

**Final Foundation Investigation  
and Design Report – Highway  
551 Mindemoya River Bridge  
Replacement, Site 49-026**

G.W.P. 5153-12-00

Geocres No. 41G-22



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Ministry of Transportation Ontario

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**FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT – HIGHWAY 551 MINDEMOYA  
RIVER BRIDGE REPLACEMENT, SITE 49-026**

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## 1.0 Introduction

The Ministry of Transportation of Ontario (MTO) retained Stantec Consulting Ltd. (Stantec) to undertake the foundations work required for the preliminary design of the replacement for the Mindemoya River Bridge. The site is located on Highway 551, approximately 1 km north of Government Road in the community of Providence Bay, Concession Road 12 within the Municipality of Central Manitoulin, Ontario.

This *Preliminary Foundation Investigation and Design Report* has been prepared specifically and solely for the proposed replacement of the Mindemoya River Bridge at Highway 551 in Central Manitoulin, Ontario.

Project Number: G.W.P.: 5153-12-00  
Agreement Number: 5013-E-0015  
Project Location: Mindemoya River within the Municipality of Central Manitoulin  
Site Location: Approximately 1 km north of Concession Road 12 (Government Road)

## 2.0 Site Description and Geology

### Site Location

The proposed structure location is shown on the Key Plan inset to Drawing No. 1, provided in Appendix A. The existing bridge carries Highway 551 traffic across the Mindemoya River at Structure Site No. 49-026b.

Chainage along Highway 551 increases from south to north. The proposed replacement bridge is between Stations 11+020 and 11+080.

### General Site Description

At this site, Highway 551 is oriented approximately in a north-south direction with chainage increasing from south to north. Highway 551 has a single 3.5 m wide lane of traffic in each direction with paved and gravel shoulders of varying widths. Mindemoya River flows westerly at the bridge location emptying into Providence Bay and Lake Huron at the southwestern boundary of the Municipality of Central Manitoulin.

In the vicinity of the existing bridge the surrounding area is generally flat to rolling; the ground surface elevation generally decreases from the bridge toward the west.

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## Existing Bridge

The Request for Proposal indicates that the existing Mindemoya River Bridge clear span is approximately 12 m (single span), a steel structure with a wood bridge deck. The overall deck width is approximately 8 m including two traffic lanes and respective shoulders. The structure was built around 1965 and is rated as being in fair condition. Photographs 1 through 6 show the general site features.

## Physiographic Description

The project site is located within the Canadian Shield and is characterized by rock tablelands tilted towards the southwest. Soil and bedrock rock mapping published by the Ontario Geological Survey suggests that the subsurface conditions at the site consist of clay, silt, sand, gravel, and boulders underlain by Ordovician shale and limestone, and dolostone of the Amabel Formation.

## **3.0 Investigation Procedures**

### **3.1 FIELD INVESTIGATION**

Prior to carrying out the field investigation, Stantec contacted the public utility authorities to clear the borehole locations of public and private utilities.

A geotechnical field investigation consisting of six boreholes was carried out for this assignment. The boreholes were designated BH14-1, BH14-2, BH14-3, BH14-4, BH14-5, and BH14-6. The borehole locations are shown on the Borehole Location Plan, Drawing No.1 in Appendix A.

The field drilling program was carried out between November 17 and 26, 2014. BH14-1 through BH14-4 were advanced with a track mounted drill rig equipped for soil and bedrock sampling. BH14-5 and BH14-6 were advanced manually with hand augers and split spoon.

The subsurface stratigraphy encountered in each borehole was recorded in the field by an experienced Stantec Field personnel. Split spoon samples were collected at regularly spaced intervals (typically every 760 mm) during the course of Standard Penetration Testing (ASTM D1586). Bedrock coring was carried out in boreholes BH14-1 through to BH14-4 with NQ size coring equipment.

All samples recovered were returned to Stantec's Ottawa laboratory for detailed classification and testing.

Vibrating wire piezometers (VWPs) were installed in BH14-1 and BH14-2 on November 20, 2014. Each VWP consisted of small diameter cylindrical housing containing a pressure transducer and a thermistor, and was installed following the manufacturer's installation guidelines. The installation detail included 300 mm sand below the piezometer tip and 300 mm sand above the tip, creating a collection zone of 600 mm. Bentonite seal was placed above the collection zone

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to the ground surface. The piezometer tips were installed to depths below the existing ground surface of approximately 5.5 m in BH14-1 and 1.7 m in BH14-2. Groundwater readings were carried out six days later on November 26, 2014. The calibration data sheet for the VWP transducers are provided in Appendix B.

After completion of drilling, the boreholes were backfilled with auger cuttings mixed with bentonite and sealed. Cold-mix asphalt patch was placed at surface for BH14-3 and BH14-4.

Rock core samples were logged and photographed, and the Rock Quality Designation (RQD) and core recovery were estimated for recovered samples.

## 3.2 LOCATION AND ELEVATION SURVEY

The elevation and coordinates (northing and easting) of the boreholes were determined using a Global Positioning System (GPS) navigation device, Trimble Geo XH, capable of decimeter accuracy.

The ground surface elevations and coordinates of the borehole locations are provided in Drawing 1 of Appendix A.

The ground surface elevations at the borehole locations are also shown on the Borehole Records included in Appendix B. Summary information pertaining to the boreholes included in this report is given in Table 3.1.

**Table 3.1: Borehole Information Summary**

	Borehole Location					
	BH14-1	BH14-2	BH14-3	BH14-4	BH14-5	BH14-6
MTM Zone 11 Coordinates Northing Easting	5058548 322974	5058578 322961	5058551 322989	5058584 322978	5058560 322997	5058580 322999
Ground Surface Elevation, m	178.5	177.0	178.9	179.2	177.4	176.9
Total Depth Drilled, m	22.5	20.2	22.3	22.2	1.2	1.2
End of Borehole Elevation, m	156.0	156.8	156.6	157.0	176.2	175.7
Depth Augered, m	19.3	16.9	20.1	19.4	1.2	1.2
Depth Cored, m	3.2	3.3	2.2	2.8	0	0
Number of Soil Samples	20	19	20	20	2	2

## 3.3 LABORATORY TESTING

All samples were taken to Stantec's Ottawa laboratory where they were subjected to a detailed visual examination by a Geotechnical Engineer.

The geotechnical laboratory testing program for the borehole samples is summarized in Table 3.2.

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**Table 3.2: Geotechnical Laboratory Testing Program**

Test Description	Number of Tests
Moisture Content	80
Grain Size Distribution	18
Specific Gravity	3
Unconfined Compression (rock)	8

Four soil samples were tested for pH, soluble sulphate content, chloride content, and resistivity.

Samples remaining after testing will be placed in storage for a period of one year after issuance of the final report. After the storage period, the samples will be discarded.

## 4.0 Subsurface Conditions

The details of the subsurface conditions observed in the boreholes are presented in the Borehole Records provided in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix B.

The borehole location plan and stratigraphic section of the soils encountered within the boreholes is provided in Drawing No. 1 of Appendix A.

### 4.1 OVERBURDEN

In general, the subsurface stratigraphy consisted of asphalt pavement over fill materials over silty sand over sandy silt to silty sand overlying limestone bedrock.

Where a value is provided for the percentage of clay-sized particles, the value represents the percentage of particles finer than a nominal size of 0.002 mm.

#### 4.1.1 Asphalt Pavement

A 20 mm and 40 mm thick layer of asphalt was encountered in BH14-3 and BH14-4, respectively.

#### 4.1.2 Fill

Fill was encountered immediately beneath the asphalt in BH14-3 and BH14-4 and at the surface in BH14-1, BH14-2, and BH14-5. In BH14-1 and BH14-2, the fill layer consists of poorly graded sand and was 1.5 and 0.6 m thick with base elevations of 177.0 m and 176.3 m, respectively. In BH14-3 to BH14-5, the fill layer consists of silty sand with gravel and was 2.6 m to 0.6 m thick extending to base elevations of 176.7 m and 177.5 m.

Trace amounts of organic material was observed within the fill in BH14-1 and BH14-2. The fill material was generally brown and moist to wet.



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The Standard Penetration Test (SPT) blow count (N-values) for the fill ranged from weight of hammer to 34 blows per 0.3 m suggesting a very loose to dense state.

Index tests carried out on representative samples from the fill material yielded the following:

Gravel:	3 and 23%
Sand:	58 and 93%
Fines (silt & clay):	4 and 19%
Moisture content:	6 to 31%

The Unified Soil Classification System (USCS) group symbols for the fill material includes SP (poorly graded sand) and SM (silty sand with gravel).

Representative grain size distribution plot is provided on Figure 1 of Appendix C.

## 4.1.3 Sand to Silty Sand

Silty sand deposit was encountered immediately beneath the fill in BH14-1 and BH14-4 through to BH14-6. Where penetrated, the thickness of the silty sand deposit was approximately 6.5 m to 13.7 m, and extended to approximate base elevations of 170.1 m to 163.3 m. In BH 14-5 and BH14-6, drilling was terminated within the silty sand layer.

Within BH14-3, a 1.5 m thick layer of poorly graded sand was observed extending to elevation 176.0 m.

Within BH14-1, a 600 mm thick layer of well-graded sand with silt and gravel was observed between elevation 173.2 m and 172.6 m.

The SPT N-values for this deposit ranged from 9 to greater than 100 blows per 0.3 m suggesting a loose to very dense state. The silty sand and sand deposits were generally grey or brown and moist to wet.

Index tests carried out on representative samples from this material yielded the following:

Gravel:	0 to 18%
Sand:	59 to 91%
Fines (silt & clay):	6 to 40%
Silt:	4 to 36%
Clay:	1 to 2%
Moisture content:	8 to 24%

The USCS group symbols for this deposit are SM (silty sand to silty sand with gravel), SP (poorly graded sand) and SW-SM (well graded sand with silt and gravel).

Representative grain size distribution plots for this material are given in Figures 2 through 4 of Appendix C.

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## 4.1.4 Silt

A deposit of silt with variable amounts of sand and gravel was encountered in all the boreholes with the exception of BH14-5 and BH14-6.

The silt layer was observed beneath the sand deposit in BH14-1, BH14-3, and BH14-4 and immediately beneath the fill layer in BH14-2. The silt layer appears to be bisected with a layer of silty sand in BH14-2 (between elevation 171.6 m and 161.7 m) while approximately 1.1 m of silty sand with gravel layer was observed beneath the sandy silt layer in BH14-4 (between elevation 161.0 m and 159.9 m).

The SPT N-values for this deposit ranged from 7 to 50 blows per 0.3 m suggesting a loose to dense state. The silt deposit was generally grey and wet.

Index tests carried out on representative samples from this material yielded the following:

Gravel:	0 to 5%
Sand:	15 to 45%
Silt:	41 to 70%
Clay:	1 to 11%
Moisture content:	14 to 24%

The USCS group symbol for this deposit is ML and is described as sandy silt and silt with sand.

Representative grain size distribution plots for this material are given in Figures 5 and 6 of Appendix C.

### 4.1.4.1 Bedrock

Bedrock was encountered in the boreholes immediately beneath the sand and silt deposits at approximate elevation ranging from 158.8 m to 160.1 m. The bedrock consisted of limestone with shaly partings with dolomitic seams.

The Rock Quality Designation (RQD) values ranged between 47% and 85%, indicating a poor to good rock quality. The Total Core Recovery (TCR) was 71% to 100%. A detailed description of the rock core is provided in Field Bedrock Core Logs and rock core photographs are provided in Appendix B.

Unconfined compressive strength tests were carried out on bedrock samples from four boreholes. The results of these tests are summarized in Table 4.1. The test results indicate a strength classification of very strong.

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**Table 4.1: Unconfined Compressive Strength of Rock Cores**

Borehole No	Test Elevation (m)	Unconfined Compressive Strength (MPa)
BH14-1	159.1	237
	156.7	117
BH14-2	159.5	146
	157.9	202
BH14-3	158.2	142
	156.8	124
BH14-4	158.8	147
	157.2	194

## 4.1.5 Chemical Analysis

Four representative samples retrieved from the silty sand and sandy silt deposits in BH14-1 and BH14-4 were tested for resistivity, pH, and water soluble sulphates and chloride concentrations. The results of this chemical analysis are provided in Table 4.2 and in Appendix C.

**Table 4.2: Results of Chemical Analysis**

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH14-1	SS7	4.57 to 5.18	7.9	10	86	57.5
BH14-1	SS16	13.71 to 14.32	7.7	9	122	49.6
BH14-4	SS7	4.57 to 5.18	7.9	10	113	50.1
BH14-4	SS16	12.19 to 12.80	7.8	10	102	59.5

## 4.2 GROUNDWATER

Vibrating wire piezometers were installed in BH14-1 and BH14-2 after completion of drilling on November 20, 2014. The vibrating wire piezometers were measured on November 26, 2014. The measured groundwater levels are summarized in Table 4.3 below. The water level in the Mindemoya River was measured on September 26, 2014, at approximate elevation 176.3 m.

**Table 4.3: Measured and Inferred Groundwater Levels**

Borehole No	Ground Surface Elevation (m)	Groundwater	
		Depth (m)	Elevation (m)
BH14-1	178.5	2.7	175.2
BH14-2	177.0	1.0	176.0

Fluctuations in the groundwater level due to seasonal variations or in response to a particular precipitation event should be anticipated.

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## 5.0 Discussion

### Project Purpose/Justification

The existing bridge is a single-span wood deck steel frame structure. It has a span of approximately 12 m and an overall width of 8 m including two traffic lanes and respective shoulders. The structure was built around 1965 and will be replaced.

### Proposed Replacement

Two bridge replacement options are being evaluated. Construction of a new bridge on either side of the existing bridge or construction of a detour and temporary structure on either side of the existing bridge and construction of a new bridge at the same location of the existing bridge. For both options, the new bridge is anticipated to be a single span structure with a span of approximately 18 m.

Approximate key elevations associated with the structure are as follows:

Approx. Proposed Final Grade (at centerline of structure)	179.2 m
Proposed Final Grade at North Abutment (Detour and New Bridge)	179.2 m
Proposed Final Grade at South Abutment (Detour and New Bridge)	179.2 m
Proposed Underside of Abutment Pile Caps (Detour and New Bridge)	176.6 m
Water Level Elevation in Mindemoya River (September 2014)	176.3 m
Proposed Underside of Abutment Footings	176.0 m

## 5.1 GEOTECHNICAL DESIGN PARAMETERS

The subsurface conditions encountered at this site consist of asphalt pavement over fill overlying non-cohesive deposit of sand and silt that overlies Limestone bedrock. The native soils at the site are generally compact to very dense. Bedrock was encountered at depths of 16.9 to 20.1 m below the existing ground surface. The RQD of the bedrock ranged between 47% and 85%, indicating a poor to good rock quality. The unconfined compressive strength ranged between 117 and 237 MPa (very strong).

The subsurface profile shown in Table 5.1 can be used for preliminary design purposes. The subsurface profile was developed based on the synthesis of the measured N-values and laboratory index test results (including moisture contents) of samples retrieved from the site. This profile is included in Figure 7 of Appendix D and was developed based on the information obtained from the boreholes.

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**Table 5.1: Preliminary Subsurface Profile**

Elevation (m)		Soil Type	Design Parameters			
From	To		$\gamma$ (kN/m <sup>3</sup> )	$\phi$ (°)	UCS (MPa)	E (MPa)
179.2	176.7	Fill: loose to dense silty sand with gravel	21.3	32	-	10
176.7	Varies (158.8 to 160.1)	Cohesionless deposit: compact to dense sand and silt	20.7	35	-	15
Varies (158.8 to 160.1)		Shaly limestone with dolomitic seams bedrock (poor to good quality, very strong bedrock)	24.5	-	117 to 237	16,800

Notes: (1)  $\gamma$  = total unit weight,  $\phi$  = soil friction angle, UCS = unconfined compressive strength of rock, E = soil (rock) modulus

(2) Groundwater is assumed to be at an approximate elevation of 176.3 m for preliminary design purposes. Submerged unit weight ( $\gamma'$ ) should be used below the groundwater level.

## 5.2 FROST PENETRATION

In accordance with OPSD 3090.101, the design frost penetration depth for foundations,  $f$ , at the site is 1.6 m. Therefore, footings and pile caps should be provided with a minimum of 1.6 m of soil cover or equivalent insulation for protection against frost heaving.

## 5.3 SEISMIC DESIGN CONSIDERATIONS

The soil profile at the site includes an approximately 0.6 to 2.6 m thick, layer of fill over a 16.3 to 17.2 m thick, compact to dense cohesionless deposit of sand and silt. The native deposit is underlain by limestone bedrock with shaly partings and dolomitic seams. It is recommended that a Soil Profile I, as defined in Canadian Highway Bridge Design Code (CHBDC, 2006) Section 4.4.6, be used in the seismic design of this site.

Table A3.1.1 of the CHBDC indicates that the Zonal Acceleration Ratio (ZAR) for Espanola, Ontario, which is approximately 75 km north-east of the site is 0.05. This table indicates a ZAR of 0.00 for Gore Bay, ON (approximately 30 km northeast of the site). For comparison, the peak ground acceleration (PGA) for a 10% probability of exceedance in 50 years for the site was estimated from NRCAN website to be 0.011. Hence, a ZAR of 0.011 should be used for this site.

The potential liquefaction of the site soils under seismic loading conditions was assessed. The assessment result is shown in Figure 8 of Appendix D. The assessment indicated that liquefaction of the site soils is not of a concern due to:

- (a) A very low ZAR,
- (b) Soils present are compact on average, and
- (c) Relatively high fraction of fines content within the soils.

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Even though it is not likely significant, seismically induced lateral earth pressures should be considered for this project with a ZAR of 0.011.

## 5.4 FOUNDATION OPTIONS

Table 5.2 compares the foundation options from a foundation design and constructability perspective.

