



**THURBER** ENGINEERING LTD.

**DRAFT  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
REPLACEMENT OF HIGHWAY 401 UNDERPASS AT WESTLEY ROAD  
TOWNSHIP OF LANCASTER  
SITE 31-233, G.W.P. 4066-13-00  
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 Westley 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 31-233 is located on Highway 401, approximately 33 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 113 m long, and 10.4 m wide that carries two lanes of Westley 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 be oriented east-west and Westley 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.

Westley 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 to the McIntosh and Ferguson Drains.

Site photographs showing the structure and approach embankments are presented in Appendix D.

The approach embankments are up to approximately 8.8 m high with slopes that extend down at approximately 2H:1V (Horizontal:Vertical). The embankment slopes are vegetated with long grasses, trees, and occasional shrubs. Cat-tails were also noted at the toe of the east embankment.

### 3 SITE INVESTIGATION

#### 3.1 Previous Investigations

A GEOCREST report is available for this site (Report 31G00-148, 1960). The investigation for the current structure alignment included three boreholes, all accompanied by dynamic cone penetration tests (DCPT) plus two additional DCPT tests.

The stratigraphy in the area of the bridge is generally described as medium stiff, silty clay to silty sand and clay, overlying a soft to medium stiff, silty clay layer, overlaying compact to very dense sand and gravel containing cobbles and boulders. The Borehole Records identified the thickness of the clay ranging from approximately 4.3 m to 6.7 m. The boreholes were terminated within the very dense sand and gravel with cobbles and boulders. Bedrock was not encountered during the investigation.

#### 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 18, 2015 and December 3<sup>rd</sup>, 2015. The locations and elevations of the boreholes are shown on Drawing No. 1 and are summarized in Table 3-1.

**Table 3-1: Borehole Location Summary**

Borehole	Location	Latitude (degrees)	Longitude (degrees)	Ground Surface Elevation (m)	Depth (m)
101	South Abutment	45.16226	-74.44391	55.0	23.3
102	South Abutment	45.16225	-74.44396	55.0	18.6
103	Existing Pier	45.16245	-74.44398	48.3	17.2
104	Existing Pier	45.16242	-74.44419	48.4	18.5
105	Existing Pier	45.16258	-74.44432	48.2	17.0
106	Existing Pier	45.16285	-74.44430	47.6	15.8
107	Existing Pier	45.16290	-74.44456	47.6	20.9
108	Existing Pier	45.16308	-74.44448	47.9	32.2
109	North Abutment	45.16327	-74.44472	53.9	14.3
110	North Abutment	45.16326	-74.44477	53.9	18.1

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 stiff 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 Borehole 107 with NQ size coring equipment and Borehole 108 with HQ size coring equipment following ASTM Standard D6032-08. Bedrock core samples were stored in core boxes for transport.

Borehole 108 was advanced to a termination depth of 32.2 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 107 to allow for measurement of the groundwater level at the site. Piezometer construction details are illustrated on the Record of Borehole sheet for Borehole 107, provided in Appendix B.

The boreholes without piezometer 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 27, 2015. The vertical datum used was temporary benchmark (TBM) provided by MMM located on Pier 4, near Borehole 105. The TBM had a geodetic elevation of 55.707 m. The location of the TBM is indicated on the Borehole Location and Soil Strata Drawing.

### **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.

In general, the stratigraphy in the area of the boreholes through the embankments 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 a weathered clay crust, over soft to stiff clay, over silty sand with gravel, overlying glacial till, and underlain by limestone bedrock.

More detailed descriptions of the individual strata are presented below.

#### **4.2 Topsoil**

A topsoil layer with a thickness ranging from 100 mm to 300 mm was encountered in Boreholes 103 to 108.

#### **4.3 Granular Fill**

Boreholes 101, 102, 109 and 110 were advanced through Westley Road. An asphaltic surface layer with a thickness of 50 mm and 150 mm was encountered in the boreholes advanced at the north and south abutments respectively. 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 asphaltic surface in the embankment boreholes. The top of this layer ranges from Elevation 54.9 m to Elevation 53.8 m and has a thickness ranging from 300 mm to 900 mm. The SPT 'N' values ranged from 32 to 72 blows per 0.3 m of penetration; indicating a dense to very dense condition.

The moisture content of the samples tested ranged from 3% to 5%. 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 Base Material Fill**

<b>Soil Particles</b>	<b>%</b>
Gravel	25 and 39
Sand	56 and 72
Silt and Clay	3 and 7

#### **4.4 Embankment Fill**

A granular fill layer consisting predominantly of sand and gravel with varying amounts of silt and clay was encountered beneath the granular fill. Occasional cobbles were noted in this layer. The top of this layer ranges from Elevation 54.3 m to Elevation 53.5 m and has a thickness ranging from 6.5 m and 10 m. The SPT 'N' values ranged from 5 to 61 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 3% to 13%. The results of grain size analysis conducted on samples of the embankment fill material are summarized in Table 4-2 and are illustrated on Figures 2 to 4 in Appendix C.



**Table 4-2: Gradation Results for Embankment Fill**

Soil Particles	%
Gravel	2 to 38
Sand	23 to 91
Silt and Clay	9 to 35

#### 4.5 Fill

A granular fill layer consisting predominantly of sand with varying amounts of silt and gravel was encountered beneath the topsoil layer in Boreholes 103 to 108. Occasional cobbles and boulders were noted in this layer. The top of this layer ranges from Elevation 48.2 m to Elevation 47.5 m and has a thickness ranging from 0.7 m to 1.8 m. The SPT 'N' values ranged from 1 to greater than 100 blows per 0.3 m of penetration; indicating a very loose to very dense condition; but typically compact.

The moisture content of the samples tested ranged from 6% to 31%. The results of grain size analysis conducted on samples of this fill material are summarized in Table 4-3 and are illustrated on Figure 5 in Appendix C.

**Table 4-3: Gradation Results for Fill**

Soil Particles	%
Gravel	17 to 37
Sand	42 to 45
Silt and Clay	21 to 38

#### 4.6 Silt (MI)

A thin stratum of silt with sand and varying amounts of gravel and clay was encountered beneath the fill layers in Boreholes 103, 104 and 108. The top of this layer ranges from Elevation 47.1 m to Elevation 46.4 m and has a thickness ranging from 0.3 m to 0.8 m. The SPT 'N' values ranged from 6 to 11 blows per 0.3 m of penetration; indicating a loose to compact condition.

The moisture content of the samples tested ranged from 21% to 39%. The results of a grain size analysis testing conducted on samples of this material are summarized in Table 4-4 and are illustrated on Figure 6 in Appendix C.

**Table 4-4: Gradation Results for Silt**

Soil Particles	%
Gravel	1 to 17
Sand	10 to 17
Silt	67 to 58
Clay	22 to 8

Based on the results of Atterberg Limits testing the material is a silt with clay of medium plasticity. The results of the Atterberg Limits testing are illustrated on Figure 9 in Appendix C.

#### 4.7 Clay (CL, CH) Weathered Crust

A weathered, mottled brown-grey clay crust deposit with silt and trace amounts of sand and gravel was encountered in all boreholes except Borehole 101 and 102. The clay crust was encountered beneath the silt material in Boreholes 103, 104 and 108 and beneath the fill materials in the remaining boreholes.

The top of this layer ranges from Elevation 47.0 m to Elevation 46.0 m and has a thickness ranging from 1.0 m to 2.0 m. In-situ shear vane test results indicated undrained shear strengths ranging from 54 kPa to greater than 106 kPa; indicating a stiff to very stiff consistency. The moisture content of the samples tested ranged from 27% to 69%. The results of a grain size analysis testing conducted on samples of this material are summarized in Table 4-5 and are illustrated on Figure 7 and 8 in Appendix C.

**Table 4-5: Gradation Results for Clay Crust**

Soil Particles	%
Gravel	0 to 1
Sand	0 to 19
Silt	34 to 56
Clay	24 to 66

The results of Atterberg Limits testing completed on samples of this material are summarized in Table 4-6 and are illustrated on Figures 9 to 12 in Appendix C.

**Table 4-6: Atterberg Limits Test Results**

Liquid Limit	33 to 87
Plastic Limit	17 to 30
Plasticity Index	16 to 57

#### 4.8 Clay (CH)

A grey clay deposit with silt and trace sand was encountered beneath the weathered crust in all boreholes except Boreholes 101 and 102 where it was encountered below the fill materials. The clay can generally be characterized as moderately sensitive with high plasticity.

The top of this layer ranges from Elevation 45.9 m to Elevation 44.3 m and has a thickness ranging from 2.3 m to 5.1 m. In-situ shear vane test results indicated undrained shear strengths ranging from 15 kPa to 64 kPa; indicating a soft to stiff consistency; typically soft to firm. The moisture content of the samples tested ranged from 19% to 92%. The results of a grain size analysis testing conducted on samples of this material are summarized in Table 4-7 and are illustrated on Figures 13 and 14 in Appendix C.

**Table 4-7: Gradation Results for Clay**

Soil Particles	%
Gravel	0
Sand	0 to 1
Silt	24 to 50
Clay	50 to 76

The results of Atterberg Limits testing completed on samples of this material are summarized in Table 4-8 and are illustrated on Figure 10 to 12 in Appendix C.

**Table 4-8: Atterberg Limits Test Results**

Liquid Limit	64 to 82
Plastic Limit	25 to 47
Plasticity Index	26 to 57

The results of oedometer (one-dimensional consolidation) tests carried out on an undisturbed clay sample are summarized in Table 4-9. The results of the tests indicate that the clay is slightly over-consolidated.

**Table 4-9: Consolidation Test Results**

Parameter	Value
Borehole	103
Sample	TW7
Depth / Elevation (m) (mid-sample)	4.9 / 43.4
Moisture Content, (%)	79
Unit Weight, ( $\gamma$ ) (kN/m <sup>3</sup> )	15.2
Specific Gravity ( $G_s$ )	2.771
Initial Void Ratio ( $e_o$ )	2.243
Pre-consolidation Pressure, (kPa)	108
Compression Index ( $C_c$ )	1.55
Recompression Index ( $C_r$ )	0.113

#### 4.9 Silty Sand (SM) with Gravel

A stratum of granular material consisting predominantly of sand with silt and gravel was encountered beneath the clay materials in all boreholes except Boreholes 102, 108 and 109. The top of this layer ranges from Elevation 42.9 m to Elevation 39.1 m and has a thickness from 2.2 m to 6.7 m. The SPT 'N' values ranged from 10 to 60 blows per 0.3 m of penetration; indicating a compact to very dense condition; typically compact.

The moisture contents of the samples tested were 8% and 28%. The results of grain size analysis testing conducted on samples of this material are summarized in Table 4-10 and are illustrated on Figure 15 in Appendix C.

**Table 4-10: Gradation Results for Silty Sand with Gravel**

Soil Particles	%
Gravel	24 to 42
Sand	38 to 53
Silt and Clay	5 to 30

#### 4.10 Glacial Till

A stratum of glacial till consisting predominantly of sand with silt and gravel was encountered beneath the silty sand in all boreholes except Boreholes 102, 108 and 109 where it was encountered beneath the clay materials. The top of this layer ranges from Elevation 43.7 m to Elevation 35.6 m and has a thickness where completely penetrated ranging from 7.7 m to 9.0 m. All boreholes except Boreholes 107 and 108 were terminated in this stratum. The SPT 'N' values

ranged from 9 to greater than 100 blows per 0.3 m of penetration; indicating a loose to very dense condition; but typically dense to very dense. Occasional to frequent cobbles and boulders were noted in this material.

The moisture contents of the samples tested were 5% and 15%. The results of a grain size analysis testing conducted on samples of this material are summarized in Table 4-11 and are illustrated on Figures 16 and 17 in Appendix C.

**Table 4-11: Gradation Results for Glacial Till**

Soil Particles	%
Gravel	13 to 52
Sand	38 to 70
Silt and Clay	8 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 stratum in both Boreholes 107 and 108; as proven by NQ and HQ coring respectively. The bedrock surface ranges from Elevation 30.8 m to Elevation 31.1 m in these two boreholes. Photographs of the bedrock core are provided in Appendix B.

The bedrock within the top 2.6 m in Borehole 108 and within the top 1.5 m in Borehole 107 was moderately weathered. Within the weathered layer the total core recovery (TCR) ranged from 28% to 100%, the solid core recovery (SCR) ranged from 12% to 77% and the Rock Quality Designation (RQD) ranged from 0% to 20%. Based on the RQD value the weathered bedrock is classified as very poor quality.

Below the weathered layer the TCR and SCR ranged from 59% to 100%, the RQD ranged from 66% to 100%. Based on the RQD value the bedrock is classified as fair to excellent quality, typically good to excellent.

#### **4.12 Groundwater**

The groundwater level in the piezometer installed in Borehole 107 was recorded on December 7, 2015 at a depth of 0.6 m; corresponding to Elevation 47.1 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 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.

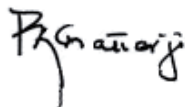
Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal 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 the replacement of the Highway 401 underpass structure at Westley 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 113 m long, and 10.4 m wide. The abutment and piers were designed to be supported on 22 inch (560 mm) diameter, Franki piles (uncased/reinforced concrete) with lengths ranging from 4.6 m to 10.7 m.

The existing bridge abutments are perched within approach embankment slopes. The highway embankments are up to approximately 8.8 m high with slopes that extend down at approximately 2H:1V (Horizontal:Vertical). The embankment slopes are vegetated with long grasses, trees, and occasional shrubs. No evidence of slope instability was noted during the site reconnaissance.

The performance of the existing structure was discussed in a technical paper prepared by MTO staff for presentation at the 20<sup>th</sup> 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:

*“More severe settlements have occurred on the south side where abutment movements were observed (Figure 23). No settlement records are available for this side. The records for the north side are shown in Figure 24”.*

A copy of Figure 24 is provided in Appendix G and indicates that 0.5 ft. (150 mm) of settlement occurred at the north approach (within the width of the roadway) during the first 750 days post construction, before slowing down.

Although no settlement data is available for the south side, the settlement was reported to be greater than the north side. This is reasonable considering that the thickness of the clay is approximately 60% greater and the height of the embankment fill is approximately 30% greater than the north side. It is noted that discussion within the historical foundation investigation and design report for this site indicates that settlement analysis for the approach embankments predicted consolidation settlement up to 2.0 ft. with a very long consolidation period and thus early construction of the embankments was recommended.

## 6.2 Proposed Structure 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 Westley Road

Based on the preliminary span configuration, Highway 401 clearance requirements and the proposed design speed for Westley Road, the vertical profile for Westley Road will be raised by an average of approximately 0.5 m across both bridge approaches, and by approximately 0.7 m and 1.2 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**

<b>Abutment</b>	<b>Existing Top of Pavement (m)</b>	<b>Approximate Grade Raise (m)</b>	<b>Proposed Top of Pavement (m)</b>
North	54.2	0.7	54.9
South	55.2	1.2	56.4

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 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 with regards to the proposed project requirements, the 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 is the Ministry of Transportation, Ontario (MTO).

It 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 ( $\Psi$ ) 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 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 ( $S_a(T)$ ) 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).



The Site Class was assessed based on the average shear wave velocity within the upper 30 m,  $V_{s30}$ , measured during the downhole seismic testing. The results of the in-situ seismic testing indicate that the  $V_{s30}$  for the project site is 336 m/s (a copy of the results is provided in Appendix E). The seismic site classification based on the measured  $V_{s30}$  corresponds to a Site Class D in accordance with Table 4.1 of the CHBDC.

## 8.2 Seismic Liquefaction

Based on the combination of the grain size distribution, and 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 0.72 m and 1.23 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).

### 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<sup>3D</sup> 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:

- $e_o = 2.24$
- $C_c = 1.55$
- $C_r = 0.113$
- $C_v = 0.103 \text{ cm}^2/\text{min} / 5.42 \text{ m}^2/\text{year}$
- $C_{vr} = 0.415 \text{ cm}^2/\text{min} / 21.8 \text{ m}^2/\text{year}$

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 settlement 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)			
		At the abutment	+ 20 m	+ 50 m	+ 75 m
North Abutment	0.72	60	40	5	NA
South Abutment	1.23	90	90	50	30

The settlement values provided in Table 9-1 are unfactored values.

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 approach fills.

## 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 0.19 equal to  $\frac{1}{2}$  of the PGA value (0.380g) was used for seismic analysis
- Existing embankment side slope geometry (2H:1V)

**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	1.3	1.4	1.0
South Abutment	1.2	1.5	0.8

The factor of safety does meet the target value of 1.3 under static conditions however does not meet the target value of 1.0 under seismic condition.

### 9.3 Evaluation of Embankment Design Options

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

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 existing embankment fill materials. 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 addresses both settlement and stability concerns, and does not result in significant time delays to the project. It is noted that since the grade raise is generally less than 1.3 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 500 mm within the zone 20 m to 50 m from the bridge abutments
- is greater than 1 m beyond 50 m from the bridge abutments

The thickness of the EPS fill should be the greater of the height of the proposed grade raise, or the thickness of EPS required to ensure global stability of the embankments as summarized Table 9-3.

**Table 9-3: EPS Thickness to Ensure Global Stability**

Location	Distance from Abutment (m)	Minimum EPS Thickness (m)
South Approach Embankment	0 to 30	3.0
	30 to 60	2.0
	60 to 75	1.0
	75 to 77.5	0.5
	> 77.5	0.0
North Approach Embankment	EPS not required for global stability	

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
- Where the thickness of the EPS is greater than 1.0 m, the width of the upper 1.0 m of EPS should be the greater of 10 m or the width of the roadway platform including shoulders and curbs. Below this level, the width of the EPS should be widened in 0.5 m thick steps to match the 2H:1V embankment side slopes

Based on the above criteria and the profile tie-in for the Long 3 Span Arrangement provided by MMM, the preliminary minimum EPS thicknesses vs. distance from the abutments are summarized in Table 9.4.

**Table 9-4: Minimum EPS Thicknesses vs. distance from the Abutment**

Location	Distance from Abutment (m)	Minimum EPS Thickness (m)
South Approach Embankment	0 to 30	3.0
	30 to 32.5	2.5
	32.5 to 60	2.0
	60 to 62.5	1.5
	62.5 to 75	1.0
	75 to 77.5	0.5
	>77.5	0
North Approach Embankment	0 to 15	1.0
	15 to 20	0.5
	> 20	0

It should be noted that the final thicknesses and limits may vary based on standard EPS block geometry and the vertical profile.

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 9.5.

**Table 9-5: Global Stability Analysis Results – Grade Raise Constructed with EPS**

Location	EPS Layer Thickness (m)	Factor of Safety		
		Static Conditions		Seismic Conditions
		Undrained	Drained	
North Abutment	1.0	1.4	1.4	1.0
South Abutment	1.5	1.3	1.5	0.9
South Abutment	3.0	1.5	1.5	1.0

A minimum EPS layer thickness of 3.0 m is required to achieve the target factor of safety under seismic conditions.

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.

General EPS Installation Notes:

- 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               | 48.3 to 47.9 m  |
| • Existing ground surface at the abutments           | 55.0 and 53.9 m |
| • Top of glacial till deposit                        | 43.7 to 35.6 m  |
| • Top of bedrock where cored (Boreholes 107 and 108) | 30.8 to 31.1 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. Coring techniques were used to penetrate the cobbles and boulders at some locations.

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

A comparison of the technical advantages and disadvantages of alternative foundation schemes is presented in Table F-2 provided 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

Due to the anticipated length of the piles and the density and the presence of boulders within the glacial till deposit, it is recommended that the design use steel HP section piles driven to practical refusal within the glacial till. It is anticipated that either HP 310x110 or HP 360x132 piles sections could be used to support both the piers and abutment foundations.

The design parameters for axial resistance of Grade 350W HP 310x110 steel piles driven to practical refusal within the glacial till deposit can be taken as:

- Factored vertical geotechnical resistances at ULS = 1,100 kN
- Factored vertical geotechnical reaction at SLS = 800 kN

The design parameters for axial resistance of Grade 350W HP 360x132 steel piles driven to practical refusal within the glacial till deposit can be taken as:

- Factored vertical geotechnical resistances at ULS = 1,400 kN
- Factored vertical geotechnical reaction at SLS = 1,100 kN

The factored geotechnical resistances provided include the following factors:

- Consequence factor ( $\Psi$ ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2):
  - $\phi_{gu} = 0.4$ , ULS (static analysis; typical degree of understanding)
  - $\phi_{gs} = 0.8$ , SLS (static analysis; typical degree of understanding)

The estimated pile tip elevations based on piles reaching practical refusal are summarized in Table 10-1.

**Table 10-1: Estimated Pile tip Elevations**

Foundation Element	Underside of Pile Cap Elevation (m)	Estimated Pile Tip Elevation (m)
North Abutment	49.0	38.0
Pier 1	45.9	30.8
Pier 2	46.8	32.0
South Abutment	50.8	35.0

The geotechnical axial resistance was selected assuming that the piles meet refusal in the till layer above the bedrock. It is possible that some of the piles will fully penetrate the till layer and refuse on or in the limestone bedrock.

### 10.2.1 Pile Lateral Resistance

The lateral load response of a single HP 310 x 110 steel pile driven to practical refusal within the glacial till was assessed using an L-Pile analysis to determine lateral load versus pile head deflection and lateral load versus maximum bending moment within the pile. The results are presented in Appendix G.

The resistance to lateral deflection should include the following factors:

- Consequence factor ( $\Psi$ ) of 1.0
- Geotechnical resistance factor (CHBDC Table 6.2):
  - $\phi_{gs} = 0.8$ ; 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 Franki Piles must be checked.

Due to the presence of cobbles and boulders within the glacial till deposit 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 no 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 and stability concerns associated with the grade raise of the embankments, the backfill behind the abutments will consist of both granular fill and of EPS material. A mechanism for drainage behind the abutment should be provided by a column of granular backfill, fully supported on the abutment pile cap.

## 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 <sup>3</sup> , $\gamma$	21	20	20
Angle of Internal Friction, $\phi$	35°	33°	32°
Interface Friction Angle, Soil to EPS, $\delta$	35°	33°	32°
Coefficient of at Rest Earth Pressure, $K_o$ (Restrained Wall)	0.43	0.46	0.47

Parameter	OPSS Granular A & OPSS Granular B Type II	Existing Fill	OPSS Granular B Type I
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:

- $k_h = \frac{1}{2} F(PGA) \cdot PGA$  for structures that allow lateral yielding, and
- $k_h = F(PGA) \cdot PGA$  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 are provided in Table 11-2 assume the following:

- Horizontal back-slope behind the wall
- Seismic Site Class of D, and a PGA with a 2% probability of exceedance in 50 years of 0.380g; 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/m <sup>3</sup> , $\gamma$	21	20	20
Angle of Internal Friction, $\phi$	35°	33°	32°
Interface Friction Angle, Soil to EPS, $\delta$	35°	33°	32°
<b>Yielding Wall</b>			
Dynamic Active Earth Pressure Coefficient, $K_{AE}$	0.38	0.41	0.42
<b>Non-Yielding Wall</b>			
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:

- $\sigma_h$  = lateral earth pressure at depth, d (kPa)
- d = depth below the top of the wall (m)
- $K_a$  = static active earth pressure coefficient
- $\gamma$  = unit weight of the backfill soil (kN/m<sup>3</sup>)
- $K_{AE}$  = 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:

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

where:

- $K'_{EPS}$  = horizontal coefficient of subgrade reaction (kN/m<sup>3</sup>)
- $E_{EPS}$  = Young's Modulus of EPS Blocks (kN/m<sup>2</sup>)
- $v_{EPS}$  = Poisson's Ratio of EPS Blocks ( $v_{EPS} = 0.10$ )
- 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. In addition a portion of the samples were submitted to TESTMARK Laboratories Ltd. in Garson Ontario for analysis of RedOx potential. 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)	Conductivity (µS/cm)	RedOx Potential (mV)
105	SS5	4.1	240	7.6	5.5	932	1830	431
106	SS3	1.8	172	7.6	4.7	1340	2150	421
107	SS3	1.8	75	7.5	9.7	491	1030	396

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 remaining properties in Table 12-1 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.

Dewatering and surface water diversion must remain operational and effective until the temporary excavation is backfilled.

The design of any dewatering system that may be required must be the responsibility of the Contractor. 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 potential for encountering debris from construction of the existing bridge during excavation or pile driving
  - 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

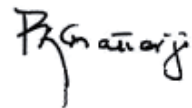
Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal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



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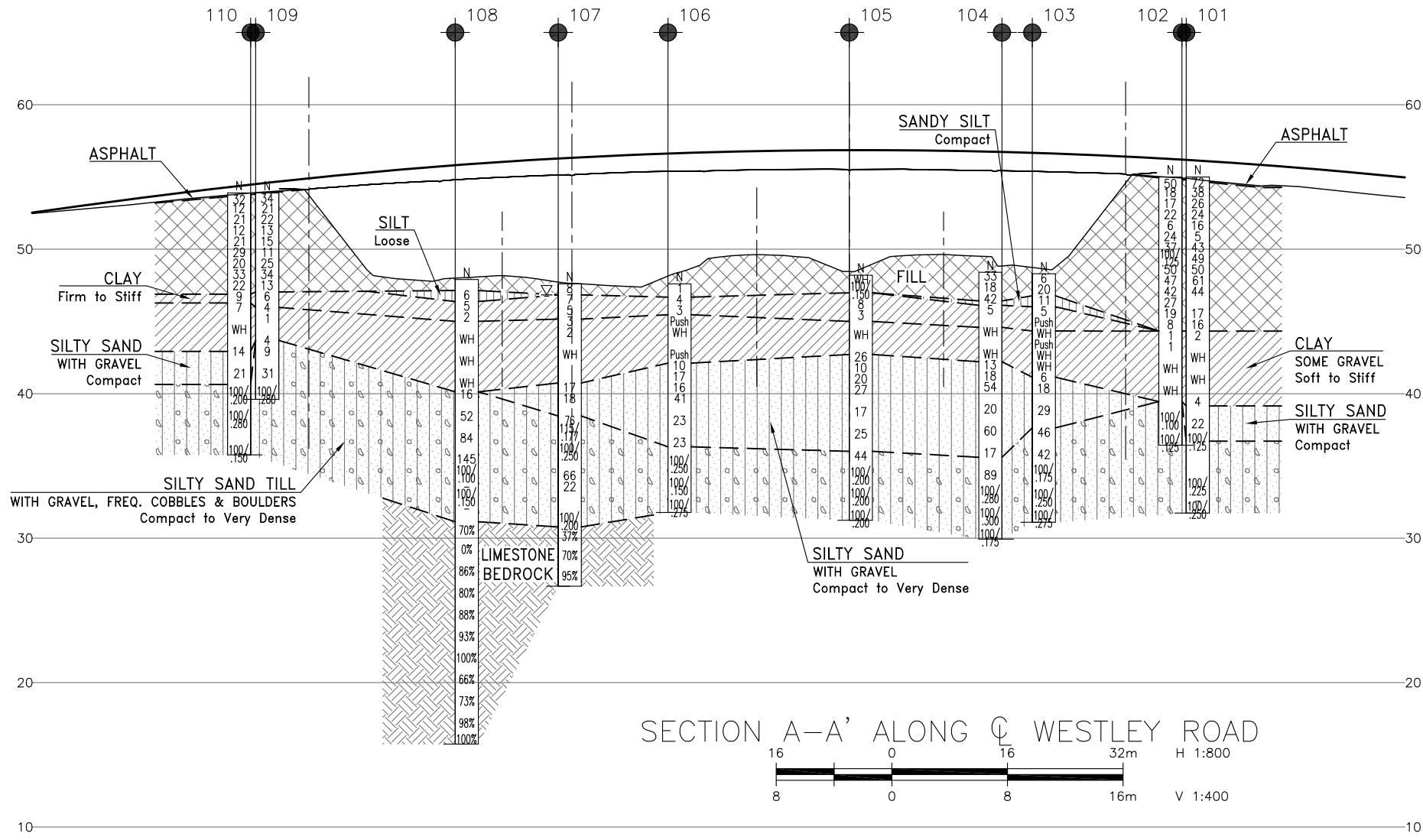
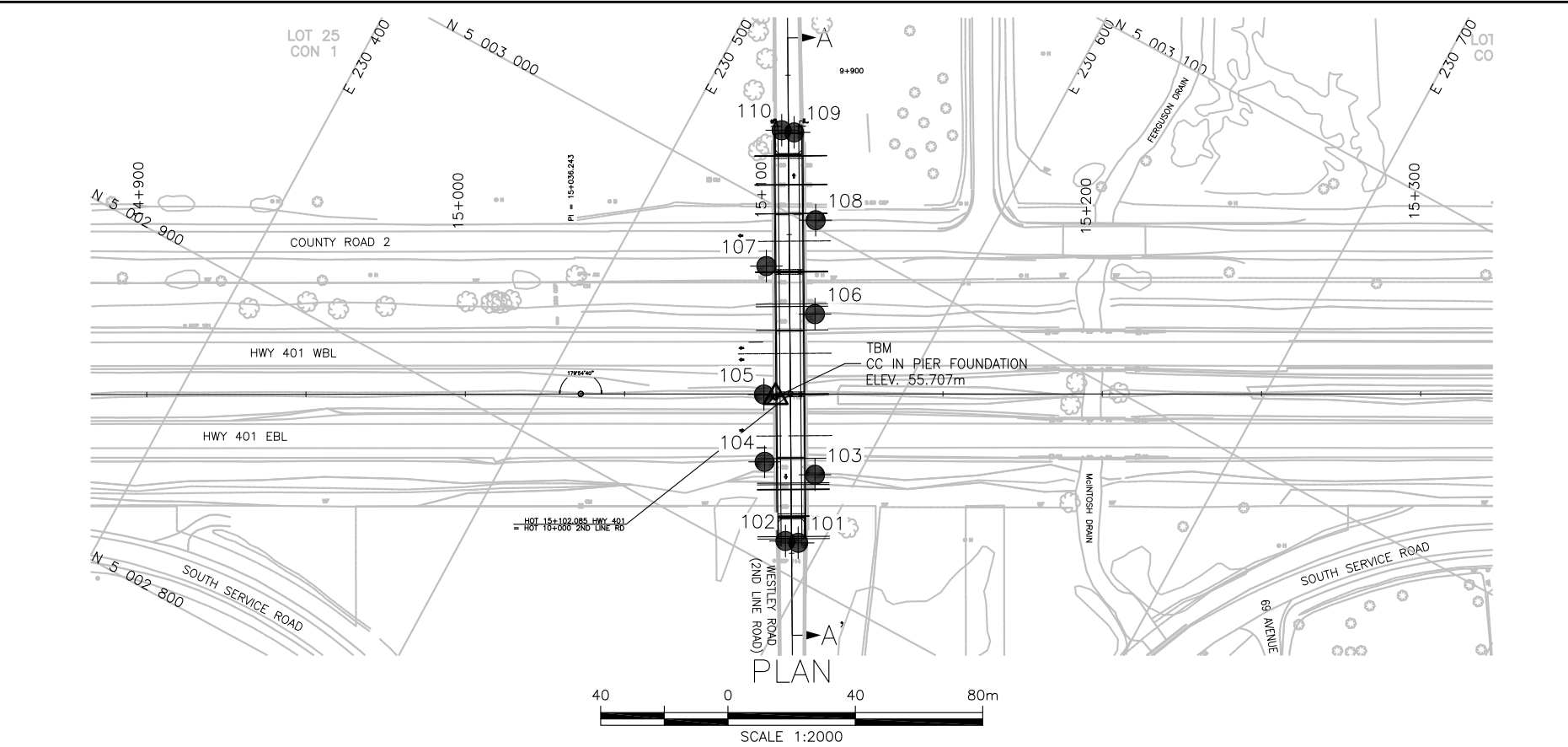
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**APPENDIX A**  
**BOREHOLE LOCATIONS AND SOIL STRATA DRAWINGS**

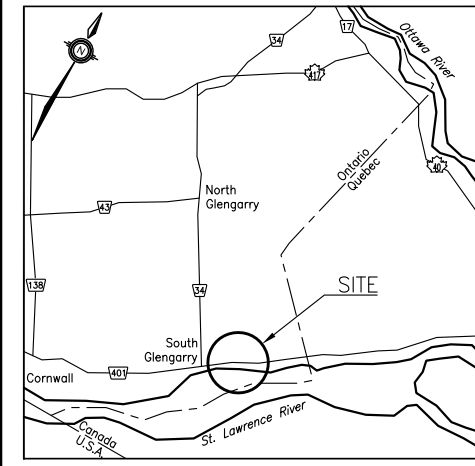
DRAFT



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

CONT No  
GWP No 4066-13-00

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



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
101	55.0	5 002 910.5	230 593.4
102	55.0	5 002 909.1	230 589.6
103	48.3	5 002 931.7	230 587.9
104	48.4	5 002 927.8	230 572.0
105	48.2	5 002 946.3	230 561.8
106	47.6	5 002 976.1	230 563.8
107	47.6	5 002 982.0	230 543.2
108	47.9	5 003 002.1	230 550.0
109	53.9	5 003 023.1	230 530.9
110	53.9	5 003 021.8	230 527.2

NOTES

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Borehole locations are shown in MTM Zone 8 coordinates.

