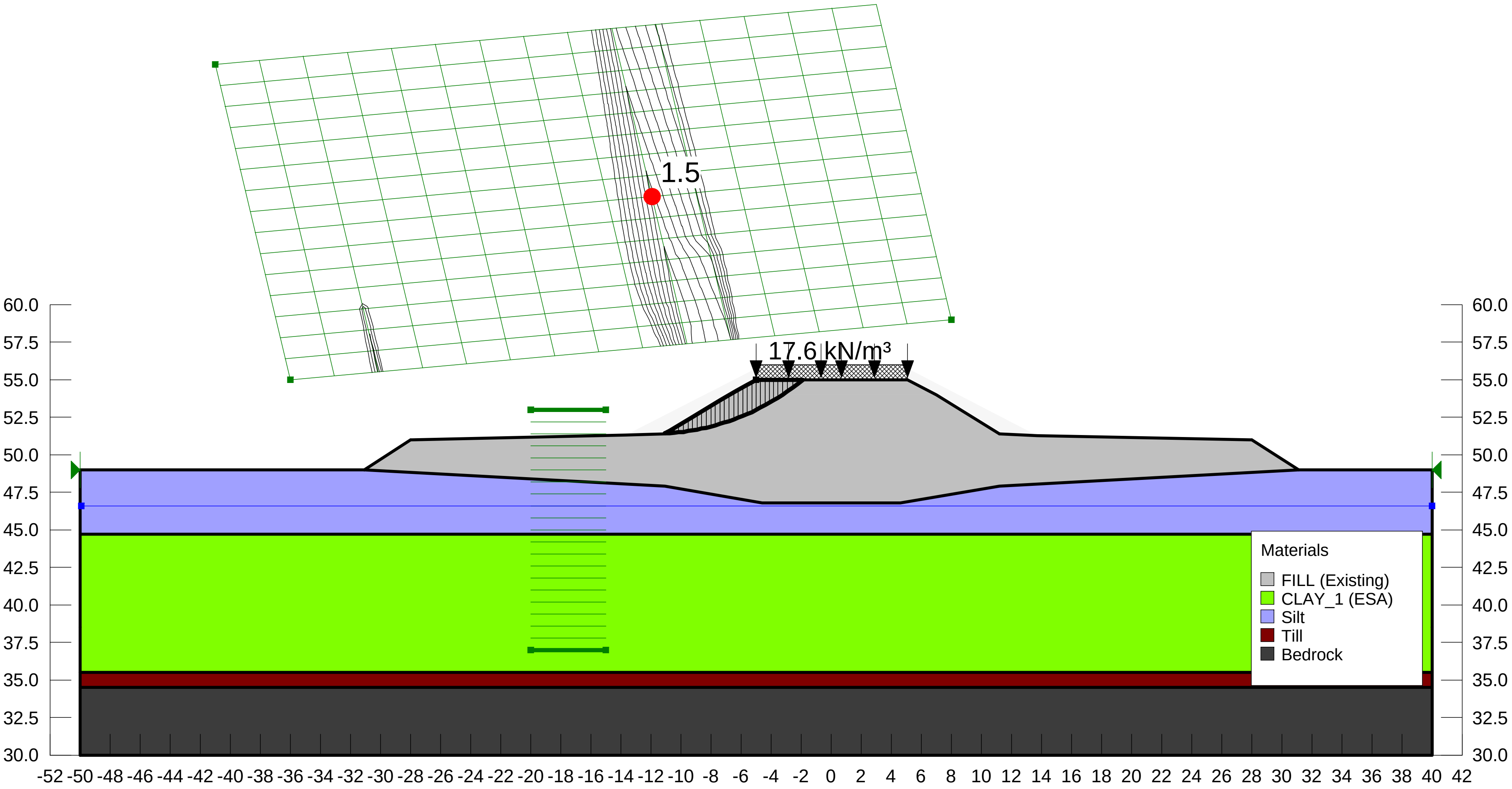


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Figure 1

Title: Highway 401 Underpass at Bainsville Road - North Abutment
Comments: Embankment Stability Assessment
Name: 2 - Existing Embankment Undrained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-10.933333, 63) w/ Radius: 11.6 m
FoS Contours: 1.2 to 2.2, ++0.1

FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	60 kPa	0 °
Silt	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

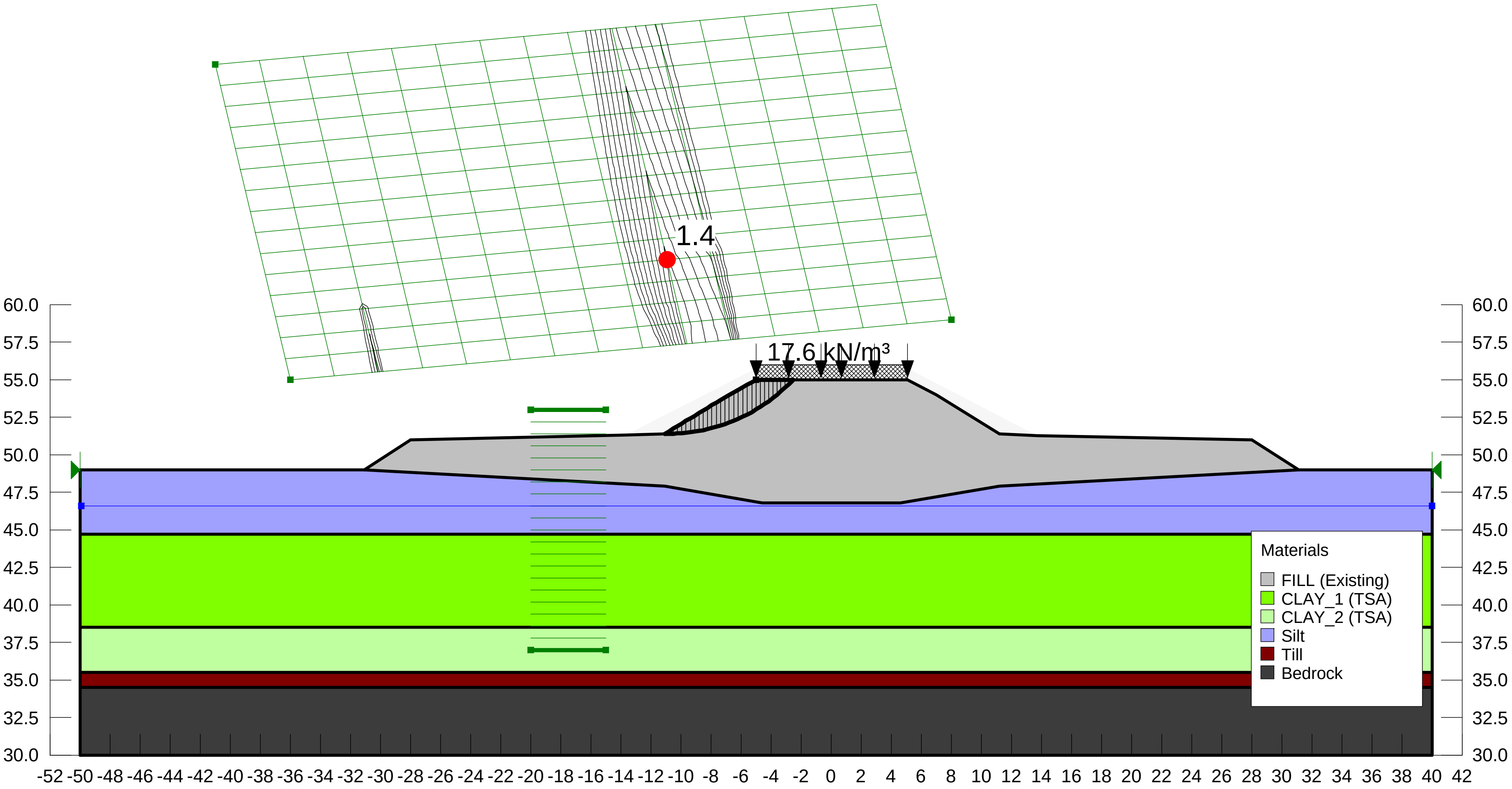

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Figure 2

Title: Highway 401 Underpass at Bainsville Road - North Abutment
Comments: Embankment Stability Assessment
Name: 3 - Existing Embankment Seismic

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0.19 V\ 0
Slip Surface Center: (-11.266667, 64.4) w/ Radius: 13 m
FoS Contours: 0.9 to 1.9, ++0.1

FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	60 kPa	0 °
Silt	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

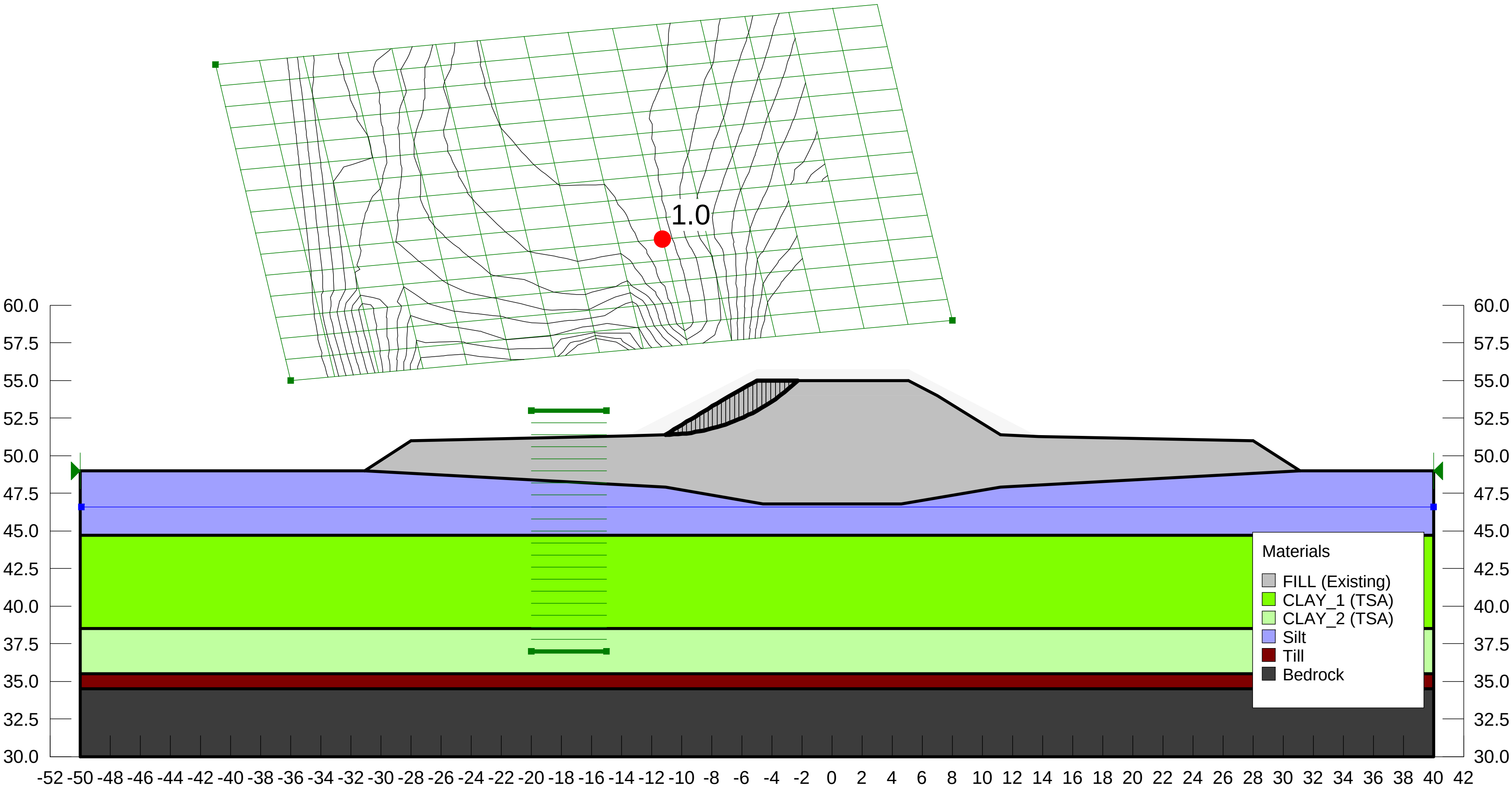

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Figure 3

Title: Highway 401 Underpass at Bainsville Road - North Abutment
Comments: Embankment Stability Assessment
Name: 4 - Embankment Granular Grade Raise Drained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-10.933333, 64) w/ Radius: 11.8 m
FoS Contours: 1.3 to 2.3, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (ESA)	17 kN/m³	4 kPa	27 °
Silt	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