**Table 5.2: Comparison of Foundation Options for Proposed Replacement Bridge and Detour Bridge**

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences	Ranking
<b>Shallow foundation on native soil</b>	<ul style="list-style-type: none"> <li>▪ Limited excavation involved</li> </ul>	<ul style="list-style-type: none"> <li>▪ May necessitate large footing area</li> <li>▪ Not suitable for integral abutment bridge construction</li> <li>▪ Sandy silt is easily disturbed, excavation and removal of unsuitable soil may be required</li> <li>▪ Fill extends to below the founding elevation which will require some subexcavation.</li> </ul>	Low to Medium	<ul style="list-style-type: none"> <li>▪ Potential differential settlement</li> <li>▪ Excavation below groundwater and basal instability</li> </ul>	1
<b>Piles Frictional in native soils</b>	<ul style="list-style-type: none"> <li>▪ Suitable for integral abutment</li> </ul>	<ul style="list-style-type: none"> <li>▪ Pile capacity may not be fully utilized</li> <li>▪ Long pile length</li> </ul>	Medium to High	<ul style="list-style-type: none"> <li>▪ Larger settlement</li> <li>▪ Piles may need to be driven to bedrock due to lower than estimated capacities</li> </ul>	4
<b>Piles End bearing on or socketed into bedrock</b>	<ul style="list-style-type: none"> <li>▪ Reduces risk of differential settlement</li> <li>▪ Suitable for integral abutment bridge</li> </ul>		Medium to High	<ul style="list-style-type: none"> <li>▪ Possible pile damage during installation; pre-drilling of very dense till and bedrock for socketing the piles may be required</li> </ul>	2
<b>Drilled Caissons</b>	<ul style="list-style-type: none"> <li>▪ Can transmit very large axial and lateral loads</li> </ul>	<ul style="list-style-type: none"> <li>▪ Not suitable for integral bridge abutment</li> <li>▪ Generally not suitable if bedrock is relatively deep</li> </ul>	High	<ul style="list-style-type: none"> <li>▪ Risk of cave-in, especially below groundwater table during drilling</li> </ul>	3

### Recommended Foundation Option

Based on the comparison presented above, the shallow foundation on undisturbed soil is recommended. The anticipated founding elevation is approximately 176.0 m which has been selected to avoid founding the footing on existing fill and loose layers.

Piles end bearing on bedrock is also a feasible foundation option. Recommendations for both options are presented.

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## Bridge Alignment

Bridge alignments on either side of the existing bridge are being evaluated. Our review of the soil conditions suggest that both shallow foundations on undisturbed soil or piles end bearing on bedrock are feasible foundation options for the bridge alignments.

## 5.5 FOUNDATION RECOMMENDATIONS

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2006).

### 5.5.1 Shallow Foundation

This section provides recommendations for the design of spread footings founded on undisturbed soils.

#### 5.5.1.1 Geotechnical Vertical Resistance

The geotechnical resistances provided in Table 5.3 may be used in the design, provided the footings are placed on undisturbed soil.

**Table 5.3: Geotechnical Resistance for Shallow Foundation (Spread Footing)**

Founding Element	Founding Elevation (m)	Footing Width (m)	Factored Geotechnical Resistance at ULS <sub>f</sub> (kPa)	Geotechnical Resistance at SLS (kPa)
Spread footing on undisturbed soil	± 176.0	1	385	180
		2	460	125
		3	500	100

In accordance with Section 6.6.2 of the CHBDC, a resistance factor of 0.5 has been applied in calculating the factored geotechnical resistance at Ultimate Limit State (ULS<sub>f</sub>).

The axial reaction at SLS corresponds to a vertical deflection (settlement) of 25 mm.

The ULS<sub>f</sub> values were estimated based on an effective friction angle of 35° with a minimum footing embedment depth of 1.6 m. The SLS values were estimated based on the elastic settlement of the native soils using E<sub>s</sub> of 15 MPa.

#### 5.5.1.2 Geotechnical Horizontal Resistance (Sliding)

The unfactored horizontal resistance of spread footings may be calculated using the following unfactored coefficients of friction:

- 0.55 Between OPSS Granular A and cast-in-place concrete
- 0.35 between native soil and cast-in-place concrete

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In accordance with Table 6.1 of the CHBDC, a resistance factor against sliding of 0.8 should be applied to obtain the resistance at  $ULS_r$ .

## 5.5.2 Foundations – Driven Piles

This section provides recommendations for the design of driven piles for the proposed bridge.

### 5.5.2.1 Geotechnical Axial Resistance

It is anticipated that a pile foundation consisting of HP310x110 piles will be used to support the proposed integral abutments. The underside of the pile caps (bottom of concrete abutments) is assumed to be at an approximate elevation of 176.6.

To provide the desired integral action, the abutment piles should be driven through a 600 mm diameter, 3 m long corrugated steel pipe (CSP) and filled with loose uniform sand. This recommendation reflects some of the high N-values observed at shallow depths in some of the boreholes.

The piles should be driven to refusal on the bedrock surface. The anticipated pile lengths for the south and north abutment are 18 m and 17 m, respectively.

A factored axial resistance in compression at  $ULS_r$  for an HP310x110 pile of 2,000 kN may be used for this site. This resistance at  $ULS_r$  assumes that the piles are driven to competent bedrock.

For piles driven to competent bedrock, settlements are anticipated to be less than the elastic shortening of the piles under loads imposed by the structure. The axial reaction at SLS is not applicable for piles successfully driven to competent bedrock.

The supply and installation of the piles should be in accordance with the OPSS 903 Construction Specification for Deep Foundations.

Axial geotechnical resistance in tension or pull-out capacities of the piles is not anticipated to be required for preliminary design purposes.

### 5.5.2.2 Downdrag

The anticipated settlement due to the placement of fill is discussed in Section 5.7.3. No significant settlement of the soils in the vicinity of the abutments is anticipated. Therefore, negligible downdrag load is anticipated to impact on the piles. It is also assumed that fill placement would be carried out prior to the installation of the piles.

### 5.5.2.3 Relaxation of Piles

For H-piles driven to refusal on competent bedrock encountered at the site, relaxation and reduction of pile capacity with time will not occur.



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## 5.5.2.4 Drivability

The soil encountered in the boreholes consisted of fill over loose to very dense silt, sand, and gravel deposits. No obstructions to pile driving are anticipated.

Piles should have Type I steel H-pile driving shoe according to Ontario Provincial Standard Drawing, OPSD 3000.100.

MTO Structural Manual (2014) Pile Driving Note 5: “Piles to be driven to bedrock” would be applicable for this site.

## 5.5.2.5 Geotechnical Lateral Resistance

The geotechnical resistance of the pile against lateral loads is mobilized due to the passive resistance of the surrounding soil.

Assessed values for horizontal passive resistance and geotechnical resistances at SLS for the proposed pile can be generated from information provided in Table C6.4 for non-cohesive materials using  $\phi' = 35^\circ$  can be used to generate geotechnical passive resistance at ULS<sub>r</sub> and SLS.

### ULS Resistance

The passive earth pressure for the pile driven through a loose uniform sand in CSP and the native soil layer was estimated using the procedure described in Section C6.8.7.1 of CHBDC (CHBDC, 2006). The resistance will be within non-cohesive soils and therefore a passive earth resistance was calculated using a bearing width of 3 times the flange width and an overburden pressure of 30 kPa at the pile head; the calculated factored lateral resistance at ULS<sub>r</sub> was 200 kPa. A geotechnical resistance factor for passive lateral resistance of 0.5 was used (Table 6.1 of CHBDC, 2006).

### SLS Resistance

The lateral geotechnical resistance at SLS was evaluated using the program LPILE Plus v6.0 developed by Ensoft, Inc. (Ensoft, 2010). The input parameters are given in Table 5.4.

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**Table 5.4: Parameters Used for Lateral Resistance at ULS and SLS for Piles**

Soil Layer	Elevation (m)		Unit <sup>(2)</sup> Weight, $\gamma$	Friction Angle, $\phi$	Undrained Shear Strength, $S_u$	Deformation Parameters <sup>(3)</sup>	
	From	To				k	$\epsilon_{50}$
Loose to compact sand <sup>(1)</sup>	177.6	174.6	20	33	-	5,400	-
Cohesionless deposit: compact to dense sand and silt	174.6	158.8	20.7	35	-	16,300	-
Granite bedrock	$\leq 158.8$		24.5	-	-	-	-

Notes:

- (1) This layer represents the loose uniform sand filled around the pile in the CSP.
- (2) Submerged unit weight will be used below groundwater level.
- (3)  $k$  = p-y modulus;  $\epsilon_{50}$  = strain corresponding to one-half the maximum principal stress difference.
- (4) Groundwater level was assumed to be at an elevation of 176.0 m.

The plots from LPILE analysis results are presented in Figures 14 and 15 of Appendix D. Figure 14 shows the deformed shape of the pile for lateral (shear) force ranging between 30 and 60 kN for the abutment piles. The analysis was carried out using the above soil profile and forcing zero rotation at the pile head with no restrictions to lateral movements which represents the conditions of integral abutments. This plot indicates that for the abutment piles, the pile undergoes negligible lateral deflection below a depth of approximately 7 m from the underside of the pile cap (at approximate elevation of 170 m).

Figure 14 of Appendix D illustrate the displacement of the pile in depth for different lateral loads. Based on this figure, lateral loads of 55 kN correspond to a pile head (top) displacement of less than 10 mm for abutment piles. Therefore, the SLS geotechnical resistance of an HP 310x110 at this site is estimated as 55 kN for abutment piles.

Figure 15 of Appendix D present the p-y plot that gives the non-linear response of the pile-soil interaction for the abutment piles. This curve was obtained from LPILE. The p-y curve values versus depth are summarized in Table D-1 of Appendix D. These plots and the p-y curve points can be used in the structural evaluation of the proposed bridge founded on H-piles.

Group action of piles (pile interaction) for lateral loading should be considered if centreline spacing of piles is less than 8 pile diameters (or least lateral dimension of pile) parallel to the direction of lateral load, or less than 5 pile diameters, perpendicular to the load. The effect of interaction between piles can be considered by applying a reduction factor to the coefficient of lateral subgrade reaction (p-y modulus); the reduction factors are applied to the p-value and may be considered as p-multipliers. The following reduction factors may be used to account for pile group action:

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**Table 5.5: Recommended Reduction Factors for Pile Groups**

Load Direction	Pile Spacing	$p$ -Multiplier Row 1	$p$ -Multiplier Row 2	$p$ -Multiplier Row 3+
Parallel	3B	0.8	0.4	0.3
Parallel	5B	1.0	0.85	0.7
Perpendicular	3B	0.8	-	-
Perpendicular	5B	1.0	-	-

## 5.6 LATERAL EARTH PRESSURES

This section provides recommendations regarding backfill, static lateral earth pressure, and seismic lateral earth pressures.

### 5.6.1 Backfill

It is recommended that the backfill behind the sub-structures for the proposed bridge replacement consist of approved earth material placed and compacted using methods and equipment appropriate to the type of structure. For the purpose of this preliminary design, the following assumptions are made:

- A backfill material meeting the requirements of OPSS Granular B Type I or Granular A and Granular B Type II material will be used, and
- The surface of the backfill will be horizontal.

### 5.6.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of abutments and any retaining walls.

The bridge abutments should be backfilled with granular material in accordance with OPSD 3101.150.

Computation of earth pressures should be completed in accordance with Section 6.9 of the CHBDC. For walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. The unfactored soil parameters provided in Table 5.6 may be used for design of walls with a horizontal backfill. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC.

The total active ( $P_A$ ), passive ( $P_P$ ) and at-rest ( $P_O$ ) thrusts can be calculated using the following equations:

$$P_A = \frac{1}{2} K_a \gamma H^2$$

$$P_P = \frac{1}{2} K_p \gamma H^2$$

$$P_O = \frac{1}{2} K_o \gamma H^2$$

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where H is the height of the wall and  $\gamma$  is the unit weight of the backfill soil. Values for  $K_a$ ,  $K_p$ ,  $K_o$  and  $\gamma$  are provided below. The thrust acts at a point one third up the height of the wall.

**Table 5.6: Recommended Non-Seismic Earth Pressure Parameters (Horizontal Backfill)**

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II	Native Soils	Embankment Fill
Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21.2	22.0	20.7	21.3
Effective Friction Angle (°)	32	35	35	32
Coefficient of Earth Pressure at Rest ( $K_o$ )	0.47	0.43	0.43	0.47
Coefficient of Active Earth Pressure ( $K_a$ )	0.31	0.27	0.27	0.31
Coefficient of Passive Earth Pressure ( $K_p$ )	3.2	3.7	3.7	3.2

## 5.6.3 Seismic Lateral Earth Pressures

The low ZAR for this site suggests that the lateral earth pressures on the bridge due to seismic loads will be very small. The following design parameters are provided, should the bridge abutment and wing walls (if any) also be designed to resist the earth pressures induced under seismic loading conditions. The seismic earth pressures may be calculated using the parameters detailed in Table 5.7 below.

The total active and passive thrusts under seismic loading conditions can be calculated using the following equations:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)$$

$$P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)$$

where:

$K_{AE}$  = active earth pressure coefficient (combined static and seismic)

$K_{PE}$  = passive earth pressure coefficient (combined static and seismic)

H = height of wall

$k_h$  = horizontal acceleration coefficient

$k_v$  = vertical acceleration coefficient

$\gamma$  = total unit weight

For this site, the following design parameters were used to develop the recommended  $K_{AE}$  and  $K_{PE}$  values.

- Zonal Acceleration Ratio, A or PGA 0.05
- Horizontal Acceleration Coefficient,  $k_h$  0.025 yielding 0.075 non-yielding
- Vertical Acceleration Coefficient,  $k_v$  0.017 yielding 0.05 non-yielding
- Horizontal Back slope to Wall 0°
- Vertical Back of Wall 0°

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The  $k_h$  value above corresponds to half of the A value for yielding walls and 1.5 times the value for non-yielding walls. The  $k_v$  value corresponds to 0.67 of the  $k_h$  value. The angle of friction between the soil and the wall has been set at  $0^\circ$  to provide a conservative estimate.

**Table 5.7: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)**

Parameter	OPSS Granular B Type I		OPSS Granular A and Granular B Type II		Native Soils	
Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21.2		22.0		20.7	
Effective Friction Angle ( $^\circ$ )	32		35		35	
Wall Type	Yielding	Non-yielding	Yielding	Non-yielding	Yielding	Non-yielding
Active Earth Pressure ( $K_{AE}$ )	0.32	0.35	0.28	0.31	0.28	0.31
Height of Application of $P_{AE}$ from base as a ratio of wall height, (H)	0.341	0.356	0.342	0.358	0.342	0.358
Passive Earth Pressure, ( $K_{PE}$ )	3.21	-	3.64	-	3.64	-
Height of Application of $P_{PE}$ from base as a ratio of wall height, (H)	0.325	-	0.325	-	0.325	-

## 5.7 EMBANKMENTS

This section provides recommendations regarding embankment construction, stability of slopes, embankment settlement, and settlement mitigation.

### 5.7.1 Embankment Construction

The proposed profile height of the bridge will be increased. To accommodate the profile raise and embankments, a combination of retaining walls and slope embankment fills are anticipated. For preliminary design purposes, it is assumed that the embankment will be constructed using either a Select Subgrade Material (SSM) or Earth Borrow material.

The expected maximum embankment height at the proposed detour is approximately 2 m to 3 m near the abutments.

### 5.7.2 Stability of Slopes

The embankment configuration (including height, side slope, etc.) has not yet been established. A preliminary slope stability evaluation was carried out, assuming a side slope of 2H:1V and maximum embankment height of 3 m as discussed above. The evaluation was carried out using a commercial program, Slope/W (Geo-Slope, 2010). The preliminary stability evaluation was carried out for two cross sections, sections A-A' and B-B' which are shown on Drawing No. 1. Preliminary slope stability evaluation results for the case of Earth Borrow are provided in Figures 9 through 12 in Appendix D.

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The slope stability results suggest a Factor of Safety of greater than 1.5 for static loading conditions and greater than 1.2 for seismic loading conditions. Results of the preliminary slope stability evaluation suggest that for the anticipated embankment configuration, constructed using SSM or Earth Borrow will be stable at a slope of 2H:1V, under both static and seismic situations.

## 5.7.3 Embankment Settlement

For the purpose of the preliminary evaluation, the following assumptions will be made in evaluating the settlement of the site soil under the proposed embankment:

- The proposed final grade of the road will be elevation 179.2 m;
- The load from the bridge abutments will be transferred to the competent bedrock and will therefore not contribute to the settlement of the site soil;
- Settlement of the site soil will be caused by the embankment fill only;
- Groundwater is assumed at elevation 176.3 m;
- The approach embankment will have 2V:1H side slopes;
- The top width of the embankment will be approximately 10.2 m (including shoulders and rounding's).
- Profile of the existing embankments will be increased by a maximum of 3 m near the location of the abutments.
- The new fills extend to approximately 60 m to the north and south of the abutments;
- The analysis does not consider interaction between the proposed and existing embankments.

Evaluation of soil settlement due to the effects discussed above was carried out using the Settle3D software (Rocscience, 2014). Settle3D is a three-dimensional computer program used for the analysis of the immediate vertical settlement and consolidation settlement of soil under surface loads such as embankments. Settlement evaluation was carried out for embankments constructed using SSM.

A plot of settlement contours from typical Settle3D preliminary analysis is presented in Figure 13 in Appendix D. The preliminary analysis result indicates that the maximum total vertical settlement of the existing materials for the conditions presented above is approximately 65 mm. The maximum settlement will take place approximately 10 m to 20 m from each abutment.

Assuming 0.5% strain under self-weight, the estimated embankment self-weight settlement is negligible.

The predicted settlement is anticipated to be completed by the end of embankment construction.

## 5.7.4 Settlement Mitigation

No settlement mitigation measures will be required for this site.

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## 5.8 TEMPORARY ROADWAY PROTECTION

Temporary roadway protection is not anticipated as part of the staged construction approach to maintain traffic flow during construction.

## 5.9 PRELIMINARY CONSTRUCTION CONSIDERATIONS

### 5.9.1 Excavation and Backfilling

The extent of soft and compressible or organic material to be removed or treated is anticipated to be negligible. Conventional embankment design and construction procedures using SSM or Earth Borrow material as described in section 5.7 is therefore suitable for this site.

Excavation backfill should be carried out in accordance with OPSS 902, Construction Specification for excavation and Backfilling – Structures.

The soils at the site should be classified according to the Occupational Health and Safety Act regulations for Construction Projects (OHSA). The soils should be classified as follows.

Embankment Fill	Type 3 Soil
Native Soil (silts and sands) above the groundwater level	Type 3 Soil
Native Soil (silts and sands) below the groundwater level	Type 4 Soil

Any vegetation, fill, organic soils, and other deleterious materials must be removed from beneath the proposed structural footing and embankment. Where deleterious materials are encountered, the materials should be excavated, removed, and replaced. The lateral extent of such excavation should include all deleterious materials within the influence zone of the embankments.

Grading work should be carried out in accordance with SP 206. Compaction should be carried out in accordance with OPSS 501.

Any side slopes for open cut excavations should conform to OHSA.