GEOCRES No.

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KP	CHK -	CODE
DRAWN	MFA	CHK KP	SITE 31-233
			LOAD
			DATE
			MAR 2016
			DWG 1



## **APPENDIX B**

### **RECORD OF BOREHOLE SHEETS BEDROCK CORE PHOTOGRAPHS SHEAR WAVE VELOCITY PROFILE**

DRAFT

# RECORD OF BOREHOLE No 101

1 OF 3

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 910.5 E 230 593.4 ORIGINATED BY SMP  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY SMP  
 DATUM Geodetic DATE 2015.11.20 - 2015.11.21 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W P                      W                      W L				GR	SA	SI	CL	
								○ UNCONFINED                      + FIELD VANE					WATER CONTENT (%)								
								● QUICK TRIAXIAL                      × LAB VANE													
55.0							20	40	60	80	100										
0.0																					
0.1																					

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 101

2 OF 3

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 910.5 E 230 593.4 ORIGINATED BY SMP  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY SMP  
 DATUM Geodetic DATE 2015.11.20 - 2015.11.21 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		<div><div>PLASTIC LIMIT</div><div>NATURAL MOISTURE CONTENT</div><div>LIQUID LIMIT</div></div> <div><div>W<sub>P</sub></div><div>W</div><div>W<sub>L</sub></div></div> <div>WATER CONTENT (%)</div>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
	Continued From Previous Page							<div>○ UNCONFINED    + FIELD VANE</div> <div>● QUICK TRIAXIAL    × LAB VANE</div>				
								<div>20 40 60 80 100</div> <div>20 40 60 80 100</div>				
44.3	Silty sand with gravel Loose to very dense Grey FILL		13	SS	16							
10.7	CLAY (CH) Firm to stiff Grey		14	SS	2		44					0 1 35 64
			17	SS	WH		43	7.5 4.2				
							42	5.2 4.2				
			20	SS	WH		41					
							40	4.0 4.4				
	- some gravel below 14.8 m		23	SS	4							
39.1							39					
15.8	Silty SAND (SM) with gravel Compact Grey		24	SS	22		38					24 50 22 4
							37					
36.7												
18.3	Silty SAND (SM) with gravel TILL - frequent cobbles and boulders Very dense Grey		25	SS	100/ 125mm		36					23 42 25 10
	- boulders from 19.5 m to 22 m											

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

## METRIC

DATUM	Geodetic	DATE	2015.11.20 - 2015.11.21	CHECKED BY	KCP
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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No 102

1 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 909.1 E 230 589.6 ORIGINATED BY SMP/JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.11.21 - 2015.11.23 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)						
								○ UNCONFINED      + FIELD VANE											
								● QUICK TRIAXIAL    × LAB VANE											
55.0							20	40	60	80	100								
0.0	50 mm ASPHALT						20	40	60	80	100								
54.3	Poorly graded sand with gravel trace silt, trace asphalt Very dense Grey FILL		1	SS	50												39 58 3 (SI+CL)		
0.7	Poorly graded sand with silt, trace gravel Compact Brown FILL		2	SS	18														
			3	SS	17														
			4	SS	22												4 87 9 (SI+CL)		
51.9																			
3.0	Silty, clayey sand with gravel Loose to dense Brown FILL		5	SS	6														
			6	SS	24														
			7	SS	37														
			8	SS	100/ 125mm												28 38 34 (SI+CL)		
			9	SS	50														
			10	SS	47														
	- grey below 7.1 m		11	SS	42														
			12	SS	27														
			13	SS	19												33 38 23 6		

Continued Next Page

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

# RECORD OF BOREHOLE No 102

2 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 909.1 E 230 589.6 ORIGINATED BY SMP/JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.11.21 - 2015.11.23 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)								
								20   40   60   80   100			w <sub>P</sub> w      w <sub>L</sub>								
Continued From Previous Page								○ UNCONFINED      + FIELD VANE											
								● QUICK TRIAXIAL      × LAB VANE											
								20   40   60   80   100			20   40   60								
44.3			14	SS	8														
10.7	CLAY (CH) Firm to stiff Grey		15	SS	1		44												
			16	SS	1		43												
							42												
				19	SS	WH		41											
							40												
				22	SS	WH		39											
39.4																			
15.5	Silty SAND (SM) some gravel TILL - occasional cobbles and boulders Very dense Grey																		
				25	SS	100/ 100mm		38											
								37											
36.4			26	SS	100/ 125mm														
18.6	End of borehole																		




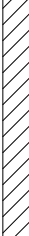
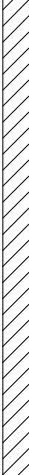

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 103

1 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 931.7 E 230 587.9 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY CAM  
 DATUM Geodetic DATE 2015.12.03 - 2015.12.03 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)					
48.3								20	40	60	80	100					
0.0	300 mm <b>TOPSOIL</b>		1	SS	6		48										17 45 31 7
0.3	Silty sand with gravel Loose to compact Brown <b>FILL</b>		2	SS	20		47										
46.8																	
1.5	<b>Silt (MI)</b> with sand and gravel trace clay Compact Brown to Grey		3	SS	11		46										17 17 58 8
46.0																	
2.3	<b>CLAY (CH)</b> - clay crust Stiff Mottled brown and grey		4	SS	5		45										0 7 43 50
			5	TW	Push												
44.3																	
4.0	<b>CLAY (CH)</b> Soft Grey		6	SS	WH		44										
			7	TW	Push		43										
			8	SS	WH		42										0 1 30 69
			9	SS	WH		41										
41.1			10	SS	6		40										
7.2	<b>Silty SAND (SM)</b> with gravel Compact Grey		11	SS	18		39										38 49 13 (SI+CL)
			12	SS	29												

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 103

2 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 931.7 E 230 587.9 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY CAM  
 DATUM Geodetic DATE 2015.12.03 - 2015.12.03 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20    40    60    80    100	W <sub>P</sub> W      W <sub>L</sub>	WATER CONTENT (%)			GR	SA		SI	CL		
								SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE											
	Continued From Previous Page							20    40    60    80    100											
37.6	Silty SAND (SM) with gravel Compact Grey						38												
10.7	Silty SAND (SM) with gravel TILL - frequent cobbles and boulders Dense to very dense Grey		13	SS	46		37						○H				19	38    33    10	
							36						○						
			14	SS	42														
							35												
	- cobbles and boulders below 13.2 m		15	SS	100/ 175mm		34						○						
							33						○				20	48    32 (SI+CL)	
							32												
31.1			16	SS	100/ 250mm								○						
17.2	End of borehole		17	SS	100/ 275mm								○						

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16




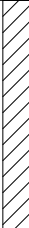
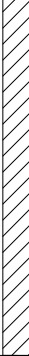



# RECORD OF BOREHOLE No 104

1 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 927.8 E 230 572.0 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.12.04 - 2015.12.07 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE							
								● QUICK TRIAXIAL	× LAB VANE							
48.4							20	40	60	80	100					
0.0	200 mm <b>TOPSOIL</b>		1	SS	33											
0.2	Silty sand with gravel - occasional cobbles Compact to dense Brown <b>FILL</b>		2	SS	18											
46.4			3	SS	42											
2.0	<b>SILT (MI)</b> with sand Compact Brown															
46.1																
2.3	<b>CLAY (CH)</b> - clay crust Stiff Mottled brown and grey		4	SS	5											
44.6																
3.8	<b>CLAY (CH)</b> Soft Grey		5	SS	WH											
			6	SS	WH											
42.2																
6.2	<b>Silty SAND (SM)</b> with gravel Compact to very dense Grey		7	SS	13											
			8	SS	18											
			9	SS	54											
			10	SS	20											

Continued Next Page

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

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 104

2 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 927.8 E 230 572.0 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.12.04 - 2015.12.07 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
					○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE																
	Continued From Previous Page							20	40	60	80	100	20	40	60						
35.6	Silty SAND (SM) with gravel Compact to very dense Grey		11	SS	60		38														
							37														
			12	SS	17		36														
12.8	Silty SAND (SM) with gravel TILL - frequent cobbles and boulders Very dense Grey		13	SS	89		35														
							34														
			14	SS	100/ 280mm		33														
							32														
			15	SS	100/ 300mm		31														
29.9			16	SS	100/ 175mm		30														
18.5	End of borehole																				

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 105

1 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 946.3 E 230 561.8 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY CAM  
 DATUM Geodetic DATE 2015.12.01 - 2015.12.01 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT		UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20    40    60    80    100	W <sub>P</sub> W                      W <sub>L</sub>	20    40    60			
48.2													
0.0	300 mm <b>TOPSOIL</b>												
47.9			1	SS	WH		48						
0.3	Sandy clay Soft Brown <b>FILL</b>		2	SS	100/								
47.0	-Boulder at 0.9 m, shifted borehole by 1 m				150mm								
1.2	<b>CLAY (CL)</b> - clay crust Stiff Mottled brown and grey		3	SS	8		47						
	- CLAY (CH) below 2.3 m		4	SS	3		46						
45.3													
2.9	<b>CLAY (CH)</b> Stiff to soft Grey						45						
			5	SS	WH		44						
42.7													
5.5	<b>Silty SAND (SM)</b> with gravel Compact Grey		6	SS	26		43						
			7	SS	10		42						
			8	SS	20		41						
			9	SS	27		40						
			10	SS	17		39						

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 105

2 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 946.3 E 230 561.8 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY CAM  
 DATUM Geodetic DATE 2015.12.01 - 2015.12.01 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W <sub>P</sub>	W		W <sub>L</sub>			
	Continued From Previous Page																		
	<b>Silty SAND (SM)</b> with gravel Compact Grey		11	SS	25														
36.0																			
12.2	<b>SAND (SM)</b> with silt and gravel <b>TILL</b> - frequent cobbles and boulders Dense to very dense Grey		12	SS	44							○					41	49	10 (Si+CL)
														</					

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 106

2 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 976.1 E 230 563.8 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY JAG  
 DATUM Geodetic DATE 2015.11.27 - 2015.12.08 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)								
								20   40   60   80   100					w <sub>p</sub> w                      w <sub>L</sub>								
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE													
	Continued From Previous Page																				
36.3	Silty SAND (SM) with gravel Compact to dense Grey						37												42   53   5 (SI+CL)		
			12	SS	23																
11.3	Silty SAND (SM) with gravel TILL - occasional cobbles and boulders Very dense Grey						36														
			13	SS	100/ 250mm																
							35														
							34														
			14	SS	100/ 150mm																
							33														
31.7	End of borehole						32											14   49   37 (SI+CL)			
15.8																					

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 107

1 OF 3

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 982.0 E 230 543.2 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.11.19 - 2015.11.20 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20    40    60    80    100	W <sub>P</sub> W                      W <sub>L</sub>	20    40    60	GR   SA   SI   CL			
47.6														
0.0	300 mm <b>TOPSOIL</b>													
0.1	Sandy clay soft		1	SS	8									
46.8	Brown <b>FILL</b>													
0.8	<b>CLAY (CH)</b> - clay crust Stiff Mottled brown and grey		2	SS	7									
	- vane shear strength exceeds 106 kPa		3	SS	5									
45.1														
2.4	<b>CLAY (CH)</b> Soft to firm Grey		4	SS	3									
			5	SS	2									
			6	SS	WH									
40.7														
6.9	<b>Silty SAND (SM)</b> with gravel Compact Grey		7	SS	17									
			8	SS	18									
38.4														
9.1	<b>Silty SAND (SM)</b> with gravel <b>TILL</b> - frequent cobbles and boulders Very dense Grey		10	SS	76									

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity



# RECORD OF BOREHOLE No 107

3 OF 3

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 002 982.0 E 230 543.2 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.11.19 - 2015.11.20 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
	Limestone <b>BEDROCK</b>		3	HQ												RUN #3 TCR=100% SCR=100% RQD=95%	
26.7							27										
20.9	End of borehole Water level in well at 0.6 m on 2015/12/07																















ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 108

1 OF 4

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 003 002.1 E 230 550.0 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring / Seismic Testing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.11.23 - 2015.11.26 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		<div><div>PLASTIC LIMIT</div><div>NATURAL MOISTURE CONTENT</div><div>LIQUID LIMIT</div></div> <div><div>W<sub>P</sub></div><div>W</div><div>W<sub>L</sub></div></div> <div>WATER CONTENT (%)</div>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
							<div>○ UNCONFINED      + FIELD VANE</div> <div>● QUICK TRIAXIAL    × LAB VANE</div>							
47.9							20	40	60	80	100			
0.0	100 mm <b>TOPSOIL</b>													
0.1	Silt and Gravel Compact Grey (FILL)													
47.1														
0.8	<b>SILT (MI)</b> with sand and clay trace gravel Loose		1	SS	6									1 10 67 22
46.4														
1.5	<b>CLAY (CH)</b> - clay crust Stiff Brown Molted grey and brown		2	SS	5									
			3	SS	2									0 0 34 66
44.4								8.0 +						
3.5	<b>CLAY (CH)</b> Soft to firm Grey Wet		4	SS	WH									
			5	SS	WH									
			6	SS	WH									0 0 50 50
40.1														
7.8	<b>Silty SAND (SM)</b> with gravel <b>TILL</b> - frequent cobbles and boulders Compact to very dense Grey		7	SS	16									
			8	SS	52									

Continued Next Page

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

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 108

2 OF 4

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 003 002.1 E 230 550.0 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring / Seismic Testing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.11.23 - 2015.11.26 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE				WATER CONTENT (%) w <sub>p</sub> w                      w <sub>L</sub>						
	Continued From Previous Page							20	40	60	80	100		20	40	60		
	- cobbles and boulders from 12.2 m to 16.8 m		9	SS	84		37										52 40 8 (SI+CL)	
			10	SS	145		36											
			11	SS	100/ 100mm		35											
			12	HQ	-		34											
			13	SS	100/ 150mm		33											
			14	HQ	-		32											
31.1																		
16.8		Limestone <b>BEDROCK</b> Moderately weathered Laminated to thinly bedded Very poor to fair quality		1	HQ			31										RUN #1 TCR=100% SCR=77% RQD=70%
			2	HQ			30										RUN #2 TCR=28% SCR=12% RQD=0%	
28.4							29											
19.4	Limestone <b>BEDROCK</b> Fresh Medium to thickly bedded Good to excellent quality						28											

Continued Next Page

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

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 108

3 OF 4

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 003 002.1 E 230 550.0 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring / Seismic Testing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.11.23 - 2015.11.26 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	Continued From Previous Page													
	Limestone <b>BEDROCK</b> Fresh Medium to thickly bedded Good to excellent quality		3	HQ										RUN #3 TCR=98% SCR=90% RQD=86%
			4	HQ										RUN #4 TCR=100% SCR=97% RQD=80%
			5	HQ										RUN #5 TCR=59% SCR=59% RQD=88%
			6	HQ										RUN #6 TCR=100% SCR=98% RQD=93%
			7	HQ										RUN #7 TCR=100% SCR=100% RQD=100%
			8	HQ										RUN #8 TCR=98% SCR=97% RQD=66%
			9	HQ										RUN #9 TCR=100% SCR=99% RQD=73%

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 108

4 OF 4

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 003 002.1 E 230 550.0 ORIGINATED BY JAG  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring / Seismic Testing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.11.23 - 2015.11.26 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	Continued From Previous Page							20 40 60 80 100						
								PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT						
								W P W W L						
								WATER CONTENT (%)						
								20 40 60						

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

## METRIC

SOIL PROFILE			SAMPLES		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES
53.9	150 mm ASPHALT				
53.5	Sand with silt and gravel Dense Grey FILL		1	SS	34
53	Sand trace silt Compact Brown FILL		2	SS	21
52			3	SS	22
51.3	Silty sand with gravel Compact Brown FILL		4	SS	13
51			5	SS	15
50			6	SS	11
49.7	Sand trace silt Compact Brown FILL		7	SS	25
48.2	Silty sand with gravel Compact Grey FILL		8	SS	34
48			9	SS	13
47.0	CLAY (CI) - clay crust Stiff Brown Molted grey and brown		10	SS	6
45.9	CLAY (CH) Firm to stiff Grey		11	SS	4
45			12	SS	1
44					

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No 109

2 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 003 023.1 E 230 530.9 ORIGINATED BY SMP  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / NW Casing COMPILED BY KCP  
 DATUM Geodetic DATE 2015.11.19 - 2015.11.19 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
	Continued From Previous Page													
43.7														
10.2	<b>Silty SAND (SM)</b> with gravel <b>TILL</b> - occasional cobbles and boulders Compact to very dense Grey		15	SS	4									
			16	SS	9		43							
							42							
			17	SS	31									
	- Cobbles and boulders 13.1 m to 13.7 m						41							
			18	SS	100/ 280mm		40							
39.6														
14.3	End of borehole													

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

# RECORD OF BOREHOLE No 110

1 OF 2

METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 003 021.8 E 230 527.2 ORIGINATED BY SMP  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring COMPILED BY JAG  
 DATUM Geodetic DATE 2015.11.18 - 2015.11.18 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)					
								20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>		
53.9																	
0.0	150 mm <b>ASPHALT</b>																
53.3	Sand with gravel trace silt Dense Grey <b>FILL</b>		1	SS	32											25 72 3 (SI+CL)	
0.4	Sand trace silt Compact Brown <b>FILL</b>		2	SS	12												
			3	SS	21											1 91 8 (SI+CL)	
			4	SS	12												
51.0	Silty sand with gravel Compact Brown <b>FILL</b>		5	SS	21											30 45 25 (SI+CL)	
2.9			6	SS	29												
49.5	Sand some silt Compact to dense Brown <b>FILL</b>		7	SS	20												
4.4	- with gravel below 5.5 m		8	SS	33												
	- grey		9	SS	22												
47.2	SILT with sand, trace organics Loose Grey <b>FILL</b>		10	SS	9												
6.7	<b>CLAY (CH)</b> - clay crust Stiff Mottled brown and grey		11	SS	7											0 1 36 63	
46.9																	
7.0	<b>CLAY (CH)</b> Soft to firm Grey		12	SS	WH											0 0 24 76	
45.6																	
8.2																	

Continued Next Page

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

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16



# RECORD OF BOREHOLE No 110

2 OF 2

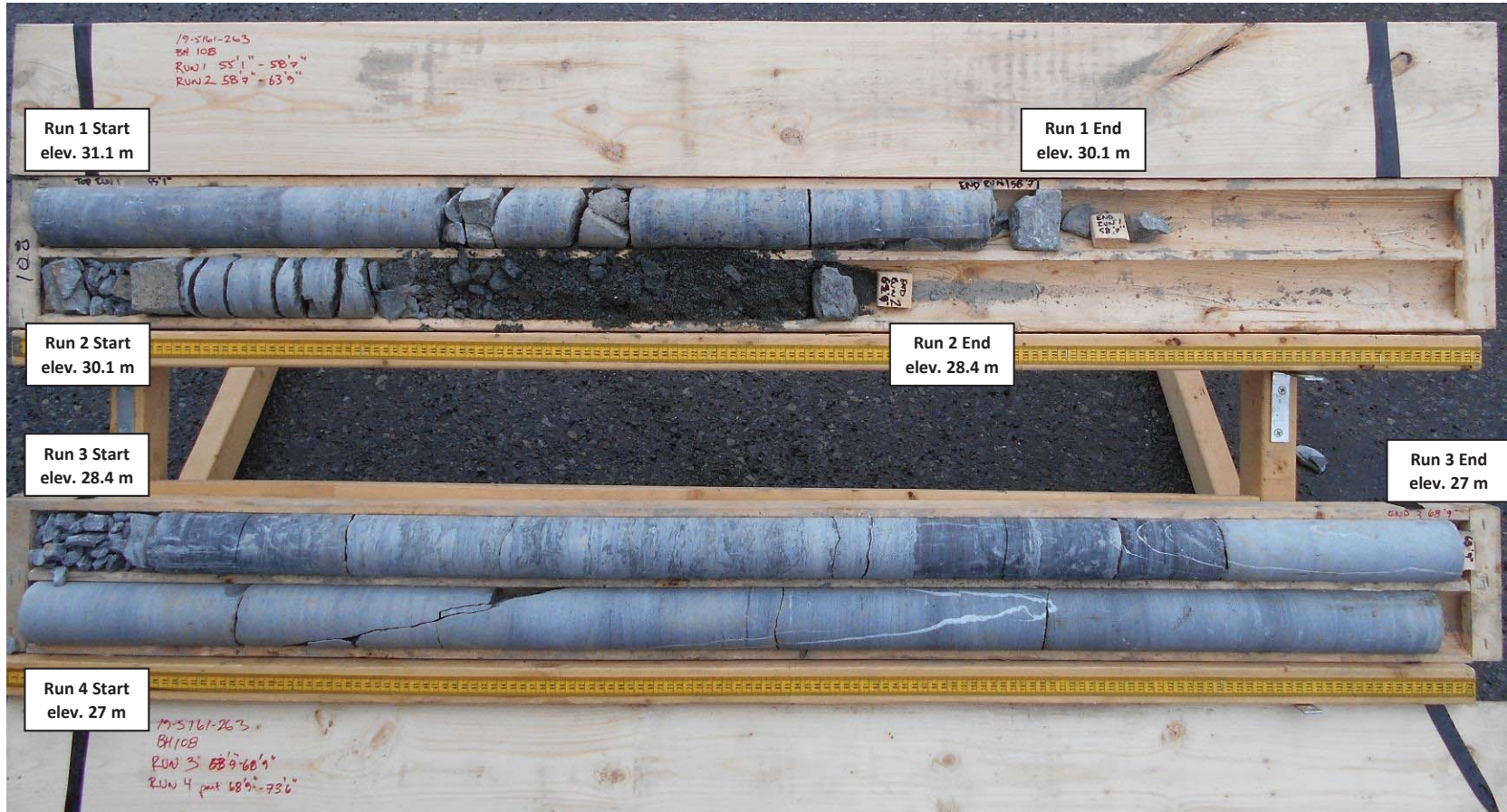
METRIC

GWP# 4066-13-00 LOCATION Highway 401 Underpass at Westley Rd., MTM Zone 8: N 5 003 021.8 E 230 527.2 ORIGINATED BY SMP  
 HWY 401 BOREHOLE TYPE Hollow Stem Auger / HQ Coring COMPILED BY JAG  
 DATUM Geodetic DATE 2015.11.18 - 2015.11.18 CHECKED BY KCP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								20 40 60 80 100				w P w w L				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page															
42.9	CLAY (CH) Soft to firm Grey		13	SS	14		43						○			
11.0	Silty SAND (SM) with gravel Compact Grey						42									
			14	SS	21		41						○			
40.6																
13.3	Silty SAND (SM) with gravel TILL Very Dense Grey - Cobbles and boulders 14.3 m to 18.0 m		15	SS	100/ 200mm		40						○		22 50 24 4	
							39									
			16	SS	100/ 280mm		38						○			
							37						○			
							36									
35.7			17	SS	100/ 150mm											
18.1	End of borehole															

ONTMT4S WESTLEY OVERPASS.GPJ 2012TEMPLATE(MTO).GDT 31/5/16

**Borehole 108**  
**Run 1 to 4 (of 11)**  
**Elevation 31.1 m to 25.5 m**





# Borehole 108

## Run 5 to 8 (of 11)

### Elevation 25.5 m to 19.5 m



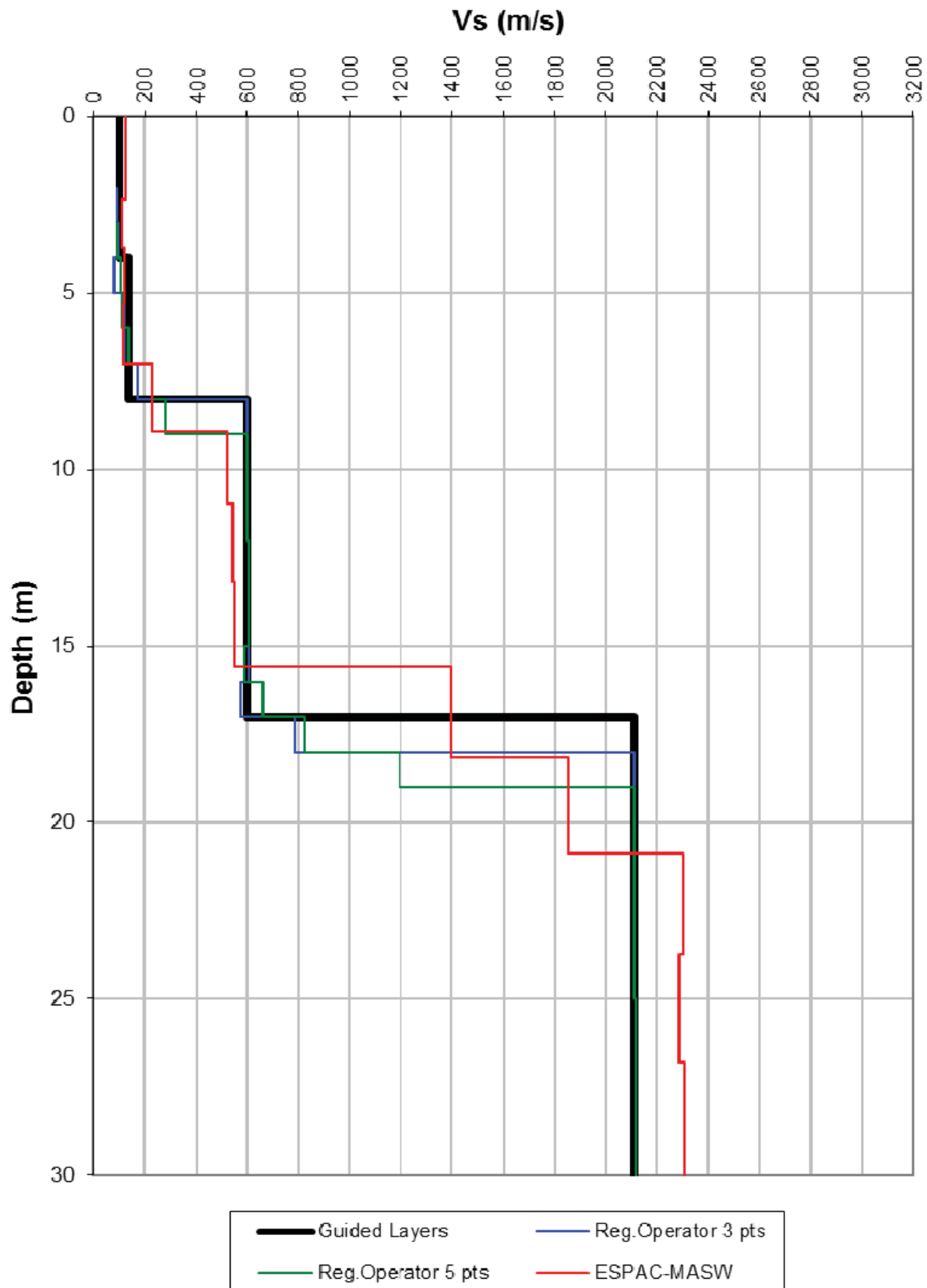
**Elevation 19.5 m to 15.7 m**



**Project No.: 19-5161-263**



# Shear Wave Velocity Profile Highway 401 – Westley Road Underpass

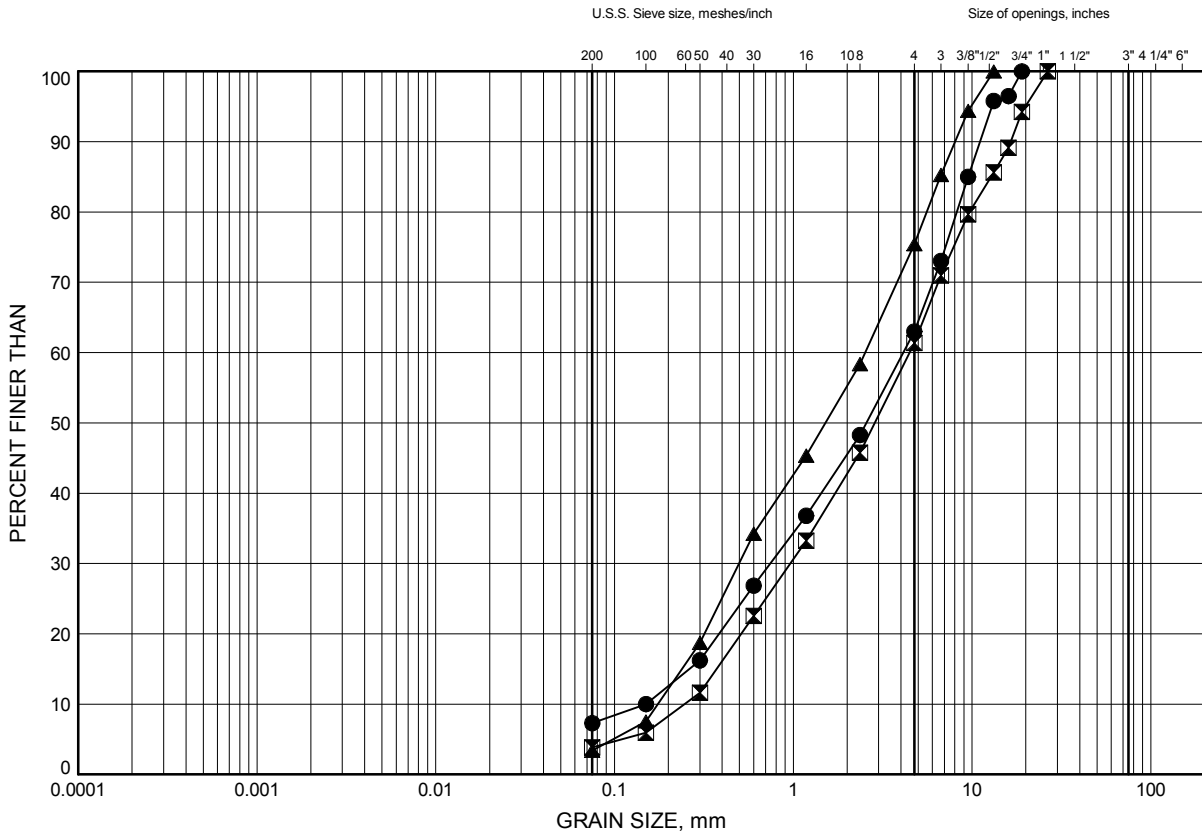


## **APPENDIX C**

### **LABORATORY TEST RESULTS**

# GRAIN SIZE DISTRIBUTION

## Granular Fill



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	101	0.38	54.59
⊠	102	0.38	54.59
▲	110	0.23	53.64

