Preliminary Foundation Investigation and Design Report
Old Highway 144 to Forest Ridge Road Underpass

Highway 144 Route Planning and
Preliminary Design Study,
Chelmsford to Dowling, ON

G.W.P. 5023-09-00

Geocres No. 41I-300

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

For

G.W.P. 5023-09-00

Old Highway 144 to Forest Ridge Road Underpass, Highway 144 Route Planning and Preliminary Design Study, Chelmsford to Dowling

1.0 Introduction

Stantec Consulting Ltd. (Stantec) was retained by the Ministry of Transportation of Ontario (MTO) to undertake the foundations work required for the planning, preliminary design and environmental assessment associated with the determination of a new Controlled Access Highway alignment for Highway 144. The study area extends from approximately 6 km south of Chelmsford to approximately 8 km north of Dowling, a distance of approximately 27 km.

The preferred alignment extends from approximate Sta. 18+656.5 in Dowling Township to Sta. 18+082.5 in Creighton Township. Chainage equations along the preferred alignment occur at the following stations:

Sta. 20+187.792 Creighton Township = Sta. 10+000 Balfour Township

Sta. 21+333.540 Balfour Township = Sta. 10+000 Dowling Township

This *Preliminary Foundation Investigation and Design Report* has been prepared specifically and solely for the proposed Old Highway 144 to Forest Ridge Road Underpass along the preferred alignment, approximately 3.5 km south of the community of Chelmsford, Ontario. Separate reports have been prepared for each of the other structures.

Project Number:	G.W.P.: 5023-09-00
Agreement Number:	5009-E-0006
Project Location:	Highway 144, from 12 km north of Highway 17, northerly 27 km
Site Location:	Approximately 3.5 km south of Chelmsford and approximately 250 m west of the existing Highway 144 alignment

2.0 Site Description and Geology

Site Location

The proposed structure location is shown on the Key Plan inset to Drawing No. 1 of Appendix A. At the project site, the proposed Highway 144 is oriented approximately in a southeast-northwest direction. The approximate location of the proposed structure is near the future Highway 144

Station 11+000 Balfour Township. An exhibit showing the preferred Highway 144 alignment along with the proposed Old Highway 144 to Forest Ridge Road Underpass site is provided in Drawing No. 2 of Appendix A.

The proposed final grade of Highway 144 at the proposed Overpass is approximately 269.0 m, based on the preliminary General Arrangement drawing. The proposed final grade of the overpass is approximately 276.9 m. The anticipated maximum height of embankment to achieve the proposed grading is approximately 10.5 m.

Chainage along the preferred alignment of Highway 144 increases from east to west.

General Site Description

The project site is located west of the existing Highway 144 and south of Lavallee Road, in an undeveloped area. The site currently consists of open fields used for agricultural purposes. The surrounding area is generally flat to undulating. Photographs 1 through 6 in Appendix A show the general site features at the proposed structure site.

A network of ditches and culverts runs along the existing Highway 144 and Lavallee Road, and streams and drainage ditches are present within the site, draining in a generally southerly direction.

Physiographic Description

The project site is located within the Canadian Shield and is characterized by frequent rock knobs. The bedrock is from the Paleoproterozoic era (1,600 to 2,500 million years ago). The bedrock forming the rock ridge outcroppings within the central portion of the study area generally consists of sedimentary rock: namely turbiditic wacke and siltstone of the Chelmsford Formation. The higher portions of the rock knobs within the southeast and northwest portions of the study area include granite and granodiorite of the Sudbury Igneous Complex. The lower portions of the rock knobs consist of fragmented rock of the Onaping Formation.

The bedrock throughout the study area is generally overlain by glacial (sands, gravels, silts, and boulder clays) deposits of variable thicknesses. In low lying areas, post-glacial, stratified, lacustrine deposits (fine sandy silts and clays) overlie the glacial deposits. Peat and organic deposits are found in some areas.

3.0 Investigation Procedures

The foundations work for this study included literature compilation and review, Geocres search, field reconnaissance, foundation investigation, as well as laboratory testing of samples taken in the field. The compiled literature and Geocres reports were documented in Stantec's Geotechnical Inventory & Constraint Memorandum (Stantec, 2010). Subsequently, a comparative foundation assessment of alternative routes was documented in Stantec's 2012 Memorandum (Stantec, 2012).

3.1 FIELD INVESTIGATION

Two boreholes were advanced beneath the footprint of both of the abutments as part of the geotechnical investigation for the preliminary design of the bridge foundations for this proposed underpass. The proposed foundation elements are located approximately 250 m west of the existing Highway 144, and approximately 600 m south of Lavallee Road. The boreholes are designated BH13-27 and BH13-28, and their locations are shown on the Borehole Location Plan in Drawing No. 1 of Appendix A.

Prior to carrying out the investigation, Stantec contacted the respective property owners to obtain their permission to access the site. Stantec also contacted the public utility authorities to clear the borehole locations of both private and public utilities.

The field drilling program was carried out on February 19 and 24, 2013. The boreholes were advanced with a track-mounted CME 850 drill rig equipped for soil and bedrock sampling.

The subsurface stratigraphy encountered in each borehole was recorded in the field by an experienced Stantec Technician. Split spoon samples were collected every 760 mm interval for up to 10 m depth below existing ground surface and every 1.5 m interval for deeper strata. Where cohesive soil was encountered, the undrained shear strength of these deposits was determined with in-situ shear vane testing and pocket penetrometer tests. Bedrock coring was carried out in both boreholes with NQ size coring equipment.

Core samples were logged and photographed and the Rock Quality Designation (RQD) and Mohs Hardness Values were estimated for recovered samples. Mohs Hardness tests were performed on representative rock samples to estimate the Mohs scale of relative hardness value of the rock for each core run. The hardness scale ranges from 1 (talc) to 10 (diamond). The hardness of a rock sample was estimated by trying to scratch it with several materials of known hardness. According to Mohs hardness rating, objects with higher Mohs numbers will scratch those lower on the scale.

The groundwater level was measured in the open boreholes.

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

After completion of drilling, boreholes were backfilled with auger cuttings mixed with bentonite.

3.2 LOCATION AND ELEVATION SURVEY

The elevation and coordinates (northing and easting) of the boreholes were determined using a Global Positioning System (GPS) apparatus, Trimble Geo XH, capable of decimeter accuracy. The accuracy of the survey results is dependent on the satellite coverage at the time of measurement.

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	
	BH13-27	**BH13-28**
MTM Zone 12 Coordinates Northing (m) Easting (m)	5156555 290350	5156599 290408
Ground Surface Elevation, m	266.6	266.5
Total Depth Drilled, m	22.9	20.3
End of Borehole Elevation, m	243.7	246.2
Depth Augered, m	19.2	17.2
Depth Cored, m	3.7	3.1
Number of Soil Samples	21	20

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.

Table 3.2: Geotechnical Laboratory Testing Program

Test Description	Number of Tests
Moisture Content	39
Atterberg Limits	5
Grain Size Distribution	13
Unconfined Compression (rock)	4

A representative rock core sample was polished and viewed with an optical microscope.

One soil sample was 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 two 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 topsoil over silty sand over clayey silt to silty clay over 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 Topsoil

The approximate thicknesses of the topsoil layers encountered in BH13-27 and BH13-28 were 180 and 250 mm, respectively. The moisture content of the topsoil was 34% in BH13-27 and 44% in BH13-28.

4.1.2 Silty Sand

A silty sand material was encountered in both boreholes immediately beneath the topsoil. The thickness of the silty sand layer was approximately 8.9 m in BH13-27 and 9.8 m in BH13-28, extending to bottom elevations of 257.5 m and 256.4 m, respectively.

The Standard Penetration Test (SPT) blow count (N-value) for the silty sand layer ranged from 3 to 33 blows/0.3 m, ranging from a very loose to dense state. The average N-value was 12, suggesting a compact state.

Flowing sands were encountered in BH13-27, at 2.3 m and 3.8 m below ground surface (elevations 264.3 m and 262.8 m, respectively). Due to the flowing sands, the remainder of the hole was cased starting at elevation 262.8 m.

Occasional cobbles were encountered within the silty sand deposit in BH13-28.