### 5.9.2 Unwatering/River Water Control

#### Unwatering

Groundwater was encountered at an elevation of approximately 176.0 m, which is approximate elevation for the proposed underside elevation for footings should this option be considered. The water level in Mindemoya River was surveyed to be at elevation 176.3 m at the time of the investigation. The groundwater level within the excavations will most likely be the same as within the creek.

Unwatering would be required to maintain dry working conditions desirable during excavation and construction of the footings option.

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The native soils within the anticipated depth of excavation are expected to have a low to moderate hydraulic conductivity ( $10^{-3}$  cm/sec to  $10^{-5}$  cm/sec).

Unwatering of the structure excavations using conventional sump and pump techniques will likely be not adequate for excavation below the water table, unless a boxed sheet pile approach driven sufficiently deep to stabilize the base is carried out. It is recommended that the Detail Design for this site include an NSSP to address issues related to groundwater control during construction.

## River Water Control

Control of the water flow in the stream will require a cofferdam or an aquadam to prevent stream flow into the excavations. If steel sheet pile is the selected option, this can be incorporated in the groundwater control scheme for the project. It is anticipated that creek flow will be diverted using pumps to allow construction of the foundation.

### 5.9.3 Reuse of Excavated Material

The native material at the site is predominantly sand with variable amounts of silt and gravel. This material will not be suitable for use as backfill within and behind the proposed structures. The material could be used as Earth Borrow for embankment construction.

## 5.10 CEMENT TYPE AND CORROSION POTENTIAL

Four samples of the site soil were tested for pH, water soluble sulphate and chloride concentrations, and resistivity. The testing 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 infrastructures. The analysis results are summarized in Table 4.2.

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. The soluble sulphate concentrations for the samples ranged between 86 and 122  $\mu\text{g/g}$ . Soluble sulphate concentrations less than 1,000  $\mu\text{g/g}$  generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH was between 7.7 and 7.9, which is within what is considered to be the normal range for soil pH of 5.5 to 9.0. The pH level of the tested soil does not indicate a highly corrosive environment. The test results provided in the Table 4.2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.



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## 5.11 FUTURE INVESTIGATIONS

The recommendations provided herein are preliminary and based on a limited foundation investigation carried out within the general area of the proposed bridge abutments. The recommendations were made based on the interpretation of a limited number of test holes. Once the final locations of the proposed structure foundations and the embankment configurations have been identified, it is recommended that additional geotechnical investigations be carried out at these locations to enable detailed recommendations for the proposed replacement bridge and the associated embankments.

## 6.0 Specifications

The following specifications are referenced in this report:

**Table 6.1: Specifications Referenced in Report**

Document	Title
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls – Abutment, Backfill – Minimum Granular Requirement
OPSS 206	Construction Specification for Grading
OPSS 501	Construction Specification for Compacting
OPSS 902	Construction Specification for Excavation and Backfilling - Structures

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## 7.0 References

- ASTM. 1999. Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). ASTM International, West Conshohocken, PA.
- ASTM. 2000. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D2487). ASTM International, West Conshohocken, PA.
- Canadian Foundation Engineering Manual (CFEM). 2006. Fourth Edition. Canadian Geotechnical Society, 488 p.
- CHBDC, 2006. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.
- Ensoft, 2010. User's Manual for Computer Program LPILE Plus Version 6.0. Ensoft, Inc., Austin, Texas.
- GEO-SLOPE International Ltd. 2010. Stability Modeling with SLOPE/W 2010©. Calgary, AB.
- Ontario Ministry of Transportation (MTO). 2011. Structural Manual. Bridge Office, St. Catharines, Ontario.
- Rocscience, 2009. Settle3D Settlement and Consolidation Analysis: Theory Manual, Rocscience, Inc.

## 8.0 Miscellaneous

The field work was carried out under the supervision of Athir Nader, M.A.Sc., Intern Geotechnical Engineer, under the direction of Chris McGrath, P.Eng., Senior Geotechnical Engineer.

The drilling equipment was supplied and operated by Landcore Drilling of Chelmsford, Ontario. Traffic control was provided by Bartletts Towing of North Bay, Ontario.

Geotechnical laboratory testing was carried out at the Stantec Ottawa laboratory. Chemical testing on soil samples was carried out by Paracel Laboratories in Ottawa.

This report was prepared by Athir Nader, and reviewed by Chris McGrath and Raymond Haché, MTO Designated Principal Contact.

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## 9.0 Closure

The discussions and preliminary recommendations provided in this report are in accordance with our present understanding of the project.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above recommendations.

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.

Respectfully submitted,

**STANTEC CONSULTING LTD.**



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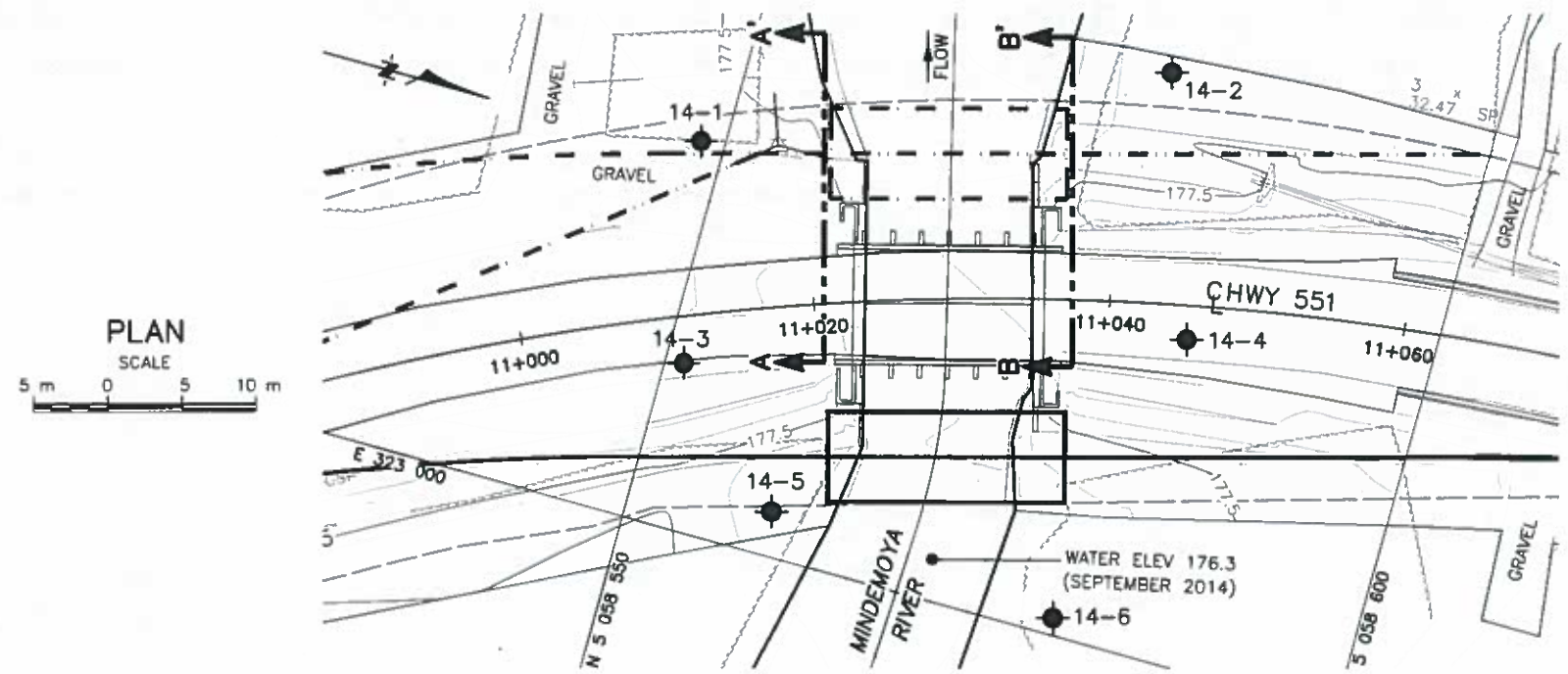
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## APPENDIX A

Drawing No. 1 – Borehole Location Plan and Soil Strata  
Site Photographs



165000912\_Plan & Section.dwg  
CREATED BY: GBB  
MODIFIED: GBB  
T:\Autocad\Drawings\Project Drawings\165000912\_Plan & Section.dwg (August 2015) Printed: Aug 24, 2015



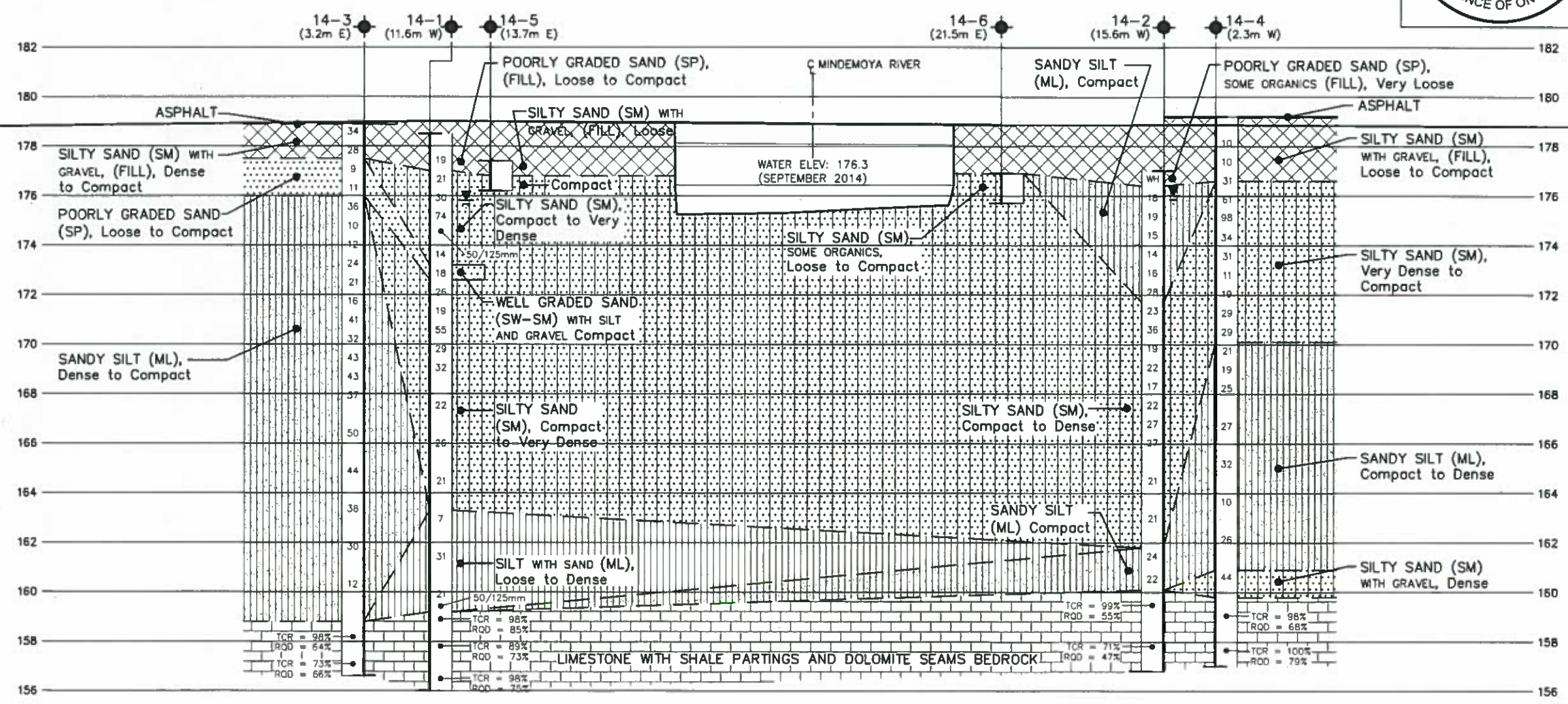
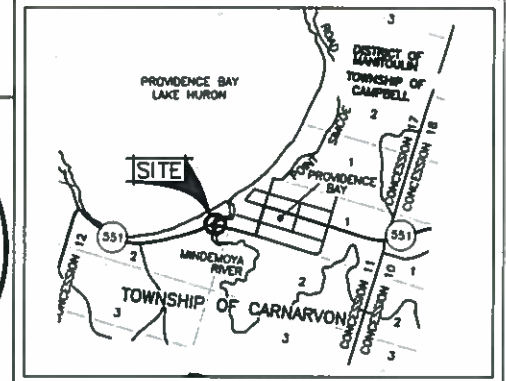
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN



PLATE No  
**CONT**  
**WP 5153-12-00**

HIGHWAY 551, MINDEMOYA  
RIVER BRIDGE REPLACEMENT  
BOREHOLE LOCATION PLAN

**SHEET**  
-



- LEGEND**
- Borehole
  - N
  - Blows/0.3m (Std Pen Test, 475 J/blow)
  - WL Measured on Month Year
  - (x.x m E) Offset East/West of Hwy 551 Centreline in meters
  - Alternative 1 Proposed Location of Bridge
  - Alternative 2 Proposed Location of Bridge
  - Cross Section for Slope Stability

No	ELEVATION	MTM ZONE 11 NORTH	COORDINATES EAST
14-1	178.5	5 058 548.4	322 974.2
14-2	177.0	5 058 577.8	322 961.3
14-3	178.9	5 058 551.3	322 988.9
14-4	179.2	5 058 583.6	322 978.3
14-5	177.4	5 058 559.7	322 997.0
14-6	176.9	5 058 579.9	322 998.9

**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.

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

REVISIONS	DATE	BY	DESCRIPTION

GEORES No 41G-22

HWY No 551	DIST
SUBMD AN	CHECKED
DRAWN GBB	CHECKED
DATE 2015-08-24	APPROVED
SITE	DWG 1





Project No.: 165000912

GWP: 5153-12-00

Site Photographs

Project Name: Highway 551, Mindemoya Bridge  
Central Manitoulin, ON

Date: Oct 7, 2014



Site Photo No.: 1

Looking east on the west side of Mindemoya River Bridge



Site Photo No.: 2

Looking north at BH15-2 on the west side of Mindemoya River Bridge





Project No.: 165000912

GWP: 5153-12-00

Site Photographs

Project Name: Highway 551, Mindemoya Bridge  
Central Manitoulin, ON

Date: Oct 7, 2014



Site Photo No.: 3

Looking south at BH15-1 on the west side of Mindemoya River Bridge



Site Photo No.: 4

Looking south at BH15-5 on the east side of Mindemoya River Bridge





Project No.: 165000912

GWP: 5153-12-00

Site Photographs

Project Name: Highway 551, Mindemoya Bridge  
Central Manitoulin, ON

Date: Oct 7, 2014



Site Photo No.: 5

Looking north near BH15-3 on Hwy 551



Site Photo No.: 6

Looking south near BH15-4 on Hwy 551



## APPENDIX B

Symbols and Terms Used on Borehole Records

Borehole Records

Field Bedrock Core Records

Rock Core Photographs

Calibration Data Sheet – Vibrating Wire Pressure Transducer

## SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

### SOIL DESCRIPTION

#### Terminology describing common soil genesis:

<i>Topsoil</i>	- mixture of soil and humus capable of supporting vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

#### Terminology describing soil structure:

<i>Desiccated</i>	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	- having cracks, and hence a blocky structure
<i>Varved</i>	- composed of regular alternating layers of silt and clay
<i>Stratified</i>	- composed of alternating successions of different soil types, e.g. silt and sand
<i>Layer</i>	- > 75 mm in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

#### Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

#### Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

#### Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

#### Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests.

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
<i>Very Soft</i>	<0.25	<12.5
<i>Soft</i>	0.25 - 0.5	12.5 - 25
<i>Firm</i>	0.5 - 1.0	25 - 50
<i>Stiff</i>	1.0 - 2.0	50 - 100
<i>Very Stiff</i>	2.0 - 4.0	100 - 200
<i>Hard</i>	>4.0	>200

## ROCK DESCRIPTION

### Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	<i>Very Poor</i>
25-50	<i>Poor</i>
50-75	<i>Fair</i>
75-90	<i>Good</i>
90-100	<i>Excellent</i>

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

### Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	<i>Extremely Wide</i>	-
2000-6000	<i>Very Wide</i>	<i>Very Thick</i>
600-2000	<i>Wide</i>	<i>Thick</i>
200-600	<i>Moderate</i>	<i>Medium</i>
60-200	<i>Close</i>	<i>Thin</i>
20-60	<i>Very Close</i>	<i>Very Thin</i>
<20	<i>Extremely Close</i>	<i>Laminated</i>
<6	-	<i>Thinly Laminated</i>

### Terminology describing rock strength:

Strength Classification	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	< 1
<i>Very Weak</i>	1 – 5
<i>Weak</i>	5 – 25
<i>Medium Strong</i>	25 – 50
<i>Strong</i>	50 – 100
<i>Very Strong</i>	100 – 250
<i>Extremely Strong</i>	> 250

### Terminology describing rock weathering:

Term	Description
<i>Fresh</i>	No visible signs of rock weathering. Slight discolouration along major discontinuities
<i>Slightly Weathered</i>	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
<i>Moderately Weathered</i>	Less than half the rock is decomposed and/or disintegrated into soil.
<i>Highly Weathered</i>	More than half the rock is decomposed and/or disintegrated into soil.
<i>Completely Weathered</i>	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

## STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders  
Cobbles  
Gravel



Sand



Silt



Clay



Organics



Asphalt



Concrete



Fill



Bedrock

## SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

## WATER LEVEL MEASUREMENT



measured in standpipe,  
piezometer, or well



inferred

## RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

## N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and N-values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

## DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability.

## OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
$\gamma$	Unit weight
$G_s$	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
$Q_u$	Unconfined compression
$I_p$	Point Load Index ( $I_p$ on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer







## RECORD OF BOREHOLE No BH14-1

1 OF 3

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 548 E: 322 974 ORIGINATED BY AN  
DIST Manitoulin HWY 551 BOREHOLE TYPE Casing - Split spoon Sampler COMPILED BY AN  
DATUM Geodetic DATE 2014 11 18 - 2014 11 20 CHECKED BY CM

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 (%)			GR	SA	SI	CL
								○ UNCONFINED	✕ FIELD VANE	● QUICK TRIAXIAL	✕ LAB VANE												
178.5 0.0	Fill: loose to compact brown poorly graded sand (SP)  -moist to wet -some organics		1	GS	-																		
			2	SS	19																		
177.0																							
1.5	SILTY SAND (SM)  Compact to very dense  grey, wet		3	SS	21																		
			4	SS	30																		
			5	SS	74																		
			6	SS	50/ 130mm																		
			7	SS	14																		
173.2 5.3	Well-graded SAND (SW-SM) wih silt and gravel  Compact		8	SS	18																		
172.6 5.9	Grey, wet SILTY SAND (SM)  Compact to very dense  Grey, wet		9	SS	26																		
					10	SS	19																
					11	SS	55																
			12	SS	29																		
			13	SS	32																		