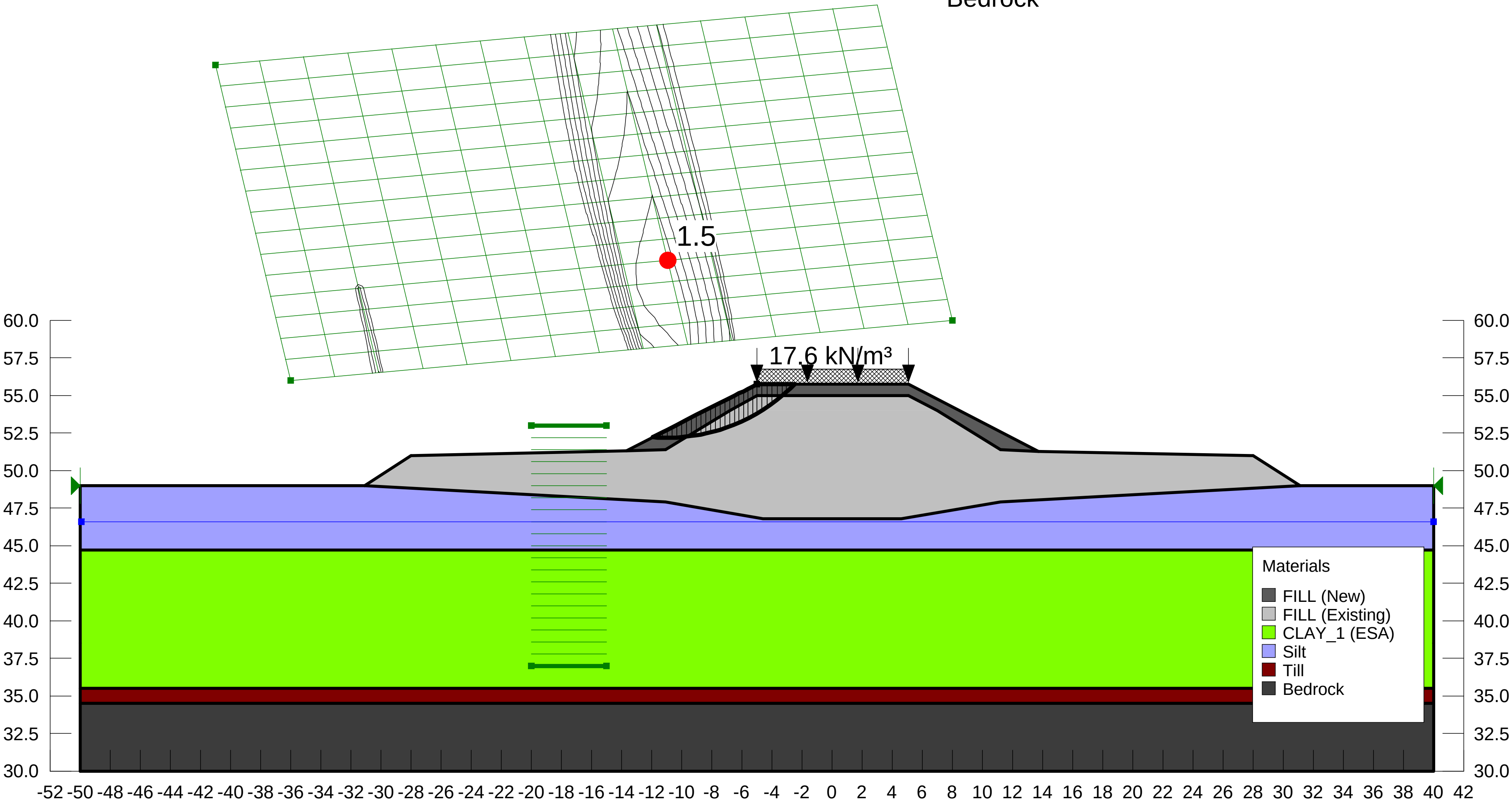

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Figure 4

Title: Highway 401 Underpass at Bainsville Road - North Abutment
Comments: Embankment Stability Assessment
Name: 5 - Embankment Granular Grade Raise Undrained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-10.266667, 61.2) w/ Radius: 9 m
FoS Contours: 1.3 to 2.3, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	60 kPa	0 °
Silt	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

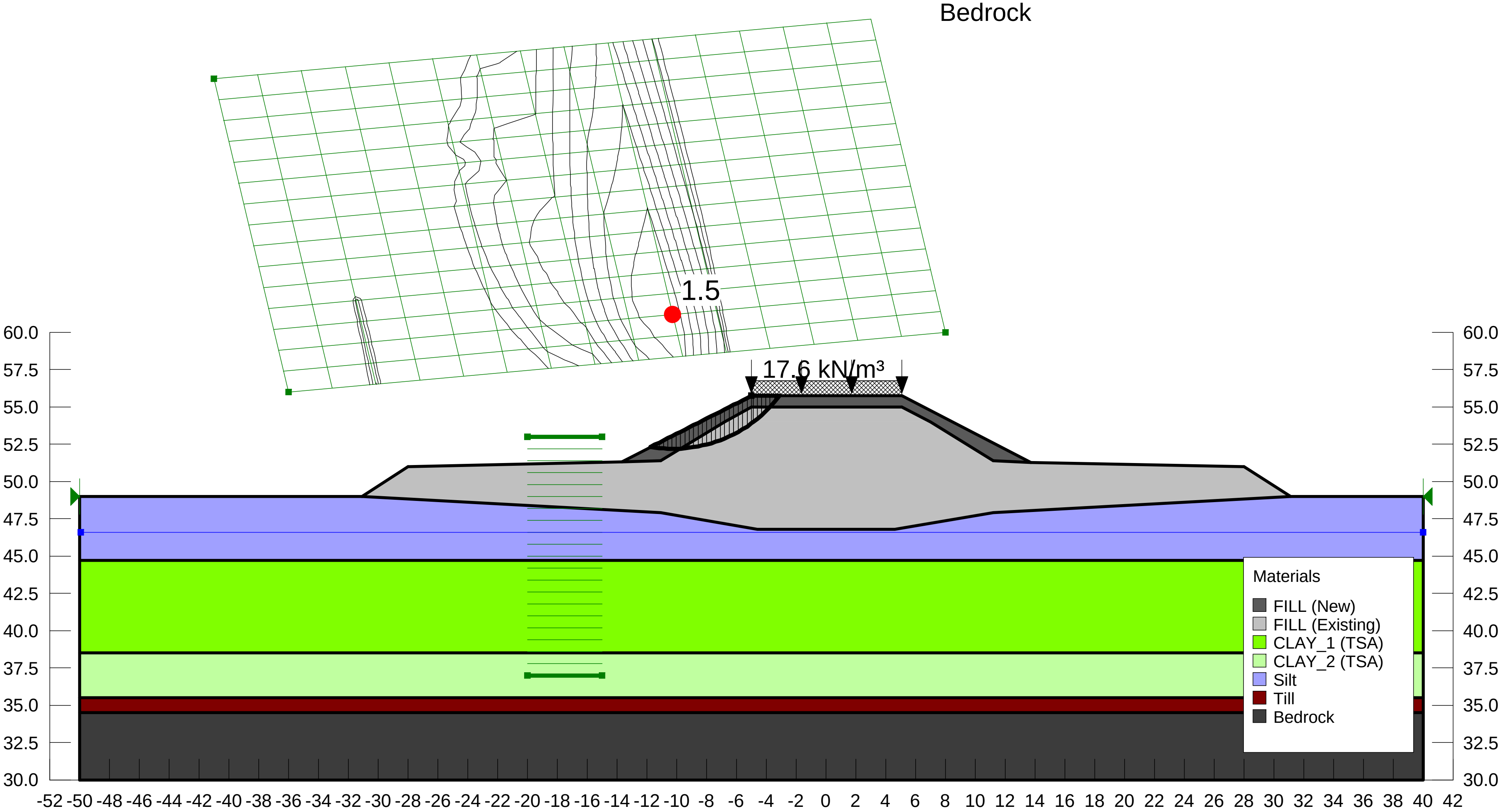

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Figure 5

Title: Highway 401 Underpass at Bainsville Road - North Abutment
Comments: Embankment Stability Assessment
Name: 6 - Embankment Granular Grade Raise Seismic

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0.19 V\ 0
Slip Surface Center: (-16.866667, 76.333333) w/ Radius: 37.733333 m
FoS Contours: 0.9 to 1.9, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	60 kPa	0 °
Silt	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