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

Gravel:	0%
Sand:	51 to 88%
Fines (Silt & Clay):	12 to 49%
Moisture Content:	17 to 26%

The Unified Soil Classification System (USCS) group symbol for the layer is SM. A representative grain size distribution plot for the silty sand layer is provided in Figure 1 of Appendix C.

4.1.3 Clayey Silt

A clayey silt layer was encountered in both boreholes immediately beneath the silty sand. The thickness of the clayey silt layer was approximately 7.7 m in BH13-27 and 7.1 m in BH13-28, and extended to approximate bottom elevation of 249.8 m and 249.3 m, respectively.

The Standard Penetration Test (SPT) blow count (N-value) for the silt layer ranged from 4 to 31 blows/0.3 m. Pocket penetrometer testing carried out on selected split-spoon samples indicated undrained shear strength measurements of 85 to 150 kPa, suggesting a stiff to very stiff consistency. N-size field vane tests carried out above elevation 253 m indicate that the undrained strength consistently exceeded 106 kPa; this value represents the maximum measurable value for the field equipment used. Below elevation 253 m, undrained shear strength ranging from 59 kPa to exceeding 106 kPa were measured using a field vane.

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

Gravel:	0%
Sand:	0 to 2%
Silt:	54 to 90%
Clay:	8 to 44%
Moisture Content:	23 to 46%

Atterberg limits tests carried out on three representative samples from this layer indicated plasticity indices of between 8 and 10. The USCS group symbol for the layer is CL (clayey silt of low plasticity).

Representative grain size distribution plots and plasticity chart for the clayey silt layer are provided in Figures 2 and 3 of Appendix C, respectively.

4.1.4 Silty Clay

A silty clay layer was encountered in BH12-27 below the clayey silt. The thickness of the silty clay layer was approximately 2.4 m, and extended to approximate bottom elevation of 247.4 m.

The Standard Penetration Test (SPT) blow count (N-value) for the clay layer were from 6 and 7 blows/0.3 m. Field shear vane testing indicated undrained shear strength measurements of 78 and 87 kPa, suggesting a stiff consistency.

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

Gravel:	0%
Sand:	0%
Silt:	38%
Clay:	62%
Moisture Content:	17 to 49%

Atterberg limits tests carried out on a representative sample from this layer indicated a plasticity index of 32. The USCS group symbol for the layer is CI (silty clay of intermediate plasticity).

Representative grain size distribution plot and plasticity chart for the silty clay layer are provided in Figures 2 and 3 of Appendix C, respectively.

4.2 BEDROCK

Bedrock was encountered in both boreholes immediately beneath the clayey silt to silty clay layer at approximate elevations ranging between of 247.4 m and 249.3 m. The bedrock was metasedimentary and consisted of black laminated argillites and siltstones of the Onwatin Formation.

The Rock Quality Designation (RQD) values ranged between 69% and 100%, indicating a fair to excellent rock quality. The Total Core Recovery (TCR) was between 69% and 100%. A detailed description of the rock core is provided in Field Core Logs. Rock core photographs, including a magnified image of a representative rock sample, are provided in Appendix B.

Unconfined compressive strength tests were carried out on two bedrock samples from each borehole. The results of these tests are summarized in Table 4.1.

Table 4.1: Unconfined Compressive Strength of Rock Cores

Borehole No	Ground Surface Elevation (m)	Test Elevation (m)	Unconfined Compressive Strength (MPa)
BH13-27	266.6	246.7	76
		244.2	47
BH13-28	266.5	248.4	104
		246.8	148

Based on the UCS test results presented above, the tested bedrock samples may be described as medium strong to very strong.

4.3 CHEMICAL ANALYSIS

One representative sample retrieved from the silty sand layer in BH13-27 was tested for pH, water soluble sulphates and chloride concentrations, and resistivity. The results of this chemical analysis are provided in Table 4.2.

Table 4.2: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH13-27	SS4	2.29 to 2.90	7.0	< 5	37	111

4.4 GROUNDWATER

The groundwater level was measured in open boreholes at the time of drilling. The groundwater levels were not stabilized at the time of measurement; hence they will be referred to as "inferred". The inferred groundwater levels are summarized in Table 4.3.

Table 4.3: Inferred Groundwater Levels (time of drilling)

Borehole No	Ground Surface Elevation (m)	Groundwater	
		Depth (m)	Elevation (m)
BH13-27	266.6	2.0	264.6
BH13-28	266.5	1.4	265.1

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

5.0 Discussion

Project Purpose/Justification

Stantec is conducting a study to determine a new route for Highway 144 from 12 km north of Highway 17, northerly for 27 km. The new route includes a four-lane divided highway and will bypass the communities of Chelmsford and Dowling.

The preferred alignment includes 11 structure sites, including the Old Highway 144 to Forest Ridge Road Underpass site.

Proposed Underpass Structure

The proposed interchange will include a new structure, linking the new alignment of Highway144 with the former alignment, and also nearby streets, including Forest Ridge Road and Lavallee Road. The proposed bridge is for the initial phase of Highway 144 which will have an interim two-lane cross-section. The ultimate configuration of the new Highway 144 will include a four-lane divided highway, with the interim Highway 144 forming the future northbound lanes of the divided highway. The proposed two-lane interchange bridge will carry traffic over the new Highway 144. The initial phase structure will also accommodate the N/S-S ramp.

The preliminary General Arrangement (GA) drawings indicate that the proposed underpass will have two spans with the centre pier through the median of the new Highway 144 and two abutments north and south of the new alignment supported on piles. The two structure options being considered are a post-tensioned concrete slab bridge with conventional abutments and a slab-on-girder bridge with integral abutments.

Approximate key elevations associated with the proposed underpass are as follows:

Post-tensioned bridge configuration

Proposed Top of Footing Elevation (East Abutment):	271.6 m
Proposed Top of Footing Elevation (West Abutment):	270.2 m
Proposed Top of Footing Elevation (Centre Pier):	266.2 m
Proposed Final Grade (Top of Bridge):	276.8 m
Proposed Final Grade of Highway 144:	269.0 m
Existing Ground Elevation at East Abutment:	266.5 m
Existing Grade Elevation at West Abutment:	266.6 m
Approximate Elevation of Highway 144 Median:	268.0 m

Integral abutment bridge configuration

Proposed Underside of Pile Cap Elevation (East Abutment):	270.4 m
Proposed Underside of Pile Cap Elevation (West Abutment):	270.5 m
Proposed Top of Footing Elevation (Centre Pier):	266.2 m
Proposed Final Grade (Top of Bridge):	276.9 m
Proposed Final Grade of Highway 144:	269.0 m
Existing Ground Elevation at East Abutment:	266.5 m
Existing Grade Elevation at West Abutment:	266.6 m
Approximate Elevation of Highway 144 Median:	268.0 m

5.1 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered at this site generally consist of a thin layer of topsoil overlying silty sand overlying a clayey silt to silty clay layer overlying metamorphosed sedimentary bedrock. The native soils at the site are generally loose (for non-cohesive soils) or stiff (for cohesive soils). Bedrock was encountered at 19.2 m depth in BH13-27 and 17.2 m depth in BH13-28 below existing ground surface, corresponding to approximate elevation of 247.4 m and 249.3 m. The RQD of the bedrock ranged between 69% and 100%, indicating a fair to excellent rock mass quality. The unconfined compressive strength ranged between 47 MPa and 148 MPa (medium strong to very strong).

The subsurface profile shown in Tables 5.1 can be used for preliminary design purposes. The subsurface profile was developed based on the synthesis of the measured N-values, in-situ shear strength measurements, and laboratory index test results of samples retrieved from the site. This profile is included in Figures 4a, 4b and 4c of Appendix D and was developed based on the information obtained from boreholes BH13-27 and BH13-28.

Table 5.1: Preliminary Subsurface Profile at Old Highway 144 to Forest Ridge Road Underpass

Elevation (m)		Soil Type	Design Parameters				
From	To		γ (kN/m^3)	ϕ (°)	S_u (kPa)	UCS (MPa)	E (MPa)
266.5	257.0	Silty sand (Very loose to dense)	20	32	-		15
257.0	248.0	Clayey silt (Stiff to very stiff)	19	See Figures 4b and 4c in Appendix D			
< 248.0		Metasedimentary bedrock (fair to excellent quality, medium strong to very strong)	24	-	-	47 to 148	1200

Notes: (1) γ= total unit weight, ϕ= soil friction angle, S_u = undrained shear strength, UCS = unconfined compressive strength of rock, E = soil modulus

(2) Groundwater is assumed to be at an approximate elevation of 265.0 m for preliminary design purposes. Submerged unit weight (γ') should be used below the groundwater level.