Date May 2016

GWP# 4066-13-00

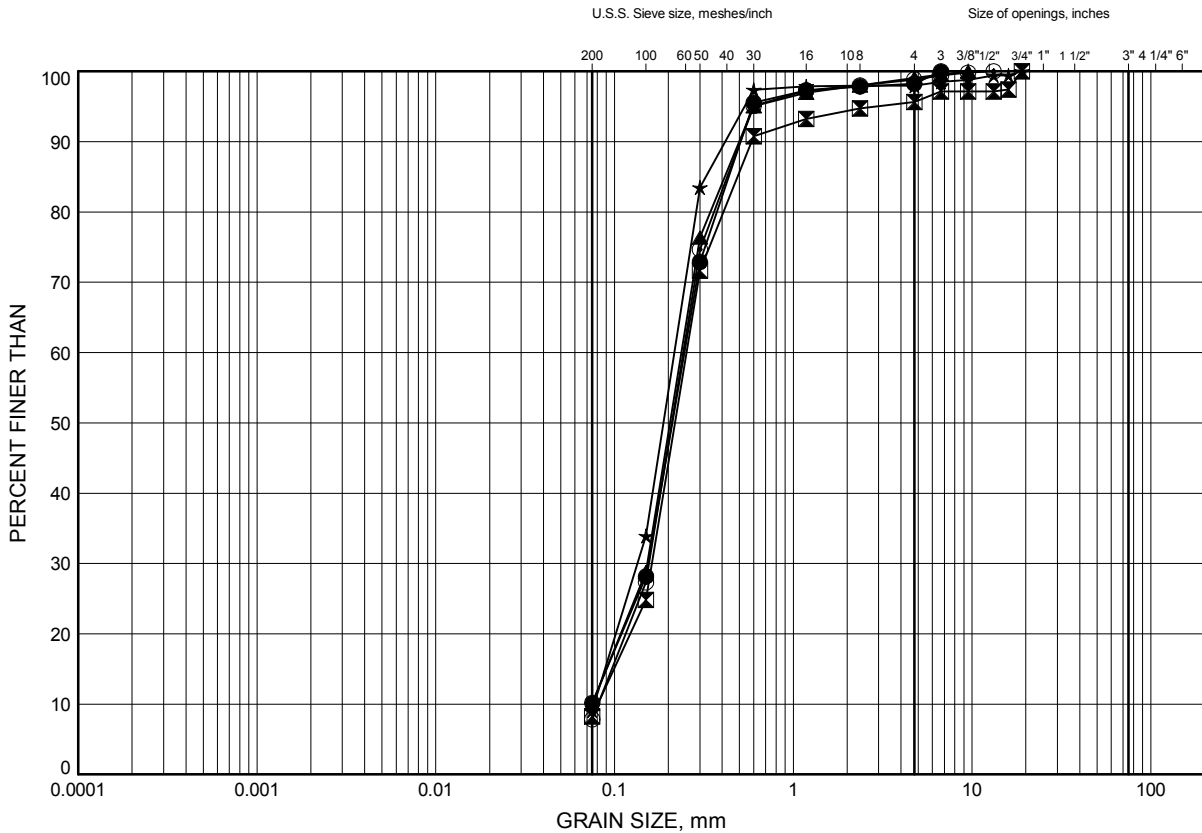


Prep'd KCP

Chkd. PC

## GRAIN SIZE DISTRIBUTION

## Embankment Fill - Sand



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	101	1.83	53.14
⊠	102	2.59	52.38
▲	109	1.83	52.04
★	109	4.88	49.00
⊙	110	1.83	52.04

Date May 2016

GWP# 4066-13-00



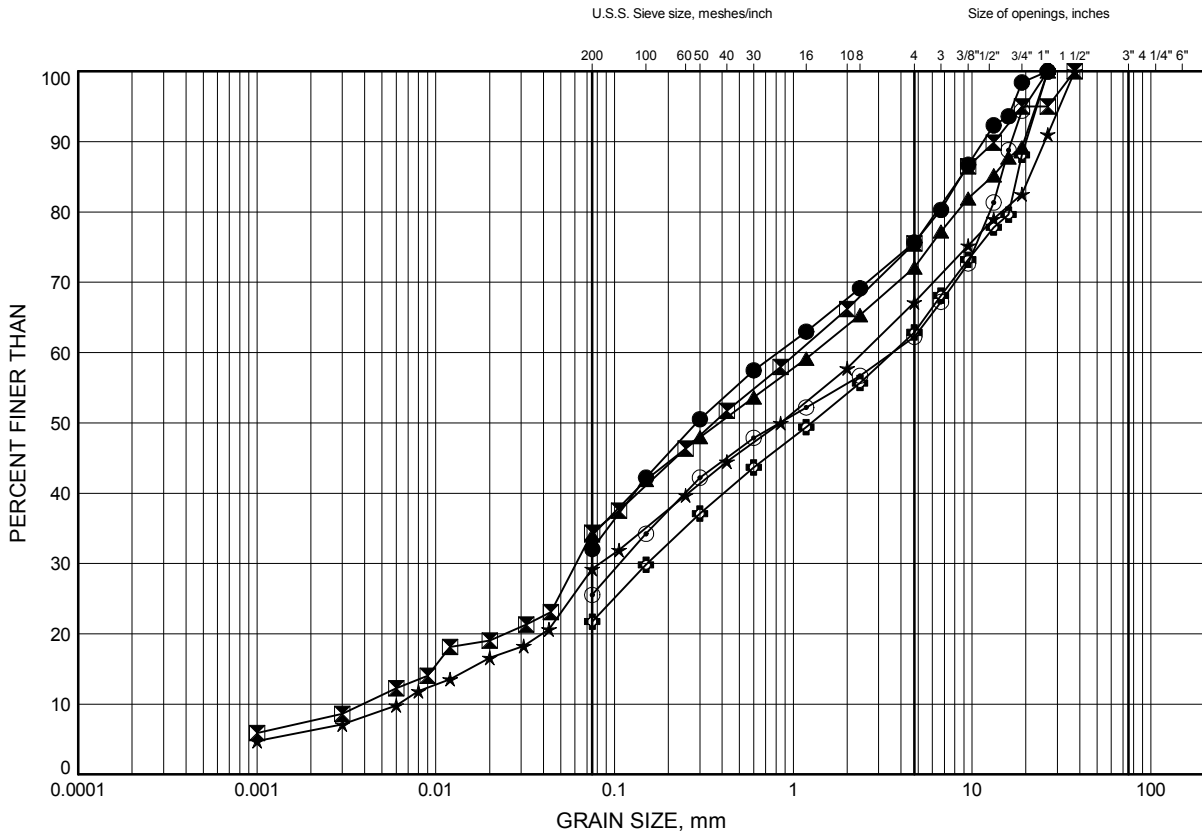
Prep'd KCP

Chkd. PC



# GRAIN SIZE DISTRIBUTION

## Embankment Fill - Silty Sand with Gravel



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	101	4.88	50.10
⊠	101	9.45	45.52
▲	102	4.99	49.98
★	102	8.69	46.29
⊙	109	3.35	50.52
⊕	109	6.40	47.47

Date May 2016

GWP# 4066-13-00

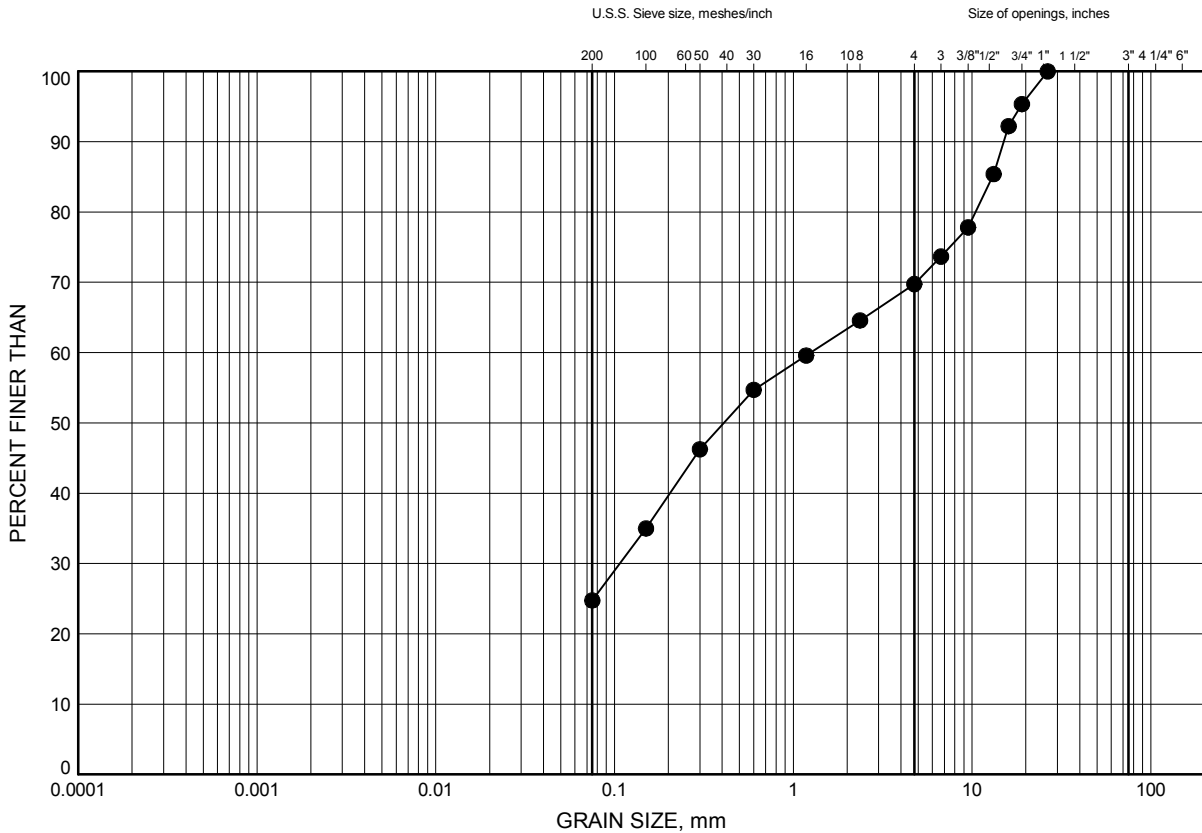


Prep'd KCP

Chkd. PC

# GRAIN SIZE DISTRIBUTION

## Embankment Fill - Silty Sand with Gravel



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	110	3.35	50.52

Date May 2016

GWP# 4066-13-00

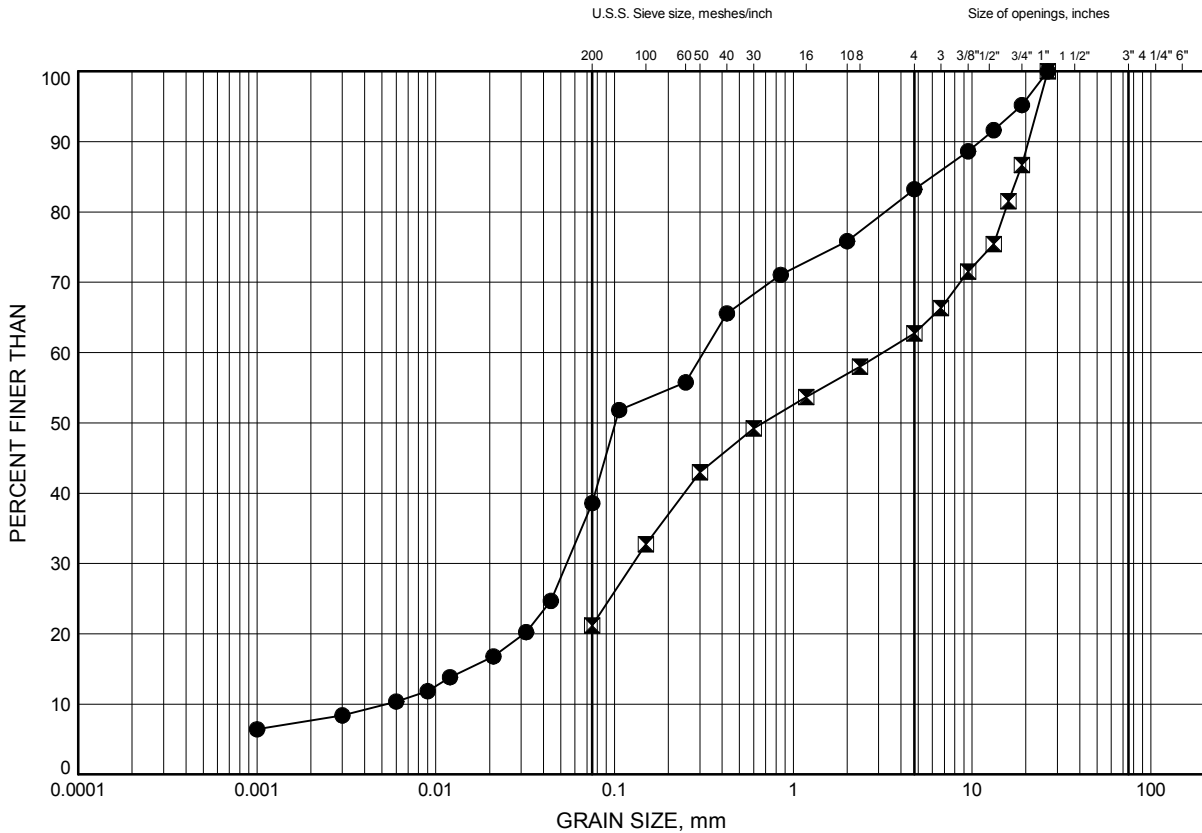


Prep'd KCP

Chkd. PC

# GRAIN SIZE DISTRIBUTION

## Silty Sand with Gravel



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	103	0.46	47.84
⊠	104	1.07	47.32

Date May 2016

GWP# 4066-13-00

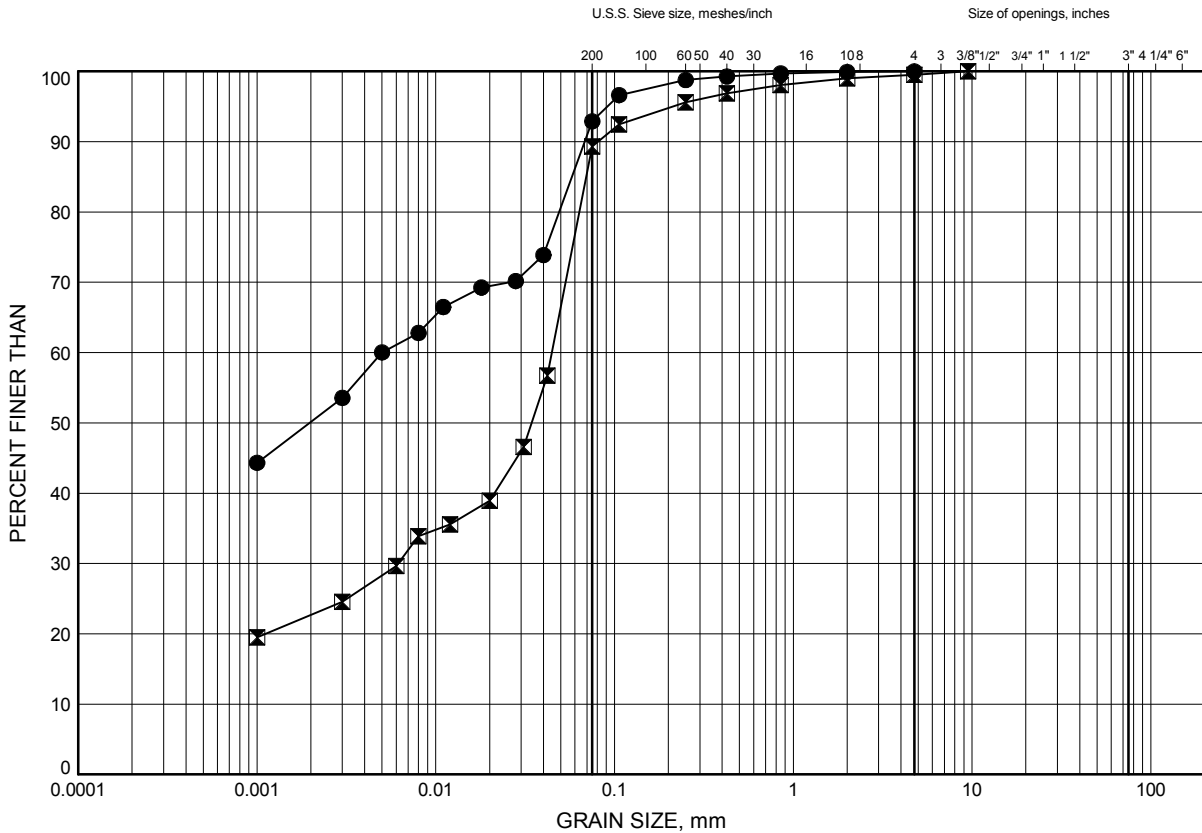


Prep'd KCP

Chkd. PC

# GRAIN SIZE DISTRIBUTION

## Silt with Sand



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	103	2.59	45.70
⊠	108	1.07	46.81

Date May 2016

GWP# 4066-13-00

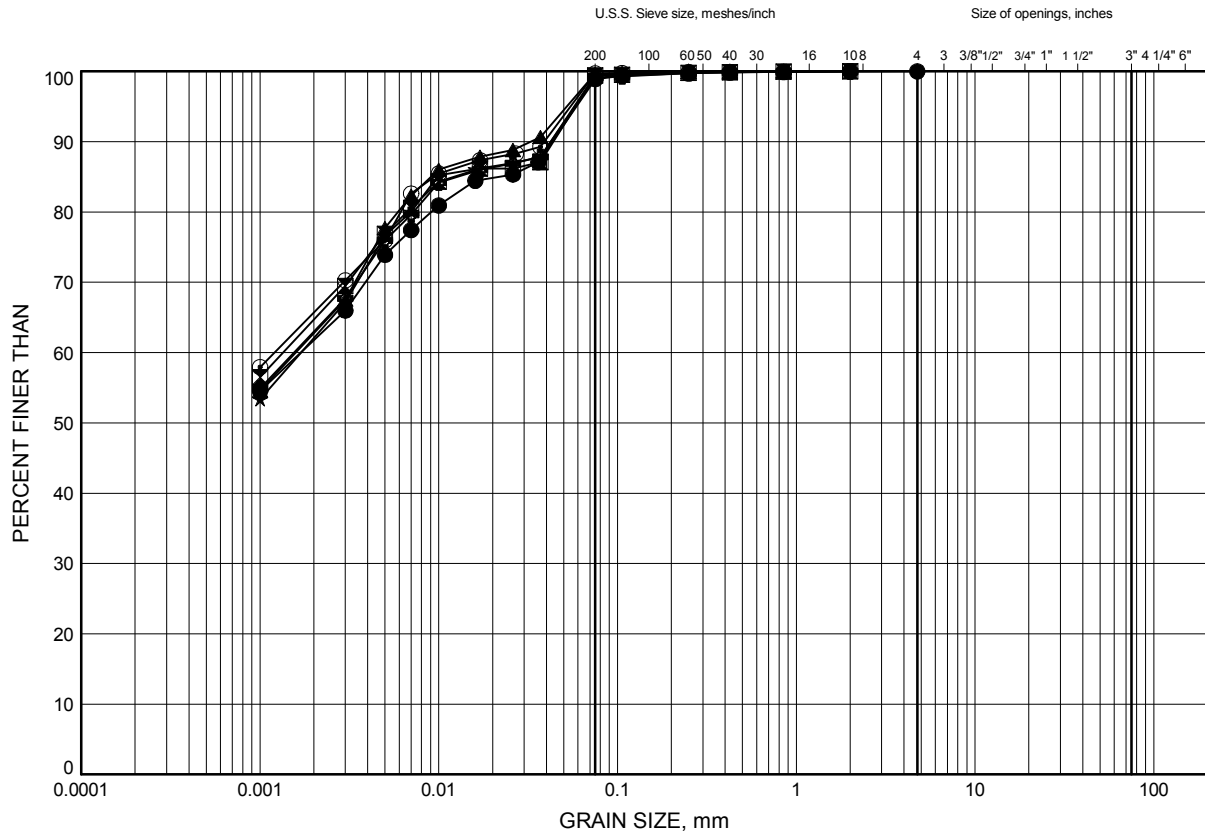


Prep'd KCP

Chkd. PC

# GRAIN SIZE DISTRIBUTION

## Clay Crust



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	104	2.59	45.80
⊠	105	2.59	45.63
▲	106	1.83	45.72
★	107	1.83	45.75
⊙	108	2.59	45.29
⊕	110	7.92	45.95

Date May 2016

GWP# 4066-13-00

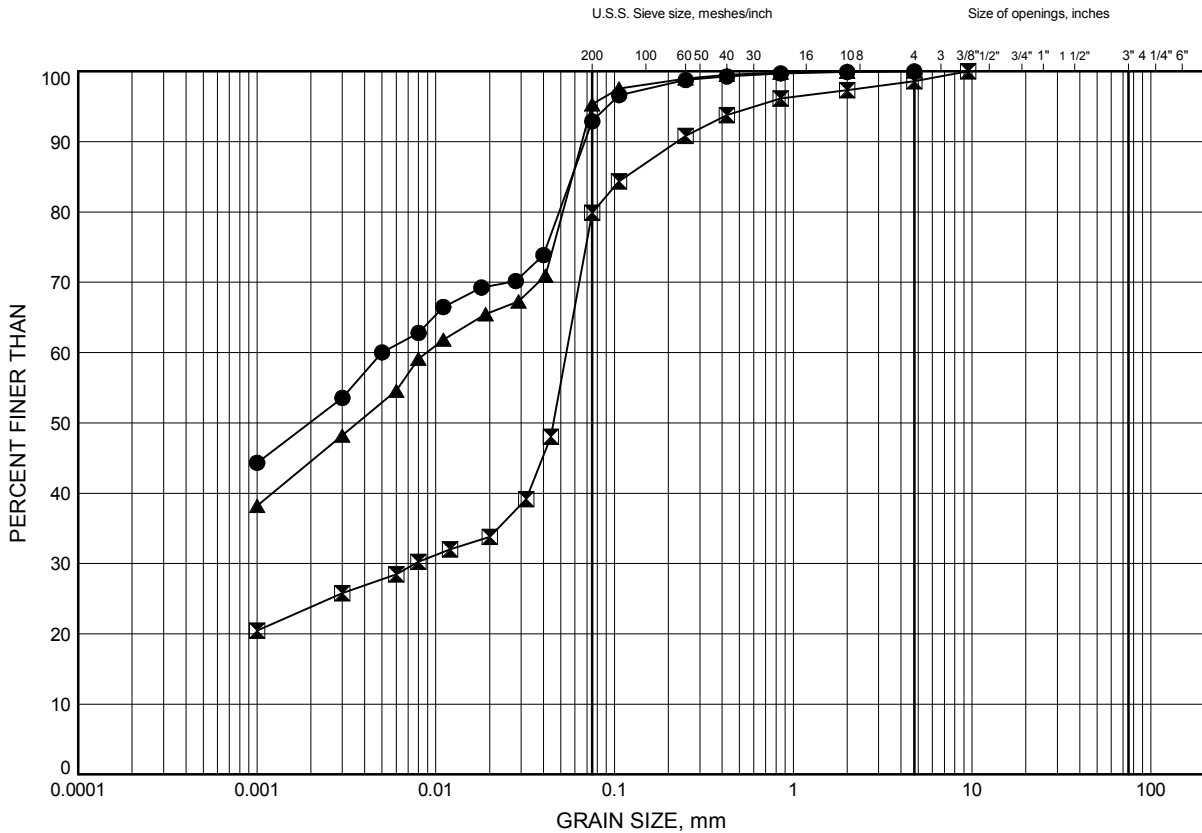


Prep'd KCP

Chkd. PC

# GRAIN SIZE DISTRIBUTION

## Clay Crust



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	103	2.59	45.70
⊠	105	1.83	46.40
▲	109	7.24	46.63

Date May 2016

GWP# 4066-13-00

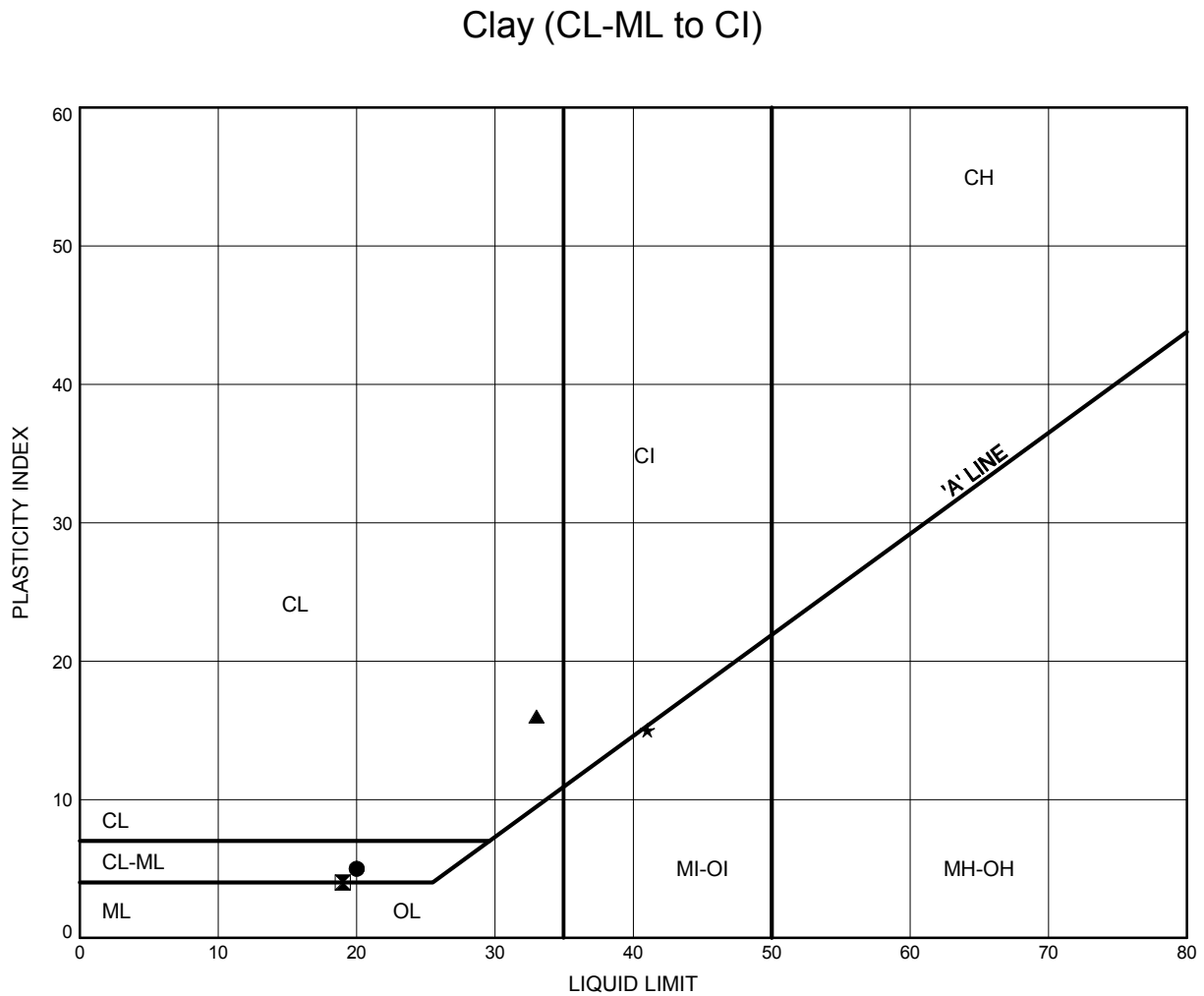


Prep'd KCP

Chkd. PC

Site 31-233 - Highway 401 Underpass at Westley Rd.  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE 9



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	101	9.45	45.52
⊠	102	8.69	46.29
▲	105	1.83	46.40
★	108	1.07	46.81

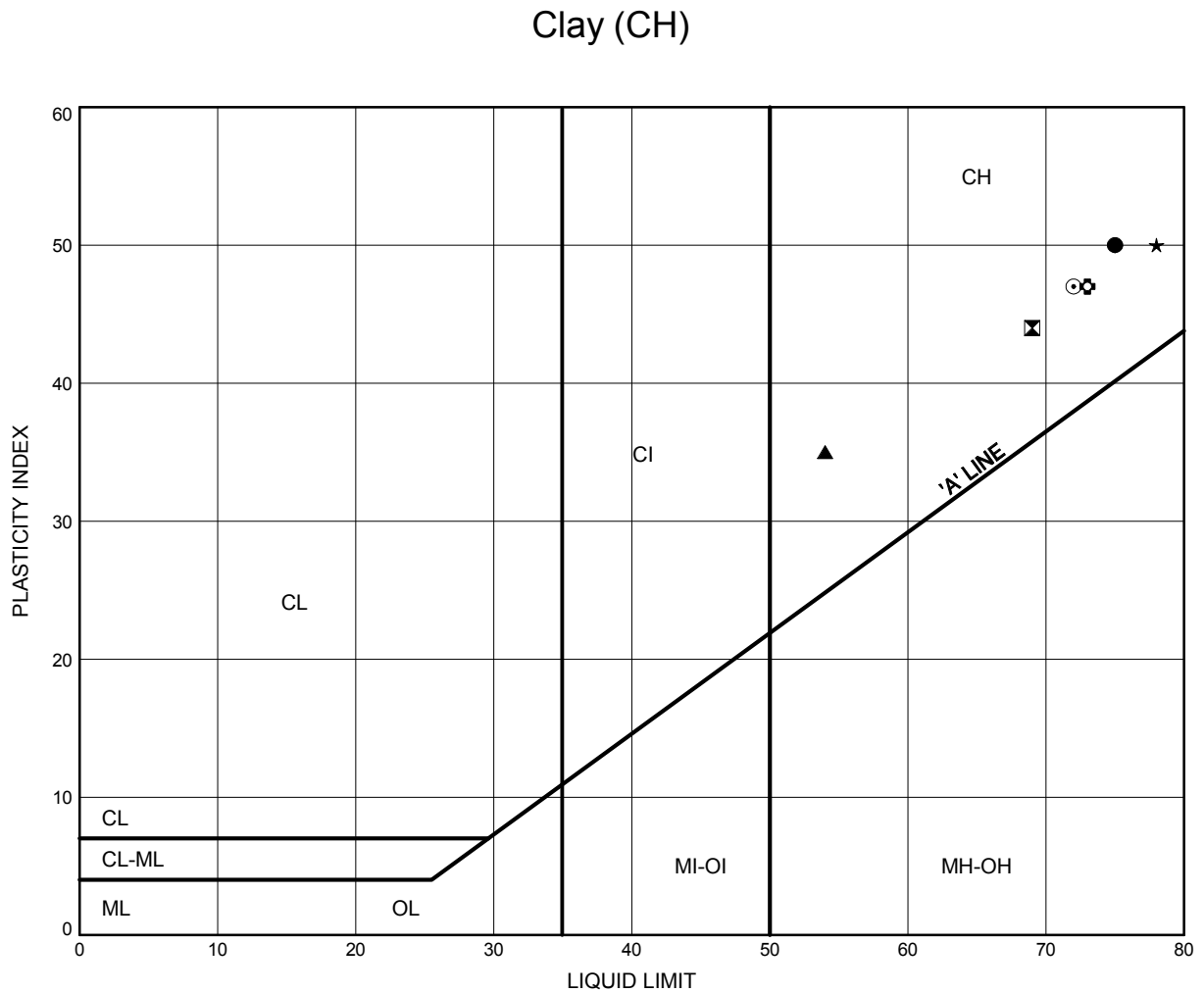
Date May 2016  
 GWP# 4066-13-00



Prep'd KCP  
 Chkd. PC

Site 31-233 - Highway 401 Underpass at Westley Rd.  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE 10