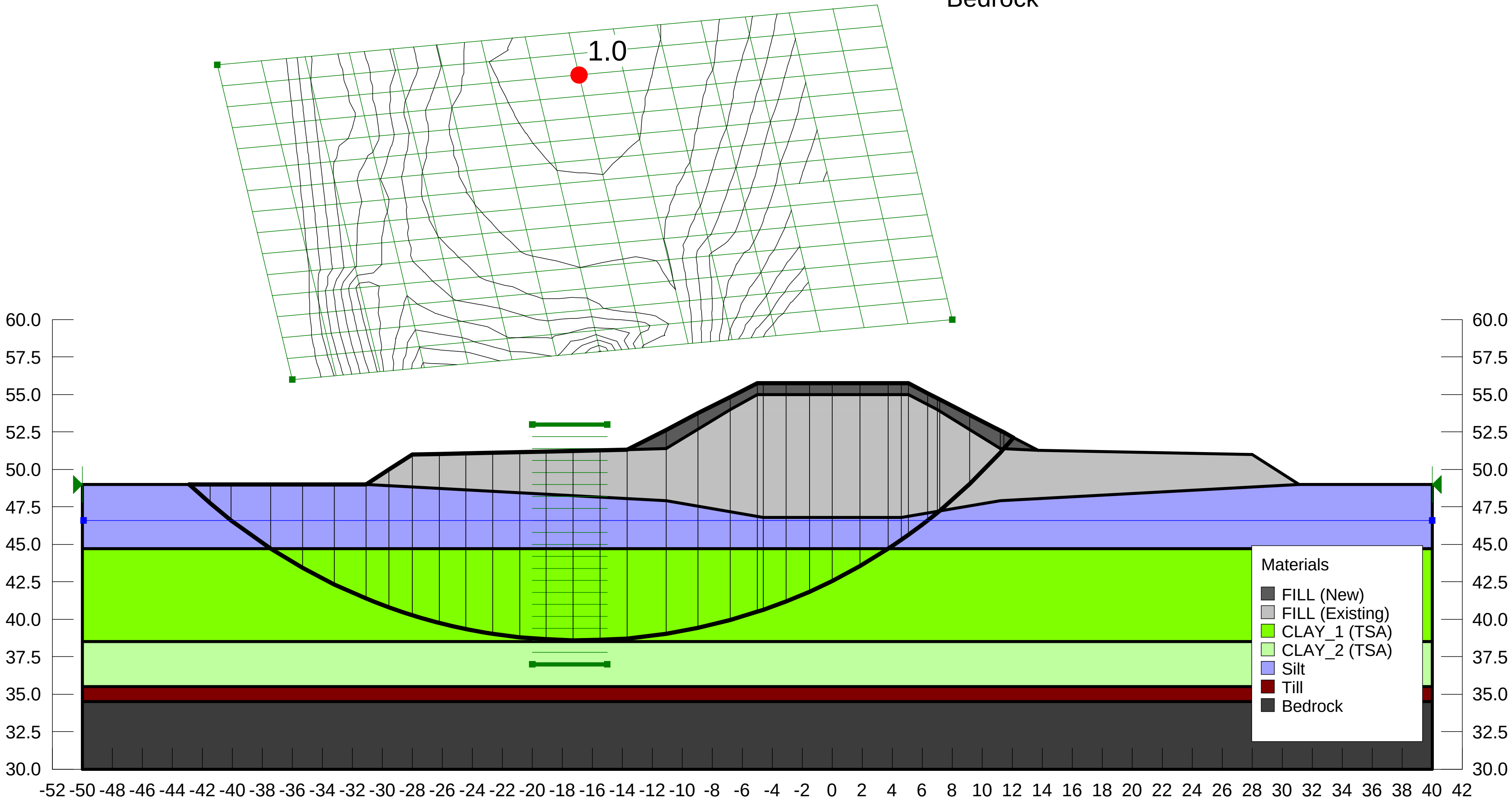

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Figure 6

Title: Highway 401 Underpass at Bainsville Road - North Abutment
Comments: Embankment Stability Assessment
Name: 7 - Embankment EPS Grade Raise Drained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-11.6, 66.8) w/ Radius: 14.6 m
FoS Contours: 1.0 to 2.0, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
EPS	1 kN/m³	1 kPa	0 °
CLAY_1 (ESA)	17 kN/m³	4 kPa	27 °
Silt	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

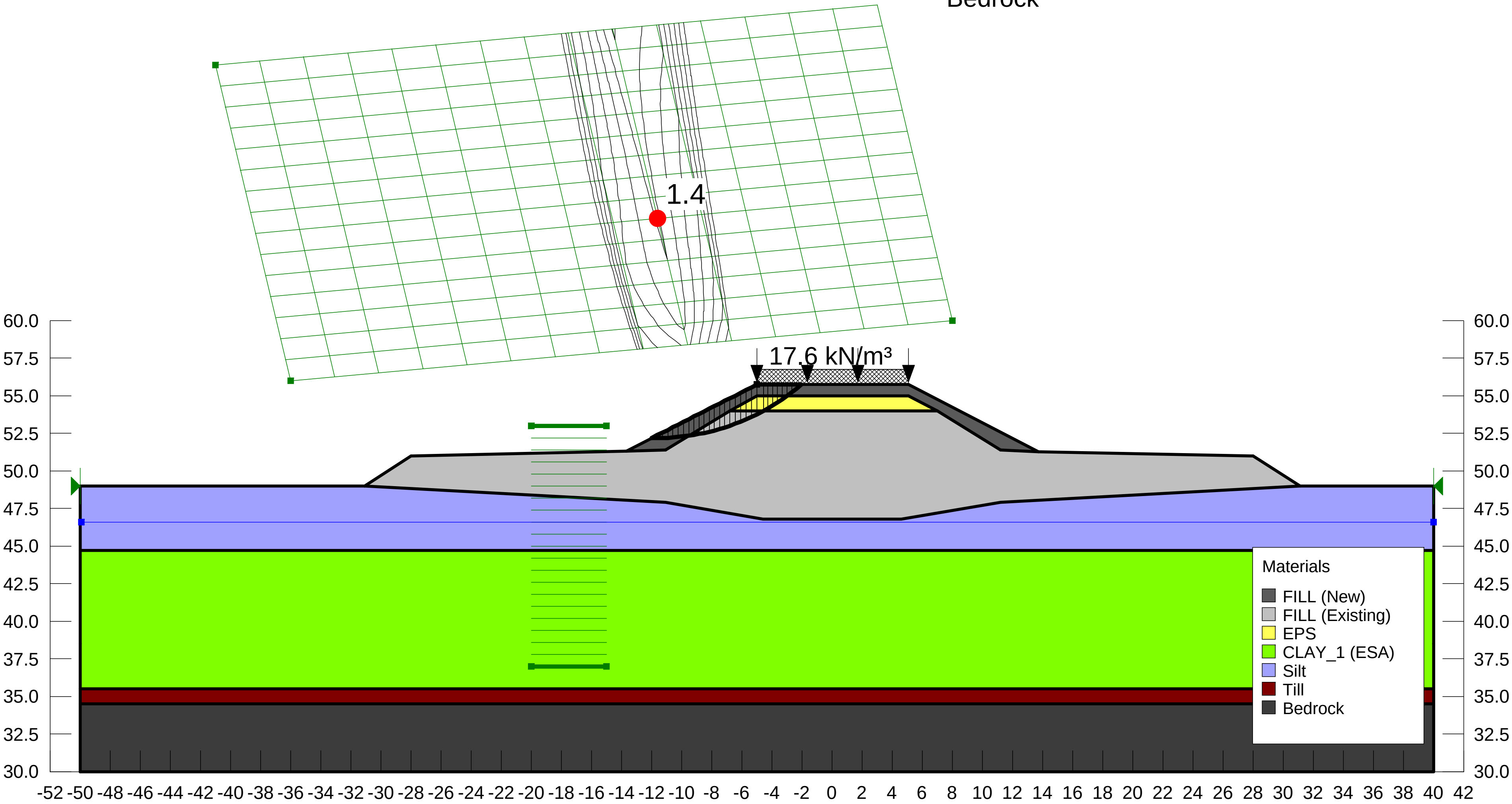

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Figure 7

Title: Highway 401 Underpass at Bainsville Road - North Abutment
Comments: Embankment Stability Assessment
Name: 8 - Embankment EPS Grade Raise Undrained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-12.6, 71) w/ Radius: 18.8 m
FoS Contours: 1.0 to 2.0, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
EPS	1 kN/m³	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	60 kPa	0 °
Silt	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