5.2 FROST PENETRATION

In accordance with OPSD 3090.100, the design frost penetration depth for foundations, f, at the site is 2.1 m. Therefore, footings and pile caps should be provided with a minimum of 2.1 m of soil cover or equivalent insulation for protection against frost heaving.

5.3 SEISMIC DESIGN CONSIDERATIONS

The soil profile at the site includes approximately 9.5 m of silty sand, over approximately 9.0 m stiff to very stiff clayey silt to silty clay, over fair to excellent quality bedrock. It is recommended that a Soil Profile III, 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 Sudbury, Ontario, which is approximately 15 km east of the site and the nearest location for which the ZAR value is available, is 0.05. Hence, a ZAR of 0.05 should be used for this site.

The potential liquefaction of the site soil under seismic loading conditions was assessed. For the Sudbury area a moment magnitude of 3.0 was assumed based on historical earthquake data. The assessment results are indicated in Figure 5 in Appendix D. The assessment indicated that liquefaction of the foundation soils is not a concern for this site.

Even though it is not very likely, seismically induced lateral earth pressure should be considered for this project with a ZAR of 0.05.

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 Old Highway 144 to Forest Ridge Road Underpass

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences	Rank*
Shallow Foundation on Soil	▪ Limited excavation involved ▪ Generally suitable to support bridge piers	▪ Low geotechnical resistance will necessitate ground improvement techniques ▪ Not suitable for integral abutment bridge construction	Low to Medium	▪ Potential differential settlement ▪ Basal instability during excavation	3
Piles End bearing on bedrock	▪ Reduces risk of differential settlement ▪ Suitable for integral abutment bridge	▪ If cobbles are encountered in the overburden, may require pre-augering (glacial till was not observed in the boreholes)	High	▪ Possible pile damage during installation	1
Drilled Caissons	▪ Can transmit very large axial and lateral loads ▪ Generally suitable if bedrock is relatively shallow	▪ Not suitable for integral bridge abutment ▪ Depth to competent soil strata	Medium to High	▪ Risk of cave-in, especially below groundwater table during drilling, would require temporary liners	2

*Based on qualitative criteria.

Based on the comparison presented above, the following foundation options are recommended:

- For both the post-tensioned and integral abutment bridge options, deep foundations consisting of H-piles should be used to support the abutments and the centre pier.
- The piles should be driven to bedrock.
- For the integral abutment bridge configuration, the pile configuration should result in flexible H-piles driven to bedrock for the abutments and the centre pier.

Spread footings are not considered an option for the bridge foundations due to the generally loose sands observed within the upper 5 m in BH13-27. Consideration was given to soil improvement methods such as dynamic compaction, or rammed aggregate piers. However, these are not considered suitable for high bridge foundation loads.

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 Driven Piles – Abutment and Pier Foundations

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

5.5.1.1 Geotechnical Axial Resistance

Pile loads have not been established yet. It is anticipated that a pile foundation consisting of HP310x110 piles will be used to support the proposed abutments as well as the centre pier. Based on the preliminary GA plan drawing, the underside of the pile caps (bottom of concrete abutments) will be at approximate elevations of 270.5 m for the integral abutment bridge option and at 271.6 m (east) and 270.2 m (west) for the post-tensioned bridge optioned. The underside of the centre pier pile cap would be at elevation 266.2 m for both options.

To provide the desired integral action for the integral abutments, the piles should be driven through a 600 mm diameter, 3 m long corrugated steel pipe (CSP) and filled with loose uniform sand.

The piles should be driven to the bedrock. The anticipated pile length at the abutments will be between 23.1 m and 21.2 m. At the centre pier, the pile length would be in the in the order of 16 m.

A factored axial resistance in compression at Ultimate Limit State (ULS_f) for an HP310x110 pile of 2000 kN may be used for this site. This resistance at ULS_f 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.

5.5.1.2 Downdrag

The proposed underpass will require an approximately 10.5 m high approach embankment fill to raise the final grade of the proposed new interchange. The anticipated settlement due to the placement of the approach embankment is discussed in Section 5.7.3 (Embankment Settlement). The estimated maximum settlement at the abutment is approximately 210 mm, of which approximately 125 mm is anticipated to be immediate settlement associated with the silty sand layer.

The amount of settlement anticipated at the abutments after pile driving is in the order of 80 mm; this lower amount of settlement reflects that the lower 4 m of the embankment fill would be placed prior to pile driving.

The thickness of the potentially compressible clayey silt to silty clay layer varies between 7.1 m and 10.1 m. The settlement within the clayey silt layer below the abutments is anticipated to be in the order of 40 mm. The maximum unfactored downdrag load was estimated to be 1000 kN; the downdrag loads are developed within both the sand and clayey silt layers which would be moving downwards with respect to the pile. This value is to be added to the dead loads to confirm that, in combination with the load, it does not exceed the structural capacity of the pile. Downdrag loads and live loads are not combined since the compression due to live loads tends to cancel out the downdrag loads.

Since the pile capacity derives from end-bearing on bedrock, consideration should be given to the application of suitable coatings along the pile in the compressible portion of the soil profile.

5.5.1.3 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.

5.5.1.4 Drivability

The soil encountered at the site consisted of silty sand overlying stiff to very stiff clayey silt to silty clay. No obstructions to pile driving are anticipated. However, this should be confirmed during Detail Design.

Piles should have reinforced Type I tips according to Ontario Provincial Standard detail, OPSD 3000.100.

Pile Driving Note 5: “Piles to be driven to bedrock” would be applicable for this site.

5.5.1.5 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 of the Commentaries to the *Canadian Highway Bridge Design Code* (CHBDC, 2006) for noncohesive material with an effective friction angle (ϕ') of 30°.

<u>ULS Resistance</u>

The passive earth pressure for the pile (driven though a loose uniform sand in CSP, followed by a very loose to dense silty sand layer, followed by a clayey silt layer) was estimated using the procedure described in Section C6.8.7.1 of CHBDC (CHBDC, 2006). The pressure was converted into a passive resistance by using a bearing width equal to the three times the flange width of HP310x110, for a depth of six times the flange width. A geotechnical resistance factor for passive lateral resistance of 0.5 was used (Table 6.1 of CHBDC, 2006). The estimated factored lateral resistance at ULS$_f$ was 145 kN for the abutments, and 40 kN for the centre pier.

It should be noted that the above estimated resistance value represents the passive resistance of the pile and not the actual force that will be mobilized. In designing the pile, it should be confirmed that the available flexural resistance at ULS$_f$ is not exceeded.

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 5.3.

Table 5.3: Parameters Used for Lateral Resistance at ULS and SLS for Piles

Soil Layer	Elevation (m)		Unit Weight, γ	Friction Angle, ϕ	Undrained Shear Strength, S_u	Deformation Parameters(3)	
						k	ε_{50}
	From	To	kN/m^3	Degrees	kPa	kN/m^3	-
Loose to compact sand(1)	270.5	267.5	20	33	-	6,800	-
Embankment fill (SSM)	267.5	266.5	20	32	-	30,000	-
Silty sand	266.5	257.0	20(2)	32	-	20,000 (above WT) 15,000 (below WT)	-
Clayey silt	257.0	248.0	19(2)	-	75	-	0.005
Bedrock	< 248.0		24(2)	-	N/A	-	-

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; ε_{50} = strain corresponding to one-half the maximum principal stress difference.

(4) Groundwater level was assumed to be at an elevation of 265.0 m.

Two plots from LPILE are presented in Figures 6 and 7 of Appendix D. Figure 6 shows the deformed shape of the pile for lateral (shear) force ranging between 80 and 170. This plot indicates that the pile undergoes negligible lateral deflection below a depth of approximately 7.5 m from the underside of the pile cap (at approximate elevation of 263.0 m).