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$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ$  3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000912 - HWY 551, MINDEMOYA BRIDGE - MTO.GPJ ONTARIO MOT.GDT 5/6/15

W.P.	5153-12-00	LOCATION	Mindemoya River Bridge, Central Manitoulin, ON	N: 5 058 548 E: 322 974	ORIGINATED BY	AN	
DIST	Manitoulin	HWY	551	BOREHOLE TYPE	Casing - Split spoon Sampler	COMPILED BY	AN
DATUM	Geodetic	DATE	2014 11 18 - 2014 11 20		CHECKED BY	CM	

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<sup>3</sup>, <sup>3</sup>: Numbers refer to Sensitivity
 <sup>3%</sup> STRAIN AT FAILURE

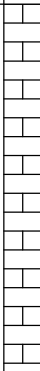


## RECORD OF BOREHOLE No BH14-1

3 OF 3

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 548 E: 322 974 ORIGINATED BY AN  
DIST Manitoulin HWY 551 BOREHOLE TYPE Casing - Split spoon Sampler COMPILED BY AN  
DATUM Geodetic DATE 2014 11 18 - 2014 11 20 CHECKED BY CM

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 (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	● QUICK TRIAXIAL	✕ FIELD VANE	✕ LAB VANE	20						40	60	80
20.0	- close to very close joint spacing  (Refer to Field Bedrock Core Log) Light grey shaly limestone with dolomite seams BEDROCK  - good quality rock - moderately to slightly weathered		22	NQ	-	158										TCR = 89% RQD = 73%				
			23	NQ	-		157											UCS = 117 MPa TCR = 98% RQD = 75%		
156.0	End of Borehole																			
22.5	-Vibrating Wire Piezometer tip installed at 5.48 m below ground surface																			

STN13-ONTARIO MTO STANTEC 165000912 - HWY 551, MINDEMOYA BRIDGE - MTO.GPJ ONTARIO MOT.GDT 5/6/15






# RECORD OF BOREHOLE No BH14-2

1 OF 3

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 578 E: 322 961 ORIGINATED BY AN  
 DIST Manitoulin HWY 551 BOREHOLE TYPE Casing - Split spoon Sampler COMPILED BY AN  
 DATUM Geodetic DATE 2014 11 20 - 2014 11 20 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					w <sub>p</sub>	w	w <sub>L</sub>		GR	SA	SI	CL		
								○ UNCONFINED	✕ FIELD VANE													
								● QUICK TRIAXIAL	✕ LAB VANE													
						20	40	60	80	100												
177.0	0.0	Fill: very loose brown poorly graded sand (SP)		1	SS	WH																
176.3	0.6	-moist -some organics																				
		SANDY SILT (ML)																				
		Compact		2	SS	18																
		Grey, wet																				
				3	SS	19																
				4	SS	15																
				5	SS	14																
				6	SS	16																
				7	SS	28																
171.6	5.3	SILTY SAND (SM)																				
		Compact to dense		8	SS	23																
		Grey, wet																				
				9	SS	36																
				10	SS	19																
				11	SS	22																
				12	SS	17																
				13	SS	22																
167.0																						

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$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ$  3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000912 - HWY 551, MINDEMOYA BRIDGE - MTO.GPJ ONTARIO MOT.GDT 5/6/15



W.P.	5153-12-00	LOCATION	Mindemoya River Bridge, Central Manitoulin, ON	N: 5 058 578 E: 322 961	ORIGINATED BY	AN	
DIST	Manitoulin	HWY	551	BOREHOLE TYPE	Casing - Split spoon Sampler	COMPILED BY	AN
DATUM	Geodetic	DATE	2014 11 20 - 2014 11 20		CHECKED BY	CM	

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<sup>3</sup>, <sup>3</sup>: Numbers refer to Sensitivity
 <sup>3%</sup> STRAIN AT FAILURE

○ 3% STRAIN AT FAILURE



## RECORD OF BOREHOLE No BH14-2

3 OF 3

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 578 E: 322 961 ORIGINATED BY AN  
DIST Manitoulin HWY 551 BOREHOLE TYPE Casing - Split spoon Sampler COMPILED BY AN  
DATUM Geodetic DATE 2014 11 20 - 2014 11 20 CHECKED BY CM

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 (%)				
							20	40	60	80	100		10	20	30		GR SA SI CL
20.0 156.8 20.2	Light grey shaly limestone BEDROCK with dolomite seams  - poor to fair quality rock - moderately to slightly weathered End of Borehole  -Vibrating Wire Piezometer tip installed at 1.65 m below ground surface																

STN13-ONTARIO MTO STANTEC 165000912 - HWY 551, MINDEMOYA BRIDGE - MTO.GPJ ONTARIO MOT.GDT 5/6/15



# RECORD OF BOREHOLE No BH14-3

1 OF 3

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 551 E: 322 989 ORIGINATED BY AN  
 DIST Manitoulin HWY 551 BOREHOLE TYPE Casing - Split spoon Sampler COMPILED BY AN  
 DATUM Geodetic DATE 2014 11 25 - 2014 11 26 CHECKED BY CM

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	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									
178.9																				
178.0	20 mm Asphalt Fill: dense to compact brown silty sand (SM) with gravel -moist		1	SS	34															
			2	SS	28															
177.5	Poorly graded SAND (SP) Loose to compact Brown, wet		3	SS	9															
1.4			4	SS	11															
176.0	SANDY SILT (ML) Dense to compact Grey, wet		5	SS	36															
2.9			6	SS	10															
			7	SS	12															
			8	SS	24															
			9	SS	21															
			10	SS	16															
			11	SS	41															
			12	SS	32															
			13	SS	43															
168.9																				

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$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ$  3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000912 - HWY 551, MINDEMOYA BRIDGE - MTO.GPJ ONTARIO MOT.GDT 5/6/15



## RECORD OF BOREHOLE No BH14-3

2 OF 3

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 551 E: 322 989 ORIGINATED BY AN  
DIST Manitoulin HWY 551 BOREHOLE TYPE Casing - Split spoon Sampler COMPILED BY AN  
DATUM Geodetic DATE 2014 11 25 - 2014 11 26 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>		WATER CONTENT (%)			
								○ UNCONFINED      × FIELD VANE									
						● QUICK TRIAXIAL      × LAB VANE											
						20   40   60   80   100											
10.0	SANDY SILT (ML)  Dense to compact  Grey, wet		14	SS	43												
			15	SS	37		168										
							167										
			16	SS	50		166										
							165										
			17	SS	44												
							164										
			18	SS	38		163										
							162										
			19	SS	30												
							161										
			20	SS	12		160										
							159										

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×<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000912 - HWY 551, MINDEMOYA BRIDGE - MTO.GPJ ONTARIO MOT.GDT 5/6/15



## RECORD OF BOREHOLE No BH14-3

3 OF 3

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 551 E: 322 989 ORIGINATED BY AN  
DIST Manitoulin HWY 551 BOREHOLE TYPE Casing - Split spoon Sampler COMPILED BY AN  
DATUM Geodetic DATE 2014 11 25 - 2014 11 26 CHECKED BY CM

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 (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	● QUICK TRIAXIAL	✕ FIELD VANE	✕ LAB VANE	20						40	60	80
188.6 20.1	SANDY SILT (ML)  Dense to compact  Grey, wet Limestone BEDROCK with shaly partings and dolomite seams  - fair quality rock - Grey - moderately to slightly weathered - Close to very close joint spacing  (Refer to Field Bedrock Core Log)		21	NQ	-	158										UCS = 142 MPa TCR = 98% RQD = 64%				
156.6 22.3	End of Borehole		22	NQ	-		157											TCR = 73% RQD = 66% UCS = 124 MPa		

$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ$  3% STRAIN AT FAILURE



# RECORD OF BOREHOLE No BH14-4

1 OF 3

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 584 E: 322 978 ORIGINATED BY AN  
 DIST Manitoulin HWY 551 BOREHOLE TYPE Hollow Stem Augurs - Split spoon Sampler COMPILED BY AN  
 DATUM Geodetic DATE 2014 11 17 - 2014 11 25 CHECKED BY CM

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							PLASTIC LIMIT  W <sub>P</sub>	NATURAL MOISTURE CONTENT  W	LIQUID LIMIT  W <sub>L</sub>
179.2																	
178.8	40 mm Asphalt																
	FILL: loose to compact brown silty sand (SM) with gravel		1	GS	-												
	-moist		2	SS	10											23 58 (19)	
			3	SS	10												
			4	SS	31												
176.7	SILTY SAND (SM)		5	SS	61												
2.6	Very dense to compact		6	SS	98												
	Grey, wet		7	SS	34											0 62 36 2	
			8	SS	31												
			9	SS	11												
			10	SS	19												
			11	SS	29												
			12	SS	29												
170.1	SANDY SILT (ML)		13	SS	21											0 45 53 2	
9.1	Compact to dense																
	Grey, wet																
169.2																	

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$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ$  3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165000912 - HWY 551, MINDEMOYA BRIDGE - MTO.GPJ ONTARIO MOT.GDT 5/6/15




## RECORD OF BOREHOLE No BH14-4

2 OF 3

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 584 E: 322 978 ORIGINATED BY AN  
DIST Manitoulin HWY 551 BOREHOLE TYPE Hollow Stem Augurs - Split spoon Sampler COMPILED BY AN  
DATUM Geodetic DATE 2014 11 17 - 2014 11 25 CHECKED BY CM

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 <sub>P</sub> W                      W <sub>L</sub>			GR	SA	SI	CL	
								20   40   60   80   100						20   40   60   80   100							
10.0	SANDY SILT (ML)  Compact to dense  Grey, wet		14	SS	19		169														
			15	SS	25		168														
			16	SS	27		167														
			17	SS	32		166														
			18	SS	10		165														










## RECORD OF BOREHOLE No BH14-5

1 OF 1

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 560 E: 322 997 ORIGINATED BY AN  
DIST Manitoulin HWY 551 BOREHOLE TYPE Split spoon Sampler COMPILED BY AN  
DATUM Geodetic DATE 2014 11 26 - 2014 11 26 CHECKED BY CM

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 (%)			GR	SA	SI	CL
													20					40	60	80				
177.4 0.0	FILL: loose brown silty sand (SM) with gravel  -moist		1	SS	-																			
176.7 0.6	SILTY SAND (SM)  Compact		2	SS	-																			
176.1 1.2	Grey, wet End of Borehole																							

$\times^3, \times^3$ : Numbers refer to Sensitivity  $\circ^3$  3% STRAIN AT FAILURE



## RECORD OF BOREHOLE No BH14-6

1 OF 1

METRIC

W.P. 5153-12-00 LOCATION Mindemoya River Bridge, Central Manitoulin, ON N: 5 058 580 E: 322 999 ORIGINATED BY AN  
DIST Manitoulin HWY 551 BOREHOLE TYPE Split spoon Sampler COMPILED BY AN  
DATUM Geodetic DATE 2014 11 26 - 2014 11 26 CHECKED BY CM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	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>	kN/m <sup>3</sup>	GR SA SI CL
176.9 0.0	SILTY SAND (SM)  Loose to compact  Grey, moist to wet -some organics		1	SS	-											
			2	SS	-											
175.7 1.2	End of Borehole															

**Client:** Ministry of Transportation Ontario  
**Project:** Hwy 551, Mindemoya Bridge  
**Contractor:** Landcore Drilling

**Project No.:** 165000912  
**Date:** December 3, 2014  
**Borehole No.:** BH14-1  
**Logger:** Athir Nader

DEPTH FROM	RUN NO.	% CORE RECOVERY	% RQD	DEPTH TO	GENERAL DESCRIPTION (Rock Type/s, %, Colour, Texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES							OCCASIONAL FEATURES	DRILLING OBSERVATIONS
								NO. OF SETS	TYPE/S	ORIENTATION	SPACING	ROUGHNESS	APERTURE	FILLING		
19.25	21	98%	85%	19.86	Shaly Limestone with dolomatic seams, Light grey, Fine grained	MS-VS	M-S	2	B	F	C-VC	SU	-	T		
19.86	22	89%	73%	21.43	Shaly Limestone with dolomatic seams, Light grey, Fine grained	VS	S	1	B	F	VC	SU	-	T		
21.43	23	98%	75%	22.50	Shaly Limestone with dolomatic seams, Light grey, Fine grained	VS	S	1	B	F	VC	SU	-	T		
<div> <div> <b>STRENGTH (MPa)</b>            EH = Extremely Strong = &gt; 250            VS = Very Strong = 100-250            S = Strong = 50-100            MS = Medium Strong = 25-50            W = Weak = 5 - 25         </div> <div> <b>WEATHERING</b>            U = Unweathered = No Signs            S = Slightly = Oxidized            M = Moderately = Discoloured            H = Highly = Friable            C = Completely = Soil-like         </div> <div> <b>DISCONTINUITY TYPE</b>            B = Bedding Joint            J = Cross Joint            F = Fault            S = Shear Plane         </div> <div> <b>SPACING</b>            VW = Very Wide = &gt;3m            W = Wide = 1-3 m            M = Moderate = 0.3-1 m            C = Close = 5-30 cm            VC = Very Close = &lt;5 cm         </div> <div> <b>ORIENTATION</b>            F = Flat = 0-20°            D = Dipping = 20-50°            V = n-Vertical = &gt;50°         </div> <div> <b>ROUGHNESS</b>            RU = Rough Undulating            RP = Rough Planar            SU = Smooth Undulating            SP = Smooth Planar            LU = Slickensided Undulating            LP = Slickensided Planar         </div> <div> <b>FILLING</b>            T = Tight, Hard            O = Oxidized            SA = Slightly Altered, Clay Free            S = Sandy, Clay Free            Si = Sandy, Silty, Minor Clay            NC = Non-softening Clay            SC = Swelling, Soft Clay         </div> </div>																

<b>Project No.:</b>	165000912
<b>Date:</b>	December 3, 2014
<b>Borehole No.:</b>	BH14-2
<b>Logger:</b>	Athir Nader

Page 1 of 1

**Client:** Ministry of Transportation Ontario  
**Project:** Hwy 551, Mindemoya Bridge  
**Contractor:** Landcore Drilling

**Project No.:** 165000912  
**Date:** December 3, 2014  
**Borehole No.:** BH14-3  
**Logger:** Athir Nader

DEPTH FROM	RUN NO.	% CORE RECOVERY	% RQD	DEPTH TO	GENERAL DESCRIPTION (Rock Type/s, %, Colour, Texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES							OCCASIONAL FEATURES	DRILLING OBSERVATIONS
								NO. OF SETS	TYPE/S	ORIENTATION	SPACING	ROUGHNESS	APERTURE	FILLING		
20.11	21	98%	64%	21.32	Shaly Limestone with dolomatic seams, Light grey, Fine grained	MS-VS	M-S	2	B	F	C-VC	SU	-	T		
21.32	22	73%	66%	22.30	Shaly Limestone with dolomatic seams, Light grey, Fine grained	VS	S	1	B	F	VC	SU	-	T		
<div> <div> <b>STRENGTH (MPa)</b>            EH = Extremely Strong = &gt; 250            VS = Very Strong = 100-250            S = Strong = 50-100            MS = Medium Strong = 25-50            W = Weak = 5 - 25  <b>WEATHERING</b>            U = Unweathered = No Signs            S = Slightly = Oxidized            M = Moderately = Discoloured            H = Highly = Friable            C = Completely = Soil-like         </div> <div> <b>DISCONTINUITY TYPE</b>            B = Bedding Joint            J = Cross Joint            F = Fault            S = Shear Plane  <b>SPACING</b>            VW = Very Wide = &gt;3m            W = Wide = 1-3 m            M = Moderate = 0.3-1 m            C = Close = 5-30 cm            VC = Very Close = &lt;5 cm         </div> <div> <b>ORIENTATION</b>            F = Flat = 0-20°            D = Dipping = 20-50°            V = n-Vertical = &gt;50°  <b>ROUGHNESS</b>            RU = Rough Undulating            RP = Rough Planar            SU = Smooth Undulating            SP = Smooth Planar            LU = Slickensided Undulating            LP = Slickensided Planar         </div> <div> <b>FILLING</b>            T = Tight, Hard            O = Oxidized            SA = Slightly Altered, Clay Free            S = Sandy, Clay Free            Si = Sandy, Silty, Minor Clay            NC = Non-softening Clay            SC = Swelling, Soft Clay         </div> </div>																

<b>Project No.:</b>	165000912
<b>Date:</b>	December 3, 2014
<b>Borehole No.:</b>	BH14-4
<b>Logger:</b>	Athir Nader

Page 1 of 1



Project No.: 165000912

Project Name: Hwy 551, Mindemoya Bridge

Rock Core Photographs



Rock Core Photo No.: 1

Borehole: BH14-1

Depth: 19.25 - 22.50



Rock Core Photo No.: 2

Borehole: BH14-2

Depth: 16.86 - 20.20





Project No.: 165000912

Project Name: Hwy 551, Mindemoya Bridge

Rock Core Photographs



Rock Core Photo No.: 3

Borehole: BH14-3

Depth: 20.11 - 22.30



Rock Core Photo No.: 4

Borehole: BH14-4

Depth: 19.38 - 22.20





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geotechnical  
instrumentation

## Calibration Record

RST Instruments Ltd., 11545 Kingston St., Maple Ridge, British Columbia, Canada V2X 0Z5  
Tel: 604 540 1100 • Fax: 604 540 1005 • Toll Free: 1 800 665 5599 (North America only)  
e-mail: info@rstinstruments.com • Website: www.rstinstruments.com

### Vibrating Wire Piezometer

Customer: Hoskin Scientific Ltd  
Model: VW2100-0 35  
Serial Number: VW30079  
Mfg Number: 1423519  
Range: 350.0 kPa  
Temperature: 24.0 °C  
Barometric Pressure: 991.1 millibars  
Work Order Number: 204804  
Cable Length: 10 meters  
Cable Markings: 615748 m - 615757 m  
Cable Colour Code: Red / Black (Coil) Green / White (Thermistor)  
Cable Type: EL380004  
Thermistor Type: 3 kΩ