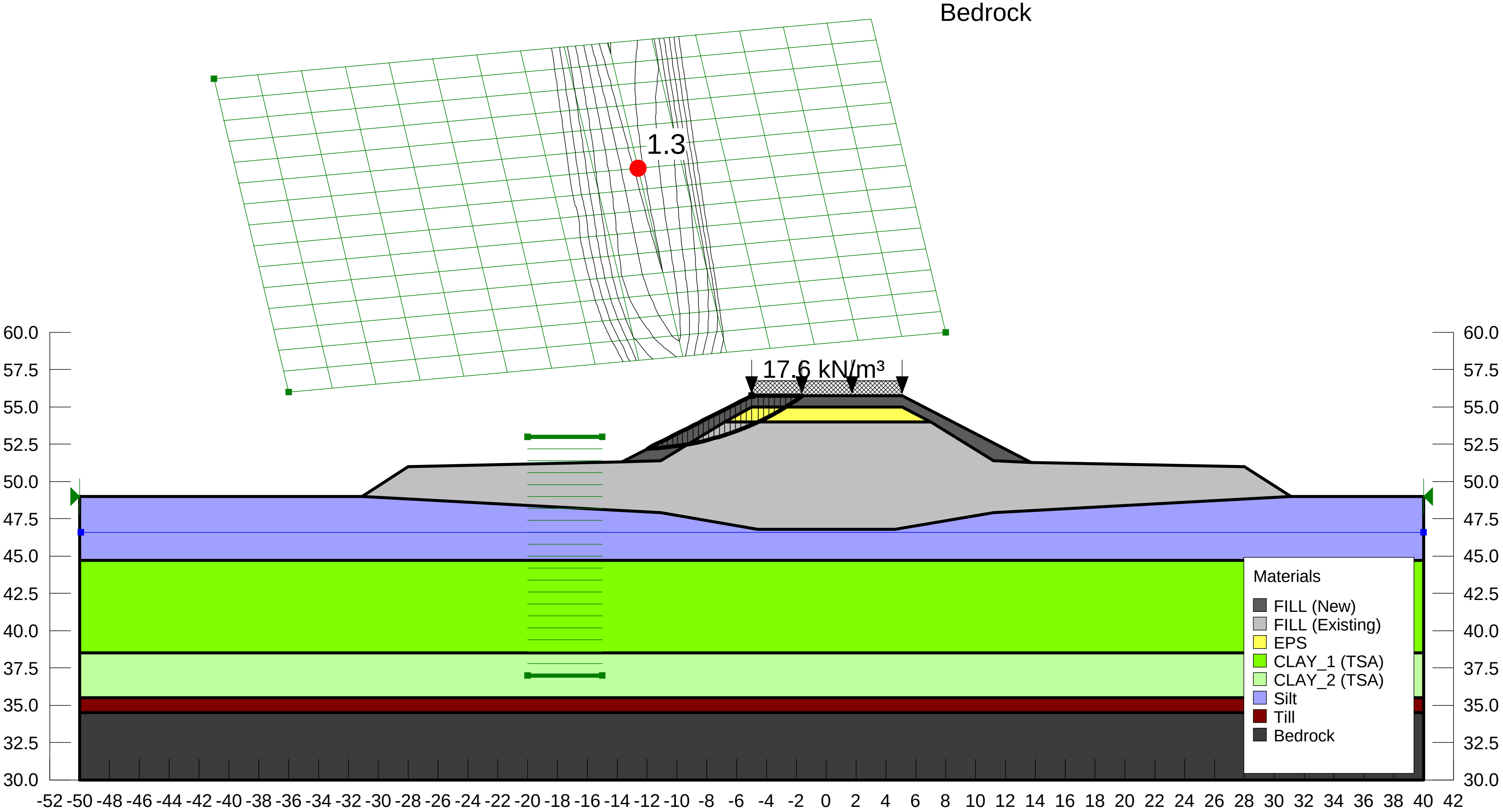

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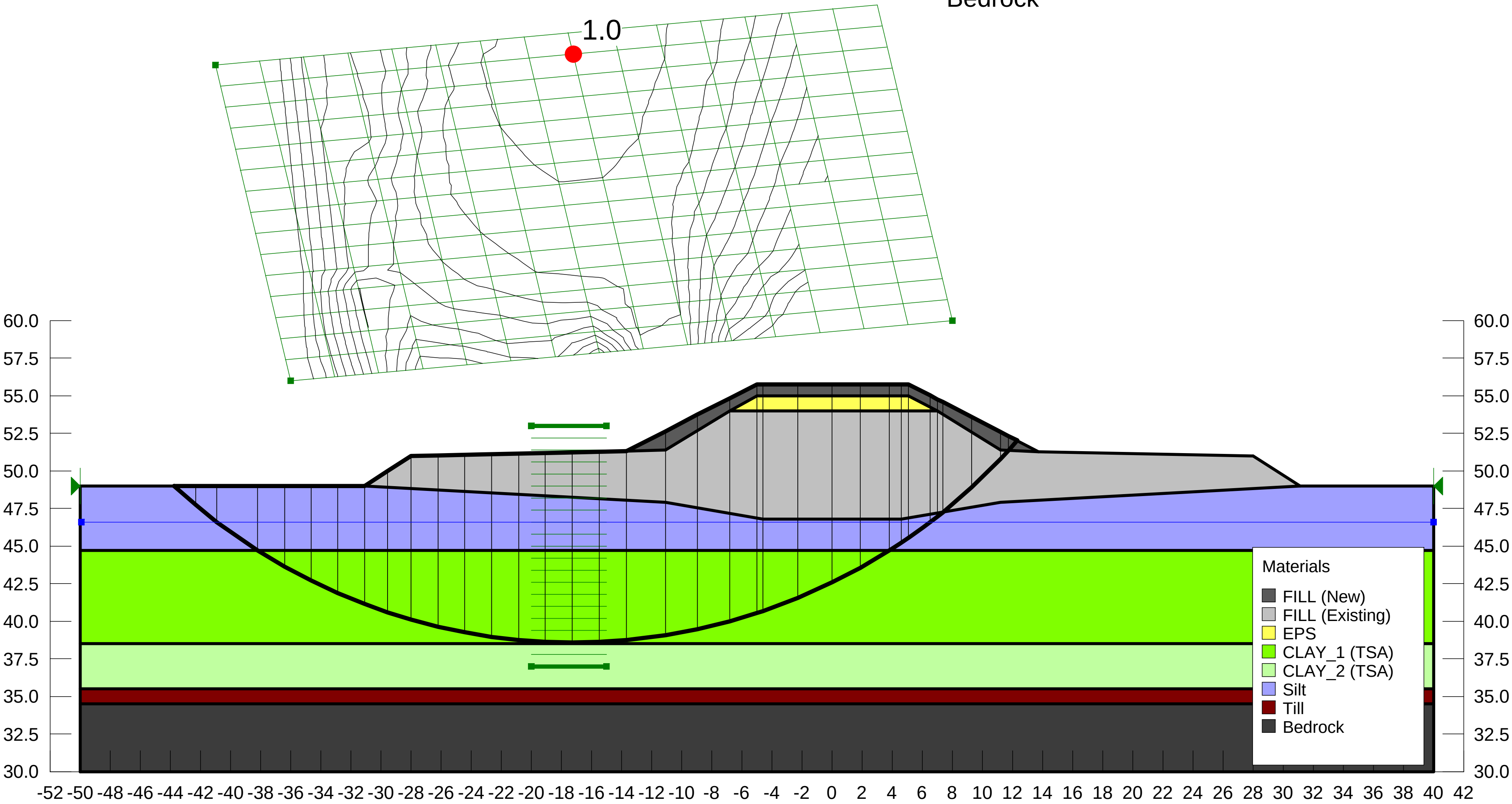
Figure 8

Title: Highway 401 Underpass at Bainsville Road - North Abutment
Comments: Embankment Stability Assessment
Name: 9 - Embankment EPS Grade Raise Seismic

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0.19 V\ 0
Slip Surface Center: (-17.2, 77.733333) w/ Radius: 39.133333 m
FoS Contours: 1.0 to 2.0, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
EPS	1 kN/m³	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	60 kPa	0 °
Silt	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			



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Tool Version: 8.15.5.11777
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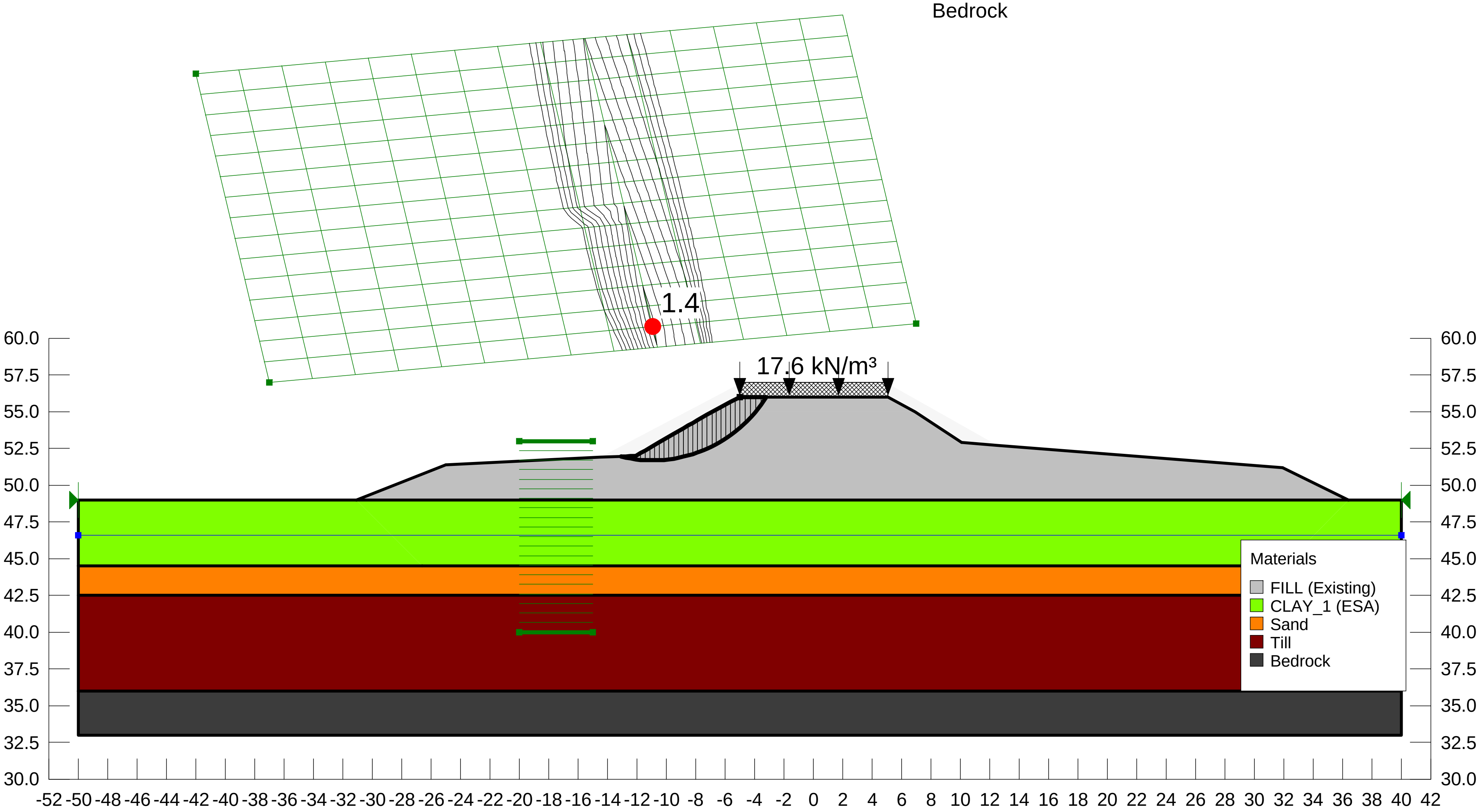
Directory: H:\Projects\19\5161\263 - ER Mega 5\Bridges\Site 31-241-Bainsville Rd Underpass\Foundations\Analysis\Stability\Bainsville North embakment complete.gsz

Figure 9

Title: Highway 401 Underpass at Bainsville Road - South Abutment
Comments: Embankment Stability Assessment
Name: 10 - Existing Embankment Drained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-10.933333, 60.8) w/ Radius: 9.1 m
FoS Contours: 1.2 to 2.2, ++0.1

FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (ESA)	17 kN/m³	4 kPa	27 °
Sand	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			



Reviewed By: ________________
Tool Version: 8.15.5.11777
Last Solved Date: 20/06/2016, 9:54:19 AM
Directory: C:\Users\KPOWER\Desktop\Bainsville South embankment complete.gsz

Figure 10

Title: Highway 401 Underpass at Bainsville Road - South Abutment
Comments: Embankment Stability Assessment
Name: 11 - Existing Embankment Undrained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-10.933333, 60.8) w/ Radius: 9.1 m
FoS Contours: 1.2 to 2.2, ++0.1

FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	15.5 kN/m³	20 kPa	0 °
Sand	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

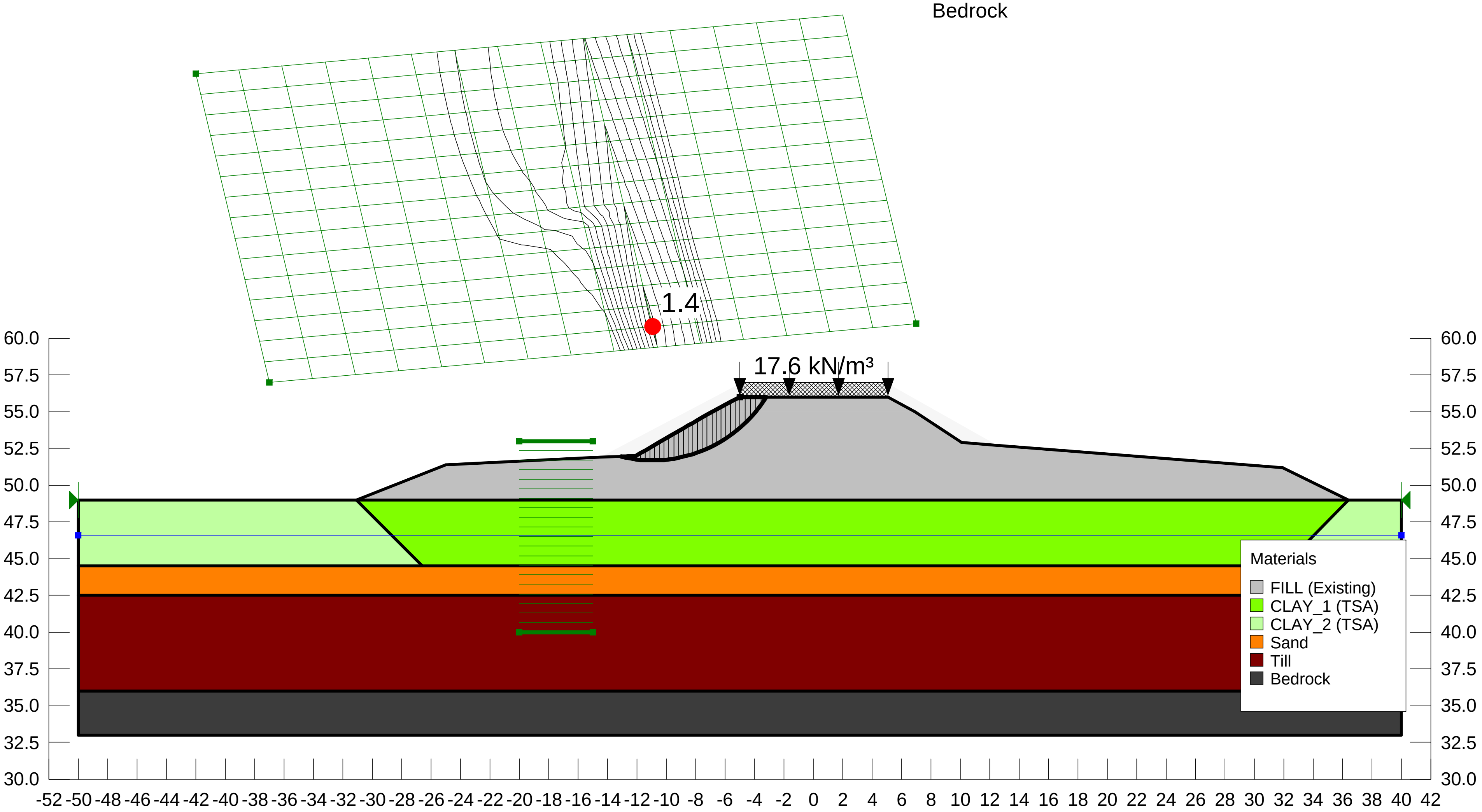


Figure 11

Reviewed By: _______________
Tool Version: 8.15.5.11777
Last Solved Date: 20/06/2016, 9:54:29 AM
Directory: C:\Users\KPOWER\Desktop\Bainsville South embankment complete.gsz