Figure 76 in Appendix D illustrates the displacement of the pile in depth for different lateral loads. Based on Figure 7, a lateral load of 150 kN corresponds to a pile head (top) displacement of less than 10 mm. Therefore, the SLS geotechnical resistance of an HP 310x110 at this site is estimated as 150 kN.

Figures 8a to 8c in Appendix D present the p-y plots that give the non-linear response of the pile-soil interaction. They provide a series of curves obtained from LPILE and generated every 1 m depth below the pile head. Estimates of p-y modulus k values versus depth are summarized in

Table D-1 of Appendix D. These plots and the p-y modulus k values 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 4 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 following reduction factors may be used to account for pile group action:

Table 5.4: Recommended Reduction Factors for Pile Groups

Pile Spacing / Pile Diameter	Reduction Factor	Pile Spacing / Pile Diameter	Reduction Factor
Load Parallel to Pile Spacing		Load Perpendicular to Pile Spacing	
7	1.0	4	1.0
4	0.8	3	0.9
3	0.7	2	0.75
2	0.6	-	-

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 within and behind structures for the proposed underpass 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 retained soil systems.

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 retaining 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

$$P_A = \tfrac{1}{2} K_a \gamma H^2$$
$$P_P = \tfrac{1}{2} K_p \gamma H^2$$
$$P_O = \tfrac{1}{2} K_o \gamma H^2$$

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

Table 5.5: Recommended Non-Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II	Native Silty Sand	Native Clayey Silt to Silty Clay
Bulk Unit Weight, γ (kN/m^3)	21.2	22.0	20.0	19.0
Effective Friction Angle (º)	32	35	32	28
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43	0.47	0.53
Coefficient of Active Earth Pressure (K_a)	0.31	0.27	0.31	0.36
Coefficient of Passive Earth Pressure (K_p)	3.2	3.7	3.2	2.8

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 wingwalls (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:

$$P_{AE} = \tfrac{1}{2} K_{AE} \gamma H^2 (1 - k_V)$$
$$P_{PE} = \tfrac{1}{2} K_{PE} \gamma H^2 (1 - k_V)$$

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
γ = 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 Backslope to Wall	0°	
Vertical Back of Wall	0°	

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° to provide a conservative estimate.

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

Parameter	OPSS Granular B Type I		OPSS Granular A and Granular B Type II		Native Silty Sand		Native Clayey Silt to Silty Clay	
Bulk Unit Weight, γ (kN/m^3)	21.2		22.0		20.0		19.0	
Effective Friction Angle (º)	32		35		32		28	
Wall Type	Yielding	Non-yielding	Yielding	Non-yielding	Yielding	Non-yielding	Yielding	Non-yielding
Active Earth Pressure (K_{AE})	0.32	0.35	0.28	0.31	0.32	0.35	0.38	0.41
Height of Application of P_{AE} from base as a ratio of wall height, (H)	0.341	0.356	0.342	0.358	0.341	0.356	0.340	0.354
Passive Earth Pressure, (K_{PE})	3.21	-	3.64	-	3.21	-	2.73	-
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.325	-	0.325	-	0.325	-	0.325	-

5.7 RETAINED SOIL SYSTEM (RSS)

A Retained Soil System (RSS) is being considered for the abutments (false abutment) to contain the soil behind the abutment. The RSS extends immediately from the abutments back into the approach embankments effectively serving as wingwalls (for both west and east abutments).

Retained soil systems are listed in the MTO Designated Sources for Materials (DSM) and under Special Provisions 599S22 and 599S23. The RSS should be tendered with the following attributes:

Application:	False Abutment
Geometry:	Vertical (GV)
Performance:	High
Appearance:	High

Site Specific Geotechnical Considerations

A 300 mm thick Granular A Leveling Pad should be constructed beneath the RSS.

The overall external stability of RSS founded at an approximate elevation of 266.5 m (approximate existing ground elevation) was evaluated. The evaluation indicated that no sliding or general slope stability failure will be expected for the site soil condition. The evaluated factor of safety against general stability failure is in excess of the required factor of safety of 1.5.

The factored geotechnical resistance at ULS for the RSS built on the site soil is 780 kPa. This geotechnical resistance was evaluated based on assumed RSS dimensions, i.e., having a length equal to the width of the bridge (≈25 m) and a width of 0.8H where H is the height of RSS wall (based on MTO's RSS Design Guidelines, MTO (2007)). A wall height of 10.5 m was assumed. The SLS resistance for 25 mm total settlement was estimated to be 50 kPa.

Unit weight and effective friction angles provided in Section 5.6 of this report may be used for design of the RSS.

5.8 EMBANKMENTS

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

5.8.1 Embankment Construction

The proposed underpass requires approach embankments for the interchange to be built north and south of the structure. For preliminary design purposes, it is assumed that the embankment will be constructed using a Select Subgrade Material (SSM) or Earth Borrow material.

The expected maximum embankment height at the proposed interchange is approximately 10.5 m, based on the preliminary design plan.

5.8.2 Stability of Slopes

A preliminary slope stability evaluation was carried out, assuming a side slope of 2H:1V and maximum embankment height of 10.5 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 three loading situations: drained static (long-term), undrained static (short-term), and seismic. Typical preliminary slope stability evaluation results for the case of Earth Borrow are provided in Figures 9a through 9c in Appendix D.

Results of the preliminary slope stability evaluation suggest that for the anticipated configuration, the embankment constructed at the site using SSM will be stable at a slope of 2H:1V, under both static (short- and long-term) and seismic situations.

5.8.3 Embankment Settlement

A rigorous settlement analysis of the underlying soil due to the embankment construction can be evaluated once the proposed embankment geometry has been identified. For the purpose of the present preliminary evaluation, the following assumptions will be made in evaluating the settlement of the site soil under the proposed embankment.

- A simplified soil profile at the site, as presented above in Table 5.1, will be considered representative;
- 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;
- Both immediate (elastic) settlement (for granular fill soil) and consolidation settlement (for clayey silt soil) will be considered;
- A Poisson's ratio of 0.35 will be used for all soil types;
- Groundwater is assumed at 1.5 m below existing ground surface;
- The maximum embankment height will be approximately 10.5 m (in the immediate vicinity of both abutments);
- The approach embankment will have a 5% longitudinal slope and 2H:1V side slopes;
- The embankment extends approximately 220 m north and south from the respective abutments of the proposed underpass;
- The top width of the embankment will be approximately 20 m (including shoulders and roundings); and
- The distance between the abutments will be approximately 90 m.

Evaluation of soil settlement due to the effects discussed above was carried out using the Settle3D software (Rocscience, 2009). 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 a typical Settle3D preliminary analysis is presented in Figure 10 of Appendix D. The preliminary analysis result indicates that the maximum total vertical settlement of the existing materials for the conditions presented above is approximately 210 mm, under a 10.5 m high embankment. The maximum settlement will take place approximately 20 m back from each bridge abutment. The estimated settlement at the abutments is 105 mm.

Approximately 60% of the settlement beneath the embankments, and 65% of the settlement at the abutments is anticipated to be due to compression of the silty sand layer. It is anticipated that this settlement will be complete by the end of construction. The remaining settlement (due to the clayey silt to silty clay layer) will take place over a longer period of time. Post-construction settlement beneath the abutment is expected to be less than 80 mm.

Assuming 0.5% strain under self-weight, the estimated embankment self-weight settlement is approximately 50 to 55 mm. This settlement is anticipated to be completed by the end of embankment construction.

5.8.4 Settlement Mitigation

The maximum post-construction settlement predicted herein is less than 100 mm and therefore, less than the Maximum Limit During Pavement Design Life for freeways on compressible soils as defined in MTO's document titled Embankment Settlement Criteria for Design dated July 2010 (MTO, 2010). Therefore, ground improvement techniques such as preloading, surcharging or the use of wick drains are not strictly required as part of the final design. However, it is recommended that the construction project staging include placing the embankment fill up to the underside of the pile cap for all areas within 30 m of the abutment prior to driving the abutment piles. With this staging approach, the remaining settlement anticipated at the abutments would be in the order of 40 mm, of which 20 mm would occur as the remainder of the embankment fill is placed.

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 as described in section 5.7 are therefore suitable for this site.