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	101	10.94	44.03
⊠	102	13.26	41.72
▲	103	2.59	45.70
★	103	5.64	42.66
⊙	104	2.59	45.80
⊕	104	5.64	42.75

Date May 2016  
 GWP# 4066-13-00

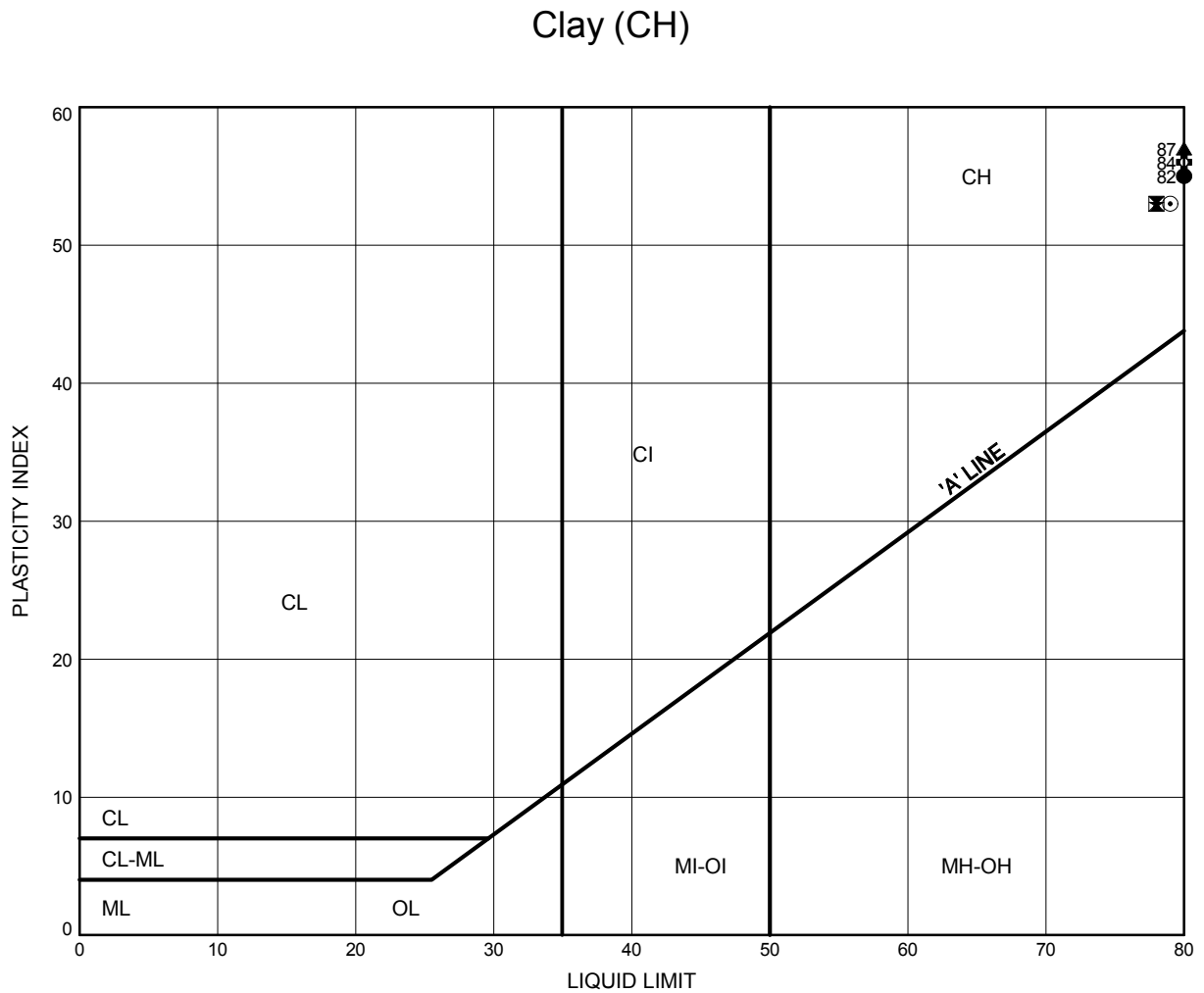


Prep'd KCP  
 Chkd. PC



Site 31-233 - Highway 401 Underpass at Westley Rd.  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE 11



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	105	2.59	45.63
⊠	105	4.11	44.11
▲	106	1.83	45.72
★	106	3.35	44.20
⊙	107	1.83	45.75
⊕	108	2.59	45.29

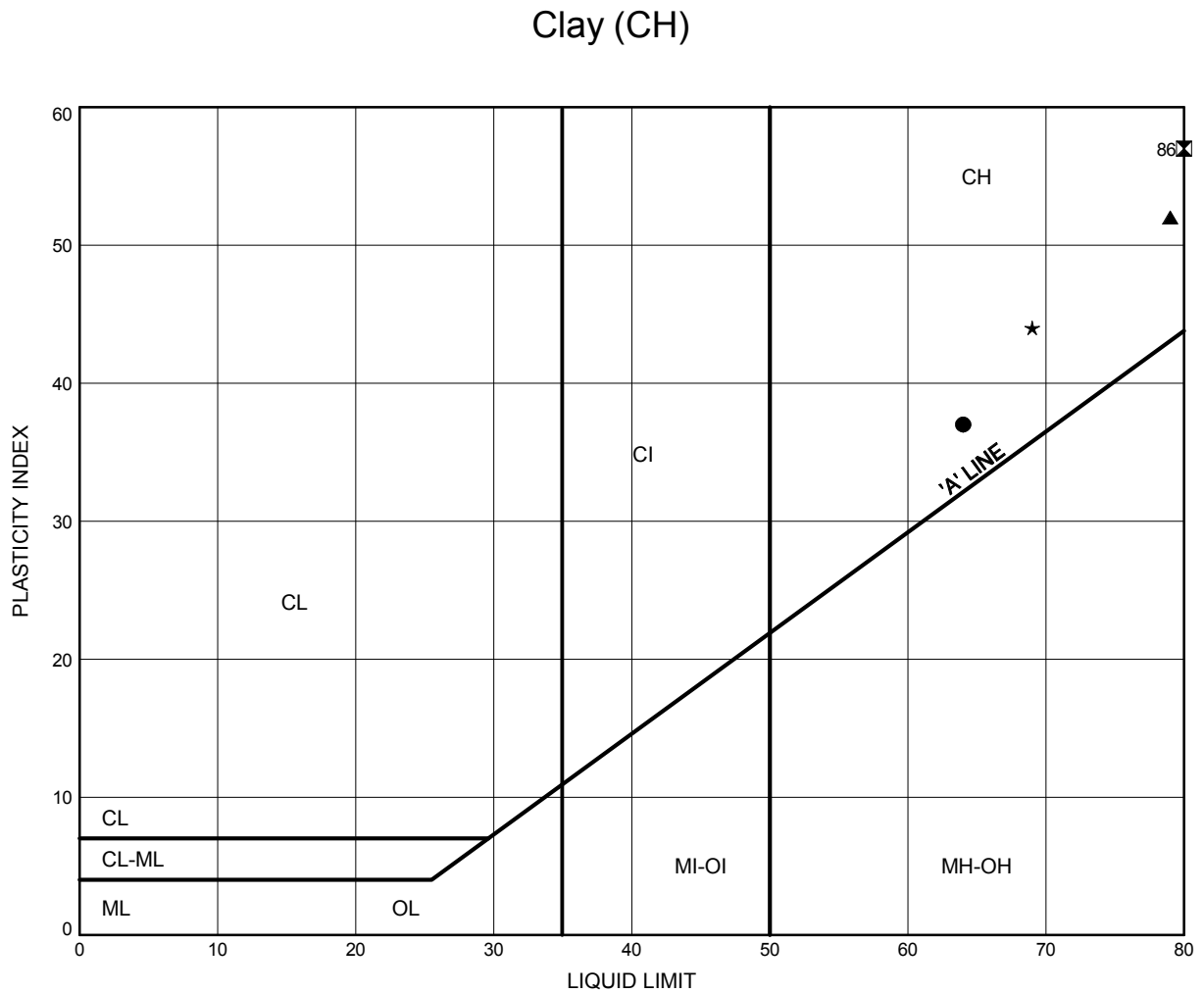
Date May 2016  
 GWP# 4066-13-00



Prep'd KCP  
 Chkd. PC

Site 31-233 - Highway 401 Underpass at Westley Rd.  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE 12



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	108	7.16	40.72
⊠	109	8.69	45.19
▲	110	7.92	45.95
★	110	9.45	44.42

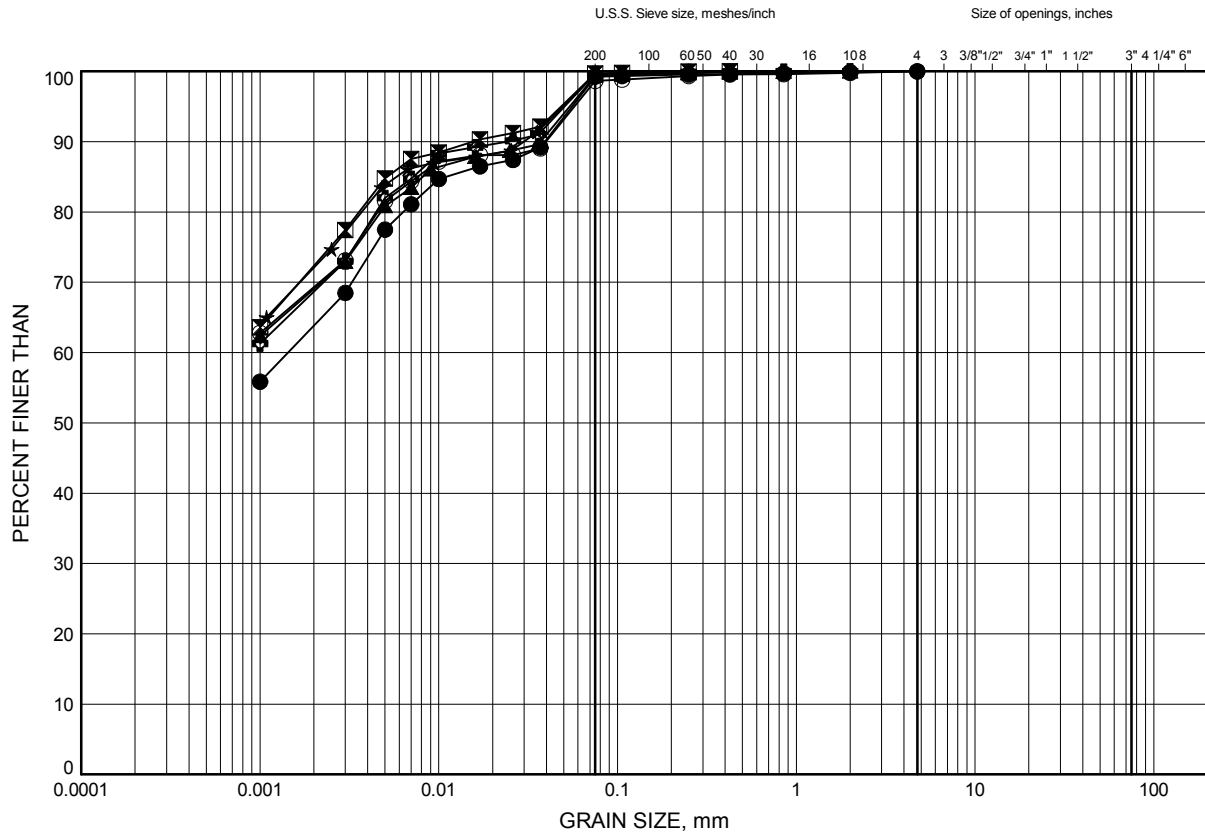
Date May 2016  
 GWP# 4066-13-00



Prep'd KCP  
 Chkd. PC

# GRAIN SIZE DISTRIBUTION

Clay (CH)



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	101	10.94	44.03
⊠	102	13.26	41.72
▲	103	5.64	42.66
★	104	5.64	42.75
⊙	105	4.11	44.11
⊕	106	3.35	44.20

Date May 2016

GWP# 4066-13-00

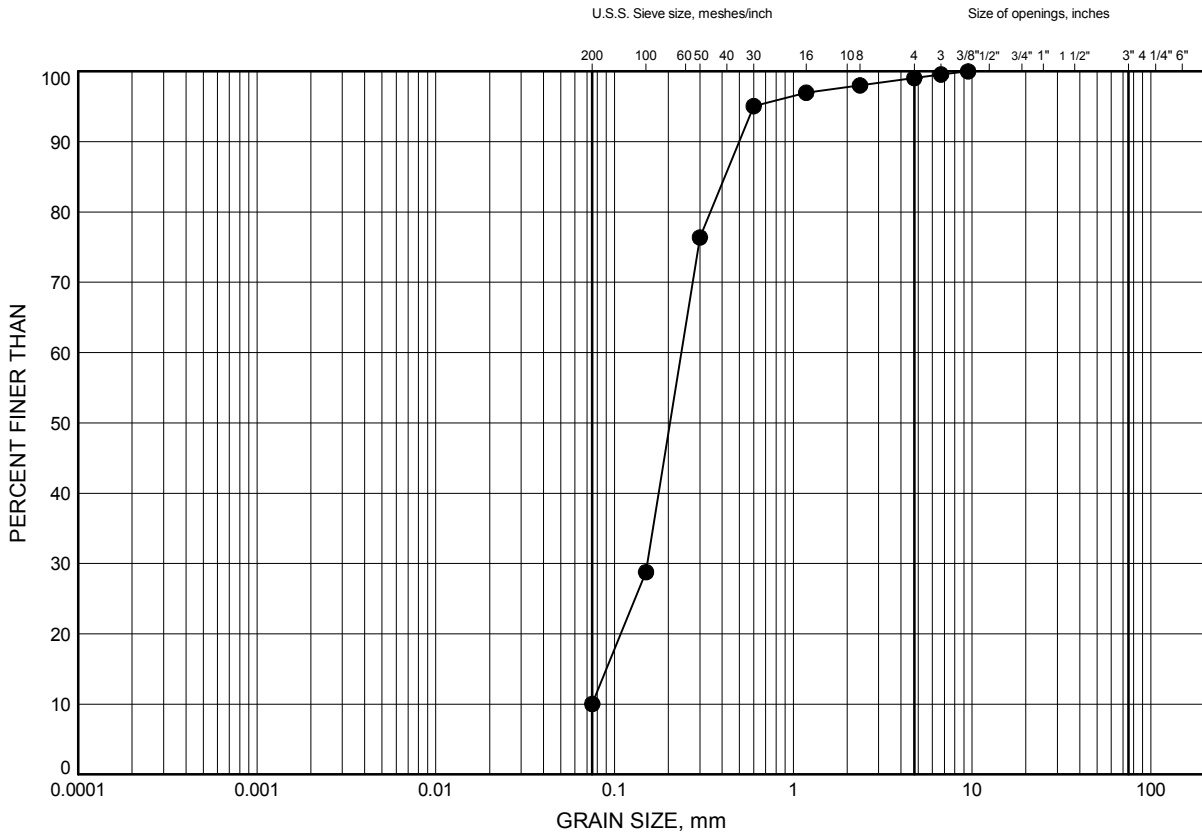


Prep'd KCP

Chkd. PC

# GRAIN SIZE DISTRIBUTION

Clay (CH)



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	109	1.83	52.04

Date May 2016

GWP# 4066-13-00

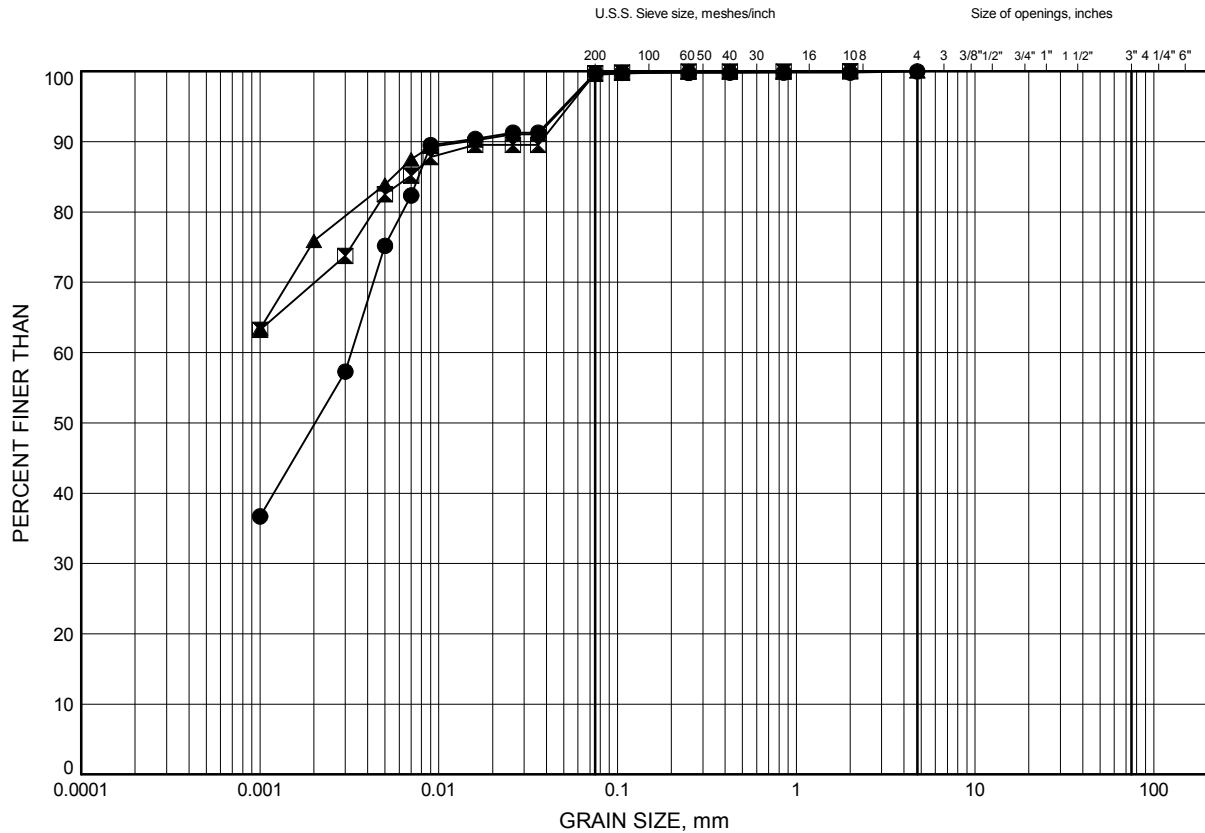


Prep'd KCP

Chkd. PC

# GRAIN SIZE DISTRIBUTION

## Clay (CH)



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	108	7.16	40.72
⊠	109	8.69	45.19
▲	110	9.45	44.42

Date May 2016

GWP# 4066-13-00

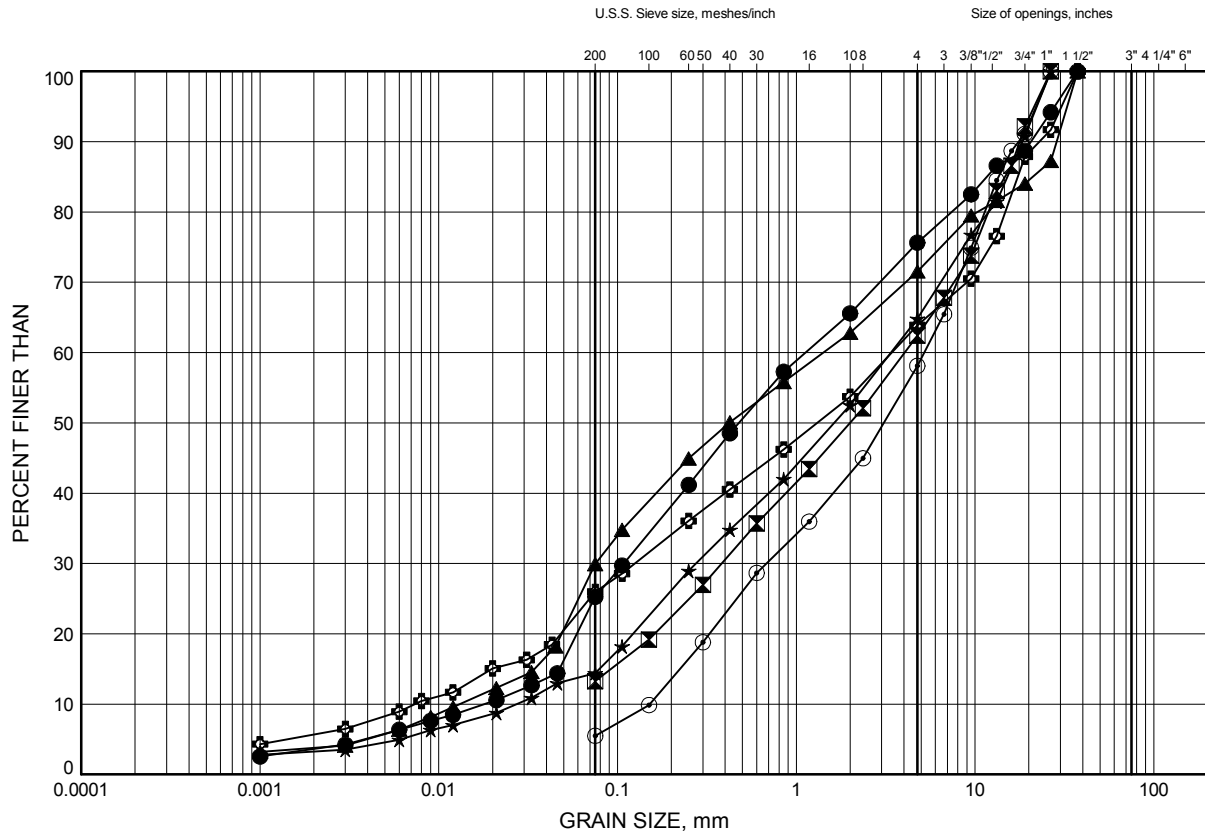


Prep'd KCP

Chkd. PC

## GRAIN SIZE DISTRIBUTION

## Silty Sand with Gravel



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	101	17.07	37.90
⊠	103	9.45	38.85
▲	104	7.16	41.23
★	105	7.16	41.06
⊙	106	10.97	36.58
⊕	107	7.16	40.41

Date May 2016

GWP# 4066-13-00

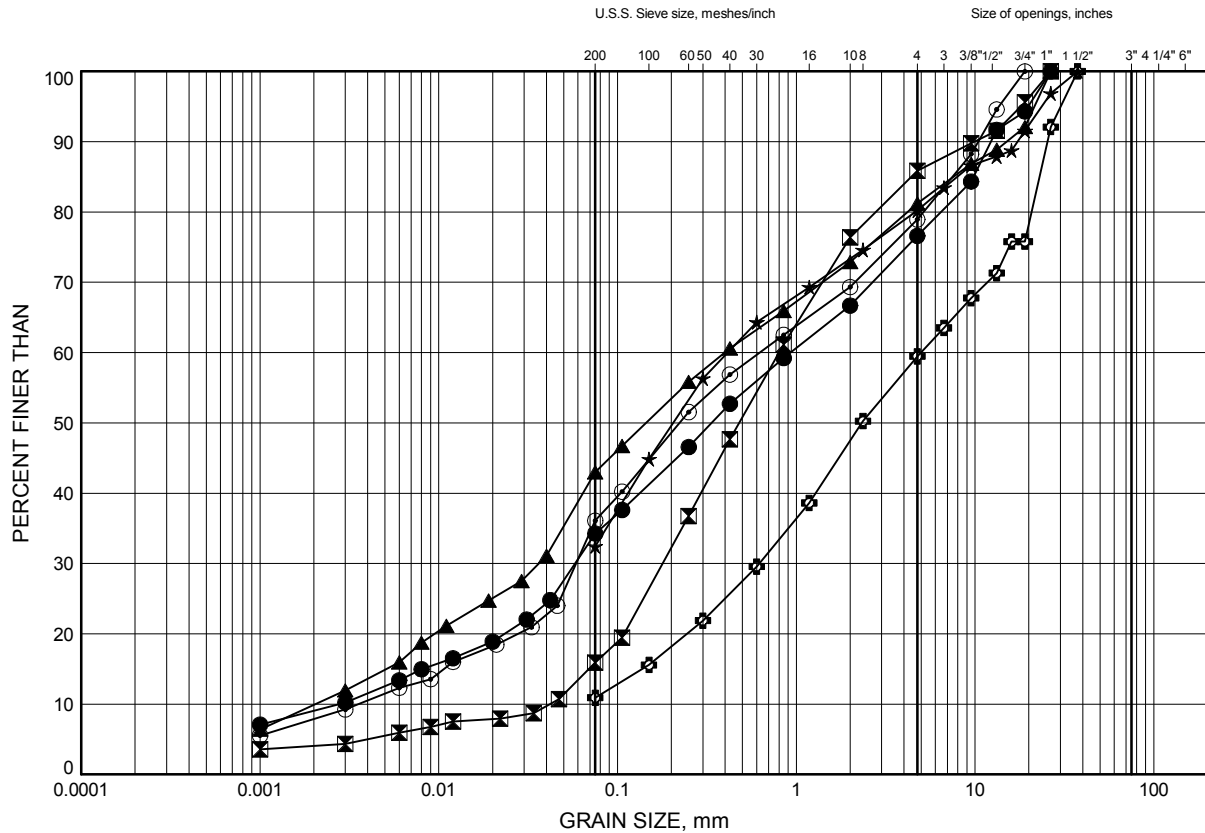


Prep'd KCP

Chkd. PC

# GRAIN SIZE DISTRIBUTION

## Glacial Till



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	101	18.35	36.62
⊠	102	16.89	38.09
▲	103	10.97	37.32
★	103	15.49	32.80
⊙	104	15.38	33.01
⊕	105	12.50	35.73

Date May 2016

GWP# 4066-13-00

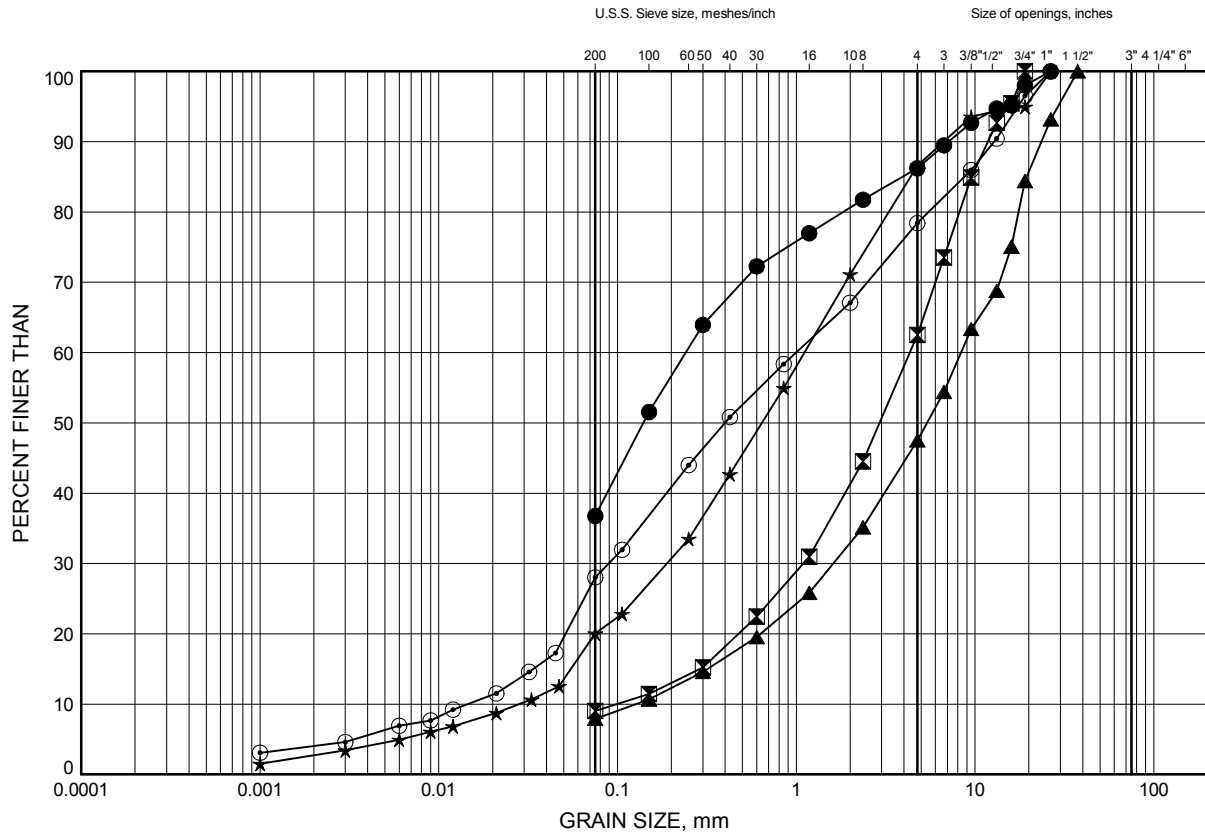


Prep'd KCP

Chkd. PC

# GRAIN SIZE DISTRIBUTION

## Glacial Till



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	106	15.39	32.16
⊠	107	11.61	35.96
▲	108	10.97	36.91
★	109	12.50	41.38
○	110	14.02	39.85

Date May 2016

GWP# 4066-13-00



Prep'd KCP

Chkd. PC





**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 Westley**  
**Thurber File# (19-5161-263): Sample 15-103 TW-7 sampled on December 3, 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.**

A handwritten signature in black ink, appearing to read "Raymond Hache".