Title: Highway 401 Underpass at Bainsville Road - South Abutment
Comments: Embankment Stability Assessment
Name: 12 - Existing Embankment Seismic

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0.19 V\ 0
Slip Surface Center: (-11.266667, 62.2) w/ Radius: 10.5 m
FoS Contours: 0.9 to 1.9, ++0.1

FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	15.5 kN/m³	20 kPa	0 °
Sand	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			

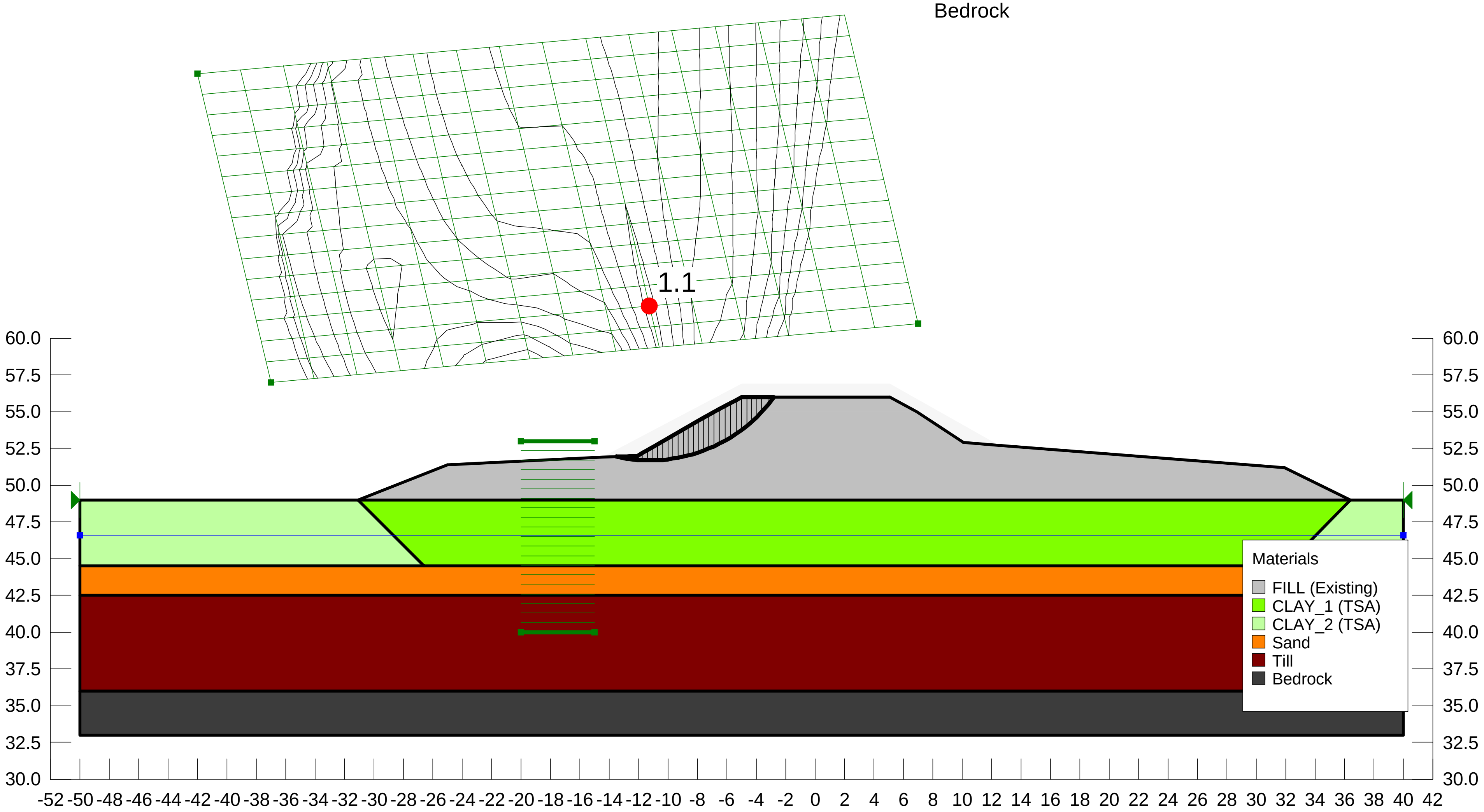


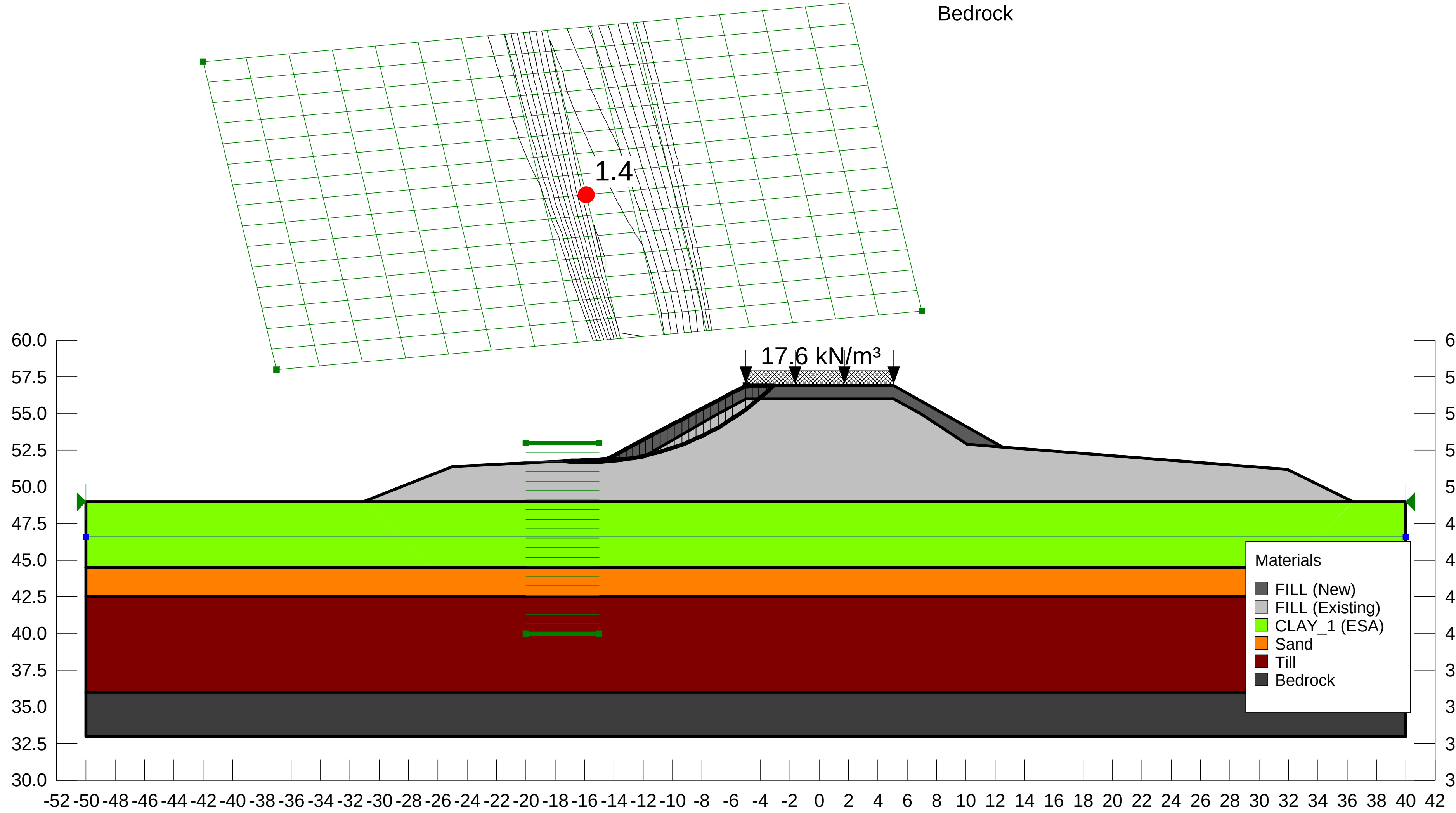
Figure 12

Reviewed By: _______________
Tool Version: 8.15.5.11777
Last Solved Date: 20/06/2016, 9:54:25 AM
Directory: C:\Users\KPOWER\Desktop\Bainsville South embankment complete.gsz

Title: Highway 401 Underpass at Bainsville Road - South Abutment
Comments: Embankment Stability Assessment
Name: 13 - Embankment Granular Grade Raise Drained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-15.866667, 69.933333) w/ Radius: 18.233333 m
FoS Contours: 1.2 to 2.2, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (ESA)	17 kN/m³	4 kPa	27 °
Sand	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			



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Tool Version: 8.15.5.11777
Last Solved Date: 20/06/2016, 9:54:36 AM
Directory: C:\Users\KPOWER\Desktop\Bainsville South embankment complete.gsz