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

The subsurface soils encountered during the geotechnical investigation within both boreholes included loose to dense silty sand over stiff to very stiff clayey silt to silty clay overlying bedrock at depths of 17.2 m and 19.2 m below existing ground surface. The surficial soils at the site should be considered as a Type 3 soil, according to the *Occupational Health and Safety Act* regulations for Construction Projects (OHSA).

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 206S03. Compaction should be carried out in accordance with OPSS 501 and SP105S21.

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

5.9.2 Unwatering

Groundwater was encountered at an elevation of approximately 265.0 m, which is approximately 1.5 m below the existing ground surface, and 0.8 m above the anticipated founding elevation of the centre pier for the post-tensioned bridge option (264.2 m). Unwatering is required to maintain dry working conditions desirable during excavation and construction of the pier footing.

The native soils within the anticipated depth of excavation are expected to have a moderate hydraulic conductivity, estimated to be in the order of $1x10^{-5}$ m/s. It is recommended that an unwatering plan be prepared for construction of the centre pier footing.

5.9.3 Reuse of Excavated Material

The native material at the site includes both a silty sand and a clayey silt to silty clay. It is anticipated that the silty sand material may be suitable for use as Earth Borrow. The depth of the clay layer will likely mean it is not practical to excavate; however, this material does not meet the Earth Borrow specification and is not suitable for use as backfill within and behind the structures.

5.10 CEMENT TYPE AND CORROSION POTENTIAL

One sample of the native clayey silt was 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 concentration for the sample was 37 µg/g. Soluble sulphate concentrations less than 1,000 µg/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 7.0, 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.

5.11 FUTURE INVESTIGATIONS

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

6.0 Specifications

The following specifications are referenced in this report:

Table 6.1: Specifications Referenced in Report

Document	Title
OPSD 3000.100	Foundation Piles Steel H-Pile Driving Shoe
OPSD 3090.100	Foundation Frost Depths for Northern Ontario
OPSD 3101.150	Walls – Abutment, Backfill – Minimum Granular Requirement
OPSS 501	Construction Specification for Compacting
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
SP 105S21	Amendment to OPSS 501, November 2010
SP 206S03	Earth Excavation, Grading
SP 599S22	Retained Soil System, False Abutment
SP 599S23	Retained Soil System, False Abutment

7.0 References

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.

Lambe, T.W. and Whitman, R.V. 1969. Soil Mechanics, John Wiley & Sons, Inc., New York.

Mesri, G. 1975. Discussion on 'New design procedure for stability of soft clays'. Journal of Geotechnical Engineering Division, ASCE, 101 (4): 409-412.

Ontario Ministry of Transportation (MTO). 2007. RSS Design Guidelines. Engineering Standards Branch, St. Catharines, Ontario.

Ontario Ministry of Transportation (MTO). 2011. Structural Manual. Bridge Office, St. Catharines, Ontario.

Rocscience, 2009. Settle3D Settlement and Consolidation Analysis: Theory Manual, Rocscience, Inc.

Stantec Consulting Limited. 2010. Highway 144 Chelmsford Bypass Route Selection Study Geotechnical Inventory & Constraint. Technical Memorandum, November 2010.

Stantec Consulting Limited. 2012. Highway 144 Chelmsford Bypass Route Selection Study Comparative Foundation Assessment of Alternative Routes. Technical Memorandum, March 2012.

8.0 Miscellaneous

The field work was carried out under the supervision of Jason Hopwood-Jones, Geotechnical Technician, under the direction of Chris McGrath, P.Eng., Senior Geotechnical Engineer.

The drilling equipment was supplied and operated by Abraflex Drilling of Val Caron, 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 Laura Bostwick, P.Eng. and Simon Gudina, P.Eng., and reviewed by Chris McGrath, P.Eng. and Raymond Haché, P.Eng., MTO Designated Principal Contact.

9.0 Closure

A soil investigation is a limited sampling of a site. The discussions and preliminary recommendations 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.

LICENSED PROFESSIONAL ENGINEER
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Mar 20, 2014
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S Gudina

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Raymond Haché, M.Sc., P.Eng.
Designated Principal MTO Foundation Contact