Applied Pressure (kPa)	First Reading (B units)	Second Reading (B units)	Average Reading (B units)	Calculated Linear (kPa)	Linearity Error (% FS)	Polynomial Error (% FS)
0.0	9035	9035	9035	0.1	0.02	0.03
70.0	8404	8404	8404	69.8	-0.06	-0.06
140.0	7767	7768	7768	140.1	0.03	0.03
210.0	7134	7135	7135	210.0	0.01	0.00
279.9	6501	6502	6502	280.0	0.02	0.02
350.0	5868	5868	5868	349.9	-0.02	-0.01
Max. Error (%):					0.06	0.06

Linear Calibration Factor: C.F. = 0.11047 kPa/B unit  
Regression Zero: At Calibration = 9035.8 B unit  
Temperature Correction Factor: Tk = -0.1202 kPa/°C rise

Polynomial Gage Factors (kPa) A: 1.1998E-08 B: -0.11065 C: 998.82

Pressure is calculated with the following equations:

Linear:  $P(\text{kPa}) = C.F. (L - L_c) - [Tk(T - T_c)] + [0.10(B - B_c)]$

Polynomial:  $P(\text{kPa}) = A(L_c)^2 + BL_c + C + Tk(T_c - T) - [0.10(B_c - B)]$

	Date (dd/mm/yy)	VW Readout Pos. B (Li)	Temp °C (Ti)	Baro (Bi)
Shipped Zero Readings:	26-Aug-14	9029	22.8	1017.3

Li, Lc = initial (at installation) and current readings

Ti, Tc = initial (at installation) and current temperature, in °C

Bi, Bc = initial (at installation) and current barometric pressure readings, in millibars

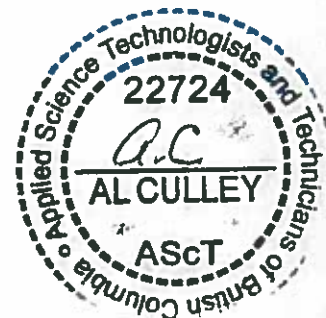
B units = B scale output of VW 2102, VW 2104, VW 2106 and DT 2011 readouts

B units =  $\text{Hz}^2 / 1000$  ie: 1700Hz = 2890 B units

Technician: H. Chang

Date: 26-Aug-14

This instrument has been calibrated using standards traceable to the NIST in compliance with ANSI Z540-1





innovation in  
geotechnical  
instrumentation

## Calibration Record

RST Instruments Ltd., 11545 Kingston St., Maple Ridge, British Columbia, Canada V2X 0Z5  
Tel: 604 540 1100 • Fax: 604 540 1005 • Toll Free: 1 800 665 5599 (North America only)  
e-mail: info@rstinstruments.com • Website: www.rstinstruments.com

### Vibrating Wire Piezometer

Customer: Hoskin Scientific Ltd  
Model: VW2100-0.35  
Serial Number: VW30080  
Mfg Number: 1423520  
Range: 350.0 kPa  
Temperature: 24.0 °C  
Barometric Pressure: 991.1 millibars  
Work Order Number: 204804  
Cable Length: 10 meters  
Cable Markings: 615738 m - 615747 m  
Cable Colour Code: Red / Black (Coil) Green / White (Thermistor)  
Cable Type: EL380004  
Thermistor Type: 3 kΩ

Applied Pressure (kPa)	First Reading (B units)	Second Reading (B units)	Average Reading (B units)	Calculated Linear (kPa)	Linearity Error (% FS)	Polynomial Error (% FS)
0.0	8851	8852	8852	-0.3	-0.08	0.00
70.0	8190	8190	8190	70.1	0.02	0.00
140.0	7530	7530	7530	140.2	0.06	0.00
210.0	6872	6872	6872	210.2	0.05	-0.01
279.9	6215	6215	6215	280.0	0.04	0.02
350.0	5560	5560	5560	349.7	-0.09	-0.01
Max. Error (%):					0.09	0.02

Linear Calibration Factor: C.F. = 0.10632 kPa/B unit  
Regression Zero: At Calibration = 8848.9 B unit  
Temperature Correction Factor: Tk = -0.1457 kPa/°C rise

Polynomial Gage Factors (kPa) A: 1.9901E-07 B: -0.10919 C: 950.90

Pressure is calculated with the following equations

Linear:  $P(\text{kPa}) = C.F. (Li - Lc) - [Tk(Ti - Tc)] + [0.10(Bi - Bc)]$

Polynomial:  $P(\text{kPa}) = A(Lc)^2 + B.Lc + C + Tk(Tc - Ti) - [0.10(Bc - Bi)]$

	Date (dd/mm/yy)	VW Readout Pos B (Li)	Temp °C (Ti)	Baro (Bi)
Shipped Zero Readings:	<u>26-Aug-14</u>	<u>8843</u>	<u>23.0</u>	<u>1017.3</u>

Li, Lc = initial (at installation) and current readings

Ti, Tc = initial (at installation) and current temperature, in °C

Bi, Bc = initial (at installation) and current barometric pressure readings, in millibars

B units = B scale output of VW 2102, VW 2104, VW 2106 and DT 2011 readouts

B units = Hz<sup>2</sup> / 1000 ie 1700Hz = 1700 B units

Technician H. Chang

Date 26-Aug-14

This instrument has been calibrated using standards traceable to the NIST in compliance with ANSI Z540-1



Document Number: ELL0130K



## APPENDIX C

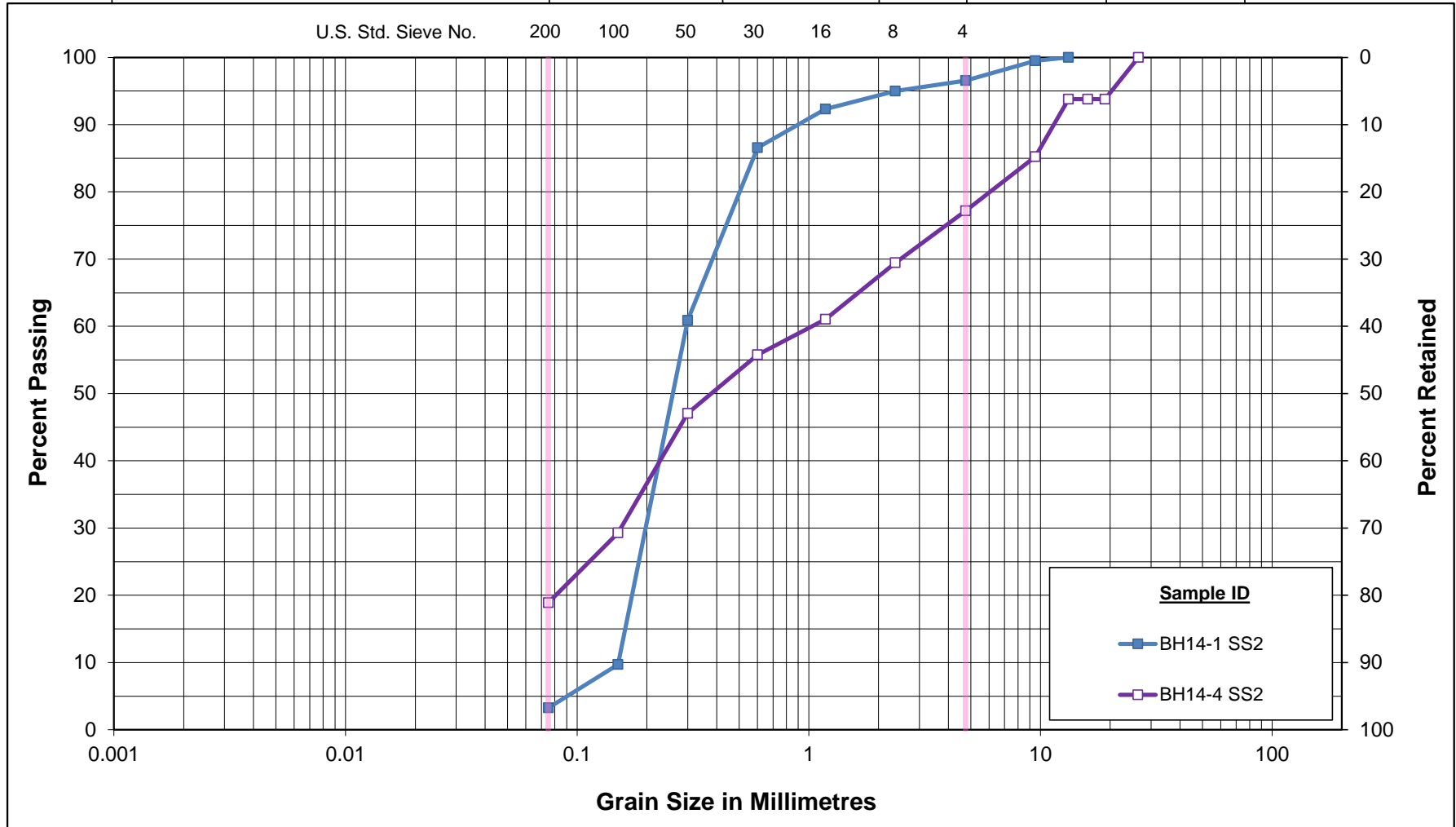
Laboratory Test Results

Figures 1 to 6: Grain Size Distribution Plots

Chemical Test Results

# Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse



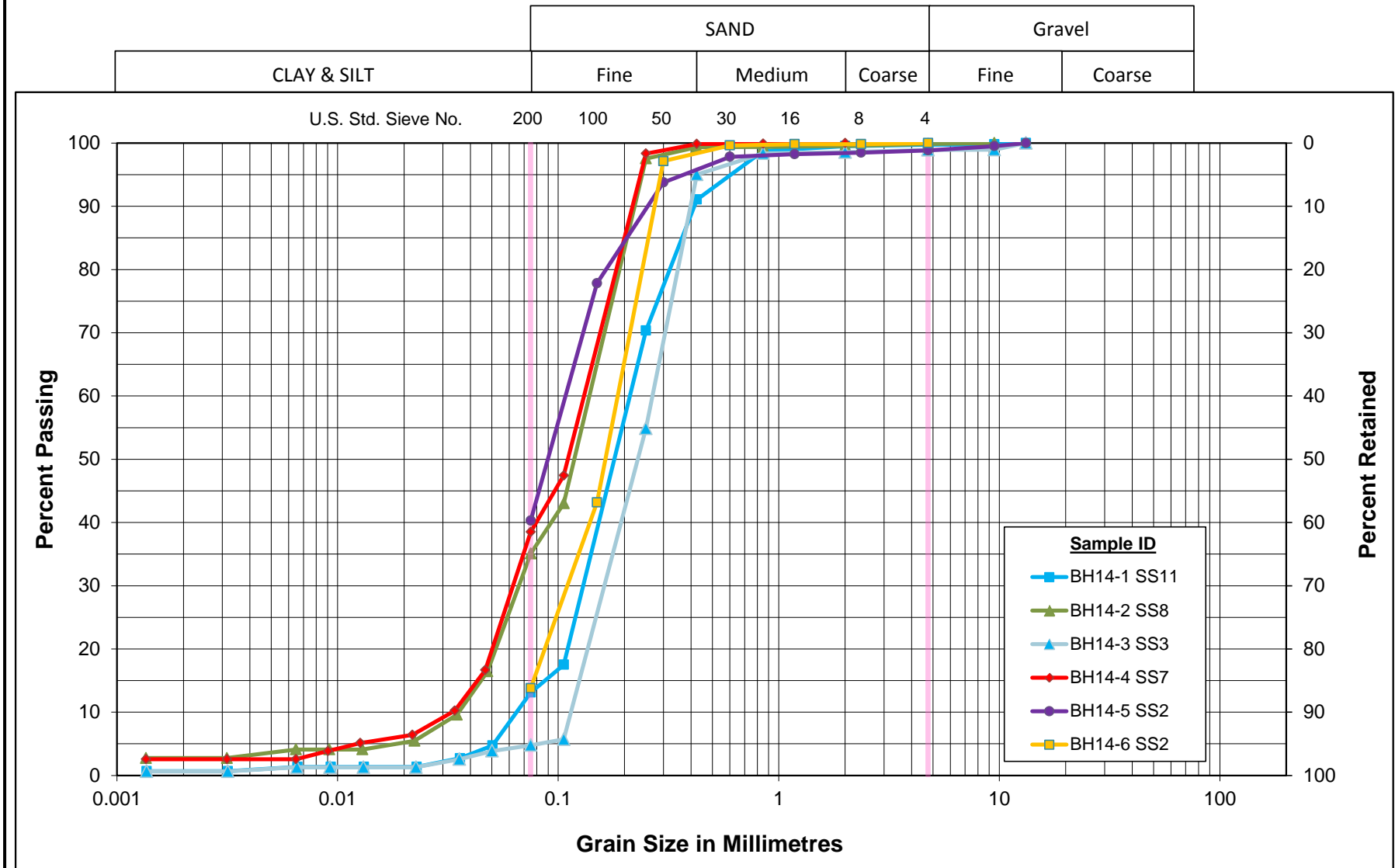
## GRAIN SIZE DISTRIBUTION

FILL: Poorly graded SAND (SP)  
to silty SAND (SM) with gravel

Figure No. 1

Project No. 165000912

# Unified Soil Classification System



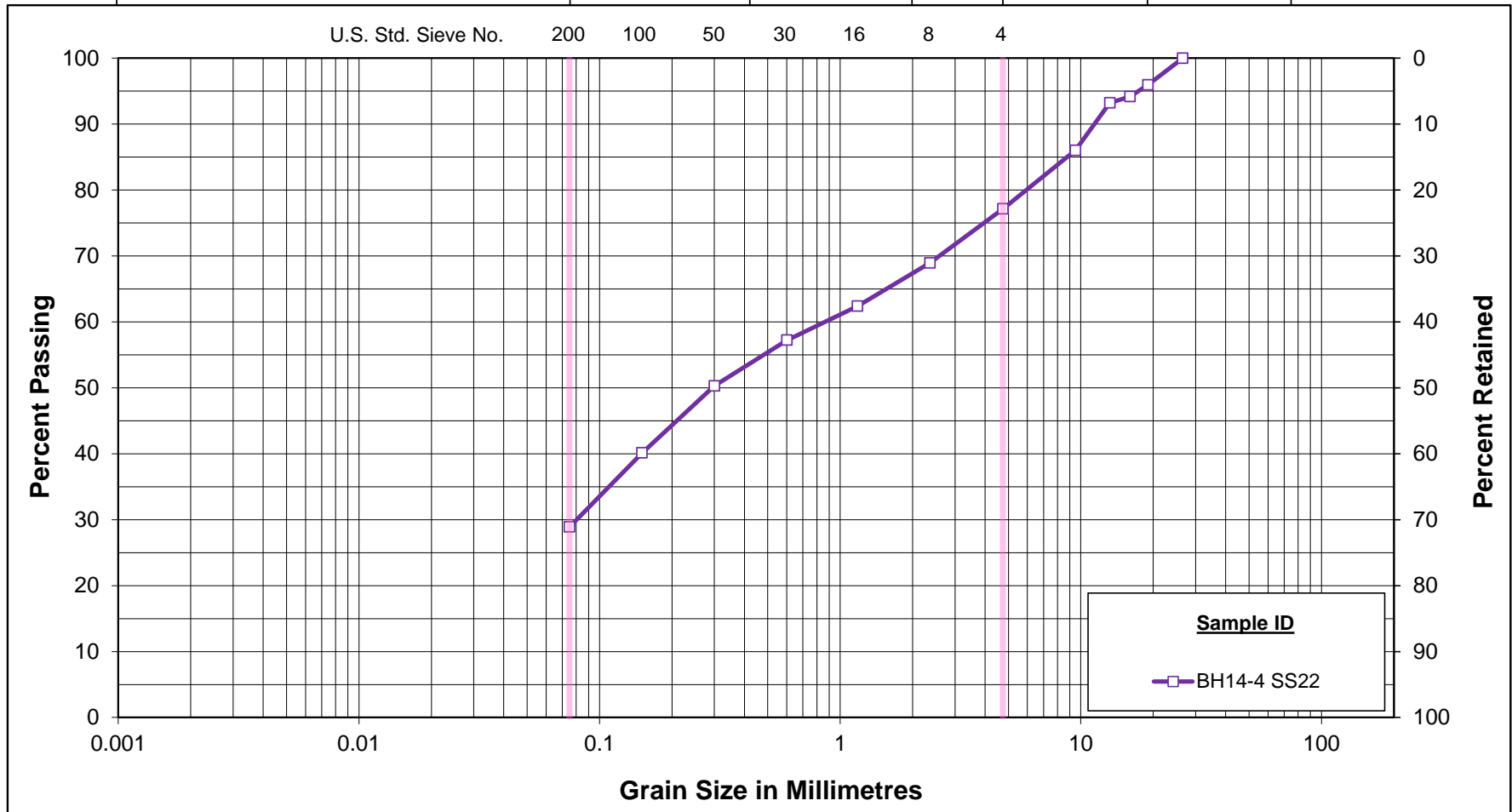
GRAIN SIZE DISTRIBUTION  
SILTY SAND (SM)  
to poorly graded SAND (SP)