Figure 13

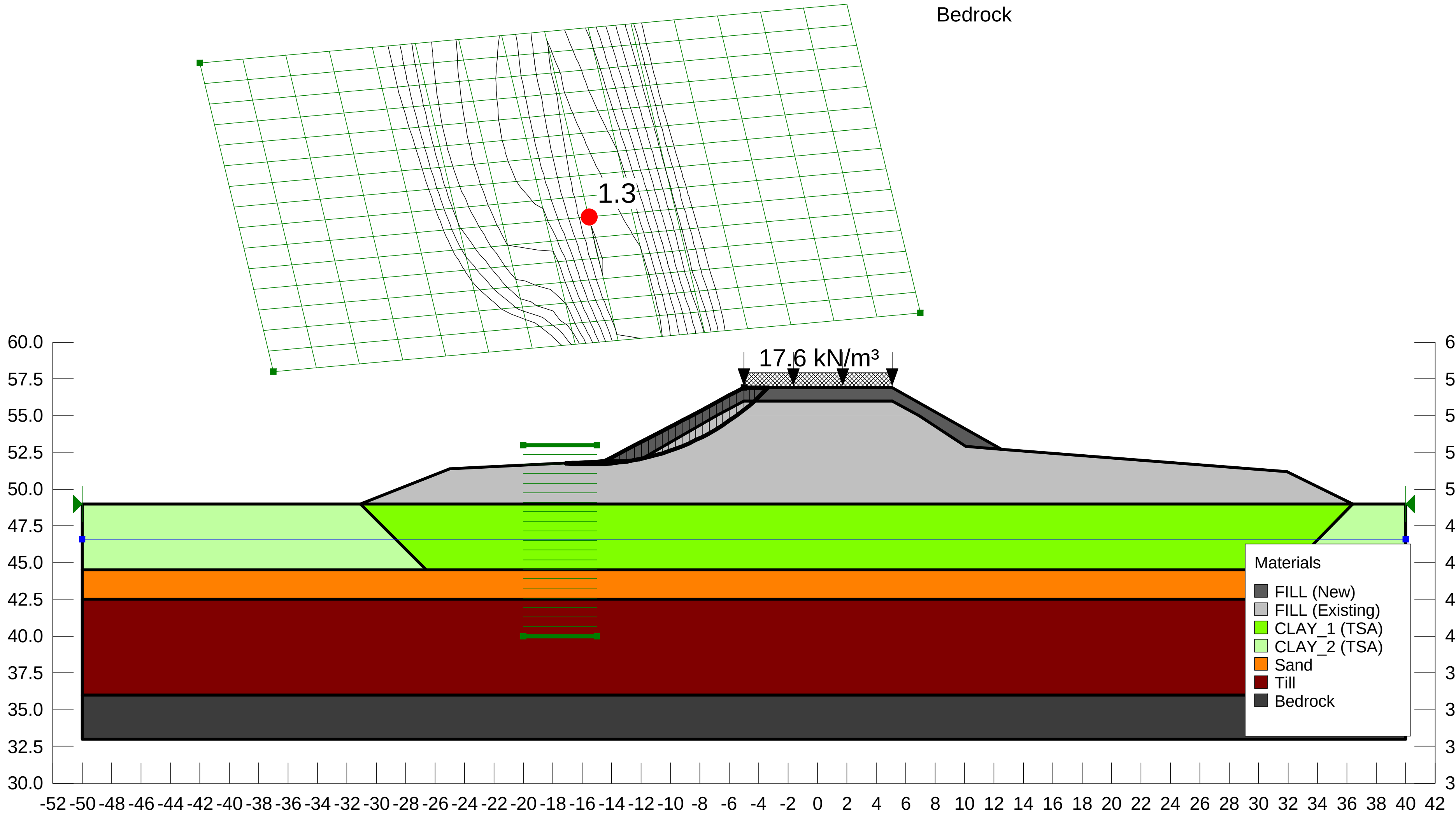
Title: Highway 401 Underpass at Bainsville Road - South Abutment

Comments: Embankment Stability Assessment

Name: 14 - Embankment Granular Grade Raise Undrained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-15.533333, 68.533333) w/ Radius: 16.833333 m
FoS Contours: 1.2 to 2.2, ++0.1

FILL (New)	20 kN/m^3	0 kPa	32 °
FILL (Existing)	20 kN/m^3	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m^3	40 kPa	0 °
CLAY_2 (TSA)	15.5 kN/m^3	20 kPa	0 °
Sand	20 kN/m^3	0 kPa	30 °
Till	22 kN/m^3	0 kPa	36 °
Bedrock			



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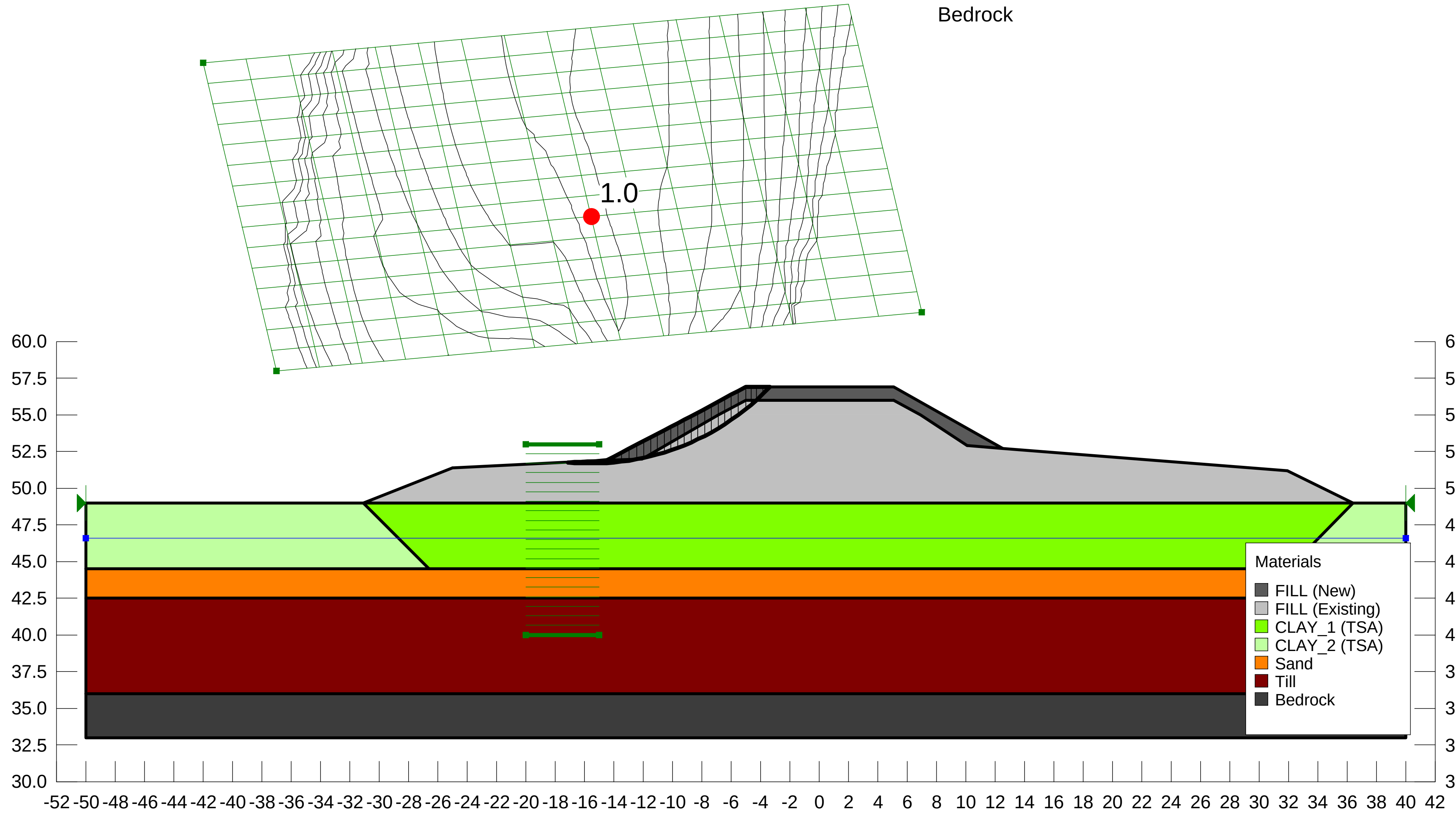
Tool Version: 8.15.5.11777
Last Solved Date: 20/06/2016, 9:54:41 AM
Directory: C:\Users\KPOWER\Desktop\Bainsville South embankment complete.gsz

Figure 14

Title: Highway 401 Underpass at Bainsville Road - South Abutment
Comments: Embankment Stability Assessment
Name: 15 - Embankment Granular Grade Raise Seismic

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0.19 V\ 0
Slip Surface Center: (-15.533333, 68.533333) w/ Radius: 16.833333 m
FoS Contours: 0.9 to 1.9, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	15.5 kN/m³	20 kPa	0 °
Sand	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			



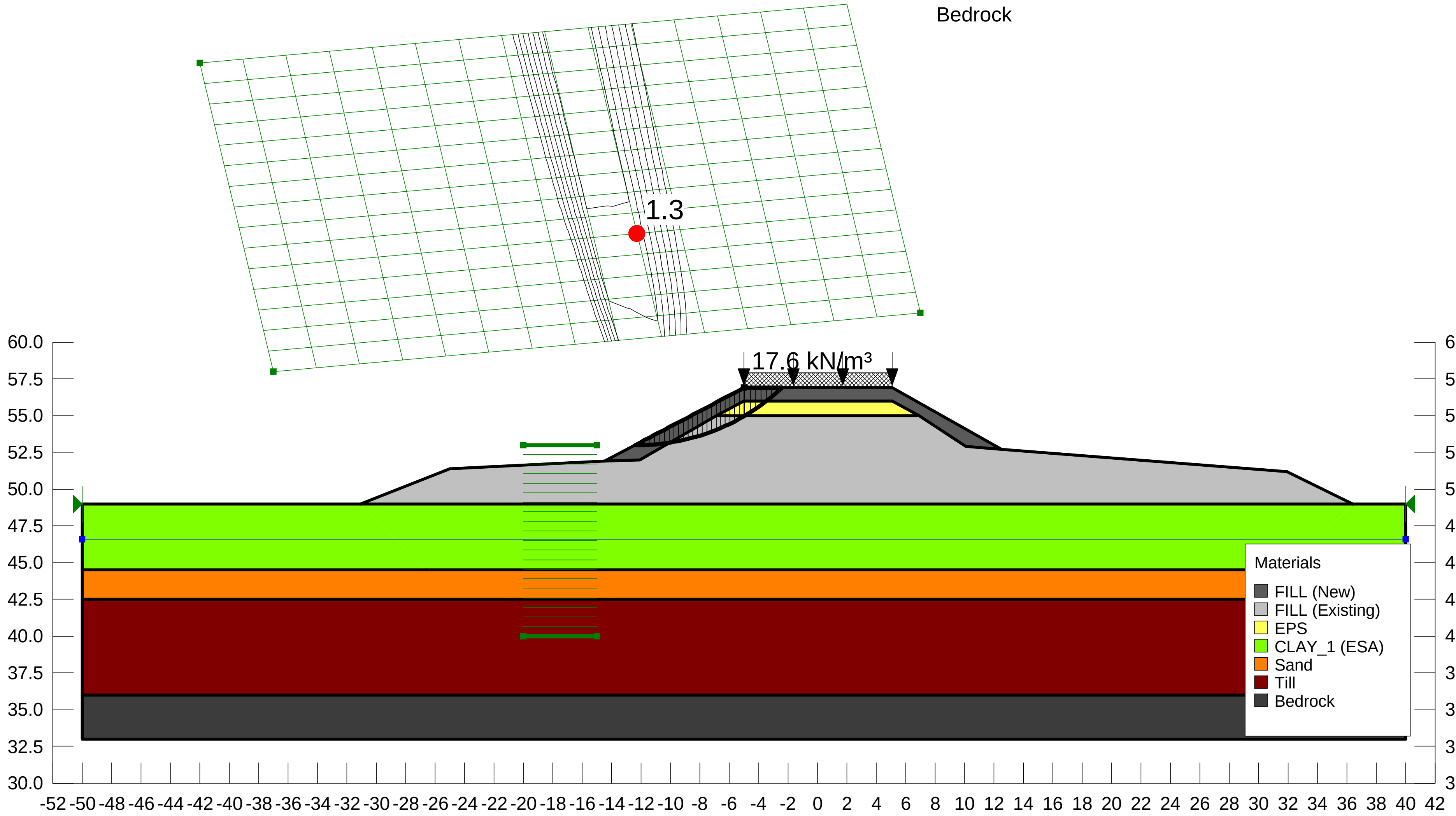
Reviewed By: _______________
Tool Version: 8.15.5.11777
Last Solved Date: 20/06/2016, 9:54:46 AM
Directory: C:\Users\KPOWER\Desktop\Bainsville South embankment complete.gsz