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APPENDIX A

Drawing No. 1 – Borehole Location Plan and Soil Strata

Drawing No. 2 – Preferred Route

Site Photographs

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

PLATE No
CONT
WP 5023-09-00

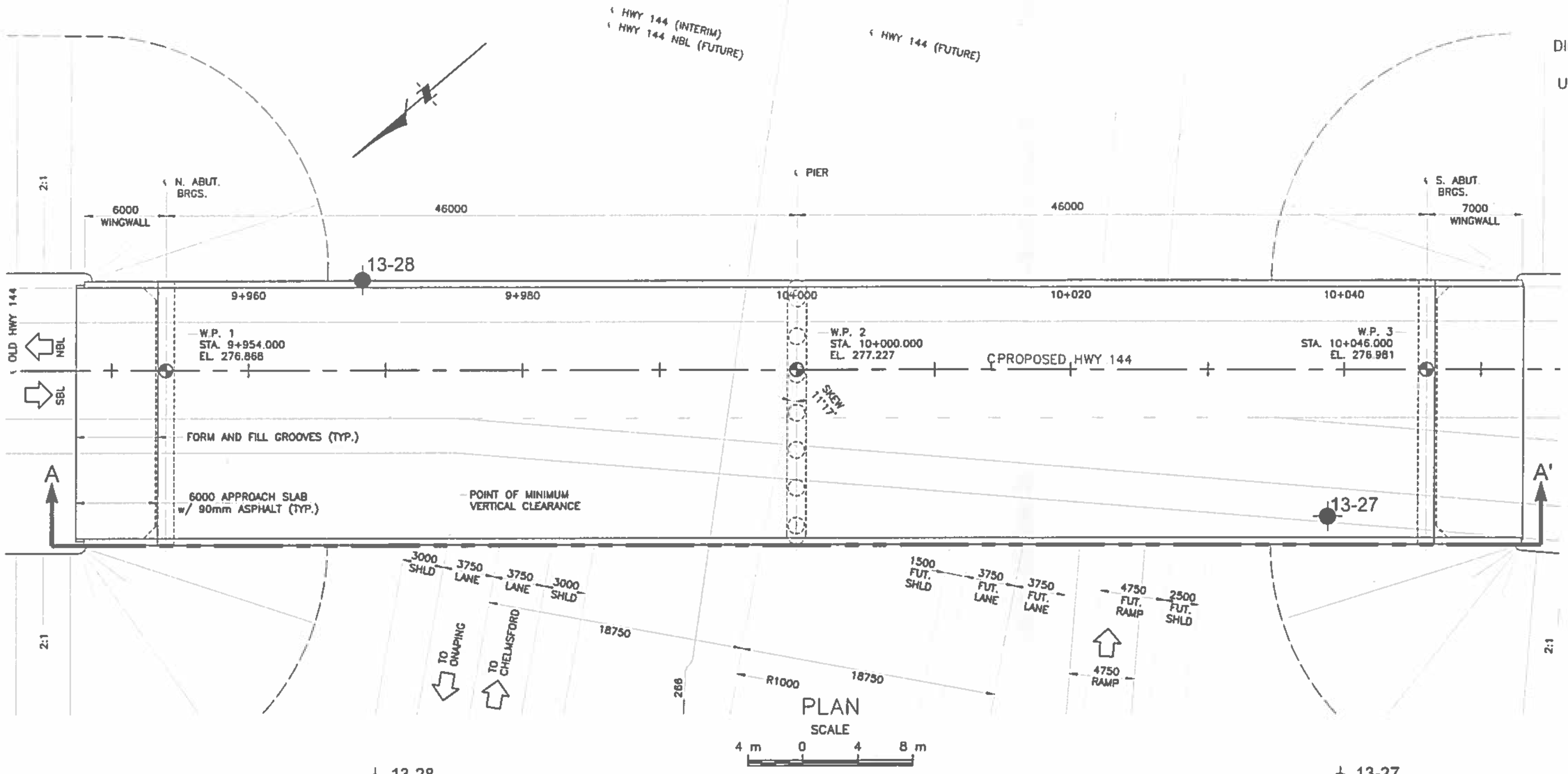
SHEET

HWY 144 ROUTE SELECTION STUDY
OLD HWY 144 TO FOREST RIDGE
BOREHOLE LOCATIONS & SOIL STRATA

Stantec

C. McGRATH
Mar 11, 2014

J.G.A.R. HACHÉ
Mar 11/14



PLAN
SCALE
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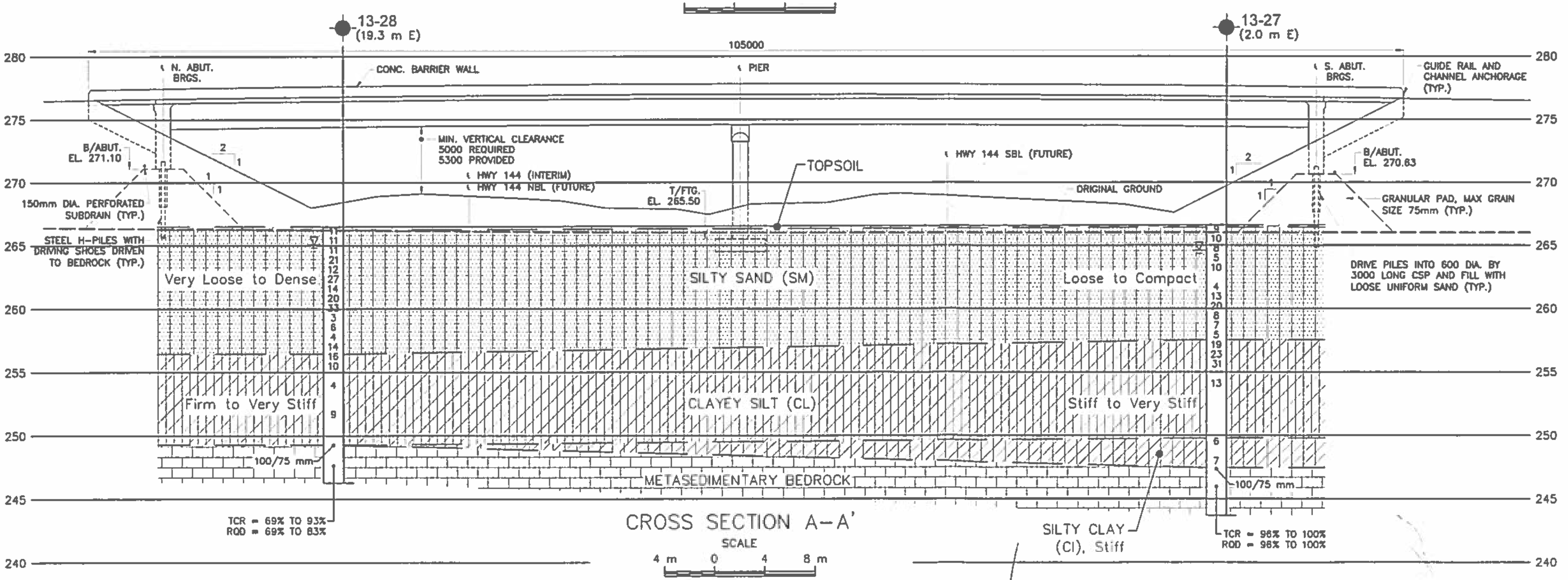

CROSS SECTION A–A'
SCALE
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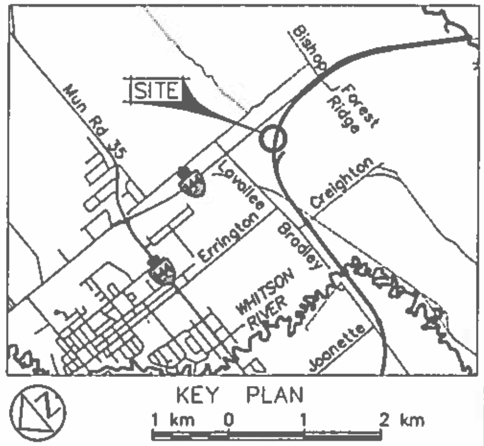

KEY PLAN
1 km 0 1 2 km

LEGEND

- ● Borehole
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- ▽ WL at time of investigation Feb 2013
- (x.x m E) Offset in meters East of Cross Section A–A'

No	ELEVATION	MTM ZONE 12 COORDINATES NORTH	MTM ZONE 12 COORDINATES EAST
13-27	266.6	5 156 555.5	290 350.3
13-28	266.5	5 156 599.0	290 408.5

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

GEOCRES No 411-300

HWY No 144			DIST
SUBM'D CM	CHECKED	DATE 2014-03-07	SITE
DRAWN GBB	CHECKED	APPROVED	DWG 1

DRAWING NAME: 165000734_S10_Old_hwy144_Int_pb1_GA_2013-11-01
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MINISTRY OF TRANSPORTATION, ONTARIO 86-05 PR-D-707

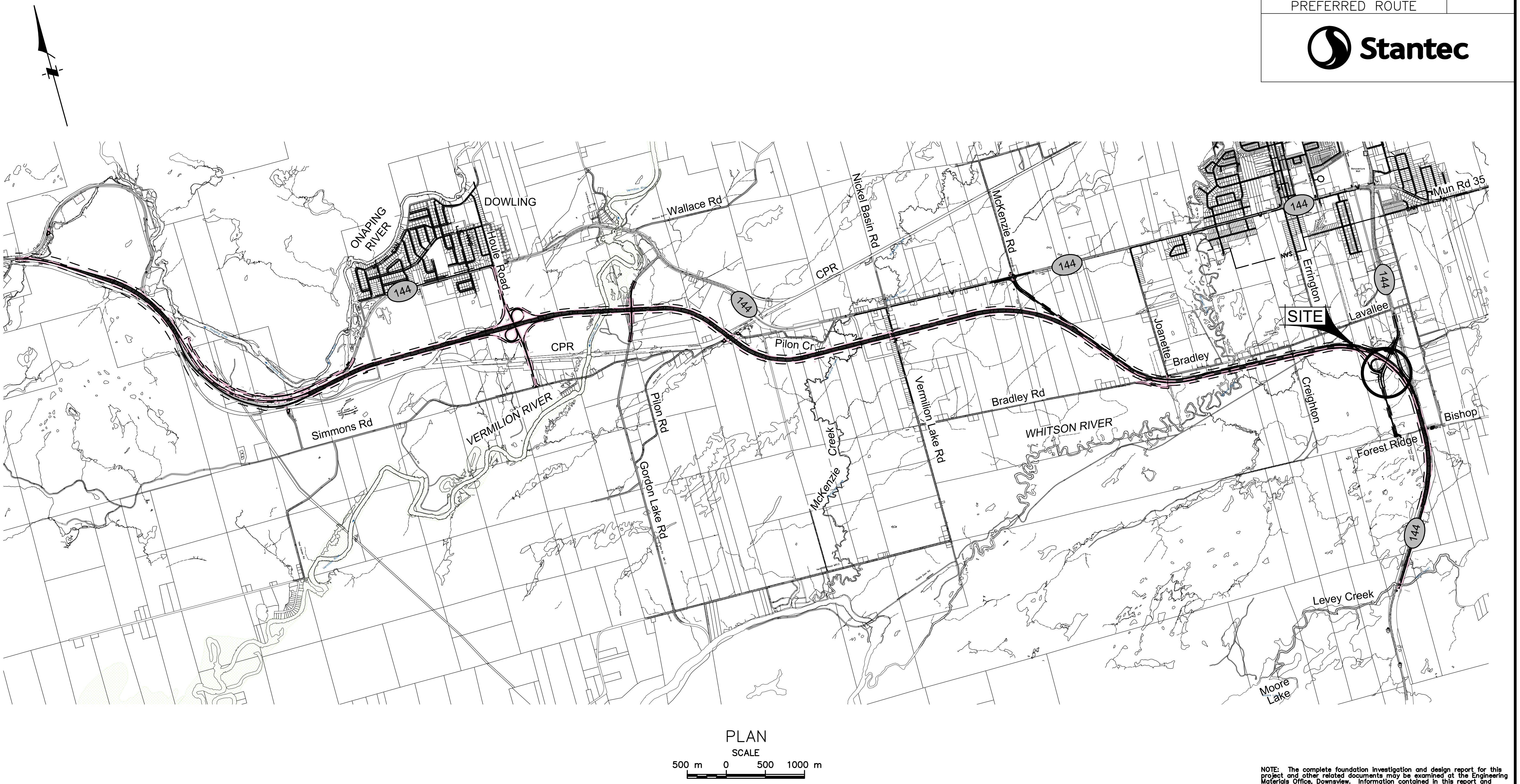
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No
CONT
WP 5023–09–00