Figure No. 2

Project No. 165000912

# Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse



## GRAIN SIZE DISTRIBUTION

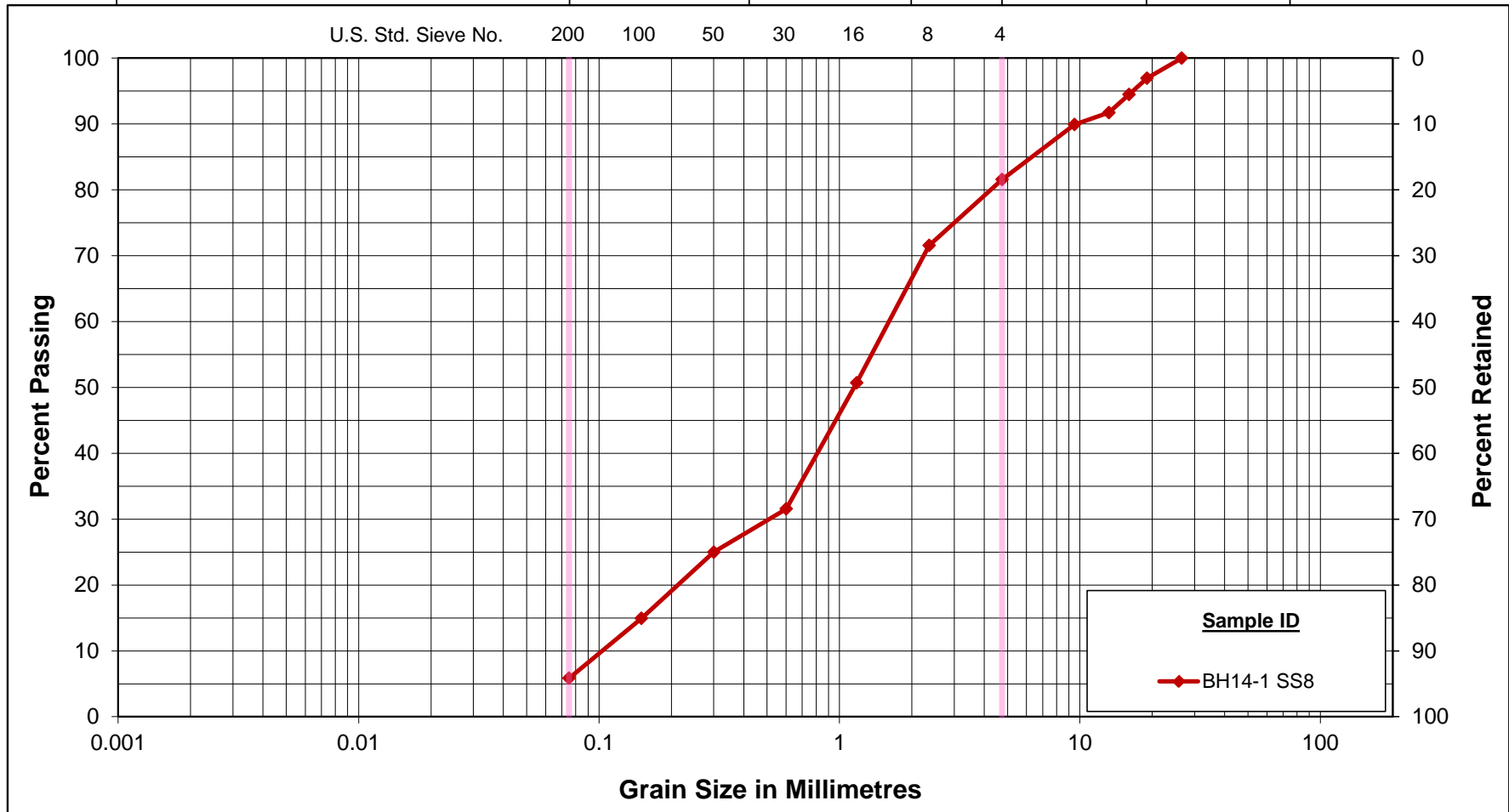
SILTY SAND (SM) with gravel

Figure No. 3

Project No. 165000912

# Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse



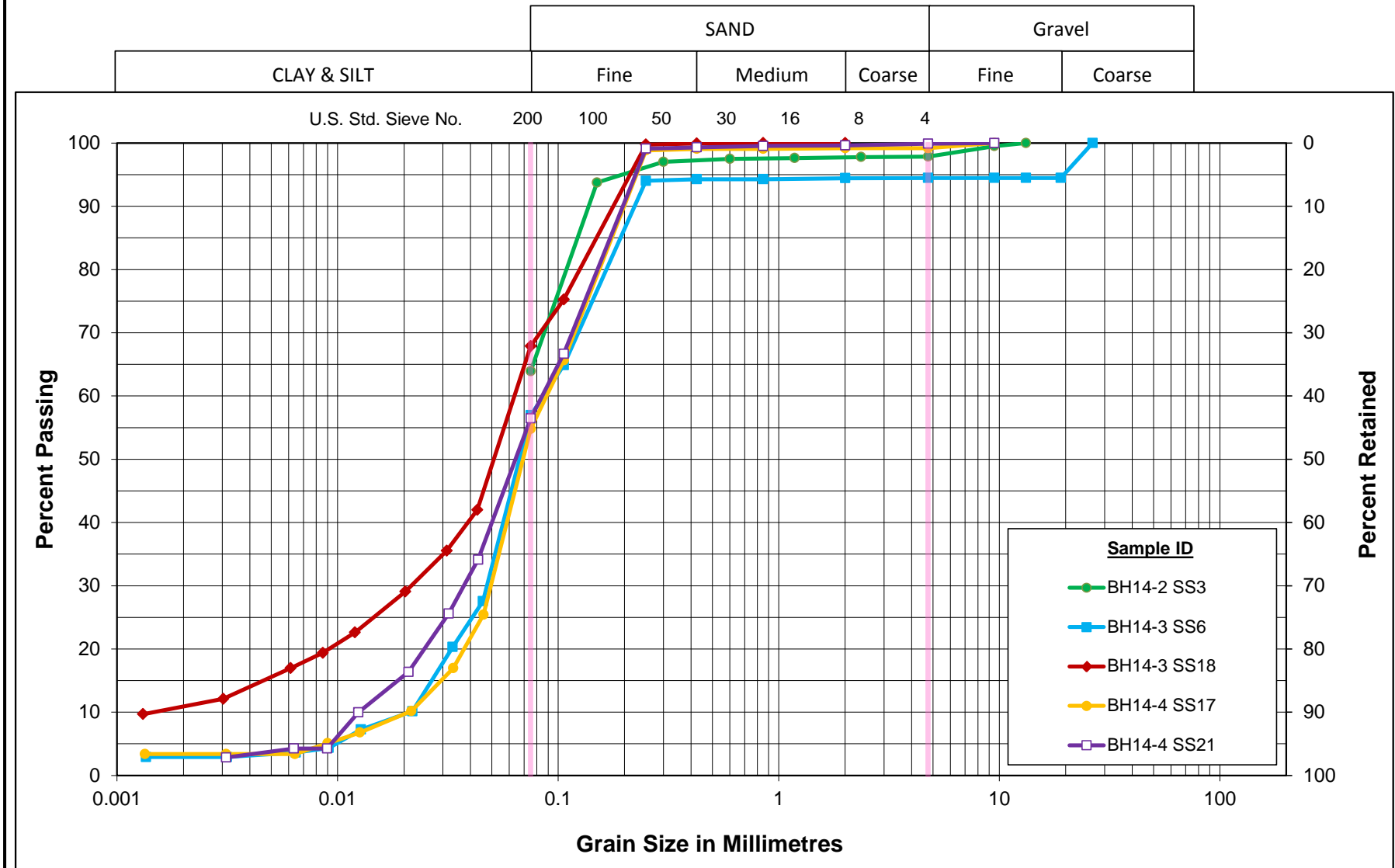
## GRAIN SIZE DISTRIBUTION

Well graded SAND (SW-SM) with silt and gravel

Figure No. 4

Project No. 165000912

# Unified Soil Classification System



GRAIN SIZE DISTRIBUTION  
SANDY SILT (ML)

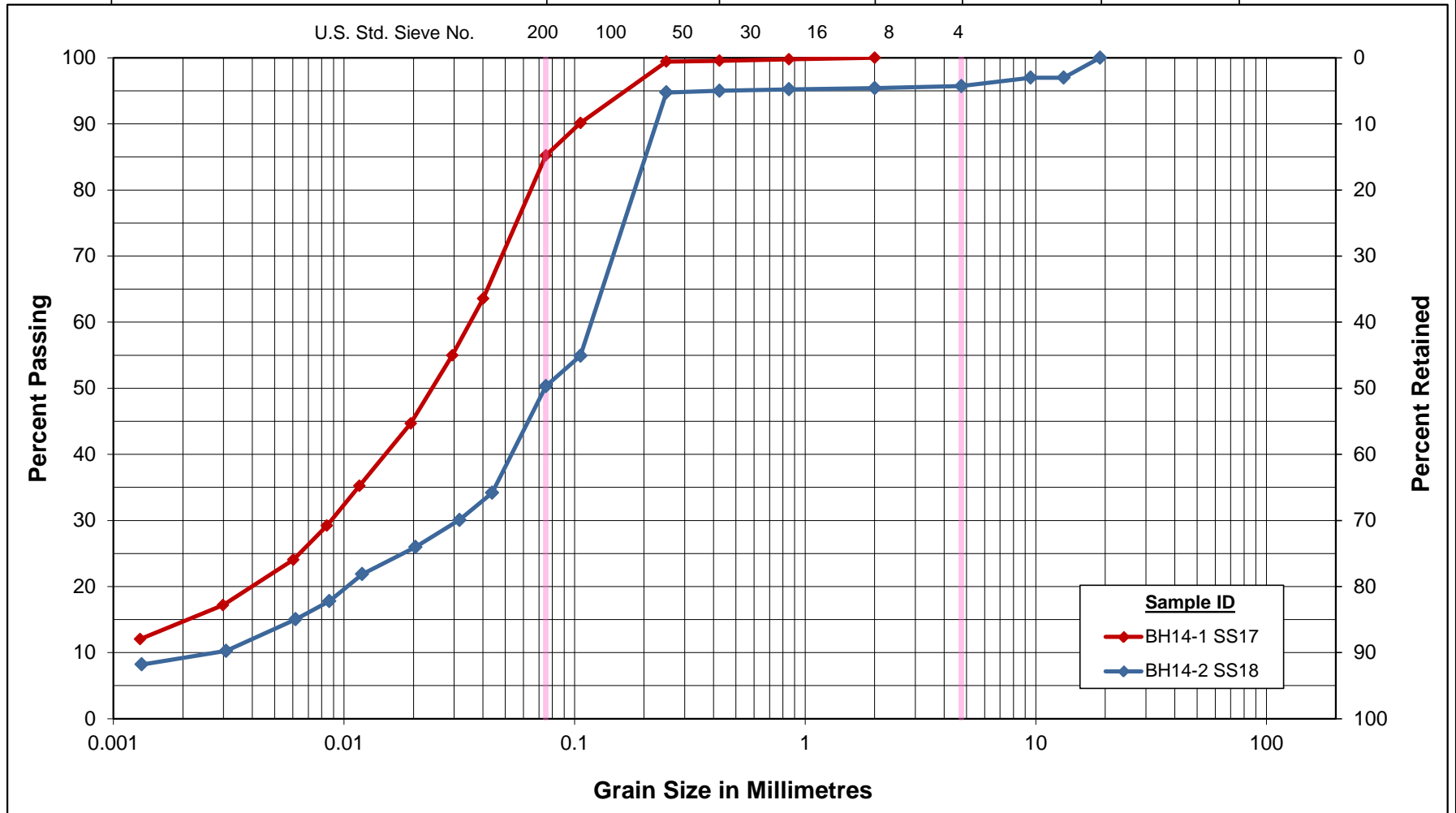
Figure No. 5

Project No. 165000912



# Unified Soil Classification System

			SAND			Gravel	
CLAY & SILT			Fine	Medium	Coarse	Fine	Coarse



## GRAIN SIZE DISTRIBUTION

SILT (ML)  
to SANDY SILT (ML)

Figure No. 6

Project No. 165000912

## Certificate of Analysis

### Stantec Consulting Ltd. (Ottawa)

1331 Clyde Avenue Suite 400  
Ottawa, ON K2C 3G4  
Attn: Athir Nader

Phone: (613) 722-4420  
Fax: (613) 738-0721

Client PO:  
Project: 165000912.200  
Custody:

Report Date: 22-Dec-2014  
Order Date: 16-Dec-2014

**Order #: 1451088**

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1451088-01	BH14-1 SS7 15'-17'
1451088-02	BH14-1 SS16 45'-47'
1451088-03	BH14-4 SS7 15'-17'
1451088-04	BH14-4 SS18 40'-42'

Approved By:



Mark Foto, M.Sc. For Dale Robertson, BSc  
Laboratory Director

Any use of these results implies your agreement that our total liability in connection with this work, however arising shall be limited to the amount paid by you for this work, and that our employees or agents shall not under circumstances be liable to you in connection with this work

**Certificate of Analysis**Client: **Stantec Consulting Ltd. (Ottawa)**

Client PO:

Project Description: 165000912.200

Report Date: 22-Dec-2014

Order Date: 16-Dec-2014

**Analysis Summary Table**

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	18-Dec-14	19-Dec-14
pH	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	16-Dec-14	17-Dec-14
Resistivity	EPA 120.1 - probe, water extraction	17-Dec-14	17-Dec-14
Solids, %	Gravimetric, calculation	17-Dec-14	17-Dec-14

P: 1-800-749-1947  
E: PARACEL@PARACELLABS.COM

WWW.PARACELLABS.COM

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300-2319 St. Laurent Blvd.  
Ottawa, ON K1G 4J8

**OTTAWA - WEST**  
104-195 Stafford Rd. W.  
Nepean, ON K2H 9C1

**MISSISSAUGA**  
6645 Kitimat Rd. Unit #27  
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**SARNIA**  
218-704 Mara St.  
Point Edward, ON N7V 1X4

**NIAGARA**  
360 York Rd. Unit 16B  
Niagara-on-the-Lake, ON L0S 1J0

**KINGSTON**  
1058 Gardiners Rd.  
Kingston, ON K7P 1R7

**Certificate of Analysis**

Client: **Stantec Consulting Ltd. (Ottawa)**

Report Date: 22-Dec-2014

Client PO:

Project Description: 165000912.200

Order Date: 16-Dec-2014

Client ID:	BH14-1 SS7 15'-17'	BH14-1 SS16 45'-47'	BH14-4 SS7 15'-17'	BH14-4 SS18 40'-42'
Sample Date:	19-Nov-14	19-Nov-14	24-Nov-14	24-Nov-14
Sample ID:	1451088-01	1451088-02	1451088-03	1451088-04
MDL/Units	Soil	Soil	Soil	Soil

**Physical Characteristics**

% Solids	0.1 % by Wt.	87.8	86.8	86.1	50.7
----------	--------------	------	------	------	------

**General Inorganics**

pH	0.05 pH Units	7.86	7.74	7.86	7.79
Resistivity	0.10 Ohm.m	57.5	49.6	50.1	59.5

**Anions**

Chloride	5 ug/g dry	10	9	10	10
Sulphate	5 ug/g dry	86	122	113	102

**Certificate of Analysis**

Client: **Stantec Consulting Ltd. (Ottawa)**

Report Date: 22-Dec-2014

Client PO:

Project Description: 165000912.200

Order Date: 16-Dec-2014

**Method Quality Control: Blank**

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
<b>Anions</b>									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
<b>General Inorganics</b>									
Resistivity	ND	0.10	Ohm.m						

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 Niagara-on-the-Lake, ON L0S 1J0

**KINGSTON**  
 1058 Gardiners Rd.  
 Kingston, ON K7P 1R7

**Certificate of Analysis**

Client: **Stantec Consulting Ltd. (Ottawa)**

Report Date: 22-Dec-2014

Client PO:

Project Description: 165000912.200

Order Date: 16-Dec-2014

**Method Quality Control: Duplicate**

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
<b>Anions</b>									
Chloride	10.5	5	ug/g dry	10.4			0.9	20	
Sulphate	90.6	5	ug/g dry	85.9			5.3	20	
<b>General Inorganics</b>									
pH	7.38	0.05	pH Units	7.40			0.3	10	
Resistivity	59.2	0.10	Ohm.m	57.5			2.9	20	
<b>Physical Characteristics</b>									
% Solids	90.0	0.1	% by Wt.	89.5			0.6	25	

**P:** 1-800-749-1947  
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**NIAGARA**  
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**KINGSTON**  
 1058 Gardiners Rd.  
 Kingston, ON K7P 1R7

**Certificate of Analysis**

Client: **Stantec Consulting Ltd. (Ottawa)**

Report Date: 22-Dec-2014

Client PO:

Project Description: 165000912.200

Order Date: 16-Dec-2014

**Method Quality Control: Spike**

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
<b>Anions</b>									
Chloride	98.0	5	ug/g	10.4	87.6	78-113			
Sulphate	184	5	ug/g	85.9	97.8	78-111			

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**KINGSTON**  
 1058 Gardiners Rd.  
 Kingston, ON K7P 1R7

**Certificate of Analysis**Client: **Stantec Consulting Ltd. (Ottawa)**

Client PO:

Project Description: 165000912.200

Report Date: 22-Dec-2014

Order Date: 16-Dec-2014

**Qualifier Notes:**

None

**Sample Data Revisions**

None

**Work Order Revisions / Comments:**

None

**Other Report Notes:**

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

P: 1-800-749-1947  
E: PARACEL@PARACELLABS.COM

WWW.PARACELLABS.COM

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NIAGARA  
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Niagara-on-the-Lake, ON L0S 1J0

KINGSTON  
1058 Gardiners Rd.  
Kingston, ON K7P 1R7



OTTAWA • KINGSTON • NIAGARA • MISSISSAUGA • SARNIA

Client Name: Ministry of Transportation Ontario

*Stantec*

Project Reference: 165000912

Contact Name: Athir Nader, Chris McGrath

Quote # 200

Address: 1331 Clyde Ave Ottawa ON K2C 3G4

Telephone: Phone: (613) 784-2244, (613) 738-6070

Email Address:

athir.nader@stantec.com, christopher.mcgrath@stantec.com

Page 1 of 1

TAT: ☐ Regular ☐ 3 Day

☐ 2 Day ☐ 1 Day

Date Required:

Criteria: ☐ O. Reg. 153/04 Table ☐ O. Reg. 153/11 (Current) Table ☐ RSC Filing ☐ O. Reg. 558/00 ☐ PWQO ☐ CCME ☐ SUB (Storm) ☐ SUB (Sanitary) Municipality: ☐ Other: ☐

Matrix Type: S (Soil/Sed.) GW (Ground Water) SW (Surface Water) SS (Storm/Sanitary Sewer) P (Paint) A (Air) O (Other)

Required Analyses

Parcel Order Number:

*1451088*

Sample ID/Location Name

Matrix

Air Volume

# of Containers

Sample Taken

Date

Time

Resistivity

pH

Sulphates & Chloride

1 BH14-1 SS7 15'-17'

soil

19-Nov-14

x

x

x

2 BH14-1 SS16 45'-47'

soil

19-Nov-14

x

x

x

3 BH14-4 SS7 15'-17'

soil

11/24/2014

x

x

x

4 BH14-4 SS18 40'-42'

soil

11/24/2014

x

x

x

5

6

7

8

9

10

*- 2TPOCK BAG*

*↓*

Comments:

Method of Delivery:

*Swift*

Relinquished By (Print & Sign):

*Kyle Polito*

*[Signature]*

Received by Driver/Depot:

Received at Lab:

*SUNEPORN DUKMAJ*

Verified By:

*My C [Signature]*

Date/Time:

Date/Time: *DEC 16, 2014*

Date/Time:

*Dec 16/14 1:39*

Temperature:  °C

Temperature: *13.1* °C

pH Verified ☐ By:

## APPENDIX D

Figure 7: Preliminary Design Parameters

Figure 8: Characterization of Liquefaction Assessment

Figure 9: Section A-A (Static)