Figure 15

Title: Highway 401 Underpass at Bainsville Road - South Abutment
Comments: Embankment Stability Assessment
Name: 16 - Embankment EPS Grade Raise Drained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-12.266667, 67.4) w/ Radius: 14.4 m
FoS Contours: 0.9 to 1.9, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
EPS	1 kN/m³	1 kPa	0 °
CLAY_1 (ESA)	17 kN/m³	4 kPa	27 °
Sand	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			



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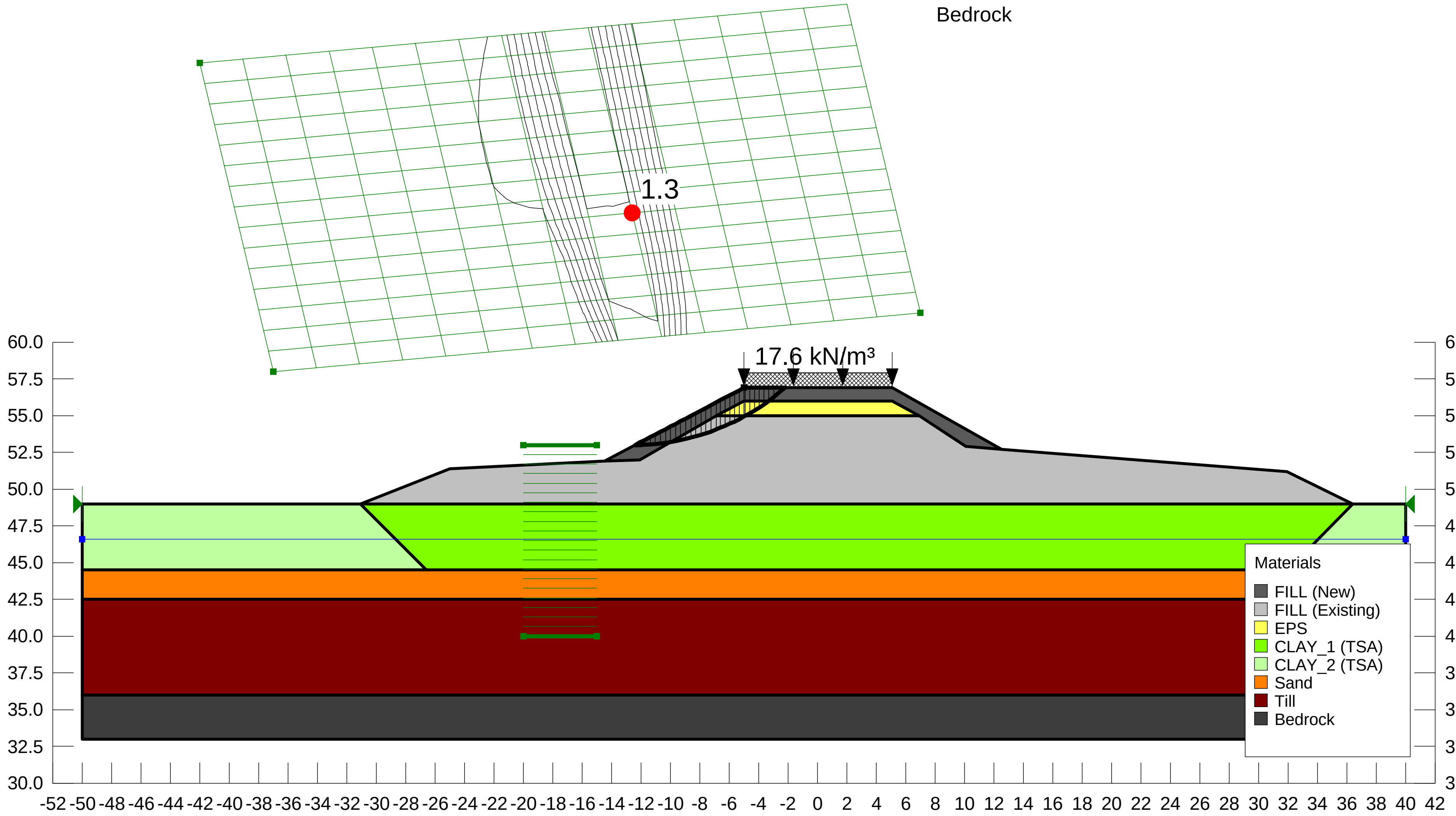
Tool Version: 8.15.5.11777
Last Solved Date: 20/06/2016, 9:54:51 AM
Directory: C:\Users\KPOWER\Desktop\Bainsville South embankment complete.gsz

Figure 16

Title: Highway 401 Underpass at Bainsville Road - South Abutment
Comments: Embankment Stability Assessment
Name: 17 - Embankment EPS Grade Raise Undrained Static

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0 V\ 0
Slip Surface Center: (-12.6, 68.8) w/ Radius: 15.8 m
FoS Contours: 0.9 to 1.9, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
EPS	1 kN/m³	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	15.5 kN/m³	20 kPa	0 °
Sand	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			



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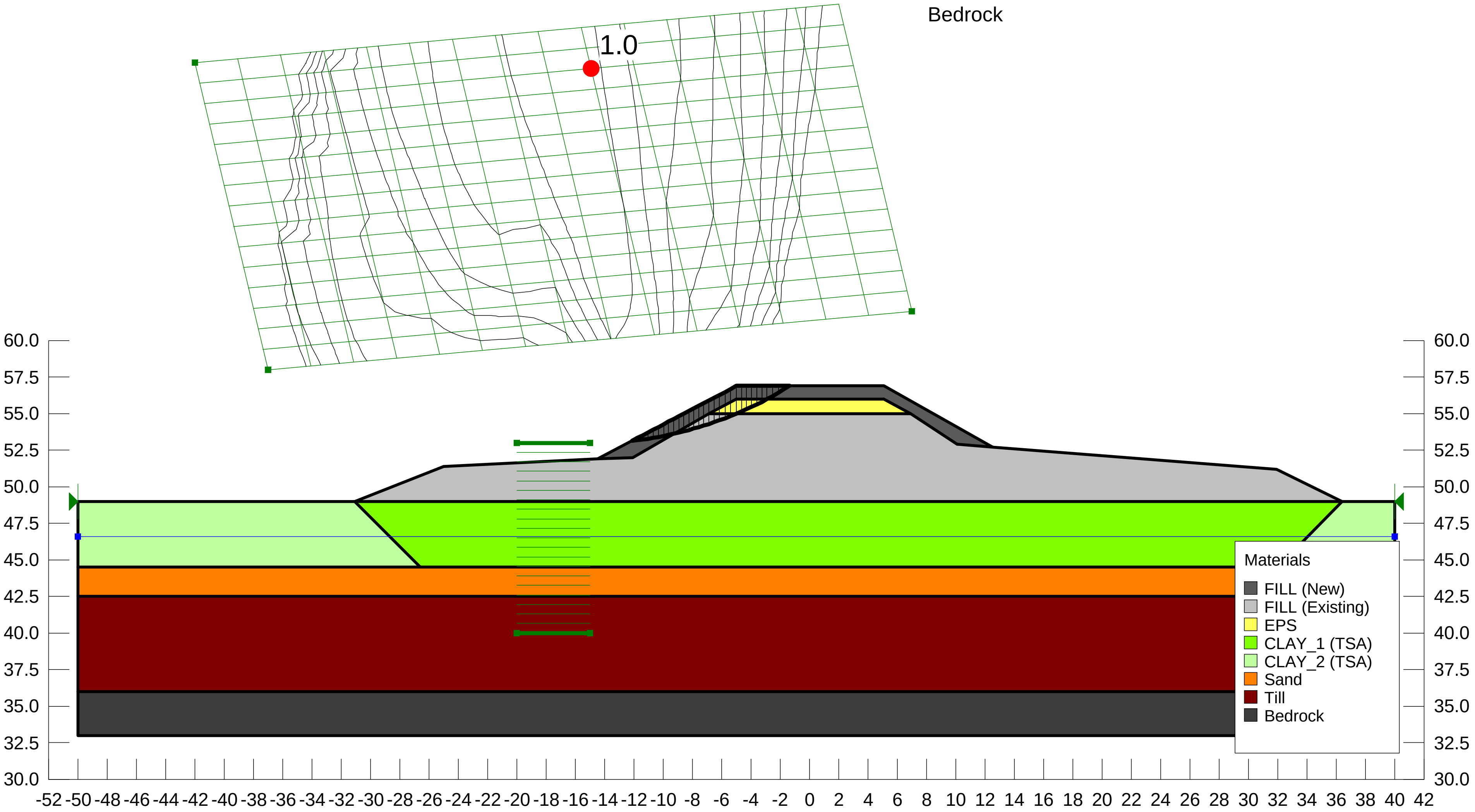
Tool Version: 8.15.5.11777
Last Solved Date: 20/06/2016, 9:54:57 AM
Directory: C:\Users\KPOWER\Desktop\Bainsville South embankment complete.gsz

Figure 17

Title: Highway 401 Underpass at Bainsville Road - South Abutment
Comments: Embankment Stability Assessment
Name: 18 - Embankment EPS Grade Raise Seismic

Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
PWP Conditions Source: Piezometric Line
Seismic: H\ 0.19 V\ 0
Slip Surface Center: (-14.933333, 78.6) w/ Radius: 25.6 m
FoS Contours: 0.8 to 1.8, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	20 kN/m³	0 kPa	32 °
EPS	1 kN/m³	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m³	40 kPa	0 °
CLAY_2 (TSA)	15.5 kN/m³	20 kPa	0 °
Sand	20 kN/m³	0 kPa	30 °
Till	22 kN/m³	0 kPa	36 °
Bedrock			



Reviewed By: _______________
Tool Version: 8.15.5.11777
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Directory: C:\Users\KPOWER\Desktop\Bainsville South embankment complete.gsz