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)

Project Thurber Engineering, File# 19-5161-263  
Sample No. 15-103, TW-7-Mega 5, Westley

Project No. 122410864  
Sample Depth (m) 4.88m

**Sample Data**

Initial Ht. of soil,  $H_i$  19.01 mm  
Initial sample volume,  $V_i$  37.36 cm<sup>3</sup>  
Specific gravity,  $G_s$  2.771 Tested  
Initial Water Content 79.4 %  
Wet mass of soil 57.82 g  
Dry mass of soil 31.92 g

Wet unit weight 15.18 kN/m<sup>3</sup>  
Dry unit weight 8.38 kN/m<sup>3</sup>  
Initial height of voids,  $H$  1.315 cm  
Ht. of solids,  $H_s$  0.586 cm  
Initial Void Ratio,  $e_o$  2.24  
Degree of Saturation 100.0 %

Odometer A  
ASTM Method A  
Load Duration 24 hours  
Start Date 8-Dec-15  
End Date 24-Dec-15

Stage	Test Type	Stress Increment (kPa)	End of Load Deformation (cm)	Strain $\epsilon = \Delta H/H_i$	$\Delta e = \Delta H/H_s$	Void Ratio $e$	End of Load Height (cm)	Corrected deformation $\Delta H50$ (cm)	Specimen height $H50$ (cm)	Time $t_{50}$ (min)	Coefficient of Consolidation $c_v$ (cm <sup>2</sup> /min)	Time $t_{90}$ (min)	Coefficient of Consolidation $c_v$ (cm <sup>2</sup> /min)
Seating	Seating	5.00	0.014			2.243							
1	Consolidation	9.76	0.0024	0.00126	0.004	2.239	1.899						
2	Consolidation	19.94	0.0142	0.00747	0.024	2.219	1.887						
3	Consolidation	42.67	0.0280	0.01473	0.048	2.195	1.873	0.0182	1.8828			1.85	0.4062
4	Consolidation	61.12	0.0428	0.02251	0.073	2.170	1.858	0.0311	1.8699			1.80	0.4118
5	Consolidation	83.84	0.0494	0.02599	0.084	2.159	1.852						
6	Consolidation	118.37	0.0674	0.03546	0.115	2.128	1.834	0.0530	1.8480			1.70	0.4259
7	Consolidation	159.55	0.2466	0.12972	0.421	1.822	1.654						
8	Consolidation	320.70	0.5228	0.27501	0.892	1.351	1.378	0.3452	1.5558			4.70	0.1092
9	Consolidation	639.90	0.6704	0.35266	1.144	1.099	1.231	0.5712	1.3298			3.90	0.0961
10	Rebound	159.55	0.6436	0.33856	1.098	1.145	1.257						
11	Rebound	42.67	0.6054	0.31846	1.033	1.210	1.296						
12	Rebound	9.76	0.5640	0.29669	0.962	1.281	1.337						

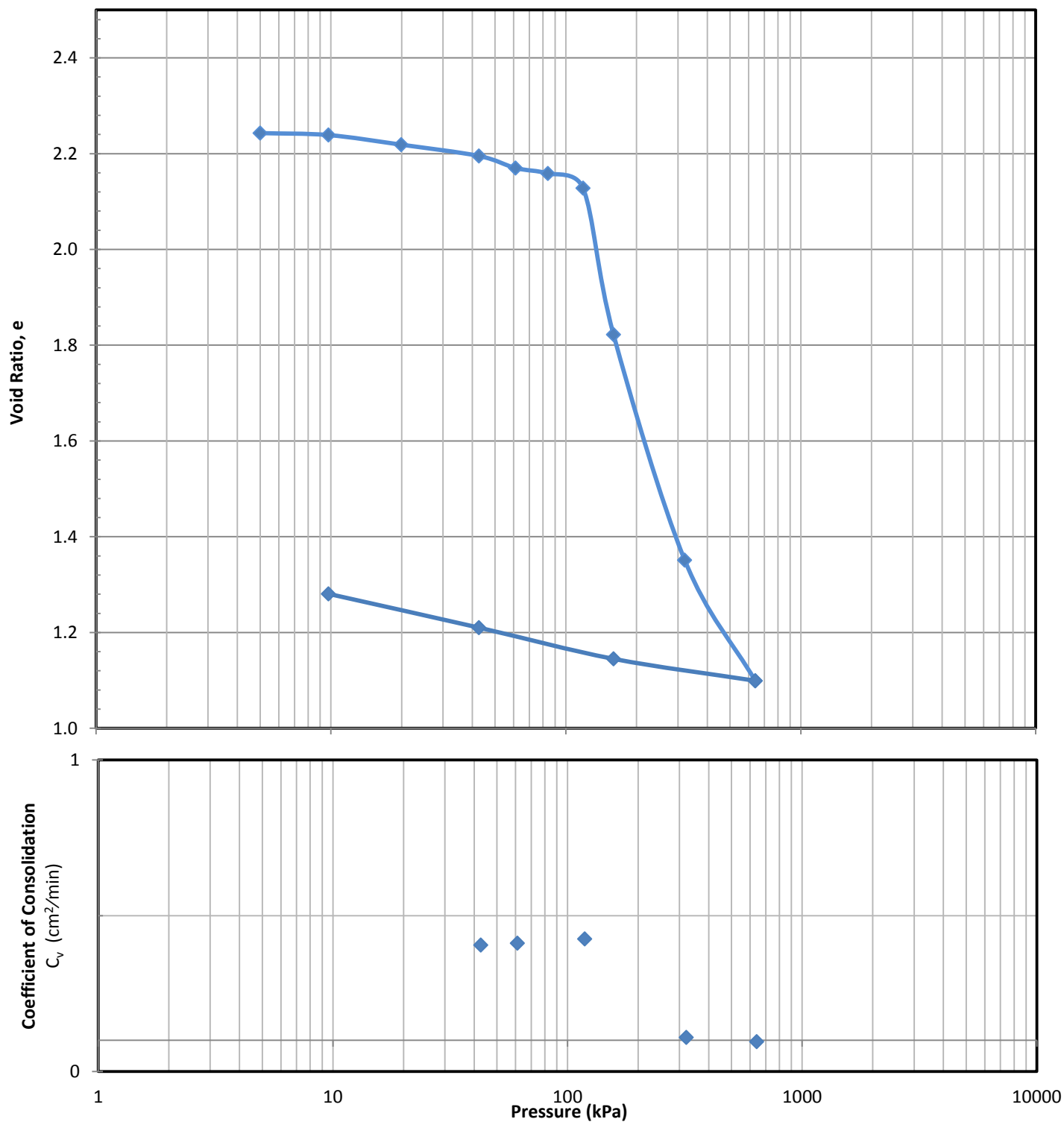
Notes: Test Method A loading  
Specimen from 310 mm from top of tube

Conducted by: DB

Checked by: AN

**Project**  
**Project No.**  
**Sample No.**  
**Sample Depth (m)**

**Thurber Engineering, File# 19-5161-263**  
**122410864**  
**15-103, TW-7-Mega 5, Westley**  
**4.88m**



## **APPENDIX D**

### **SELECTED PHOTOGRAPHS OF THE UNDERPASS LOCATION**



**Figure 1: Highway 401 eastbound showing location of the bridge crossings of the Ferguson and McIntosh Drains**



**Figure 2: Highway 401 looking westbound**





**Figure 3: Highway 401 underpass at Westley Road looking west**



**Figure 4: County Road 2 Underpass at Westley Road looking west**





**Figure 5: South embankment looking north east**



**Figure 6: South abutment foreslope looking east**





**Figure 7: South approach embankment looking south along Westley Road towards Boreholes 101 and 102**



**Figure 8: North embankment looking north east**





**Figure 9: North abutment foreslope looking east**



**Figure 10: North approach embankment looking north along Westley Road towards Boreholes 109 and 110**

**APPENDIX E**

**DOWNHOLE SHEAR WAVE VELOCITY REPORT**

DRAFT



**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 5<sup>th</sup>, 2016

Transmitted by email: [kpowers@thurber.ca](mailto:kpowers@thurber.ca)  
Our Ref.: M-15171-B

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, Westley**  
**[WP No.: 4066-13-00]**

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 Westley's overpass, to obtain the  $\bar{V}_{s30}$  value for the site (seismic) classification according with the National Building Code.

The borehole was located beside the north shoulder of County Road 2 (cf. Figure 1). The surveys were carried out on December 10<sup>th</sup>, 2015 by Mr. Charles Trottier, M.A.Sc. phys. and Mr. Maxime Boudreault, and January 27<sup>th</sup> 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 108). 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  $\mu$ s, 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 guided 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: 103 m/s from the surface to 4 meters deep; 137 m/s from 4 to 8 meters; 601 m/s from 8 to 17 meters; and 2115 m/s for the rock.

The  $\bar{V}_{S30}$  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  $\bar{V}_{S30}$  value is 335.8 m/s (Class "D"). Details of the  $\bar{V}_{S30}$  calculation are presented in Table 1. Low seismic shear wave velocities were measured and calculated from the surface to 8 meters deep.





## 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 Westley's overpass. The borehole (BH 108), 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  $\bar{V}_{s30}$  value of 336 m/s, corresponding to Site Class "D" ( $180 < \bar{V}_{s30} \leq 360$  m/s ).

Some low seismic shear wave velocities were measured and calculated for the overburden materials, from the surface to 8 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  $\bar{V}_{s30}$  value.

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

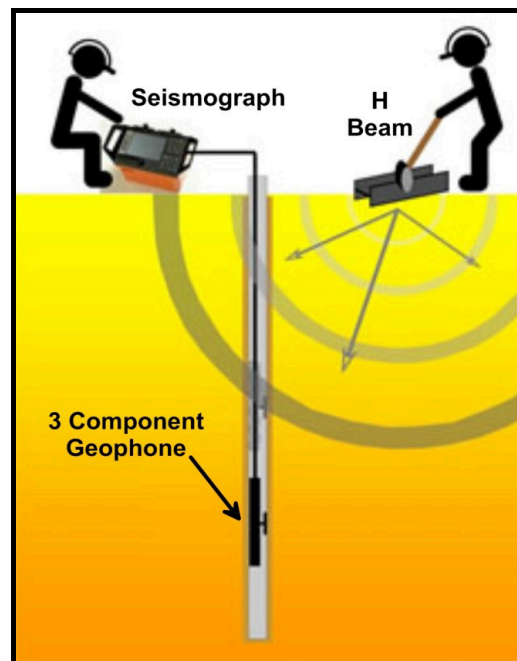
This report has been written by Jean-Luc Arsenault, M.A.Sc, P.Eng.

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**

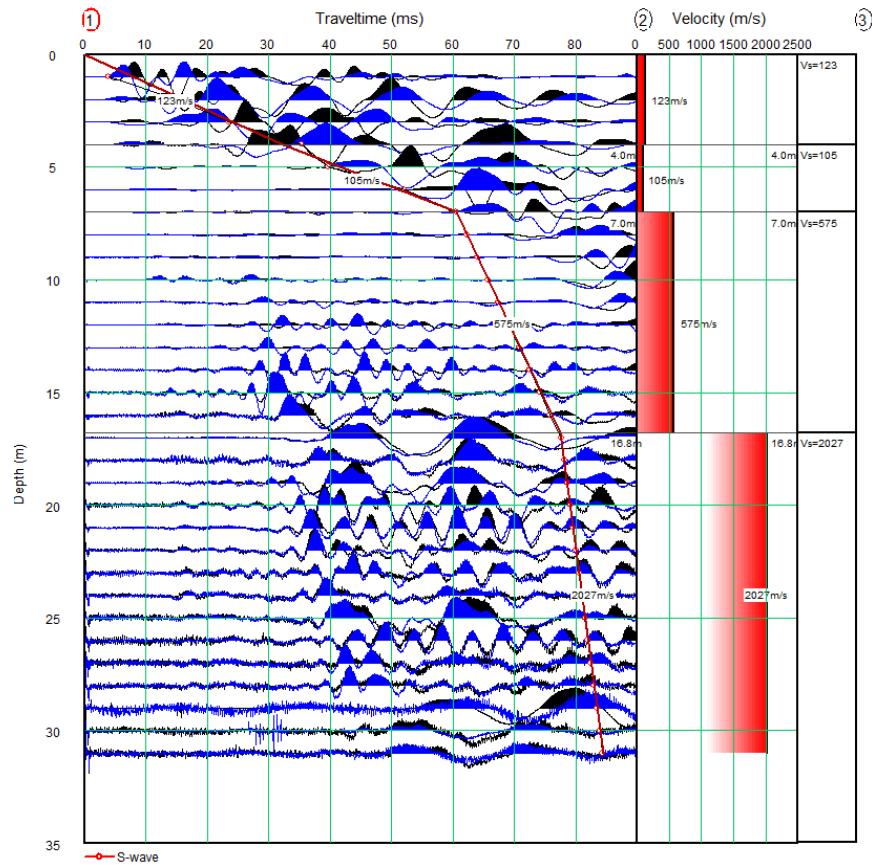


Figure 3: Downhole Polarized Seismogram (Horizontal Axis)

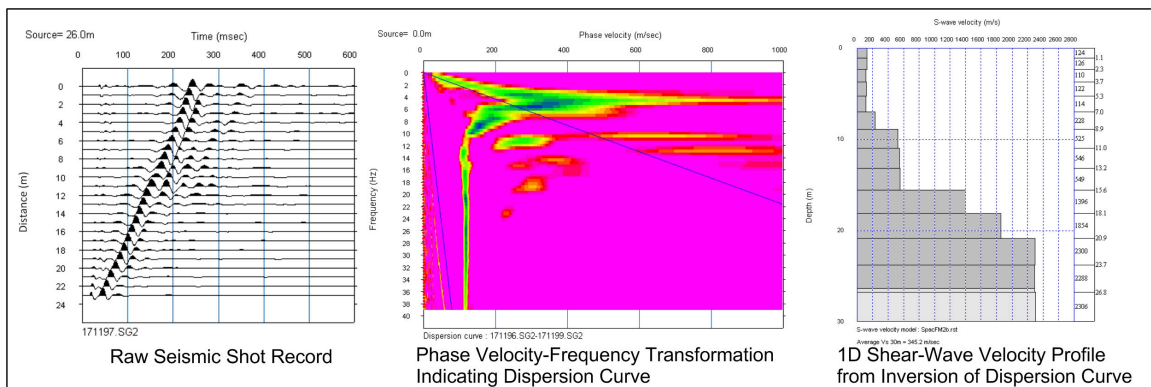


Figure 4: MASW-ESPAC Procedure Steps





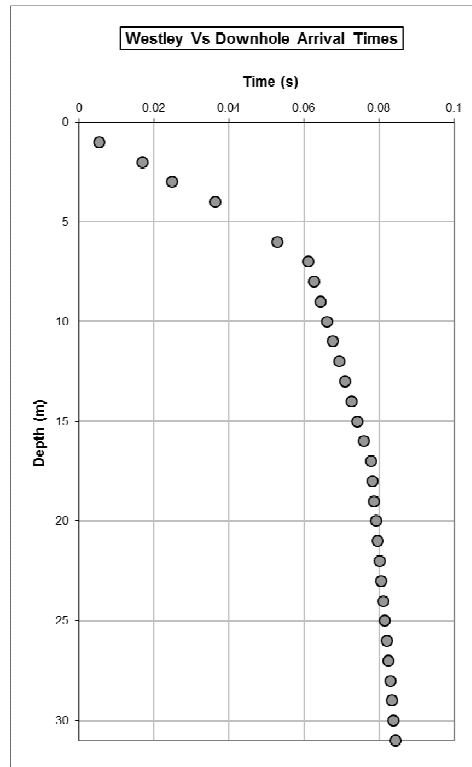


Figure 5: Shear Wave's Arrival Times Picks

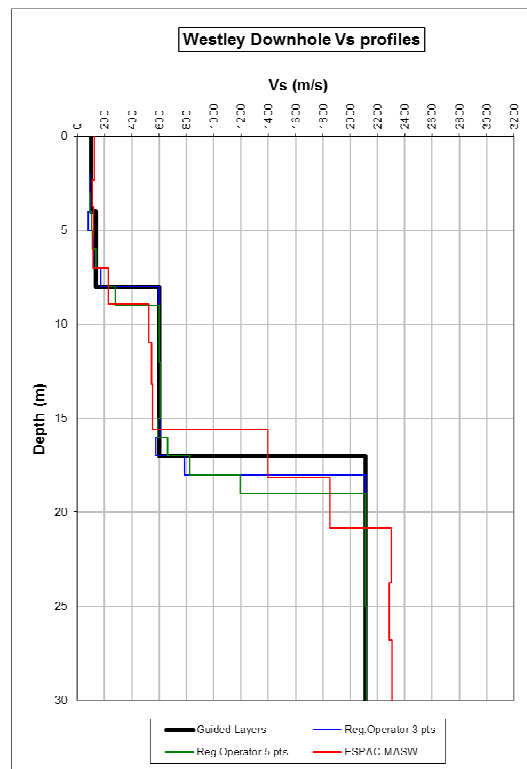


Figure 6: Downhole Survey Results (with ESPAC-MASW)



**Table 1:  $V_{s30}$  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	<b>102.6</b>					
4	<b>136.8</b>	4	0.038982	0.038982	4	102.6
8	600.9	4	0.029233	0.068216	8	117.3
17	2115.1	9	0.014978	0.083193	17	204.3
30		13	0.006146	0.089340	30	335.8

<b><math>V_{s30}</math> =</b>	335.8
<b>Site Class:</b>	D *

\*: subject to geotechnical assessment of the unconsolidated materials from surface to 8 m deep.



## **APPENDIX F**

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

F- 1: Evaluation of Embankment Grade Raise Design Options

Option	Description	Advantages	Disadvantages	Risks / Consequences	Relative Cost	Comments
1	<b>Granular Embankment</b> 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	<b>Not Recommended</b>
3	<b>Lightweight Fill</b> 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	<b>Recommended</b>
	<b>Ground Improvement</b>  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	<b>Not Recommended</b>
4	<b>Accelerated Settlement</b>  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  Does not address stability concerns  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	<b>Not Recommended</b>

**F- 2: Comparison of Deep Foundation Alternatives**

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

**APPENDIX G**  
**HISTORICAL SETTLEMENT DATA**  
**GSC SEISMIC HAZARD CALCULATION**  
**SLOPE STABILITY ANALYSIS**  
**L-PILE ANALYSIS FOR HP 310X110 STEEL PILES**

# SETTLEMENT OBSERVATIONS

WESTLEY CREEK ROAD OVER HWY. NO 401

JOB 62-F-81 W.P. 178-60

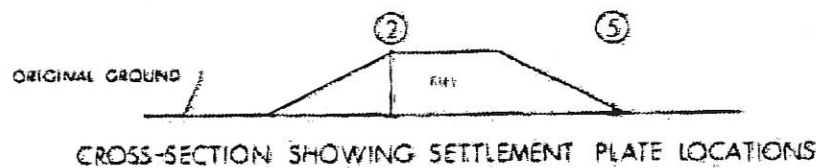
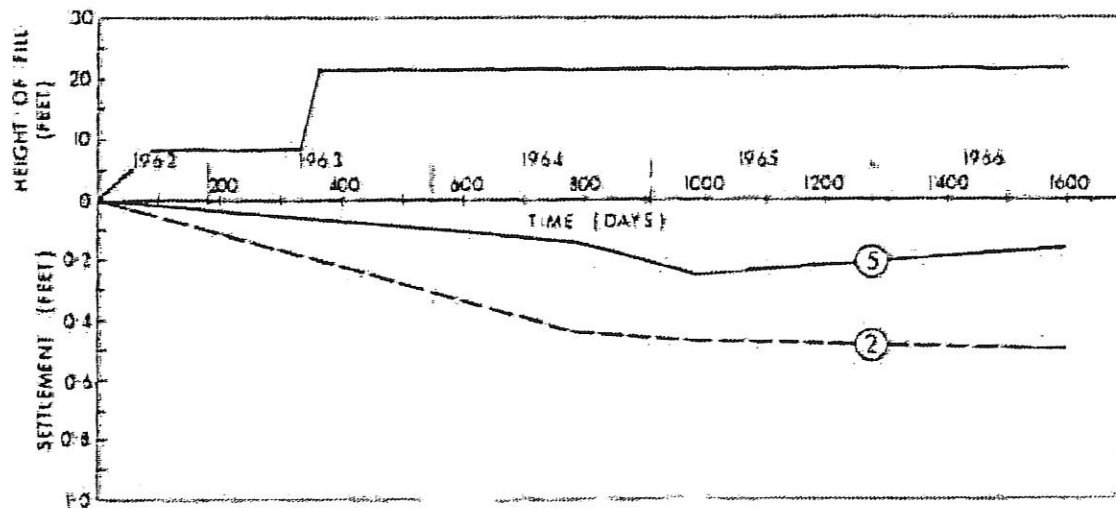


FIGURE 24, SETTLEMENT RECORDS, HIGHWAY 401 AND WESTLEY ROAD UNDERPASS (NORTH SIDE)

# 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: 45.1625 N, 74.4443 W User File Reference: Site 31-233 - Westley 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)	<b>Sa(0.2)</b>	Sa(0.3)	<b>Sa(0.5)</b>	<b>Sa(1.0)</b>	<b>Sa(2.0)</b>	<b>Sa(5.0)</b>	<b>Sa(10.0)</b>	<b>PGA (g)</b>	<b>PGV (m/s)</b>
0.633	0.725	<b>0.598</b>	0.450	<b>0.314</b>	<b>0.150</b>	<b>0.069</b>	<b>0.018</b>	<b>0.0061</b>	<b>0.380</b>	<b>0.260</b>

**Notes.** Spectral ( $S_a(T)$ , where  $T$  is the period in seconds) and peak ground acceleration (PGA) values are given in units of  $g$  ( $9.81 \text{ m/s}^2$ ). Peak ground velocity is given in  $\text{m/s}$ . Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity  $450 \text{ m/s}$ ). 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	0.162	0.261
Sa(0.5)	0.036	0.111	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

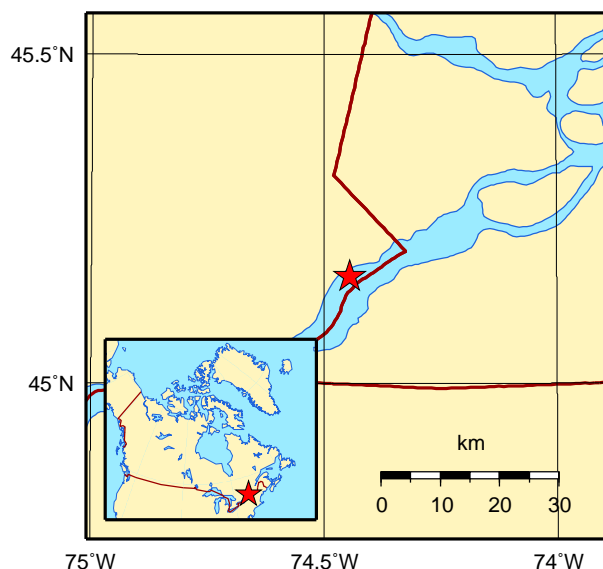
**National Building Code of Canada 2015 NRCC no. 56190;**  
**Appendix C:** Table C-3, Seismic Design Data for Selected Locations in Canada

**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](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information

Aussi disponible en français



Natural Resources  
Canada

Ressources naturelles  
Canada

Canada



Title: Highway 401 Underpass at Westley Road - North Abutment  
Comments: Embankment Stability Assessment  
Name: 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: (19.1, 69) w/ Radius: 21.6 m  
FoS Contours: 1.3 to 2.3, ++0.1

FILL (Existing)	19 kN/m³	0 kPa	32 °
CLAY_1 (ESA)	17 kN/m³	4 kPa	27 °
CLAY_2 (ESA)	15.5 kN/m³	4 kPa	27 °
Silty SAND	20 kN/m³	0 kPa	34 °
Silty SAND (TILL)	21 kN/m³	0 kPa	38 °

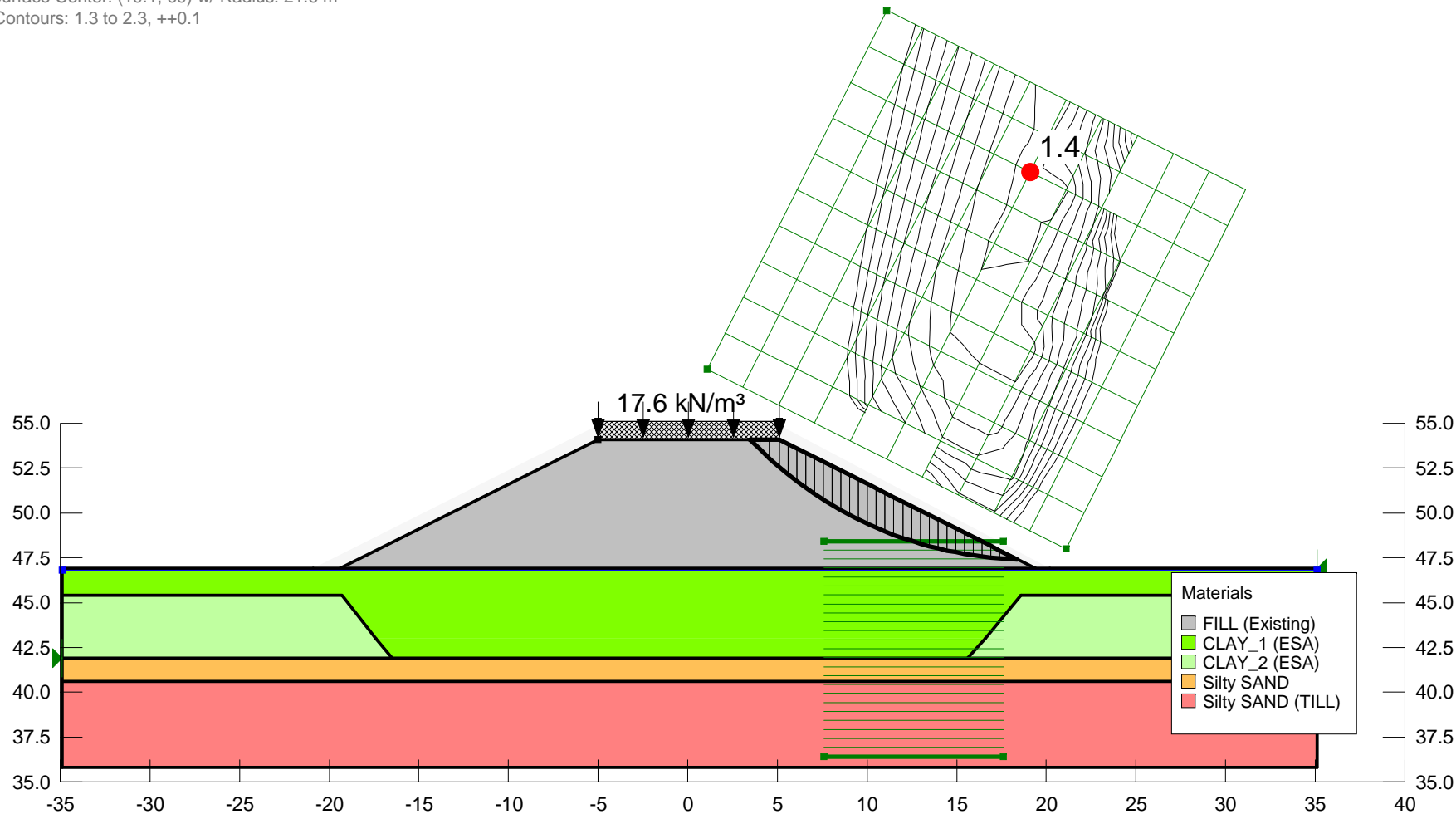


Figure 1

Title: Highway 401 Underpass at Westley Road - North Abutment  
Comments: Embankment Stability Assessment  
Name: 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: (20.1, 71) w/ Radius: 23.6 m  
FoS Contours: 1.3 to 2.3, ++0.1

FILL (Existing)	19 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	40 kPa	0 °
Silty SAND	20 kN/m³	0 kPa	34 °
Silty SAND (TILL)	21 kN/m³	0 kPa	38 °
CLAY_3 (TSA)	15.5 kN/m³	25 kPa	0 °

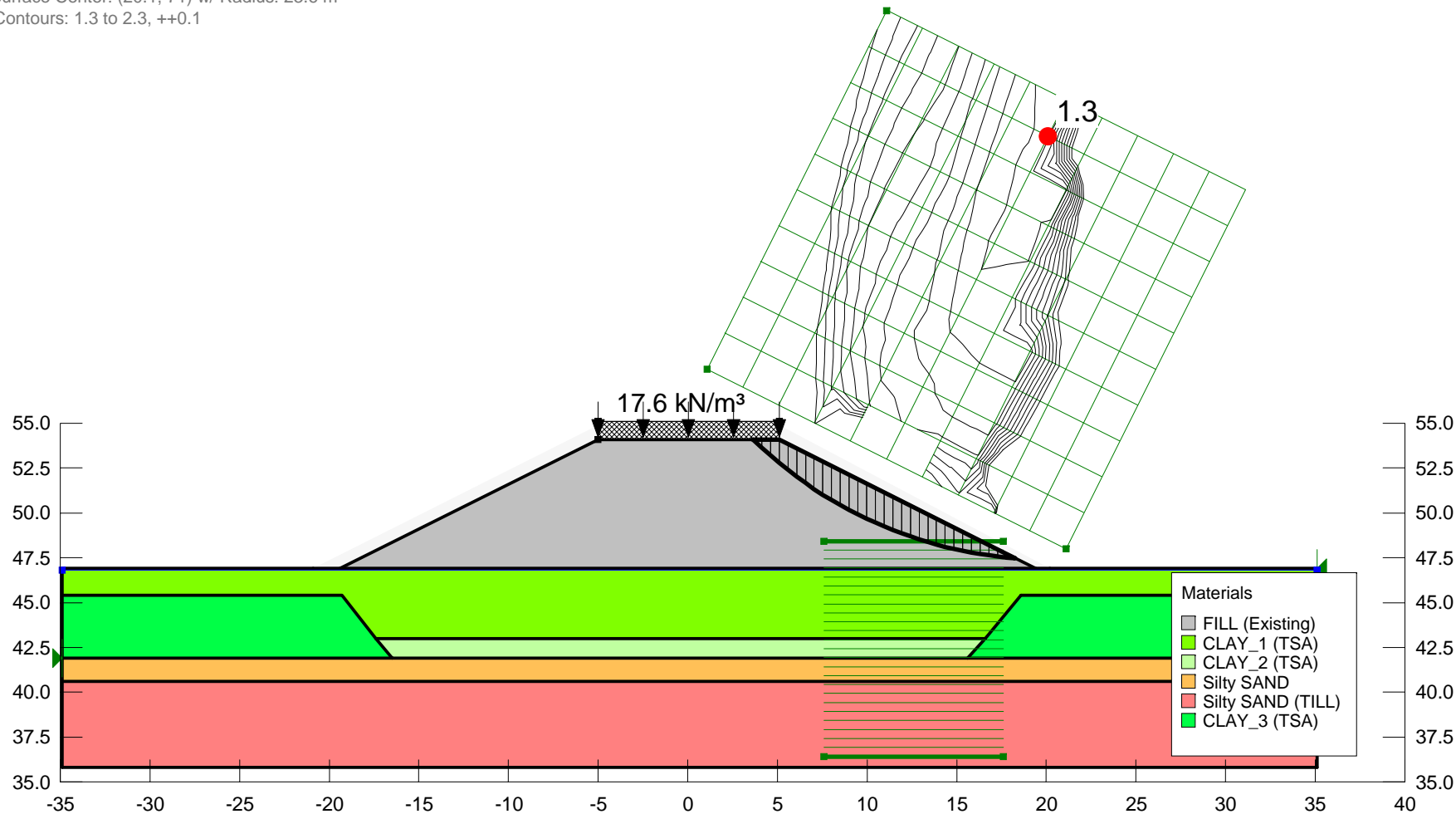


Figure 2

# Title: Highway 401 Underpass at Westley Road - North Abutment

Comments: Embankment Stability Assessment

Name: 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: (20.1, 71) w/ Radius: 23.6 m  
FoS Contours: 0.9 to 1.9, ++0.1

FILL (Existing)	19 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	40 kPa	0 °
Silty SAND	20 kN/m³	0 kPa	34 °
Silty SAND (TILL)	21 kN/m³	0 kPa	38 °
CLAY_3 (TSA)	15.5 kN/m³	25 kPa	0 °