Figure 10: Section A-A (Seismic)

Figure 11: Section B-B (Static)

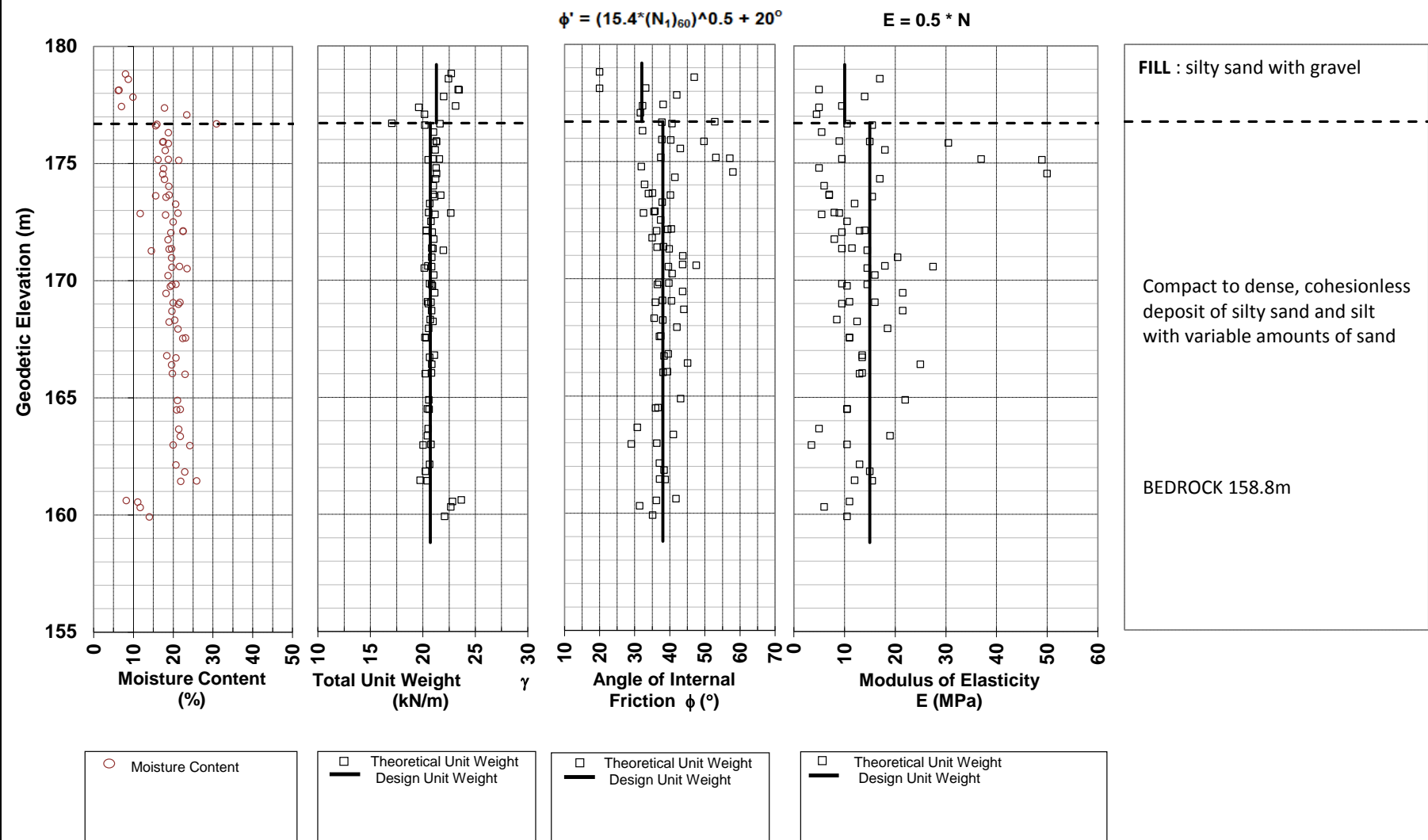
Figure 12: Section B-B (Seismic)

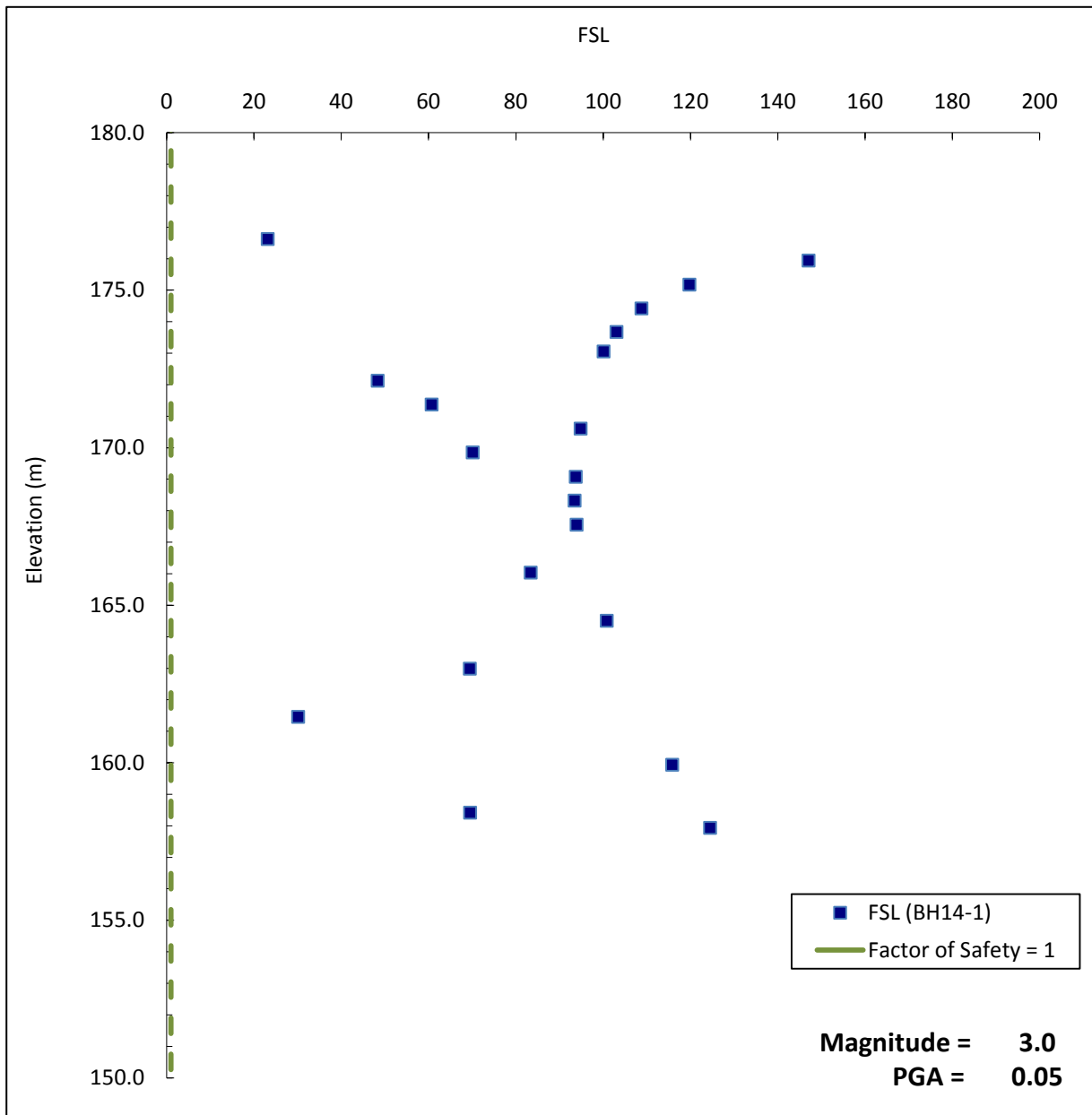
Figure 13: Preliminary Settlement Results

Figure 14: Lateral Deflection of HP 310X110 Piles

Figure 15: p-y Curves for Proposed HP 310X110 Piles

Table D1: Load Intensity  $p$  (kN/m) vs Lateral Deflection (m) of HP 310x110 Piles at  
various Depths below Pile Head



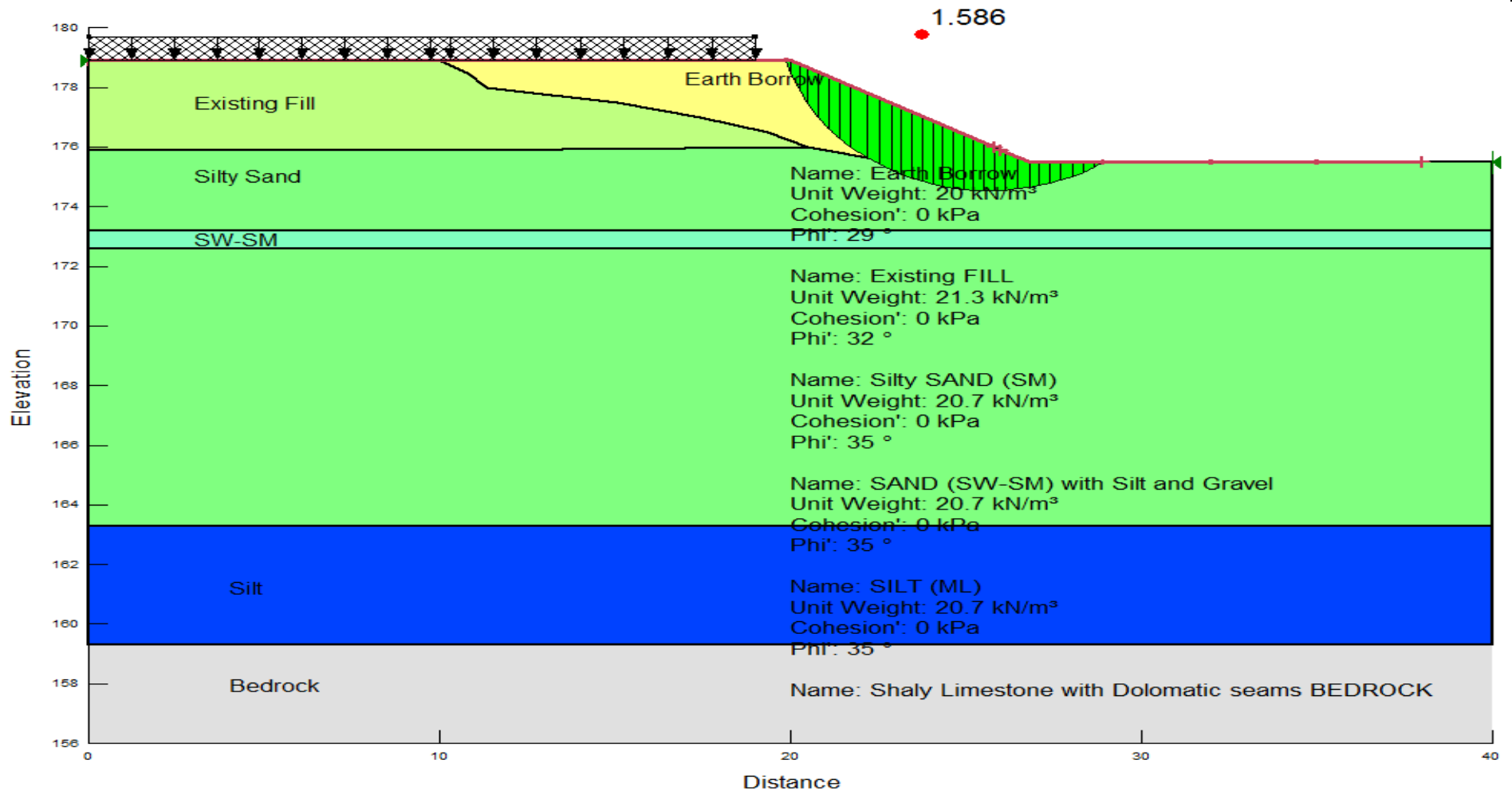


$FS_L$  = Factor of Safety against Liquefaction

The Canadian Foundation Engineering Manual defines  $FS_L$  as the "soil deposit's cyclic resistance ratio (CRR)" divided by the "earthquake induced cyclic stress ratio (CSR)"

Assessment Method based on the Summary Report from the 1996 and 1998  
NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils

Figure No. 8  
Project No. 165000912

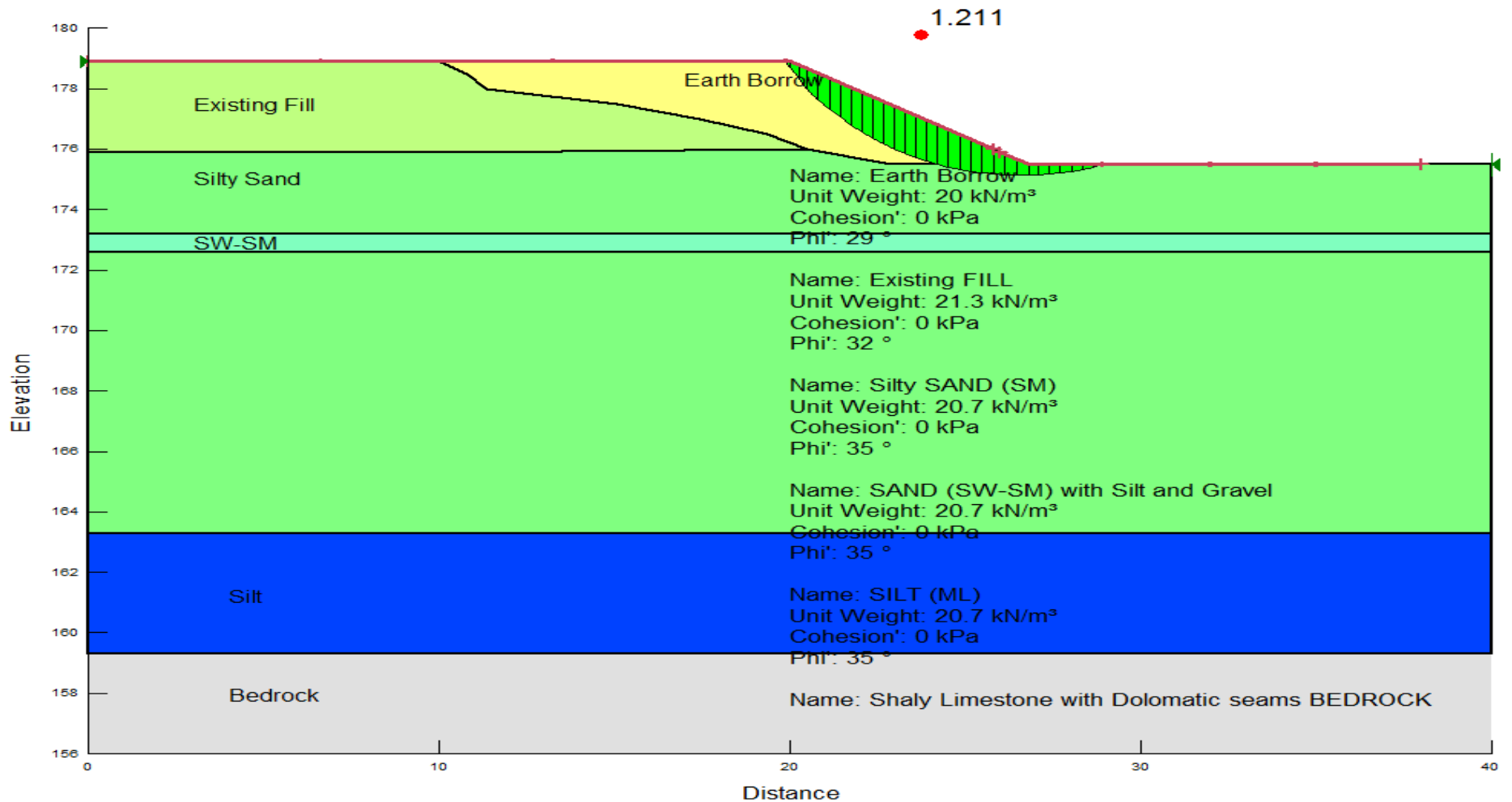


# Slope Stability Analysis Section A-A Normal Water Level, Static

Figure 9

Project No. 165000912





# Slope Stability Analysis

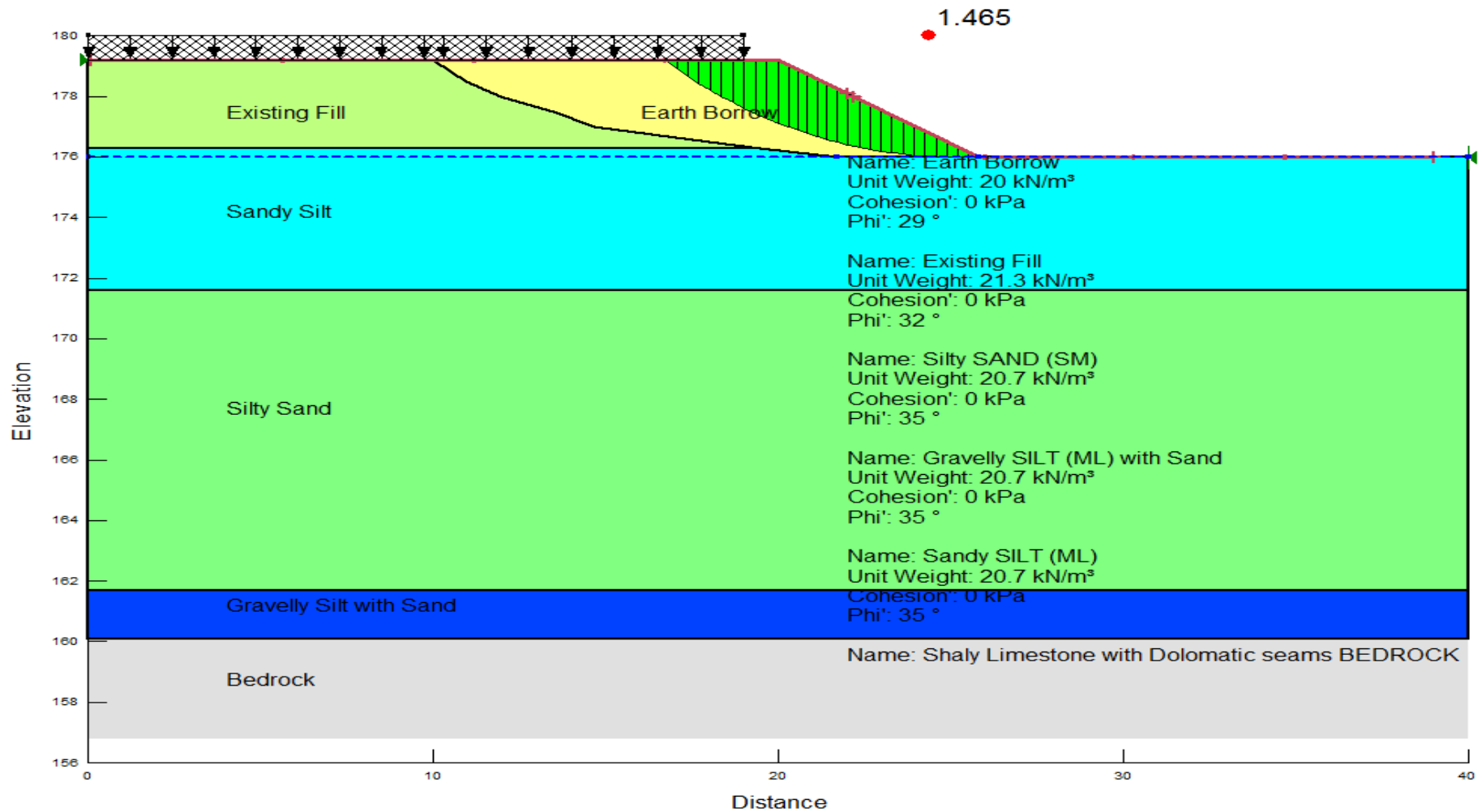
## Section A-A

### Normal Water Level, Seismic

Figure 10

Project No. 165000912





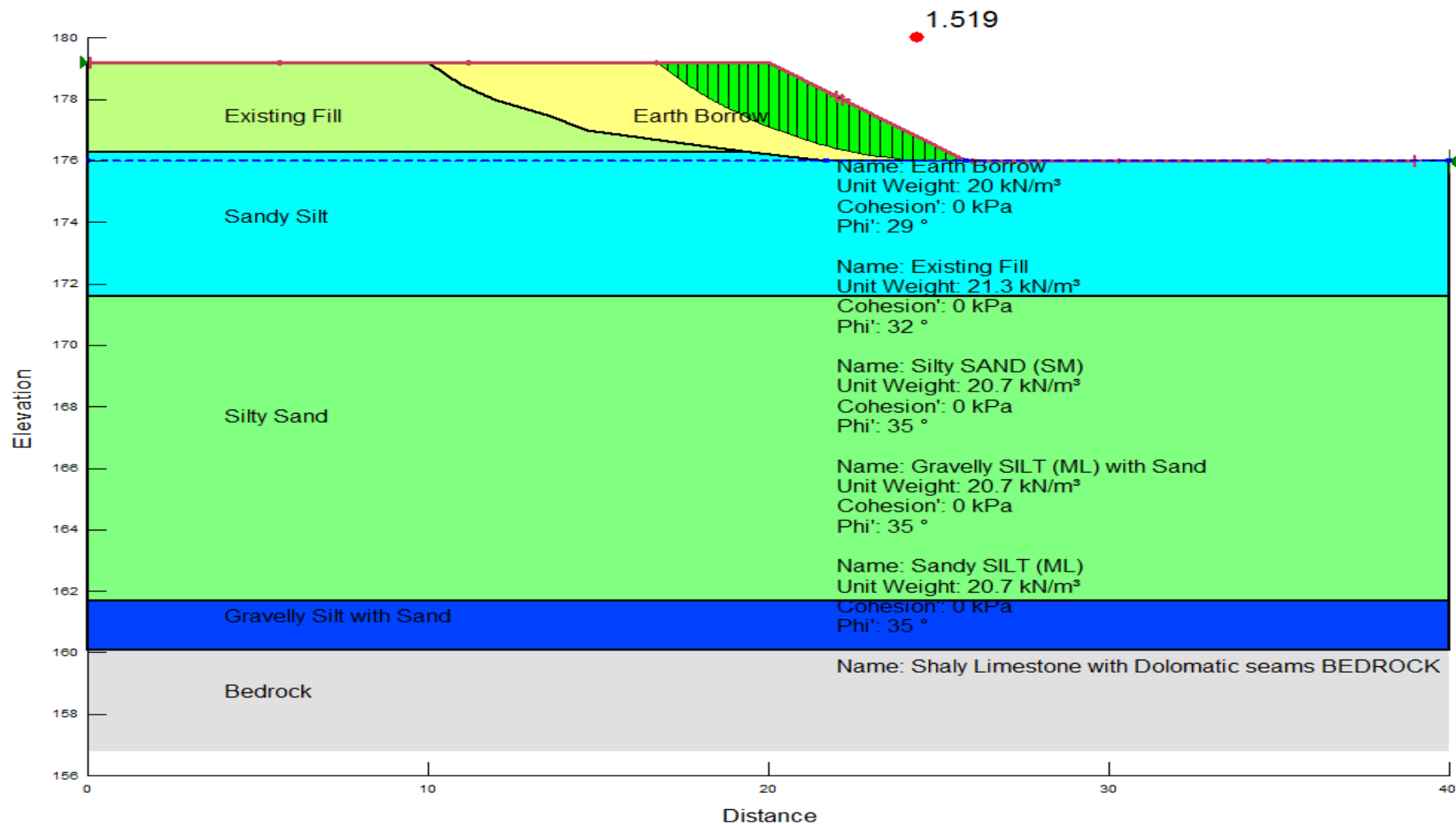
# Slope Stability Analysis Section B-B Normal Water Level, Static

Figure 11

Project No. 165000912





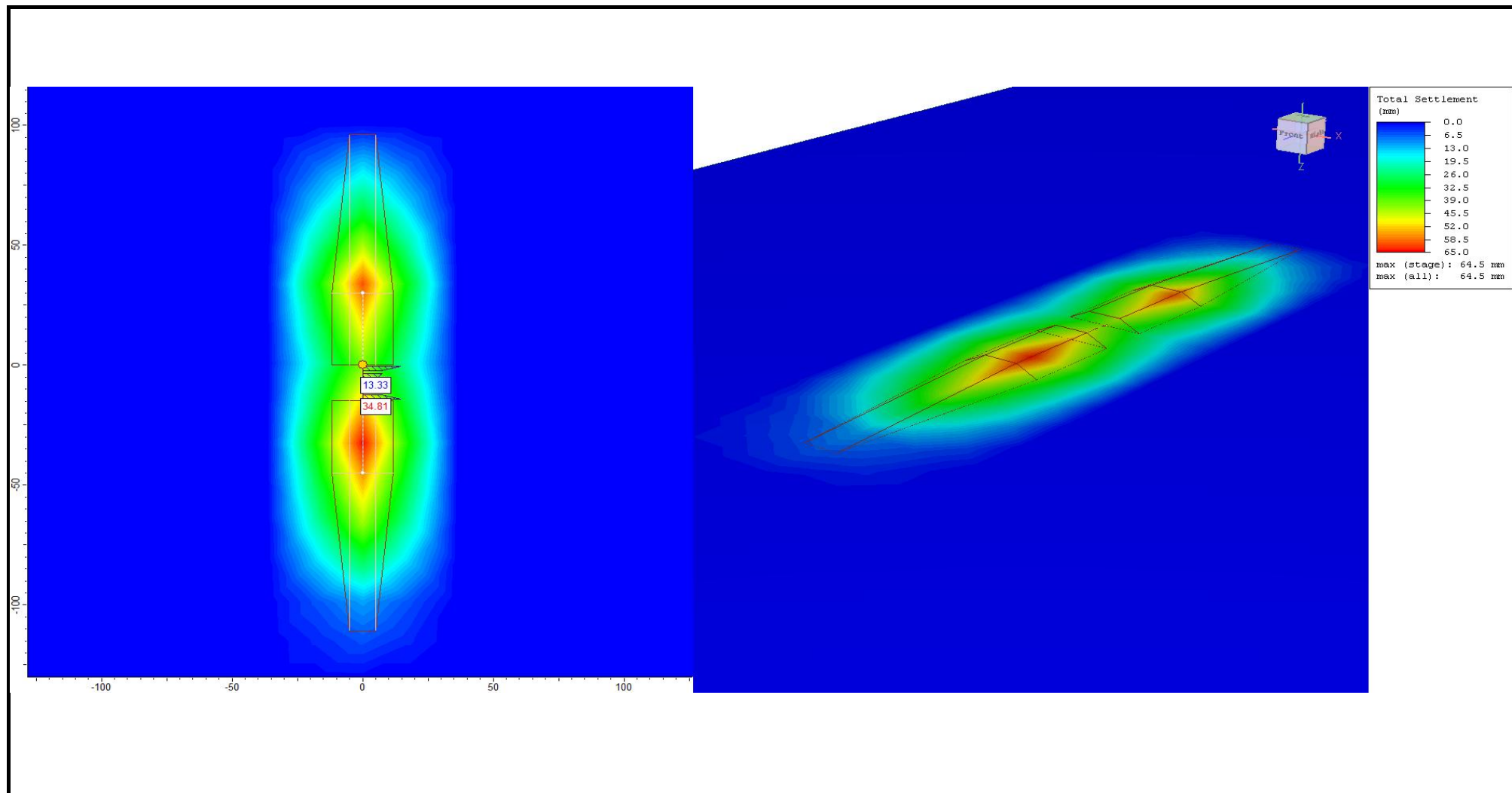


# Slope Stability Analysis Section B-B Normal Water Level, Seismic

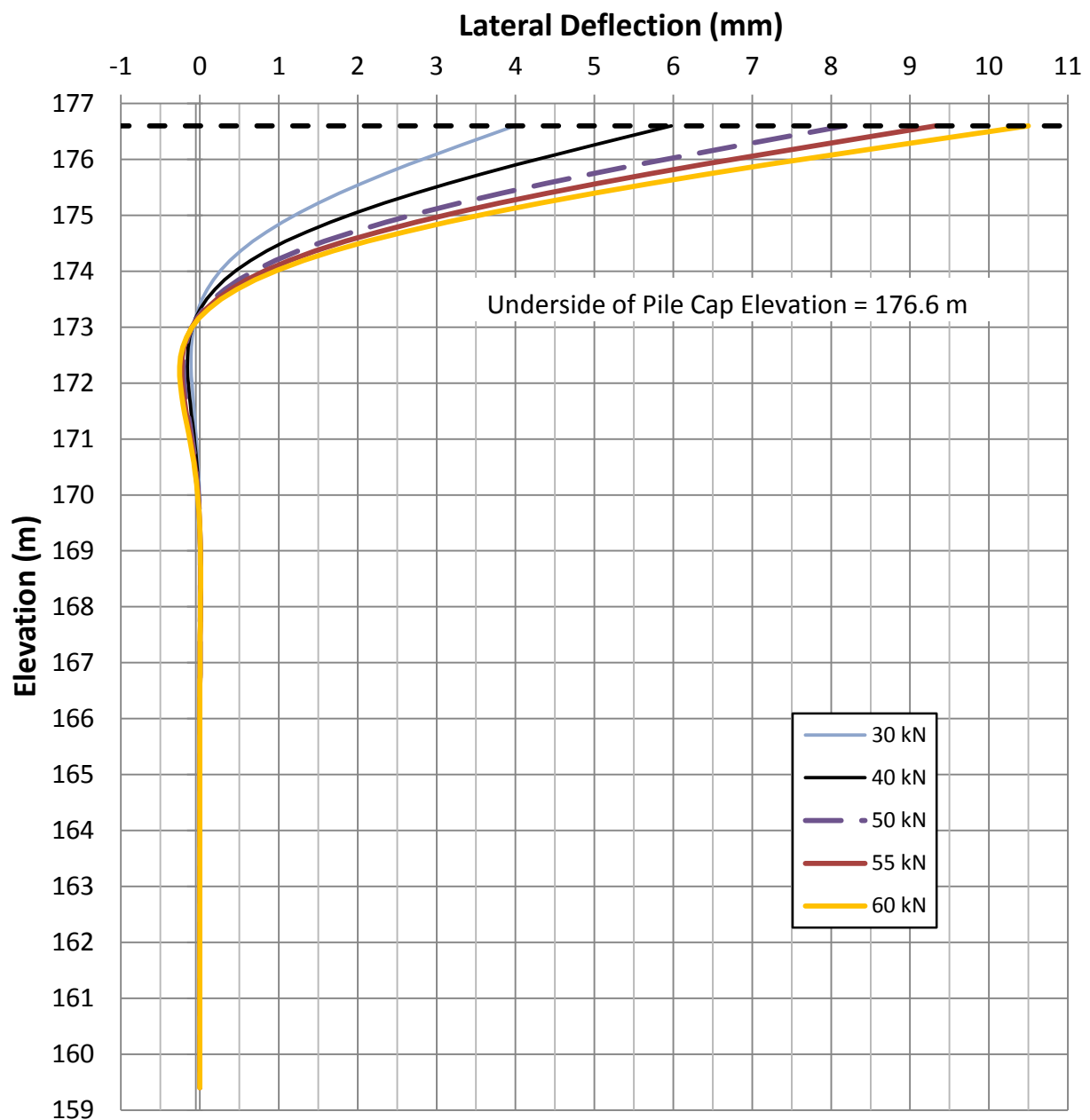
Figure 12

Project No. 165000912

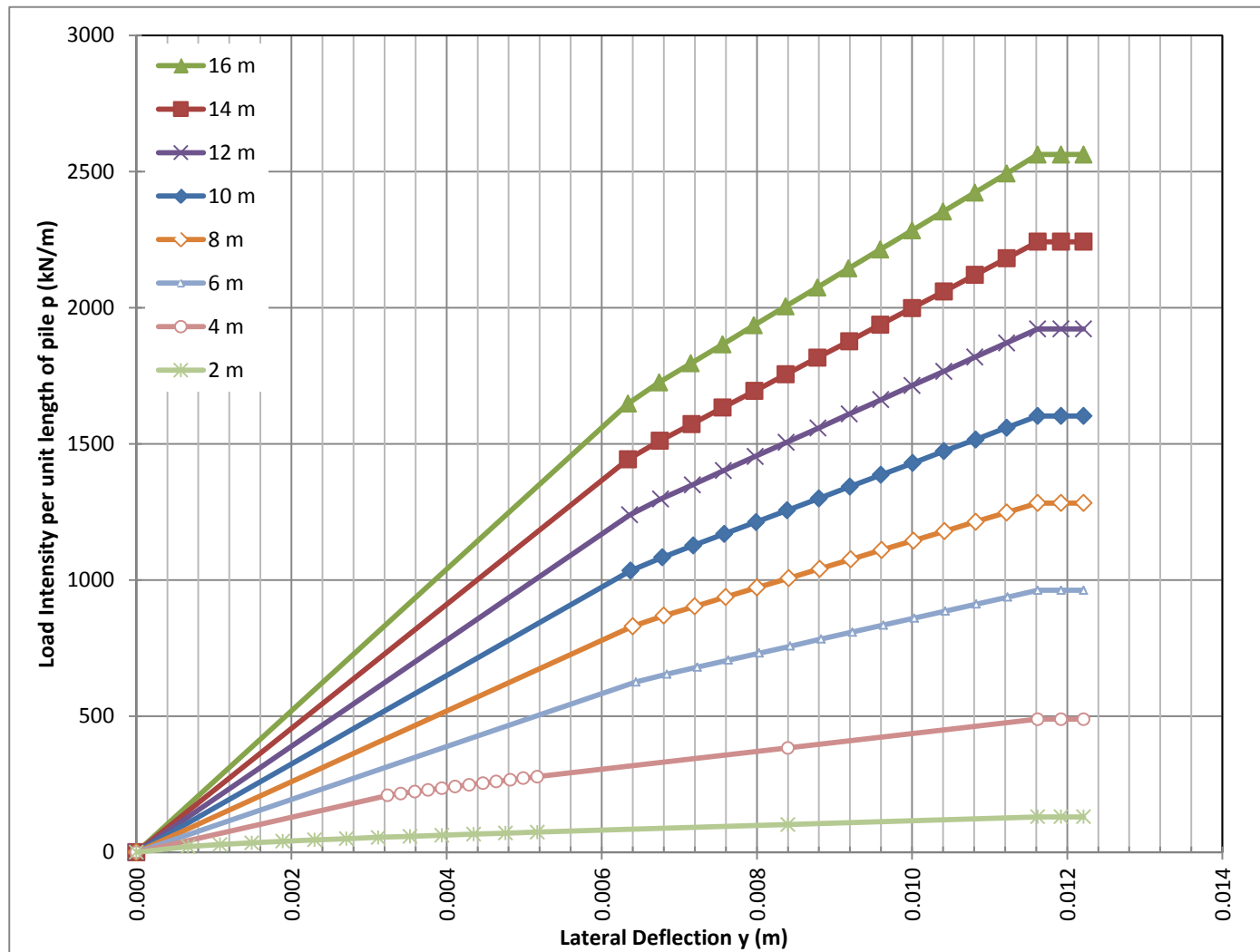




## L-Pile Results - Lateral Deflection



**Figure 14**  
**Lateral Deflection of HP 310x110 Piles**



**Figure 15**  
**p-y Curves for Proposed HP 310x110 Piles**

**Table D1**  
**Load Intensity p (kN/m) vs Lateral Deflection (m) of HP 310x110 Piles**  
**at Various Depths below Pile Head**

Depth (m) Below Pile Head		P-Y Data: Load Intensity p (kN/m) vs Lateral Deflection (m) of HP 310x110 Piles at Various Depths below Pile Head			
2	Load Intensity p (kN/m)	0	73	130	130
	Lateral Deflection (m)	0	0.0052	0.0116	0.0122
4	Load Intensity p (kN/m)	0	208	488	488
	Lateral Deflection (m)	0	0.0032	0.0116	0.0122
6	Load Intensity p (kN/m)	0	625	962	962
	Lateral Deflection (m)	0	0.0064	0.0116	0.0122
8	Load Intensity p (kN/m)	0	829	1282	1282
	Lateral Deflection (m)	0	0.0064	0.0116	0.0122
10	Load Intensity p (kN/m)	0	1034	1602	1602
	Lateral Deflection (m)	0	0.0064	0.0116	0.0122
12	Load Intensity p (kN/m)	0	1238	1922	1922
	Lateral Deflection (m)	0	0.0064	0.0116	0.0122
14	Load Intensity p (kN/m)	0	1443	2242	2242
	Lateral Deflection (m)	0	0.0064	0.0116	0.0122
16	Load Intensity p (kN/m)	0	1647	2561	2561
	Lateral Deflection (m)	0	0.0064	0.0116	0.0122

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The Table D-1 presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m) of a HP 310x110 pile. The p-y points can be used for the structural design of the pile in response to lateral loads