Figure 18

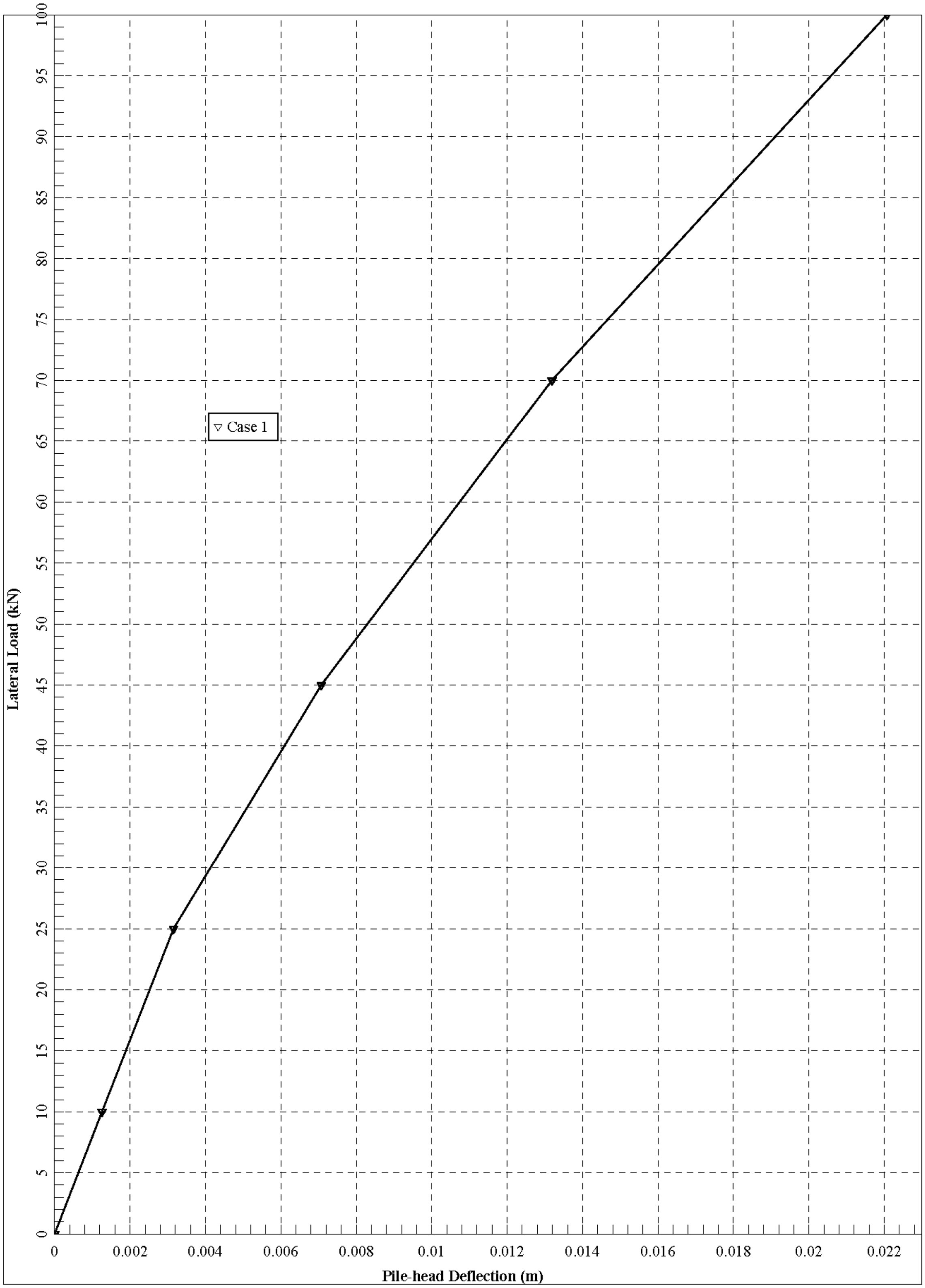

Figure 1: Lateral Load vs. Pile-head Deflection for the North Embankment

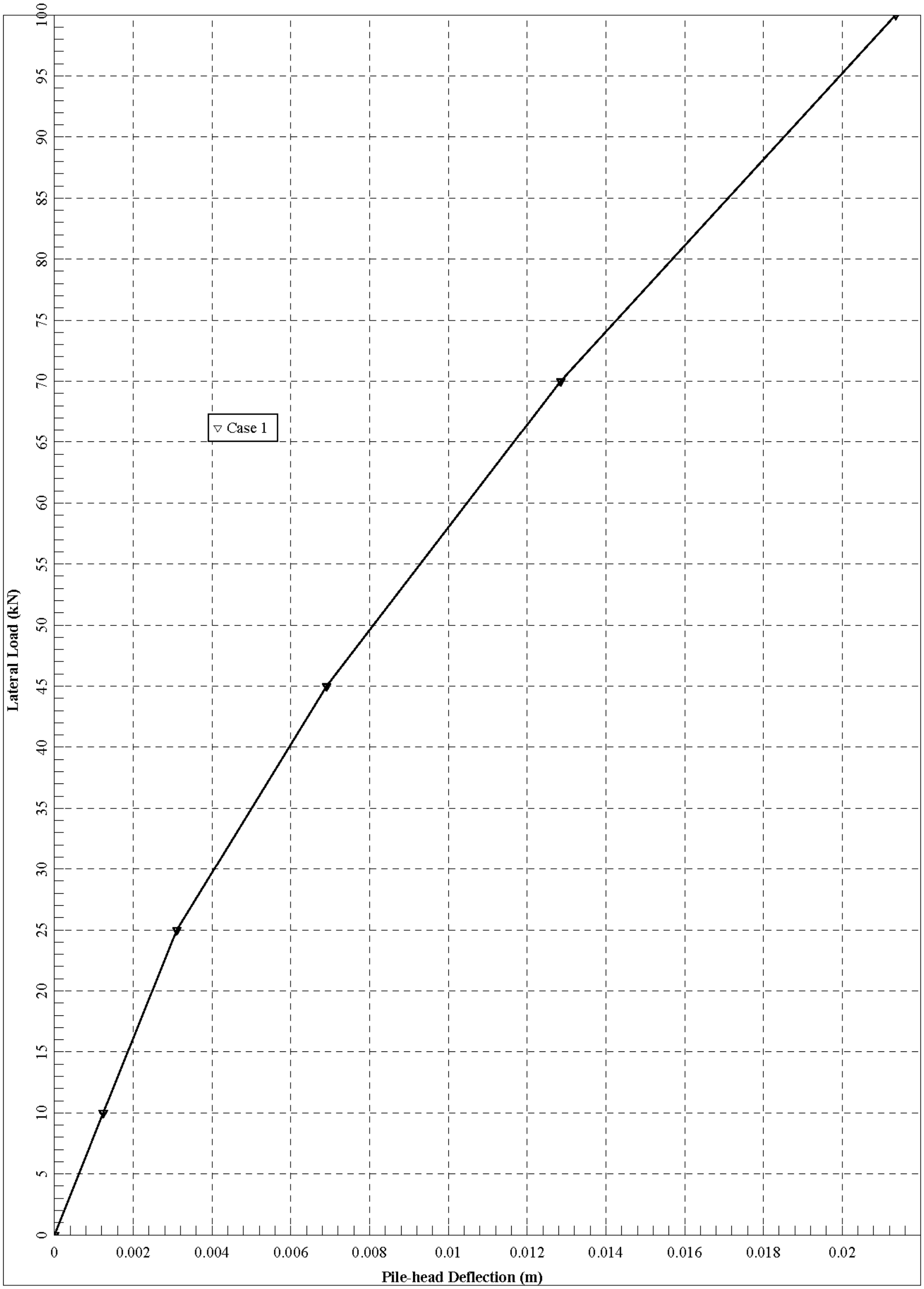

Figure 2: Lateral Load vs. Pile-head Deflection for the South Embankment

APPENDIX H

LIST OF REFERENCED SPECIFICATIONS
NON-STANDARD SPECIAL PROVISIONS - USE OF HEAVY CONSTRUCTION EQUIPMENT
NON-STANDARD SPECIAL PROVISIONS - EXPANDED POLYSTYRENE EMBANKMENT

LIST OF REFERENCED SPECIFICATIONS

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSS 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS 903	Construction Specification for Deep Foundations

RECOMMENDED WORDING FOR "NSSP- USE OF HEAVY CONSTRUCTION EQUIPMENT"

The use of heavy construction equipment and in particular heavy lift cranes may be required during removal of the existing and erection of the new bridge. The impact of the heavy equipment loads on the existing embankment, the native soft to firm soils clay underlying the embankment and the existing bridge foundations must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology, and determine requirements and/or restrictions necessary to safely support the loads. All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) - High Complexity.

The assessment shall include, but not be limited to, the following:

- Determining appropriate setbacks for heavy equipment from the bridge abutments and existing foundations;
- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment; and
- Providing recommendations for crane pad design to distribute the crane loads without causing foundation failure.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.

EXPANDED POLYSTYRENE EMBANKMENT - Item No. **

Special Provision:

1. 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. REFERENCES

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

National Standards of Canada:

- CAN/CGSB - 51.20 M87

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

OPSS - Ontario Provincial Standard Specification:

OPSS 212	Borrow
OPSS 501	Compaction
OPSS 517	Dewatering
OPSS 1010	Aggregates – Granular A,B,M, and Selected Subgrade Material
OPSS 1605	Expanded Extruded Polystyrene Pavement Insulation
OPSS 1860	Geotextiles

3. SUBSURFACE CONDITIONS

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

4. DEFINITIONS

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

Rigid Expanded Polystyrene

Molded 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.

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

Means an Engineer with a minimum of five (5) years of 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. 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. 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.

1. The method of foundation excavation and preparation.
2. Construction of levelling pad.
3. 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.
4. The method and limits of placement of polyethylene sheeting.
5. The method of placement of 125 mm reinforced concrete base pad (or equivalent).
6. The method of placement of subbase material.
7. The method of placement of side slope cover.

7. MATERIALS

7.1 Granular Levelling Pad

The levelling pad shall consist of a Granular “A” or Granular “B” material with gradation and physical requirements as specified in OPSS 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:

   - Geometry
   - Nominal Density
   - Compressive Strength
   - Flexural Strength
   - Thermal Resistance
   - Dimensional Stability
   - Flammability
   - 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 Block Production Identification

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 7-1 and as described below.

Table 7-14-1: EPS Properties Requirements

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry	mm		NA
- Linear		1200 x 600 x 300 with tolerances ± 1%	
- Flatness		10 mm in 3 m	
- Squareness		± 0.5%	
- Thickness		-3 to +5 mm	
Compressive Strength	kPa (min) at 5% Deformation	110 (EPS Type 22) 170 (EPS Type 29)	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240 (EPS Type 22) 340 (EPS Type 29)	ASTM C203 (Procedure B)
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m2.oC/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4 (EPS Type 22) 2 (EPS Type 29)	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 ± 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 for EPS Type 22 and 170 kPa for EPS Type 29 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 for EPS Type 22 and 340 kPa for EPS Type 29. The flexural strength shall be determined in accordance to ASTM C203, Method 1, Procedure B.

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.oC/W for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25mm} = \frac{R_{measured}}{\text{thickness (mm)}} \quad x25$$

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 then 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% for EPS Type 22 and 2% for EPS Type 29 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. 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. 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 501 to within ±30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place 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 90º 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 10 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.
12. The side slope of the rigid expanded polystyrene embankment shall be covered with Select Subgrade Material (SSM) as detailed elsewhere in this contract.
13. 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.
14. 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.***

10. EQUIPMENT

All cutting of polystyrene materials shall be 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. QUALITY ASSURANCE

11.1 Sampling and Testing

11.1.1 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. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

11.1.2 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.

11.1.3 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. MEASUREMENT FOR PAYMENT

12.1 Actual Measurement

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

13. PAYMENT

13.1 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.

14. SHEETING

14.1 Scope of Work

As part of the work of the above noted tender item the Contractor shall supply and install Polyethylene Sheeting as detailed elsewhere in the contract.

14.2 Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour, equipment and materials to install the Polyethylene Sheeting as detailed elsewhere in the contract and no extra payment w