HWY 144 ROUTE SELECTION STUDY
OLD HWY 144 TO FOREST RIDGE
PREFERRED ROUTE

SHEET

Stantec



PLAN
SCALE

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

GEOCRES No 411–300

HWY No 144			DIST
SUBM'D SKG	CHECKED	DATE 2014–03–07	SITE
DRAWN GBB	CHECKED	APPROVED	DWG 2

MINISTRY OF TRANSPORTATION, ONTARIO 88–05 PR–D–707

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Stantec	Project No.: 165000734	GWP: 5023-09-00	Site Photographs
	Project Name: Highway 144 Route Planning and Preliminary Design Study, Chelmsford to Dowling, ON		Date: February 25, 2013

Site Photo No.: 1 Looking north near BH13-28

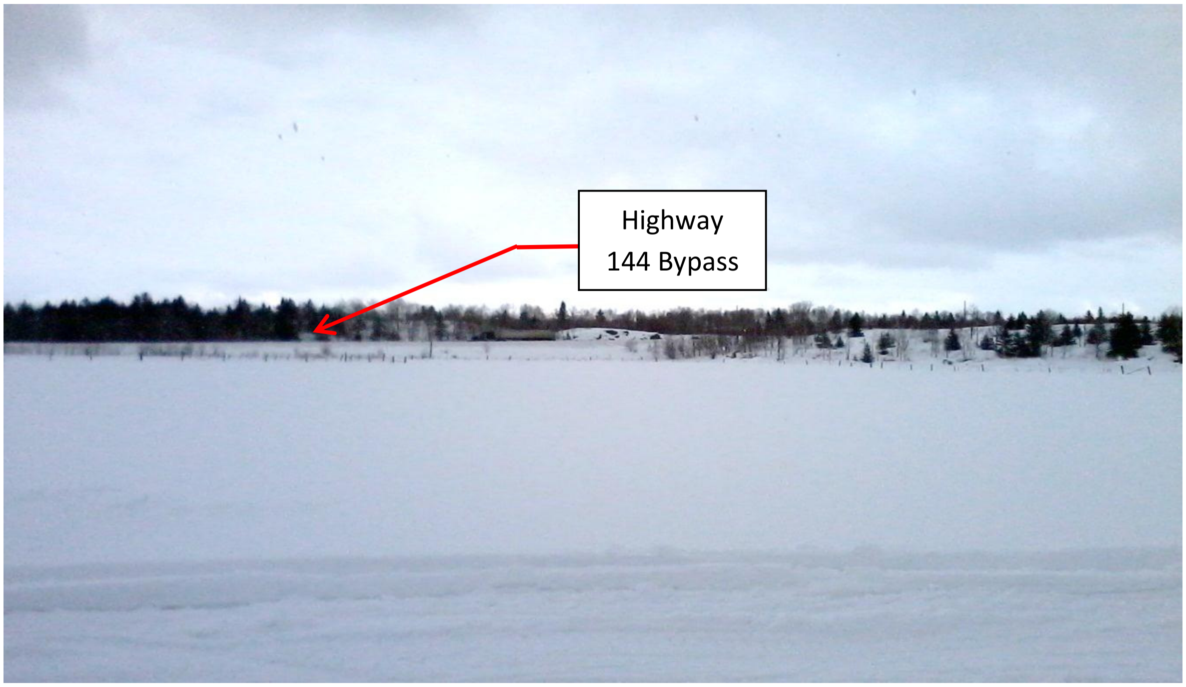
Site Photo No.: 2 Looking northwest near BH13-28

Site Photo No.: 3	Looking west near BH 13-28

Site Photo No.: 4	Looking southwest. BH13-27 shown in the background

Stantec	Project No.: 165000734	GWP: 5023-09-00	Site Photographs
	Project Name: Highway 144 Route Planning and Preliminary Design Study, Chelmsford to Dowling, ON		Date: February 25, 2013

Site Photo No.: 5	Looking south near BH 13-28



Site Photo No.: 6	Looking southeast near BH13-28. Highway 144 Bypass in background

APPENDIX B

Symbols and Terms Used on Borehole Records

Borehole Records

Rock Core Records

Rock Core Photographs

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 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
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>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
Very Soft	<0.25	<12.5
Soft	0.25 - 0.5	12.5 - 25
Firm	0.5 - 1.0	25 - 50
Stiff	1.0 - 2.0	50 – 100
Very Stiff	2.0 - 4.0	100 - 200
Hard	>4.0	>200

ROCK DESCRIPTION

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor*
25-50	*Poor*
50-75	*Fair*
75-90	*Good*
90-100	*Excellent*

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	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

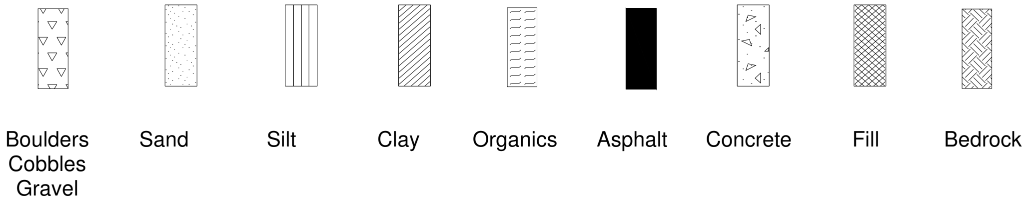
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

Terminology describing rock weathering:

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



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
γ	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

Stantec

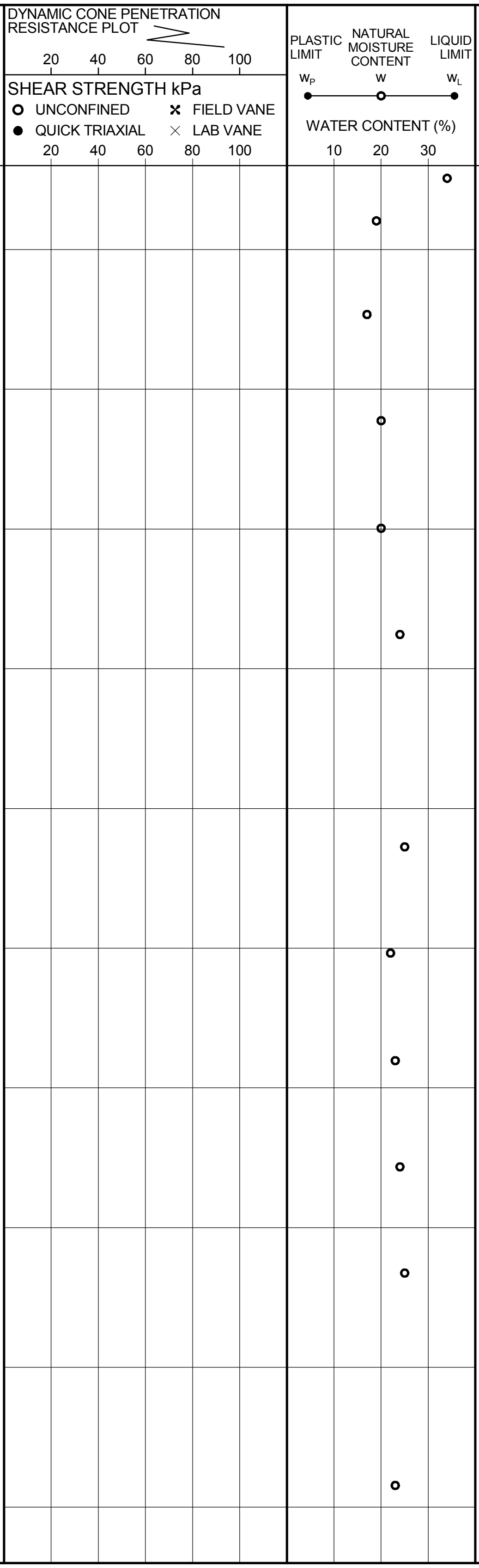
RECORD OF BOREHOLE No BH13-27

1 OF 3 **METRIC**

W.P. GWP 5023-09-00
LOCATION Old Hwy 144 to Forest Ridge Road, Greater Sudbury, ON N: 5 156 555 E: 290 350
ORIGINATED BY JHJ
DIST ____ HWY 144
BOREHOLE TYPE Hollow Stem Augers - Splitspoon Sampler, NQ Rock Core
COMPILED BY SH
DATUM Geodetic
DATE 2013 02 19 - 2013 02 24
CHECKED BY CM/SG