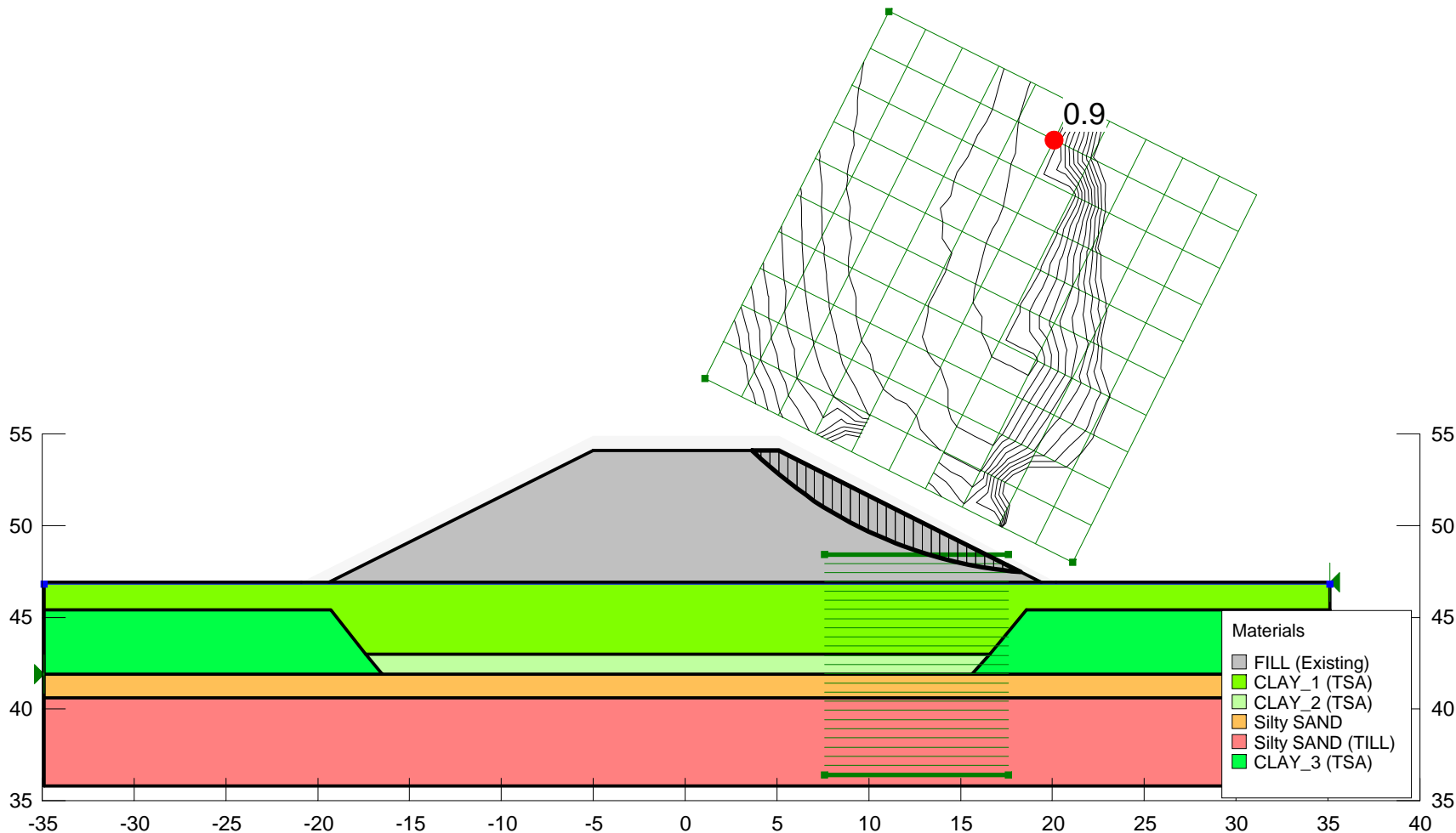


Figure 3

Title: Highwaya 401 Underpass at Westley Road - North Abutment  
Comments: Embankment Stability Assessment  
Name: 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 \ \ 0  
Slip Surface Center: (19.1, 69) w/ Radius: 21.6 m  
FoS Contours: 1.3 to 2.3, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	19 kN/m³	0 kPa	32 °
CLAY_1 (ESA)	17 kN/m³	4 kPa	27 °
CLAY_2 (ESA)	15.5 kN/m³	4 kPa	27 °
Silty SAND	20 kN/m³	0 kPa	34 °
Silty SAND (TILL)	21 kN/m³	0 kPa	38 °

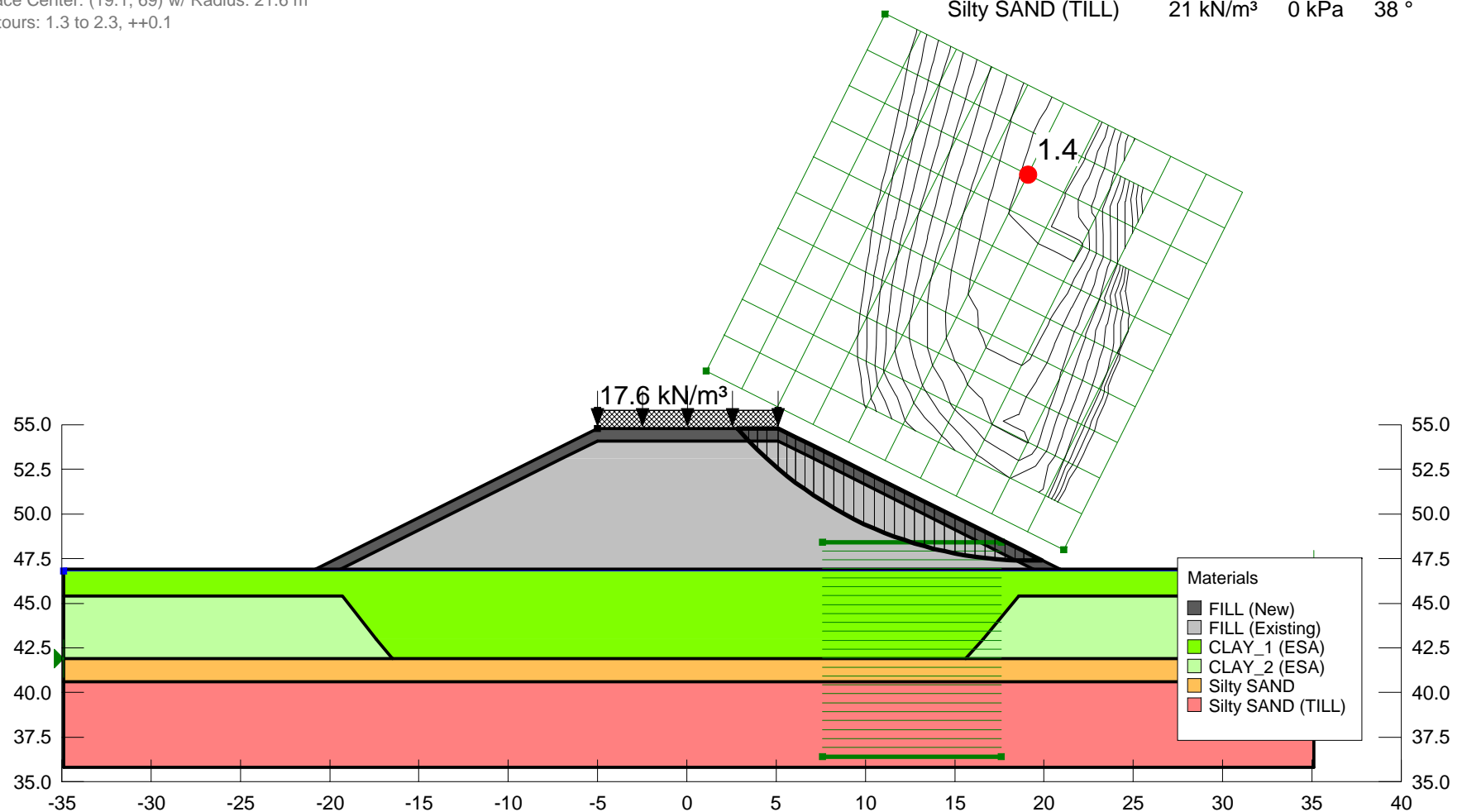


Figure 4

Title: Hwy401 Underpass at Westley Road - North Abutment  
Comments: Embankment Stability Assessment  
Name: 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: (22.1, 70) w/ Radius: 23.1 m  
FoS Contours: 1.3 to 2.3, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	19 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	40 kPa	0 °
Silty SAND	20 kN/m³	0 kPa	34 °
Silty SAND (TILL)	21 kN/m³	0 kPa	38 °
CLAY_3 (TSA)	15.5 kN/m³	25 kPa	0 °

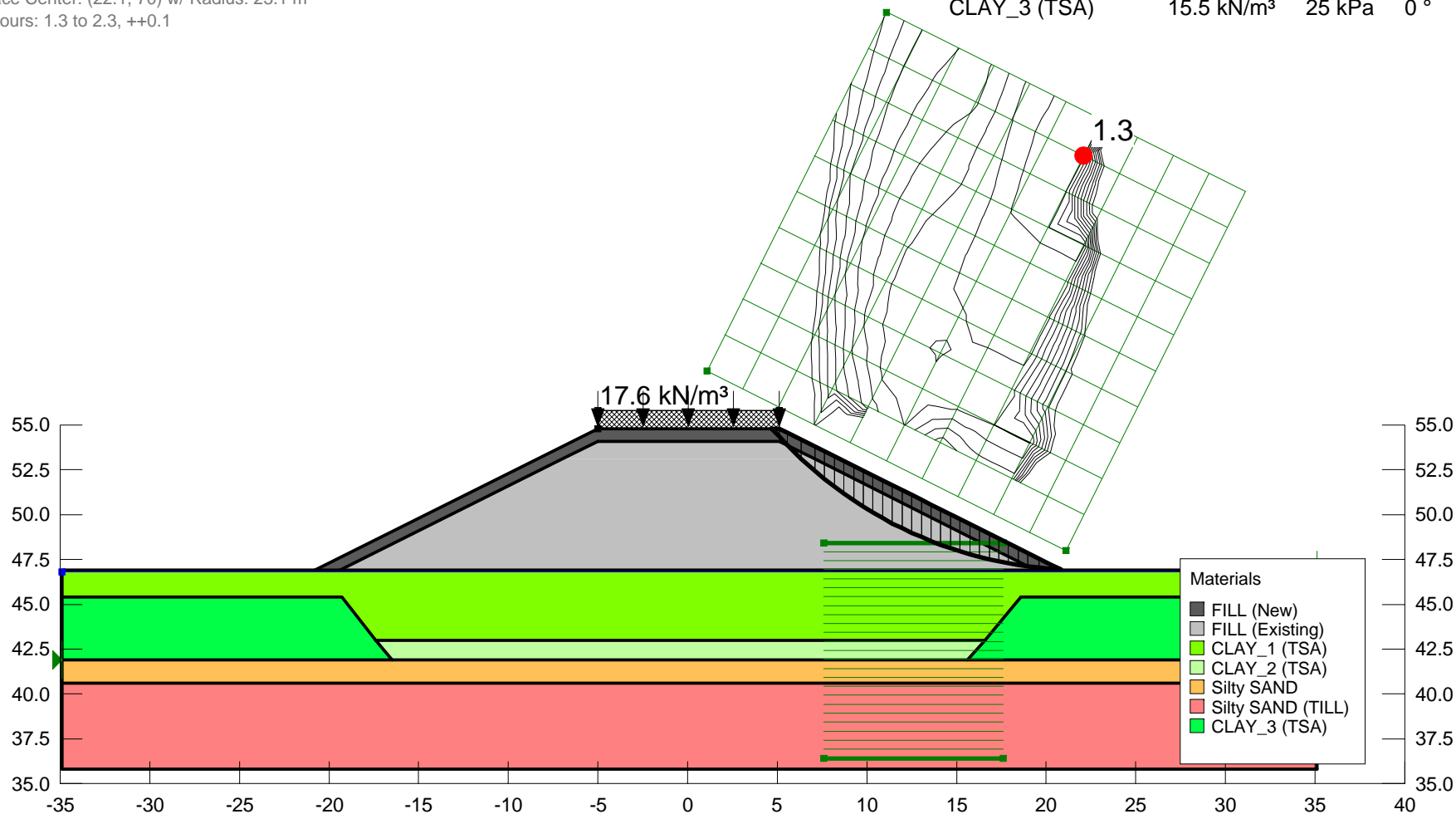


Figure 5

Title: Hwy401 Underpass at Westley Road - North Abutment  
Comments: Embankment Stability Assessment  
Name: 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: (20.1, 71) w/ Radius: 23.6 m  
FoS Contours: 0.9 to 1.9, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	19 kN/m³	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m³	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	40 kPa	0 °
Silty SAND	20 kN/m³	0 kPa	34 °
Silty SAND (TILL)	21 kN/m³	0 kPa	38 °
CLAY_3 (TSA)	15.5 kN/m³	25 kPa	0 °

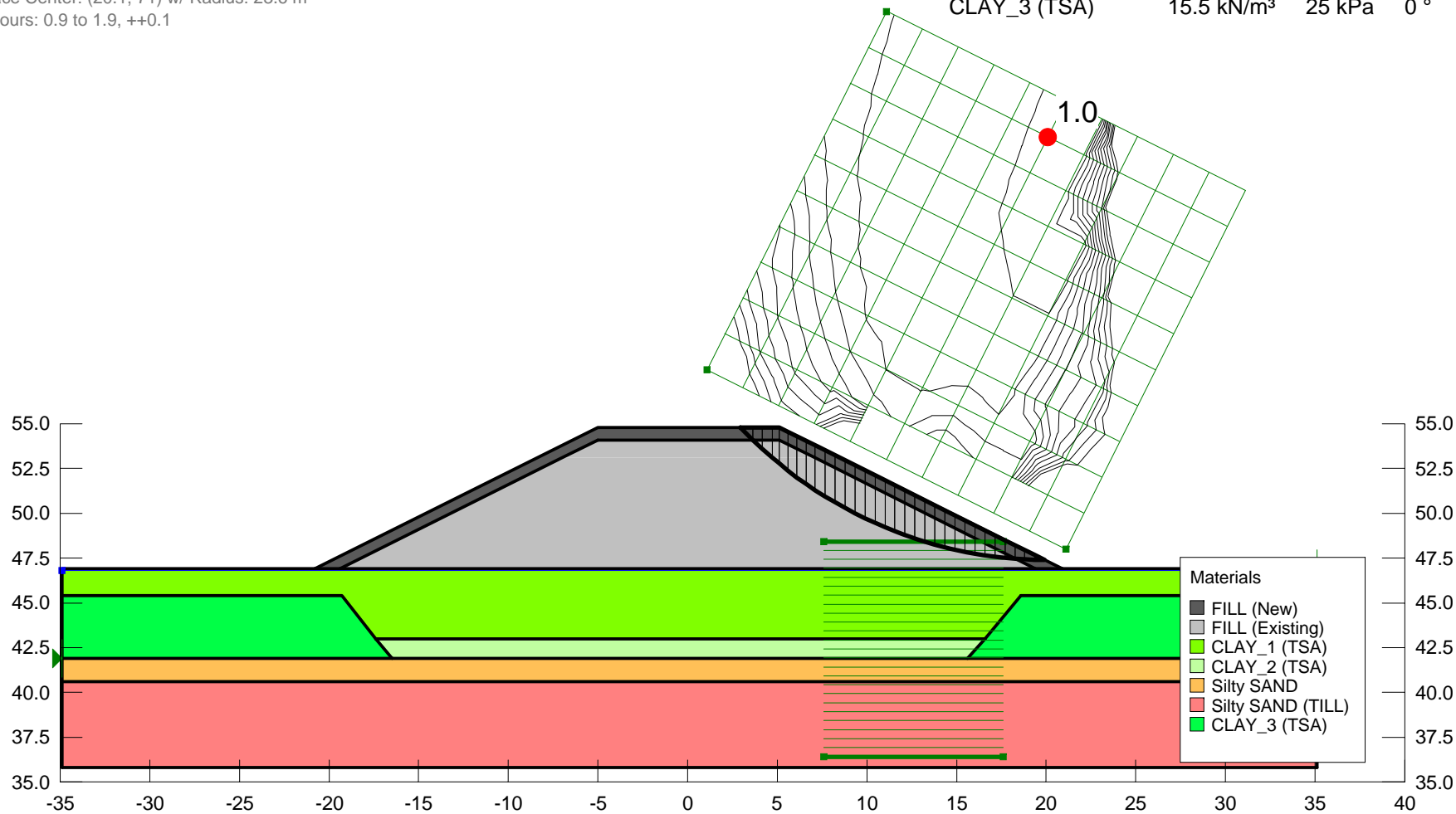


Figure 6

Title: Highwaya 401 Underpass at Westley Road - North Abutment

Comments: Embankment Stability Assessment

Name: 1m thick 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 \ 0  
Slip Surface Center: (20.1, 71) w/ Radius: 23.6 m  
FoS Contours: 1.2 to 2.2, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	19 kN/m <sup>3</sup>	0 kPa	32 °
EPS	1 kN/m <sup>3</sup>	1 kPa	0 °
CLAY_1 (ESA)	17 kN/m <sup>3</sup>	4 kPa	27 °
CLAY_2 (ESA)	15.5 kN/m <sup>3</sup>	4 kPa	27 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
Silty SAND (TILL)	21 kN/m <sup>3</sup>	0 kPa	38 °

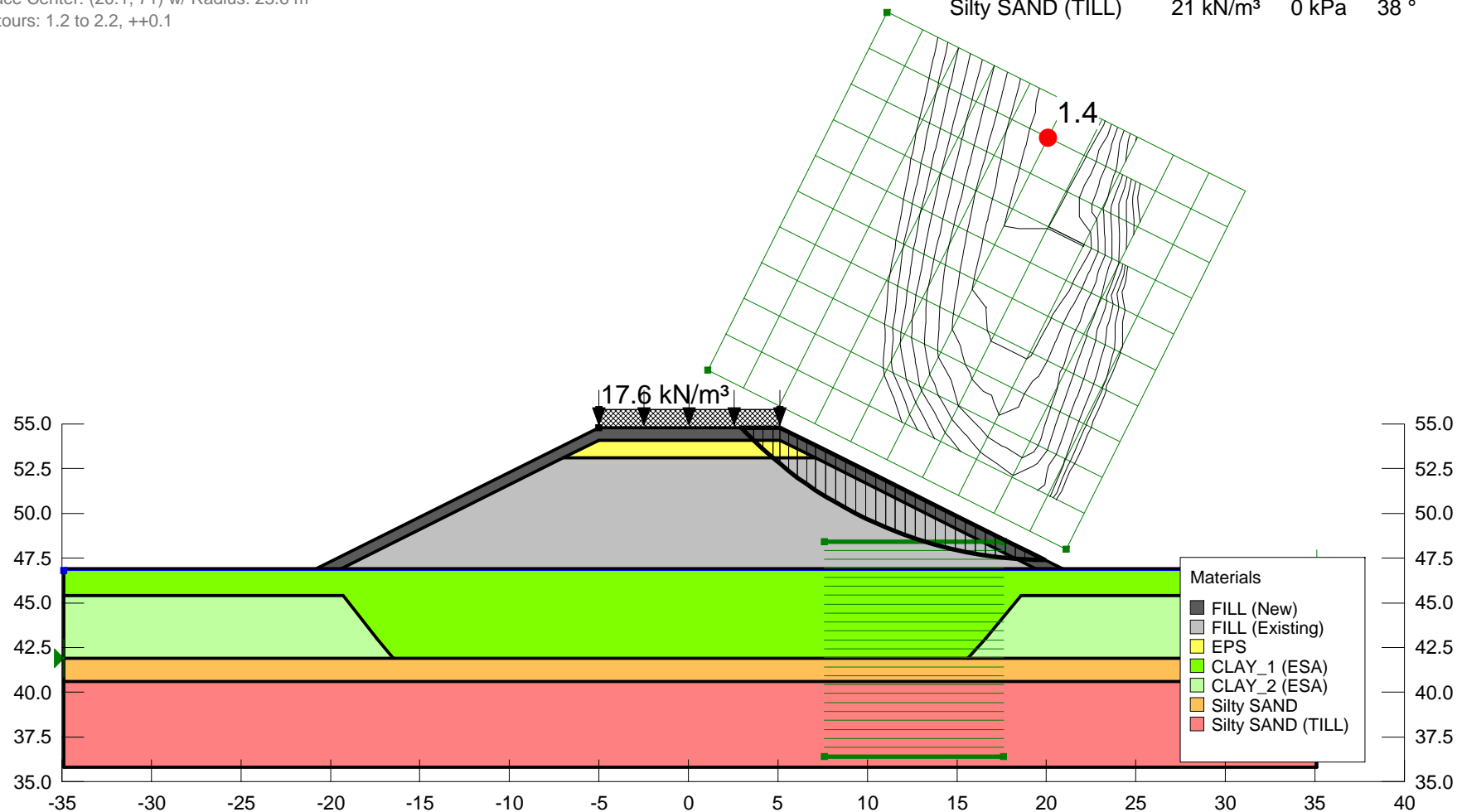


Figure 7

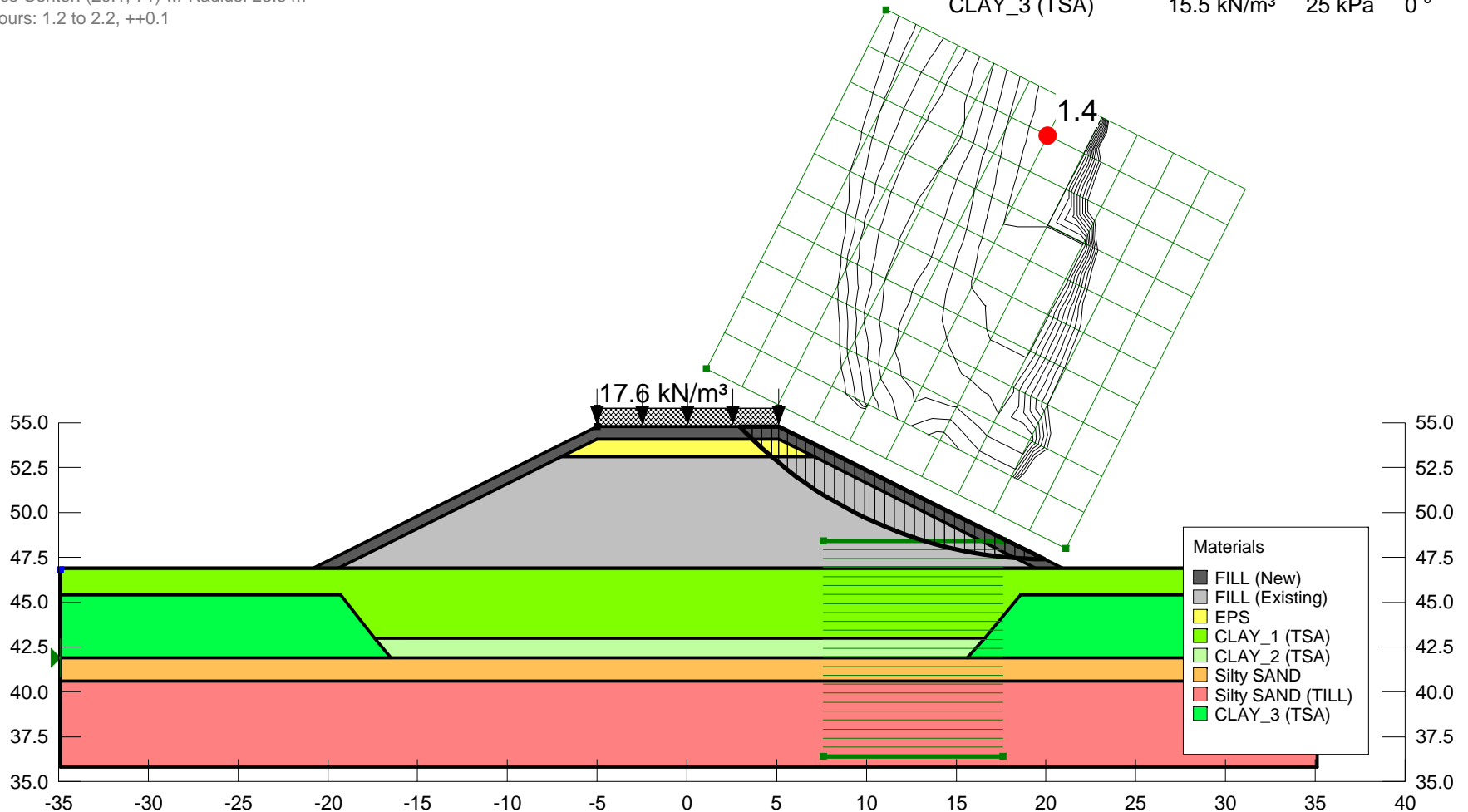
# Title: Hwy401 Underpass at Westley Road - North Abutment

## Comments: Embankment Stability Assessment

### Name: 1m thick 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: (20.1, 71) w/ Radius: 23.6 m  
FoS Contours: 1.2 to 2.2, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	19 kN/m <sup>3</sup>	0 kPa	32 °
EPS	1 kN/m <sup>3</sup>	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m <sup>3</sup>	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m <sup>3</sup>	40 kPa	0 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
Silty SAND (TILL)	21 kN/m <sup>3</sup>	0 kPa	38 °
CLAY_3 (TSA)	15.5 kN/m <sup>3</sup>	25 kPa	0 °



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Figure 8



Title: Hwy401 Underpass at Westley Road - North Abutment  
Comments: Embankment Stability Assessment  
Name: 1m thick 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: (18.1, 67) w/ Radius: 19.1 m  
FoS Contours: 0.8 to 1.8, ++0.1

FILL (New)	20 kN/m³	0 kPa	32 °
FILL (Existing)	19 kN/m³	0 kPa	32 °
EPS	1 kN/m³	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m³	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m³	40 kPa	0 °
Silty SAND	20 kN/m³	0 kPa	34 °
Silty SAND (TILL)	21 kN/m³	0 kPa	38 °
CLAY_3 (TSA)	15.5 kN/m³	25 kPa	0 °

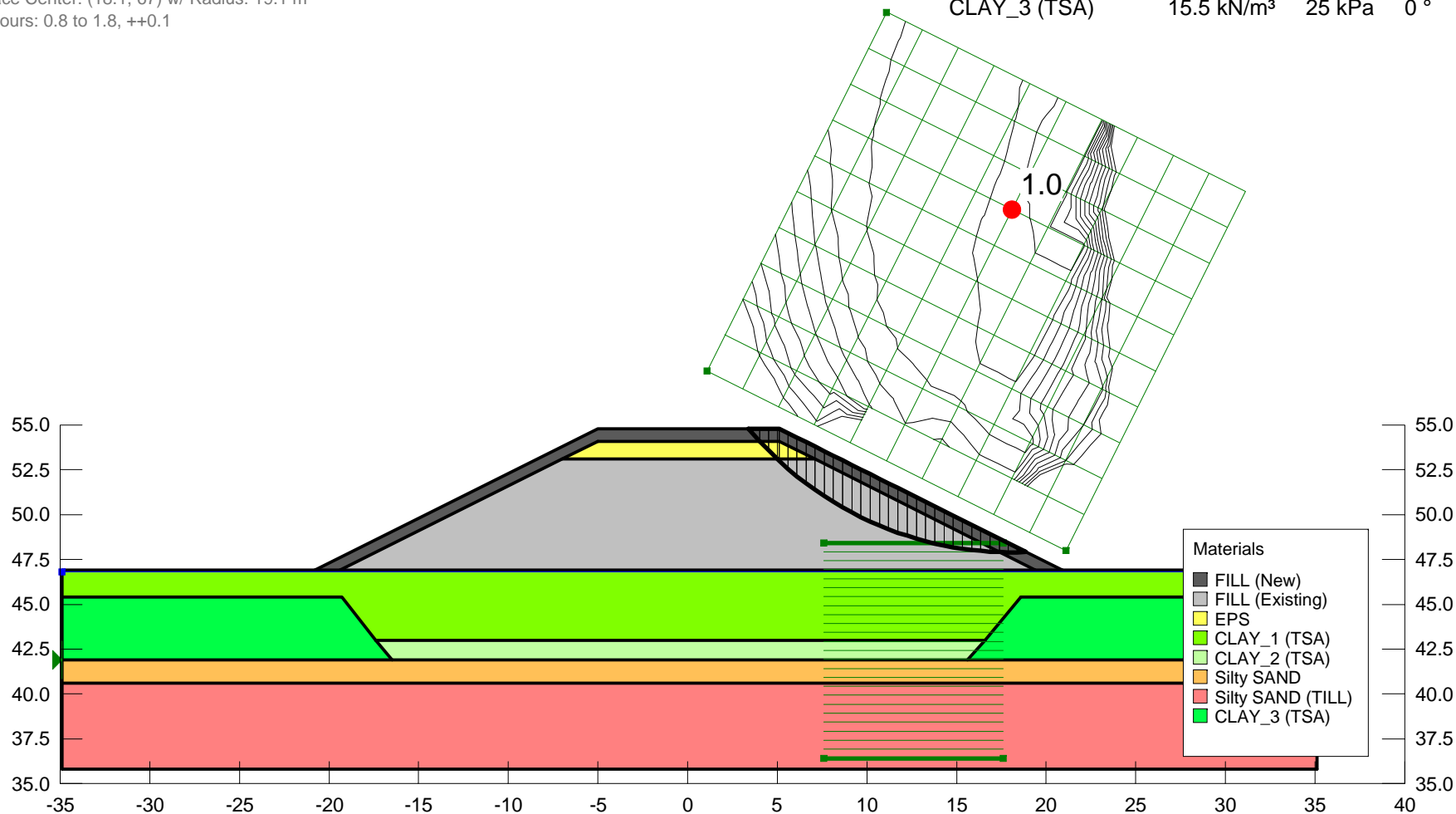


Figure 9

# Title: Highway 401 Underpass at Westley Road - South Abutment

Comments: Embankment Stability Assessment

Name: Existing Embankment Drained Static

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

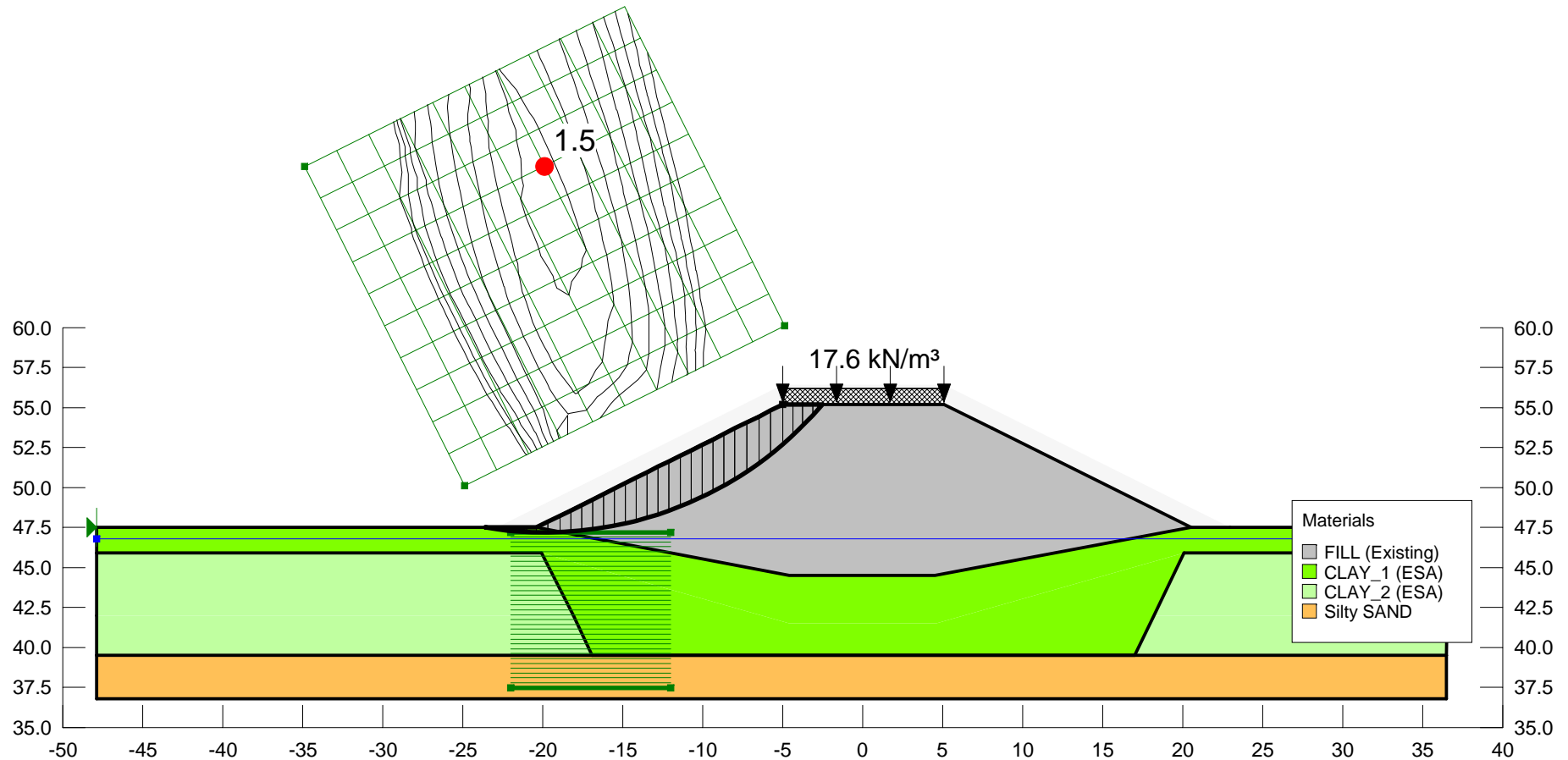
PWP Conditions Source: Piezometric Line

Seismic: H\0 V\0

Slip Surface Center: (-19.9, 70.1) w/ Radius: 22.89842 m

FoS Contours: 1.3 to 2.3, ++0.1

FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
CLAY_1 (ESA)	17 kN/m <sup>3</sup>	4 kPa	27 °
CLAY_2 (ESA)	15.5 kN/m <sup>3</sup>	0 kPa	27 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °



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Last Solved Date: 17/06/2016, 9:53:34 AM

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Figure 10

# Title: Highway 401 Underpass at Westley Road - South Abutment

Comments: Embankment Stability Assessment

Name: Existing Embankment Undrained Static

FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m <sup>3</sup>	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m <sup>3</sup>	40 kPa	0 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
CLAY_4 (TSA)	15.5 kN/m <sup>3</sup>	30 kPa	0 °
CLAY_3 (TSA)	15.5 kN/m <sup>3</sup>	20 kPa	0 °

Method: Morgenstern-Price, Half-Sine

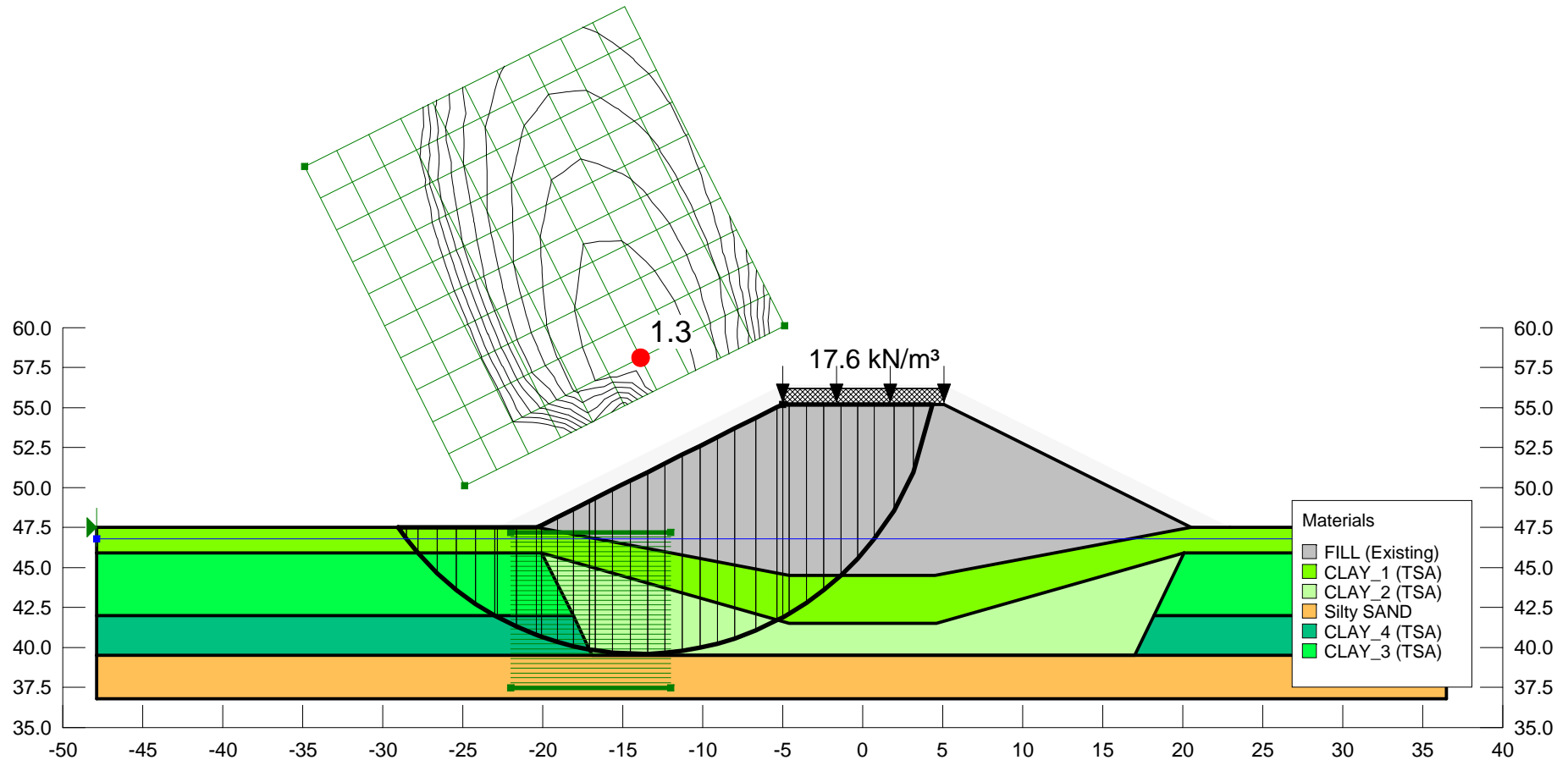
Minimum Slip Surface Depth: 1.52 m

PWP Conditions Source: Piezometric Line

Seismic: H\0 V\0

Slip Surface Center: (-13.9, 58.1) w/ Radius: 18.486936 m

FoS Contours: 1.2 to 2.2, ++0.1



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Last Solved Date: 17/06/2016, 9:53:32 AM

Directory: H:\Projects\19\5161\263 - ER Mega 5\Bridges\Site 31-233-Westley Rd Underpass\Foundations\Analysis\Stability\06 - print for report\Westley\_S10 existing ESA - TSA.gsz

Figure 11

# Title: Highway 401 Underpass at Westley Road - South Abutment

## Comments: Embankment Stability Assessment

### Name: Existing Embankment Seismic

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

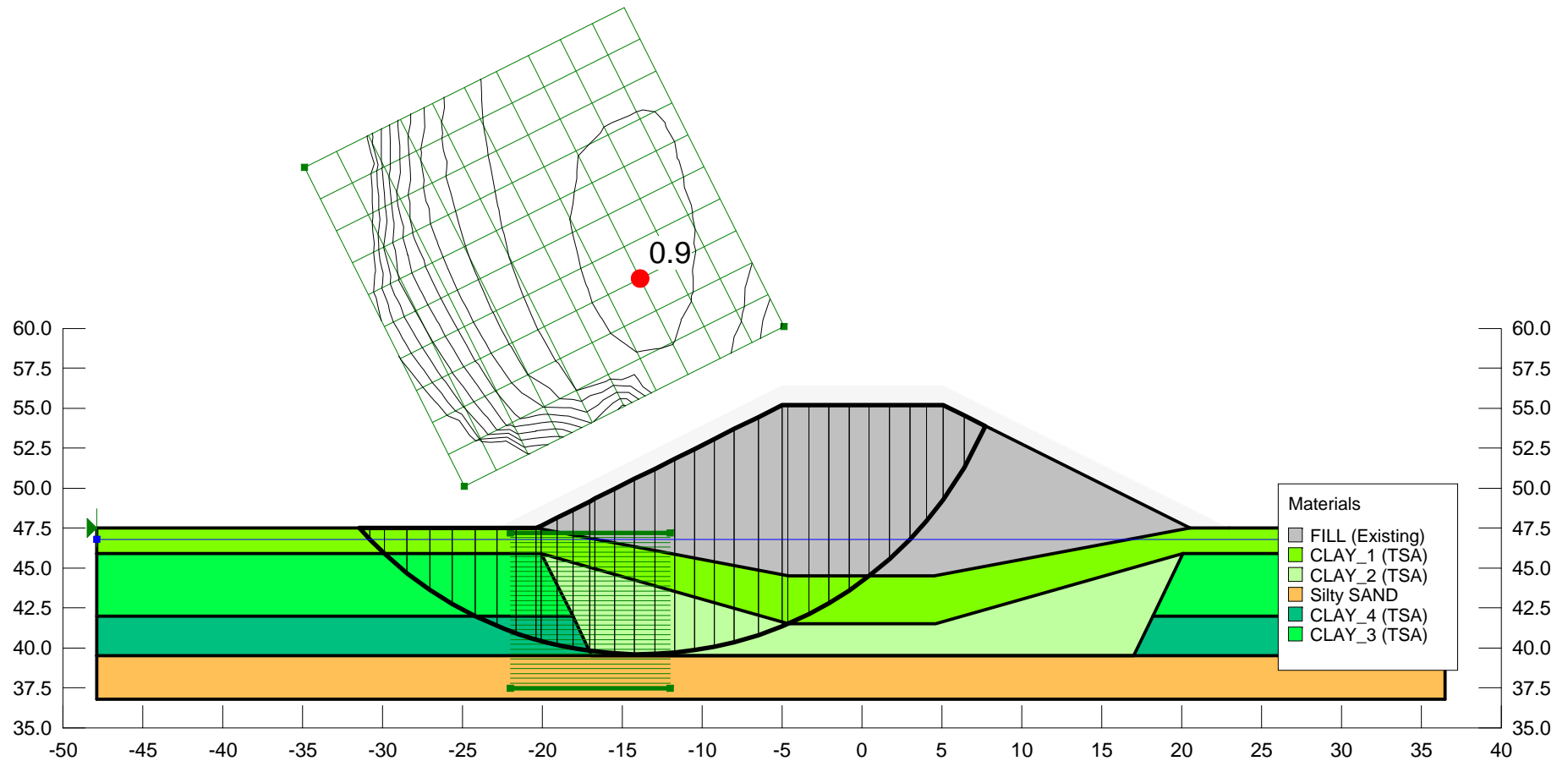
PWP Conditions Source: Piezometric Line

Seismic: H\ 0.19 V\ 0

Slip Surface Center: (-13.9, 63.1) w/ Radius: 23.486936 m

FoS Contours: 0.8 to 1.8, ++0.1

FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m <sup>3</sup>	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m <sup>3</sup>	40 kPa	0 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
CLAY_4 (TSA)	15.5 kN/m <sup>3</sup>	30 kPa	0 °
CLAY_3 (TSA)	15.5 kN/m <sup>3</sup>	20 kPa	0 °



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Tool Version: 8.15.5.11777

Last Solved Date: 17/06/2016, 10:04:35 AM

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Figure 12

# Title: Highway 401 Underpass at Westley Road - South Abutment

Comments: Embankment Stability Assessment

Name: 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: (-21.9, 69.1) w/ Radius: 21.89842 m

FoS Contours: 1.4 to 2.4, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
CLAY_1 (ESA)	17 kN/m <sup>3</sup>	4 kPa	27 °
CLAY_2 (ESA)	15.5 kN/m <sup>3</sup>	0 kPa	27 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °

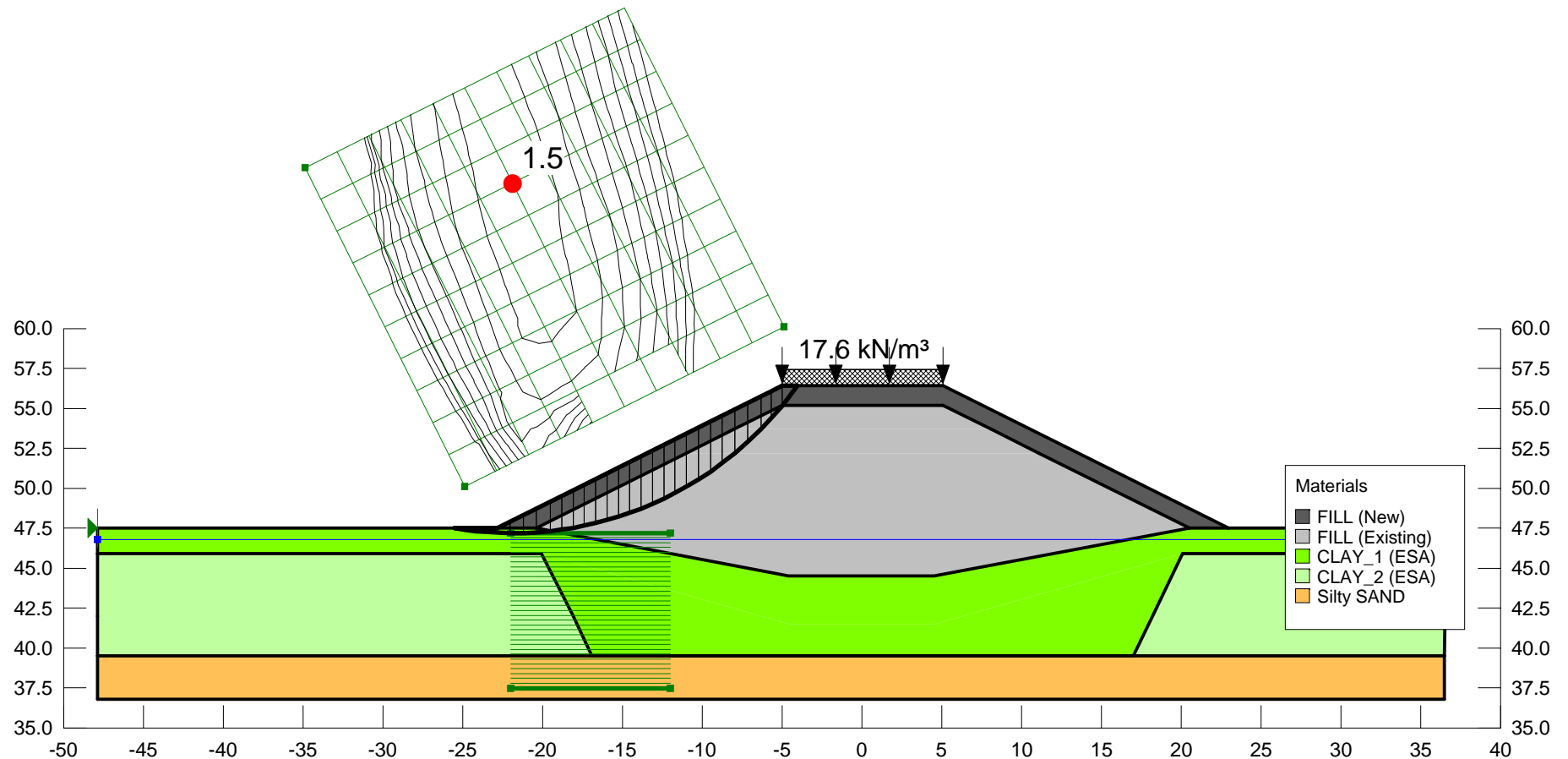


Figure 13

Reviewed By: \_\_\_\_\_

Tool Version: 8.15.5.11777

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# Title: Highway 401 Underpass at Westley Road - South Abutment

Comments: Embankment Stability Assessment

Name: 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: (-14.9, 60.1) w/ Radius: 20.486936 m

FoS Contours: 1.1 to 2.1, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m <sup>3</sup>	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m <sup>3</sup>	40 kPa	0 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
CLAY_4 (TSA)	15.5 kN/m <sup>3</sup>	30 kPa	0 °
CLAY_3 (TSA)	15.5 kN/m <sup>3</sup>	20 kPa	0 °

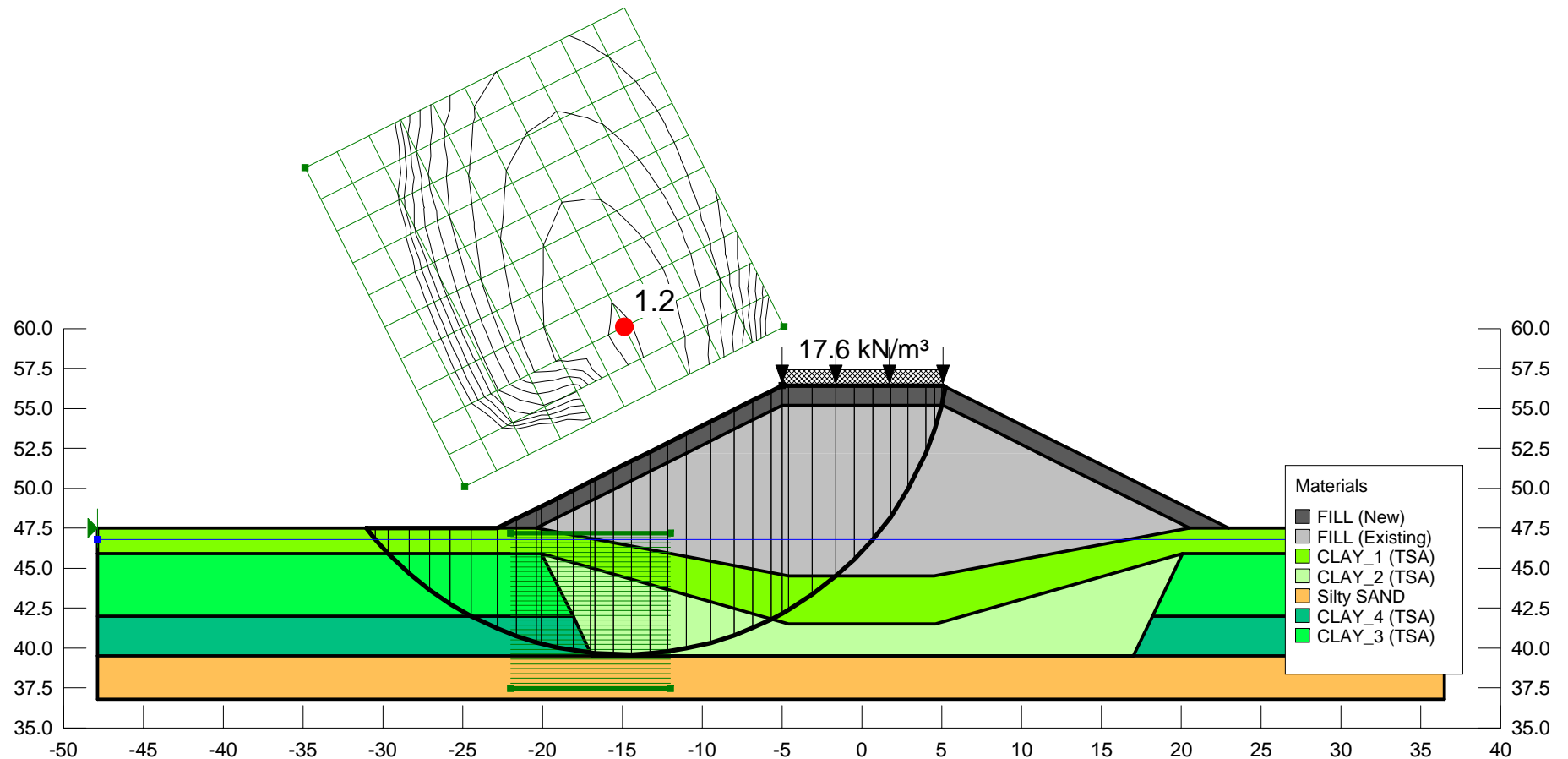


Figure 14

Reviewed By: \_\_\_\_\_

Tool Version: 8.15.5.11777

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# Title: Highway 401 Underpass at Westley Road - South Abutment

Comments: Embankment Stability Assessment

Name: Granular Grade Raise Seismic

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

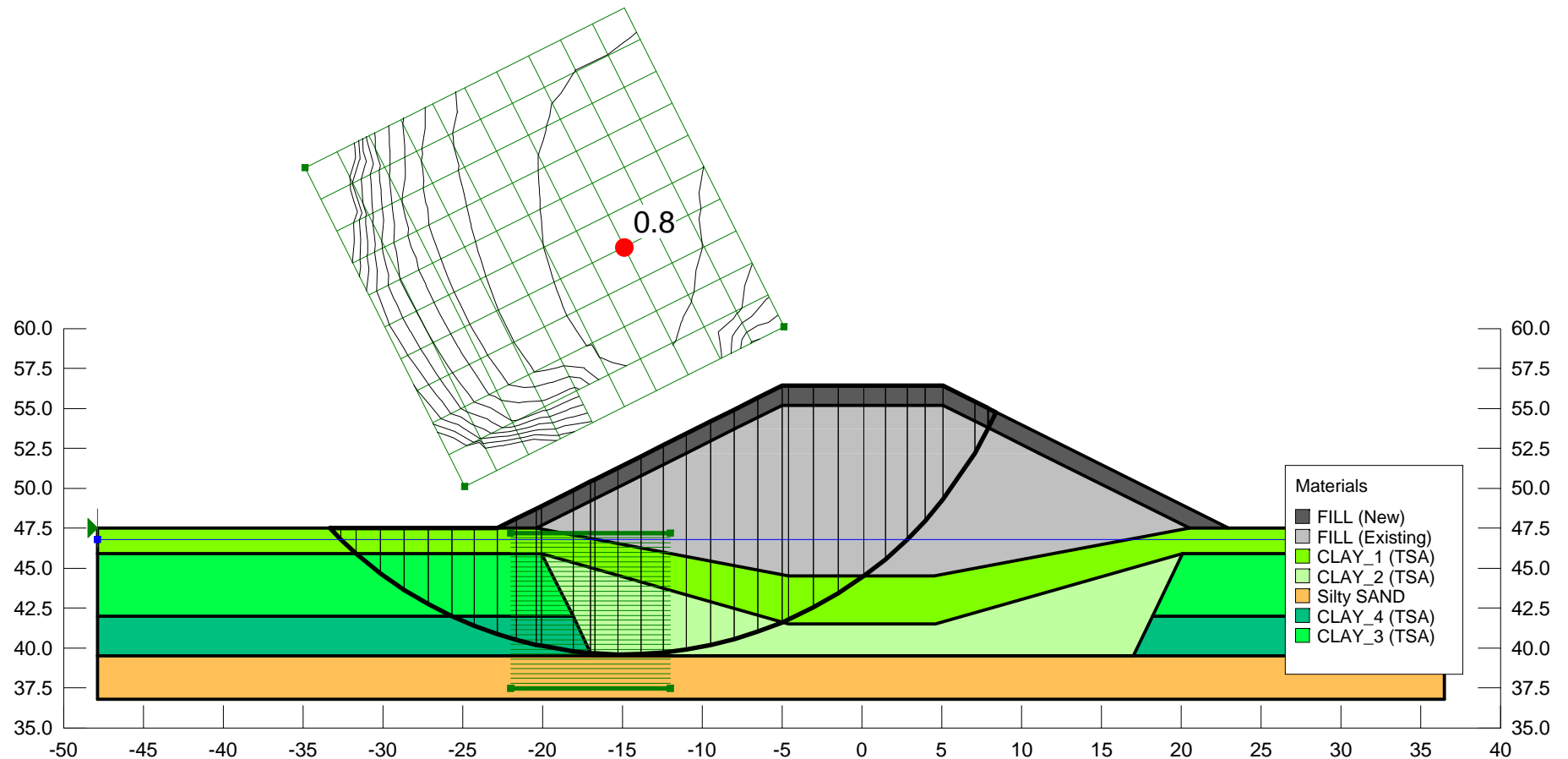
PWP Conditions Source: Piezometric Line

Seismic: H\ 0.19 V\ 0

Slip Surface Center: (-14.9, 65.1) w/ Radius: 25.486936 m

FoS Contours: 0.7 to 1.7, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
CLAY_1 (TSA)	17 kN/m <sup>3</sup>	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m <sup>3</sup>	40 kPa	0 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
CLAY_4 (TSA)	15.5 kN/m <sup>3</sup>	30 kPa	0 °
CLAY_3 (TSA)	15.5 kN/m <sup>3</sup>	20 kPa	0 °



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Tool Version: 8.15.5.11777

Last Solved Date: 17/06/2016, 11:11:26 AM

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Figure 15

# Title: Highway 401 Underpass at Westley Road - South Abutment

Comments: Embankment Stability Assessment

Name: 1.5m thick EPS Grade Raise Drained Static

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

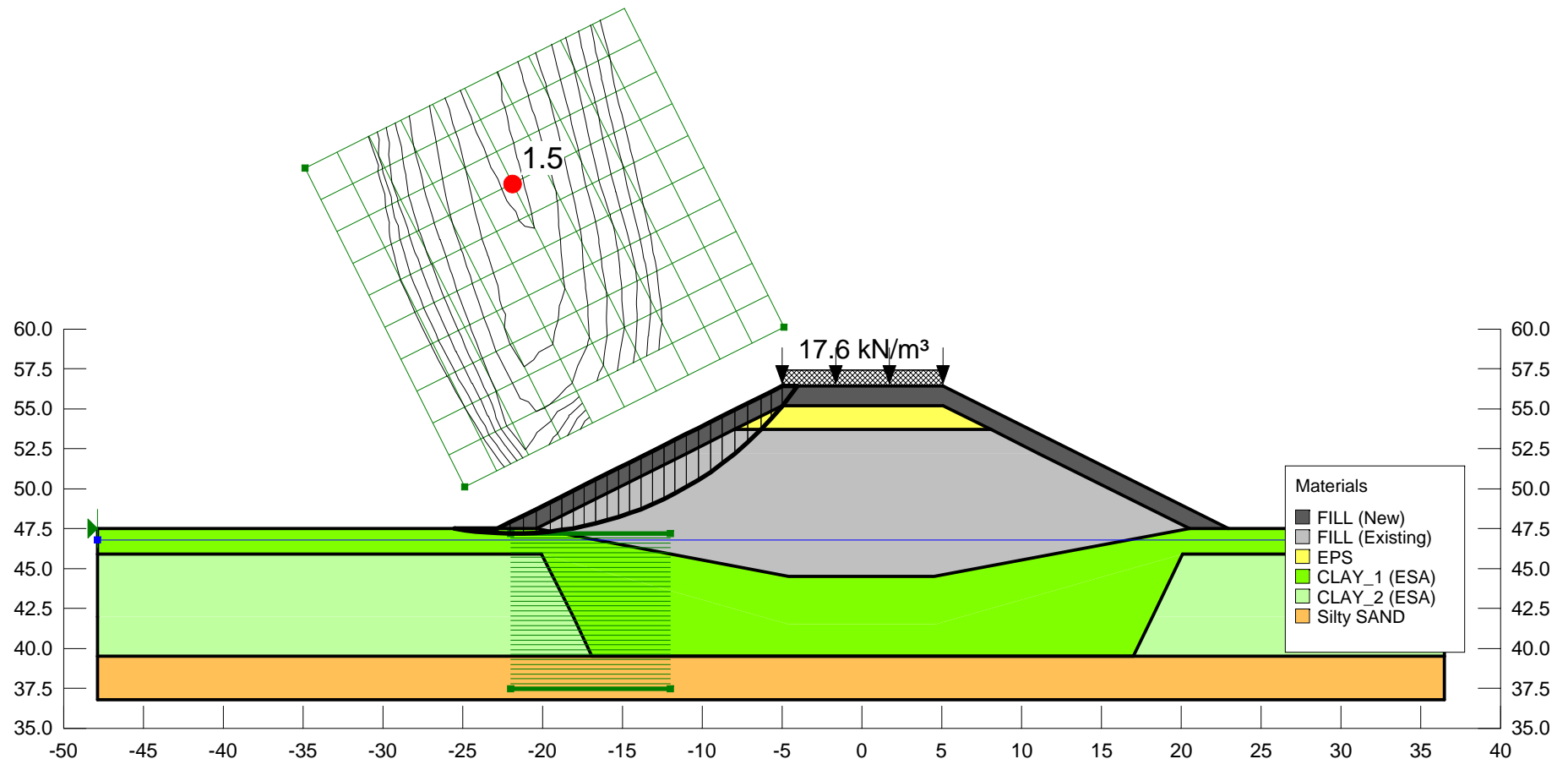
PWP Conditions Source: Piezometric Line

Seismic: H\0 V\0

Slip Surface Center: (-21.9, 69.1) w/ Radius: 21.89842 m

FoS Contours: 1.2 to 2.2, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
EPS	1 kN/m <sup>3</sup>	1 kPa	0 °
CLAY_1 (ESA)	17 kN/m <sup>3</sup>	4 kPa	27 °
CLAY_2 (ESA)	15.5 kN/m <sup>3</sup>	0 kPa	27 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °



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Tool Version: 8.15.5.11777

Last Solved Date: 17/06/2016, 10:24:31 AM

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Figure 16



# Title: Highway 401 Underpass at Westley Road - South Abutment

## Comments: Embankment Stability Assessment

### Name: 1.5m thick EPS Grade Raise Undrained Static

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

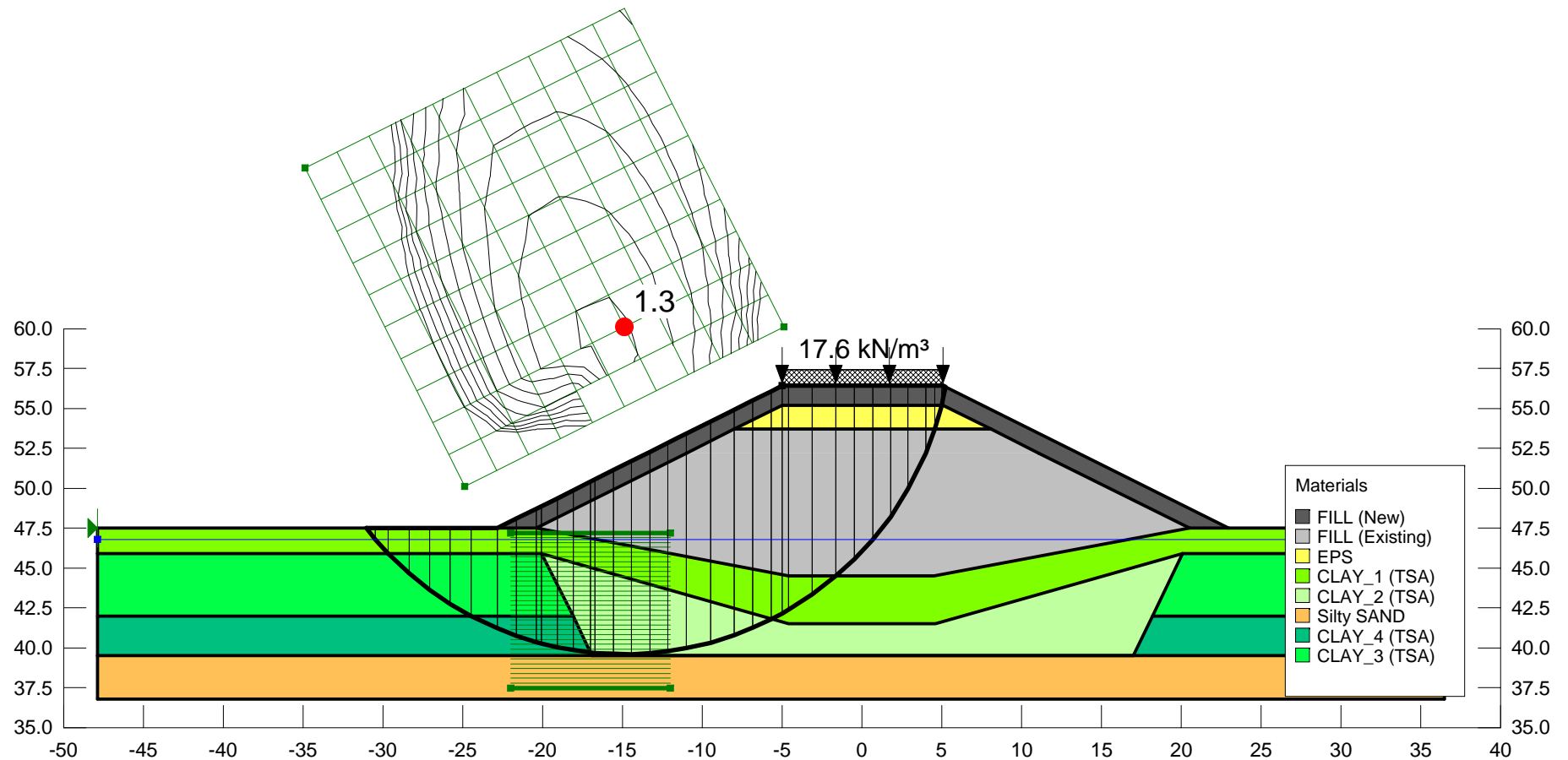
PWP Conditions Source: Piezometric Line

Seismic: H\0 V\0

Slip Surface Center: (-14.9, 60.1) w/ Radius: 20.486936 m

FoS Contours: 1.2 to 2.2, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
EPS	1 kN/m <sup>3</sup>	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m <sup>3</sup>	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m <sup>3</sup>	40 kPa	0 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
CLAY_4 (TSA)	15.5 kN/m <sup>3</sup>	30 kPa	0 °
CLAY_3 (TSA)	15.5 kN/m <sup>3</sup>	20 kPa	0 °



Reviewed By: \_\_\_\_\_

Tool Version: 8.15.5.11777

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Figure 17

# Title: Highway 401 Underpass at Westley Road - South Abutment

## Comments: Embankment Stability Assessment

### Name: 1.5m thick EPS Grade Raise Seismic

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

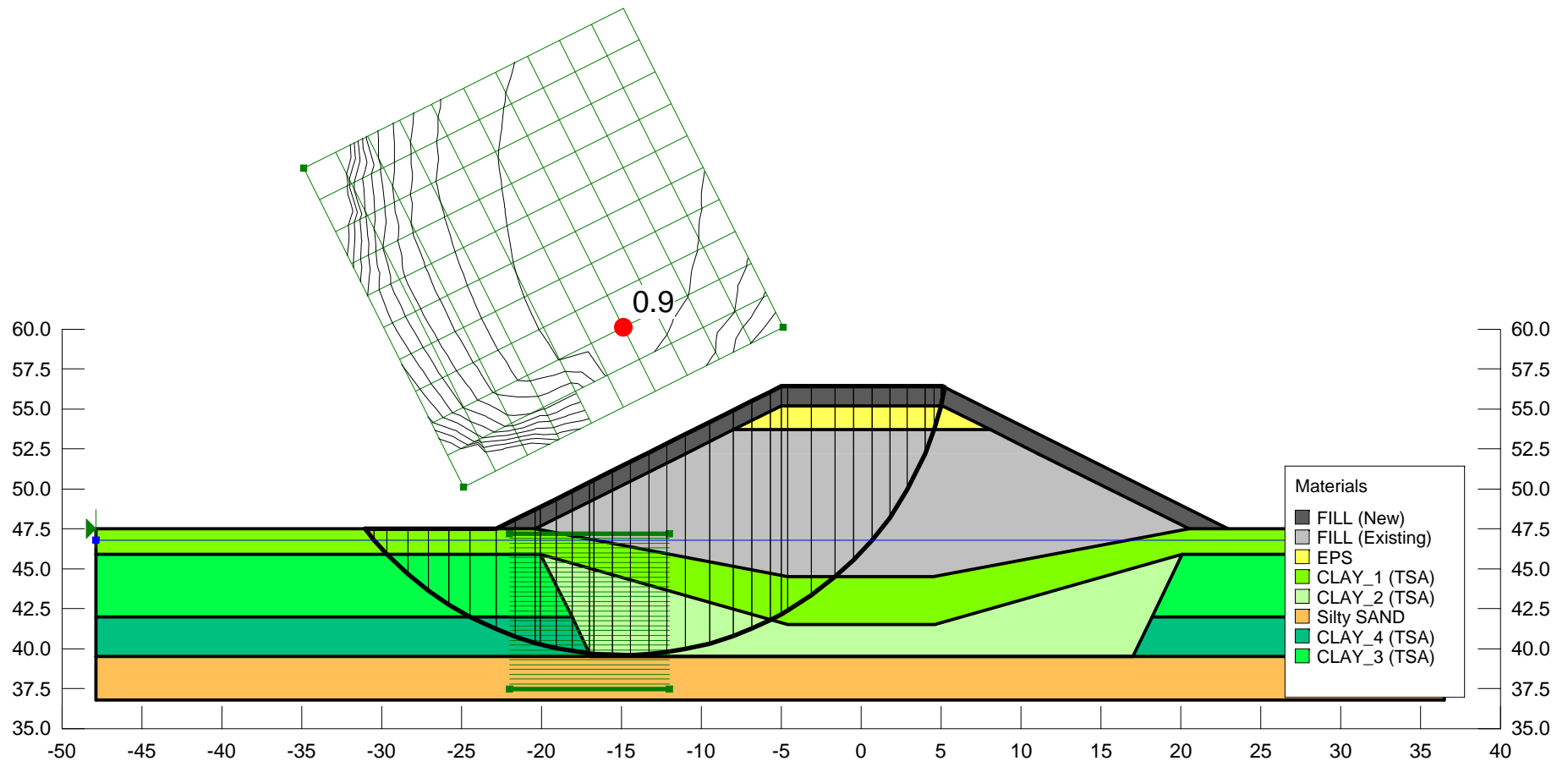
PWP Conditions Source: Piezometric Line

Seismic: H\ 0.19 V\ 0

Slip Surface Center: (-14.9, 60.1) w/ Radius: 20.486936 m

FoS Contours: 0.8 to 1.8, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
EPS	1 kN/m <sup>3</sup>	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m <sup>3</sup>	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m <sup>3</sup>	40 kPa	0 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
CLAY_4 (TSA)	15.5 kN/m <sup>3</sup>	30 kPa	0 °
CLAY_3 (TSA)	15.5 kN/m <sup>3</sup>	20 kPa	0 °



Reviewed By: \_\_\_\_\_

Tool Version: 8.15.5.11777

Last Solved Date: 17/06/2016, 10:24:44 AM

Directory: H:\Projects\19\5161\263 - ER Mega 5\Bridges\Site 31-233-Westley Rd Underpass\Foundations\Analysis\Stability\06 - print for report\Westley\_S16-21 - 1.5&3m EPS grade raise.gsz

Figure 18

# Title: Highway 401 Underpass at Westley Road - South Abutment

## Comments: Embankment Stability Assessment

### Name: 3.0m thick EPS Grade Raise Drained Static

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

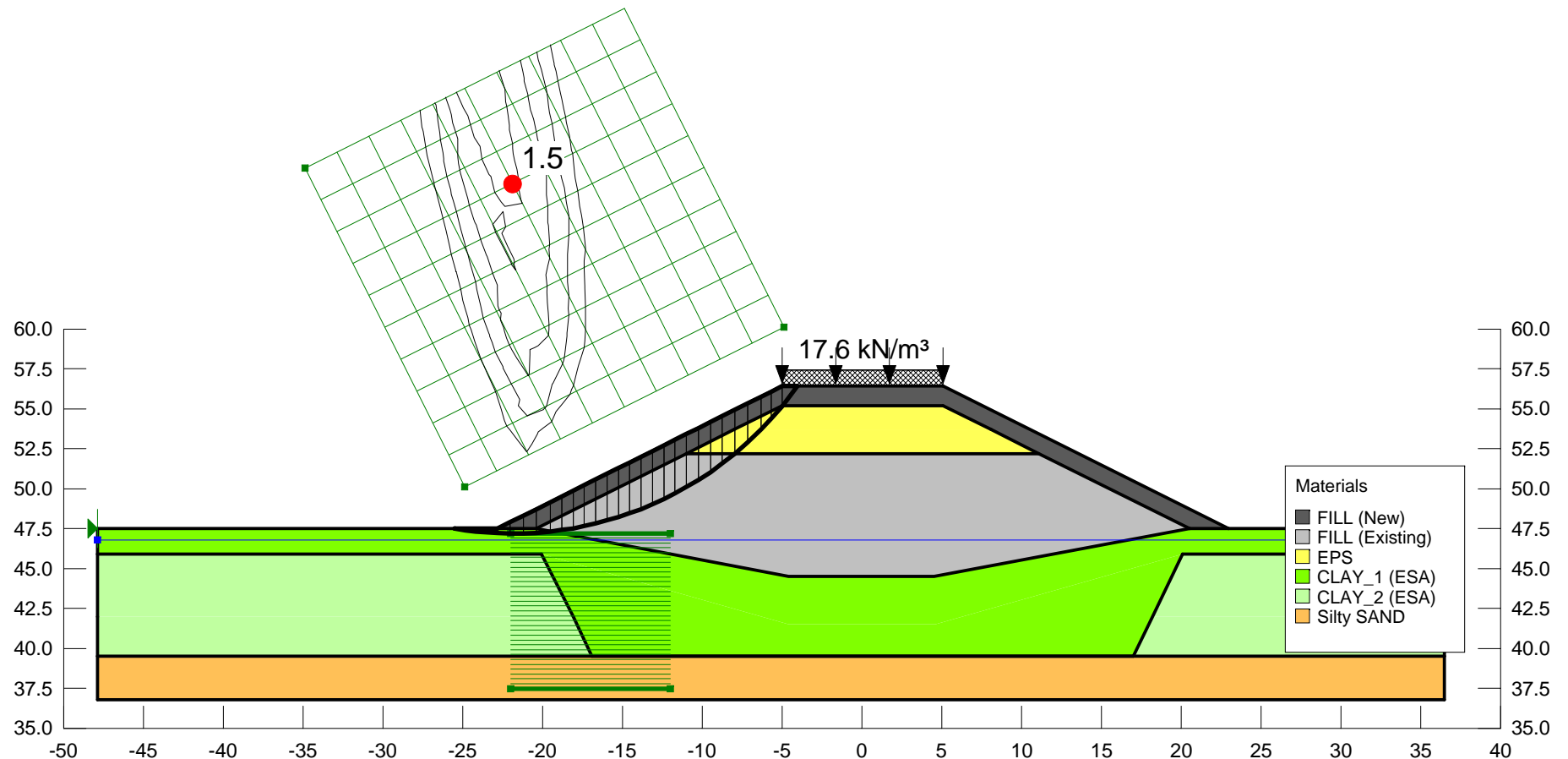
PWP Conditions Source: Piezometric Line

Seismic: H\0 V\0

Slip Surface Center: (-21.9, 69.1) w/ Radius: 21.89842 m

FoS Contours: 0.8 to 1.8, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
EPS	1 kN/m <sup>3</sup>	1 kPa	0 °
CLAY_1 (ESA)	17 kN/m <sup>3</sup>	4 kPa	27 °
CLAY_2 (ESA)	15.5 kN/m <sup>3</sup>	0 kPa	27 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °



Reviewed By: \_\_\_\_\_

Tool Version: 8.15.5.11777

Last Solved Date: 17/06/2016, 10:24:44 AM

Directory: H:\Projects\19\5161\263 - ER Mega 5\Bridges\Site 31-233-Westley Rd Underpass\Foundations\Analysis\Stability\06 - print for report\Westley\_S16-21 - 1.5&3m EPS grade raise.gsz

Figure 19

# Title: Highway 401 Underpass at Westley Road - South Abutment

## Comments: Embankment Stability Assessment

### Name: 3.0m thick 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: (-16.9, 59.1) w/ Radius: 19.486936 m

FoS Contours: 1.4 to 2.4, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
EPS	1 kN/m <sup>3</sup>	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m <sup>3</sup>	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m <sup>3</sup>	40 kPa	0 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
CLAY_4 (TSA)	15.5 kN/m <sup>3</sup>	30 kPa	0 °
CLAY_3 (TSA)	15.5 kN/m <sup>3</sup>	20 kPa	0 °

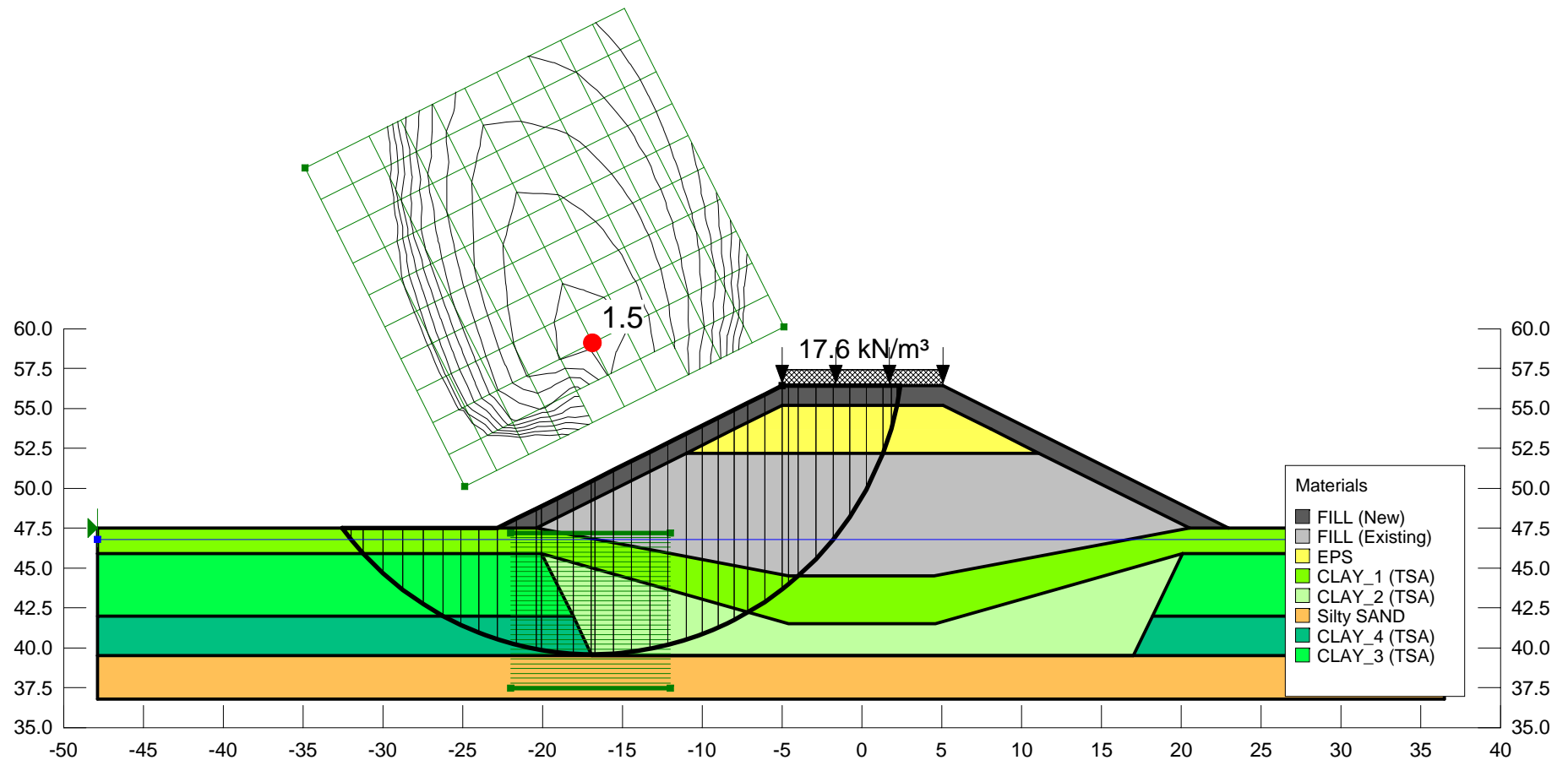


Figure 20

Reviewed By: \_\_\_\_\_

Tool Version: 8.15.5.11777

Last Solved Date: 17/06/2016, 10:24:57 AM

Directory: H:\Projects\19\5161\263 - ER Mega 5\Bridges\Site 31-233-Westley Rd Underpass\Foundations\Analysis\Stability\06 - print for report\Westley\_S16-21 - 1.5&3m EPS grade raise.gsz

# Title: Highway 401 Underpass at Westley Road - South Abutment

Comments: Embankment Stability Assessment

Name: 3.0m thick EPS Grade Raise Seismic

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

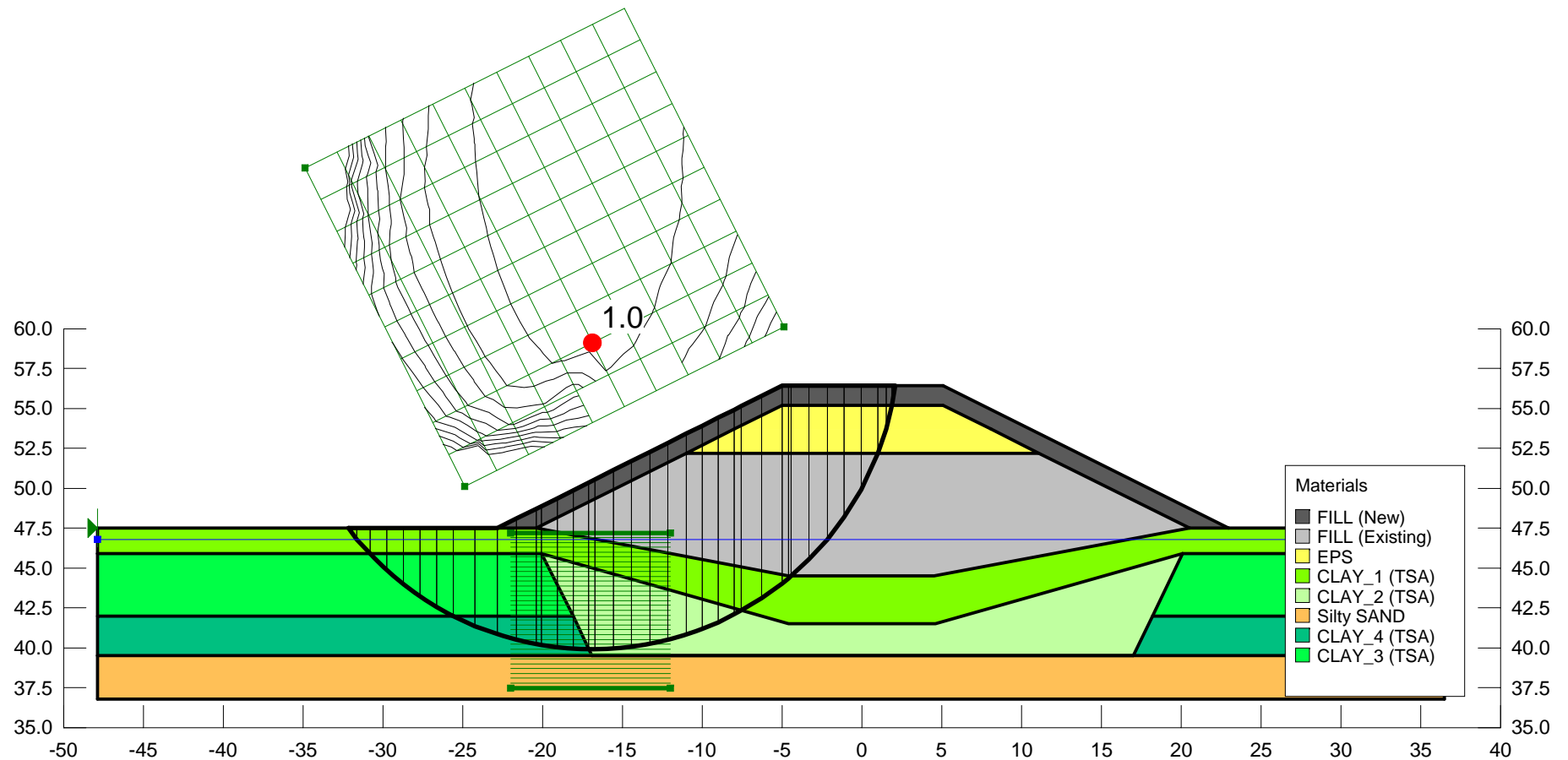
PWP Conditions Source: Piezometric Line

Seismic: H\ 0.19 V\ 0

Slip Surface Center: (-16.9, 59.1) w/ Radius: 19.183395 m

FoS Contours: 0.9 to 1.9, ++0.1

FILL (New)	20 kN/m <sup>3</sup>	0 kPa	32 °
FILL (Existing)	20 kN/m <sup>3</sup>	0 kPa	32 °
EPS	1 kN/m <sup>3</sup>	1 kPa	0 °
CLAY_1 (TSA)	17 kN/m <sup>3</sup>	60 kPa	0 °
CLAY_2 (TSA)	17 kN/m <sup>3</sup>	40 kPa	0 °
Silty SAND	20 kN/m <sup>3</sup>	0 kPa	34 °
CLAY_4 (TSA)	15.5 kN/m <sup>3</sup>	30 kPa	0 °
CLAY_3 (TSA)	15.5 kN/m <sup>3</sup>	20 kPa	0 °



Reviewed By: \_\_\_\_\_

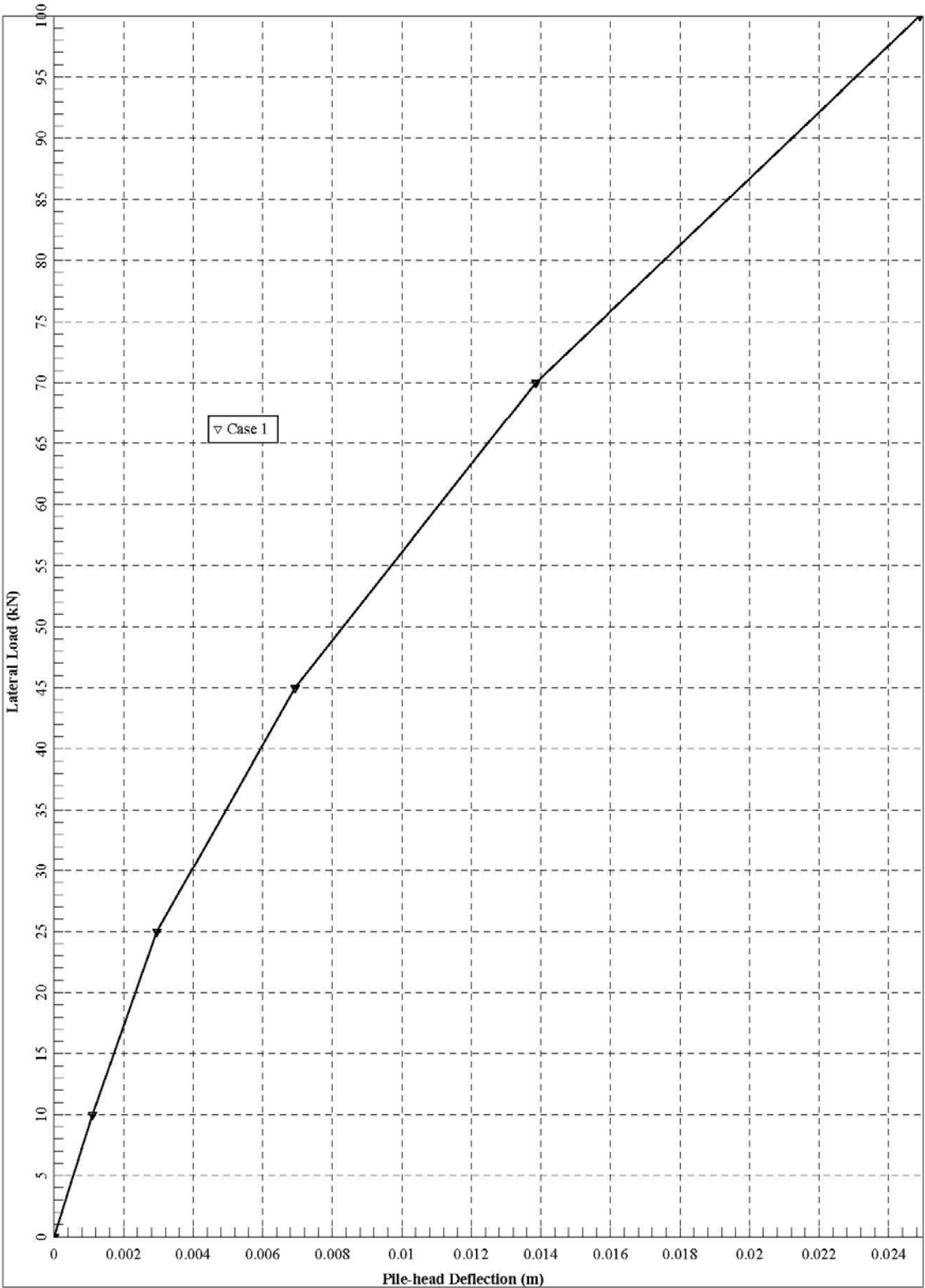
Tool Version: 8.15.5.11777

Last Solved Date: 17/06/2016, 10:24:58 AM

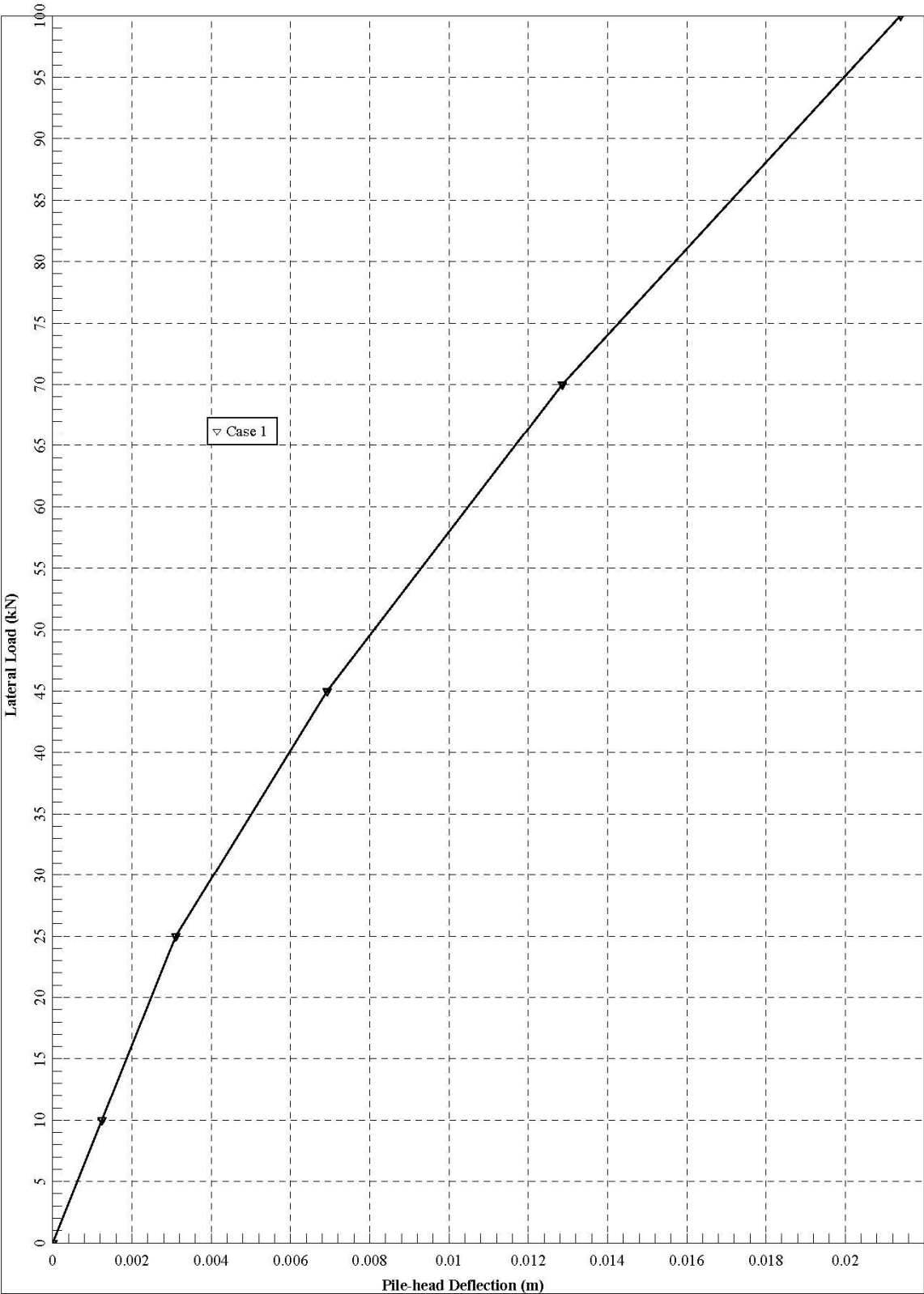
Directory: H:\Projects\19\5161\263 - ER Mega 5\Bridges\Site 31-233-Westley Rd Underpass\Foundations\Analysis\Stability\06 - print for report\Westley\_S16-21 - 1.5&3m EPS grade raise.gsz

Figure 21

Westley Road Lateral Load vs. Pile Head Deflection North Abutment



Westley Road Lateral Load vs. Pile Head Deflection South Abutment



## **APPENDIX H**

### **LIST OF REFERENCED SPECIFICATIONS**

**NON-STANDARD SPECIAL PROVISIONS - USE OF HEAVY CONSTRUCTION EQUIPMENT**

**NON-STANDARD SPECIAL PROVISIONS - EXPANDED POLYSTYRENE EMBANKMENT**

DRAFT



## **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 - Linear  - Flatness  - Squareness  - Thickness	mm	1200 x 600 x 300 with tolerances $\pm 1\%$  10 mm in 3 m  $\pm 0.5\%$  -3 to +5 mm	NA
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	m <sup>2</sup> .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  $\pm 1\%$ .

The flatness of the block faces shall be within  $\pm 10$  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 m<sup>2</sup>·°C/W for a 25 mm thickness using the following equation and using the average value from three specimens:

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

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

#### *7.2.2.6 Flammability*

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

#### *7.2.2.7 Water Absorption*

The water absorption as measured by ASTM D2842 shall be limited to 4% 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  $\pm 30$  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  $90^\circ$  to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.



- 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.
3. Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
  4. 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.
  5. 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.
  6. 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.
  7. Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
  8. Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
  9. 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.
  10. The contractor shall install the concrete base pad as detailed elsewhere in the contract.
  11. The side slope of the rigid expanded polystyrene embankment shall be covered with Granular B Type II material as detailed elsewhere in this contract.
  12. 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.
  13. 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 7-1 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 will be made.