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
266.6	Topsoil								
266.6 0.0	180 mm TOPSOIL								
266.4 0.2	SILTY SAND (SM) Loose to compact Grey, moist to wet	1	SS	9					
		2	SS	10					
	- Groundwater inferred at a depth of 2.0 m (elevation 264.6 m)	3	SS	8	▽	0	57	(43)	
	- Flowing sands encountered	4	SS	5					
		5	SS	10		0	88	(12)	
	- Flowing sands encountered. Remainder of borehole cased with N casing.	6	SS	-					
		7	SS	4					
		8	SS	13					
		9	SS	20		0	82	(18)	
		10	SS	8					
		11	SS	7					
		12	SS	5					
257.5 9.1	CLAYEY SILT (CL) Stiff to very stiff Grey, wet	13	SS	19					
256.6									

ELEVATION SCALE: 266, 265, 264, 263, 262, 261, 260, 259, 258, 257



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

✕ 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH13-27

2 OF 3

METRIC

W.P. GWP 5023-09-00
LOCATION Old Hwy 144 to Forest Ridge Road, Greater Sudbury, ON N: 5 156 555 E: 290 350
ORIGINATED BY JHJ
DIST HWY 144
BOREHOLE TYPE Hollow Stem Augers - Splitspoon Sampler, NQ Rock Core
COMPILED BY SH
DATUM Geodetic
DATE 2013 02 19 - 2013 02 24
CHECKED BY CM/SG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR SA SI CL
10.0	*(continued)* CLAYEY SILT (CL) Stiff to very stiff Grey, wet	14	SS	23		0 2 90 8 PP = 105 kPa Non-Plastic Fines
		15	SS	31		PP = 150 kPa
		16	SS	-		- s_u > 106 kPa - s_u > 106 kPa
		17	SS	7		0 0 65 35
		18	SS	-		- s_u > 106 kPa - s_u > 106 kPa
249.8 16.8	SILTY CLAY (CI) Stiff Grey	19	SS	6		
		20	SS	7		0 0 38 62
247.4 19.2	Metasedimentary BEDROCK: Laminated argillites and siltstones -excellent quality -unweathered -black colour -one joint set with close to wide spacing	21	SS	100/ 75 mm		
		22	NQ	-		TCR = 96% RQD = 96% UCS = 76 MPa
246.6						

Continued Next Page

ELEVATION SCALE

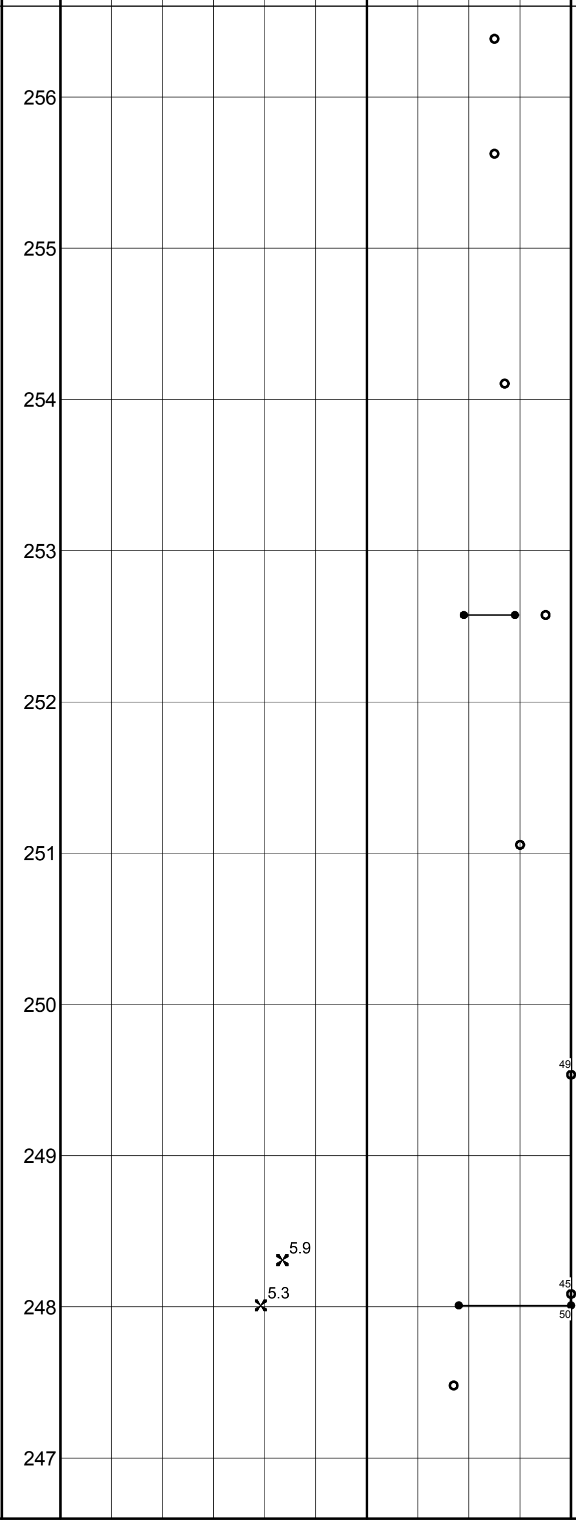
DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%)
10 20 30

UNIT WEIGHT γ kN/m^3



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

O$^{3\%}$ STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH13-27

3 OF 3 **METRIC**

W.P. GWP 5023-09-00 LOCATION Old Hwy 144 to Forest Ridge Road, Greater Sudbury, ON N: 5 156 555 E: 290 350 ORIGINATED BY JHJ

DIST HWY 144 BOREHOLE TYPE Hollow Stem Augers - Splitspoon Sampler, NQ Rock Core COMPILED BY SH

DATUM Geodetic DATE 2013 02 19 - 2013 02 24 CHECKED BY CM/SG

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
20.0	*(continued)* Metasedimentary BEDROCK: Laminated argillites and siltstones - excellent quality - medium strong to strong - black colour - unweathered - one joint set with close to wide spacing (Refer to Field Bedrock Core Log)										
							246				
			23	NQ	-						TCR = 100% RQD = 100%
							245				
			24	NQ	-						TCR = 98% RQD = 98% UCS = 47 MPa
							244				
243.7 22.9	End of Borehole										

✕ 3, × 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH13-28

1 OF 3

METRIC

W.P. GWP 5023-09-00

LOCATION Old Hwy 144 to Forest Ridge Road, Greater Sudbury, ON N: 5 156 599 E: 290 408

ORIGINATED BY JHJ

DIST

HWY 144

BOREHOLE TYPE Hollow Stem Augers - Splitspoon Sampler, NQ Rock Core

COMPILED BY SH

DATUM Geodetic

DATE 2013 02 24 - 2013 02 25

CHECKED BY CM/SG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
266.5	Topsoil									
0.0	250 mm TOPSOIL									
266.3 0.3	SILTY SAND (SM) Very loose to dense Brown to light grey, moist to wet	1	SS	11						
						266				
		2	SS	11			0	51	(49)	
	-Groundwater inferred at a depth of 1.4 m (elevation 265.1 m)				▽	265				
		3	SS	11			0	69	28	3
						264				
		4	SS	21						
		5	SS	12			0	70	(30)	
						263				
		6	SS	27						
	- Occasional cobbles					262				
		7	SS	14						
						261				
		8	SS	20						
		9	SS	33			0	83	(17)	
						260				
		10	SS	3						
						259				
		11	SS	6						
						258				
		12	SS	4						
		13	SS	14		257	0	65	(35)	

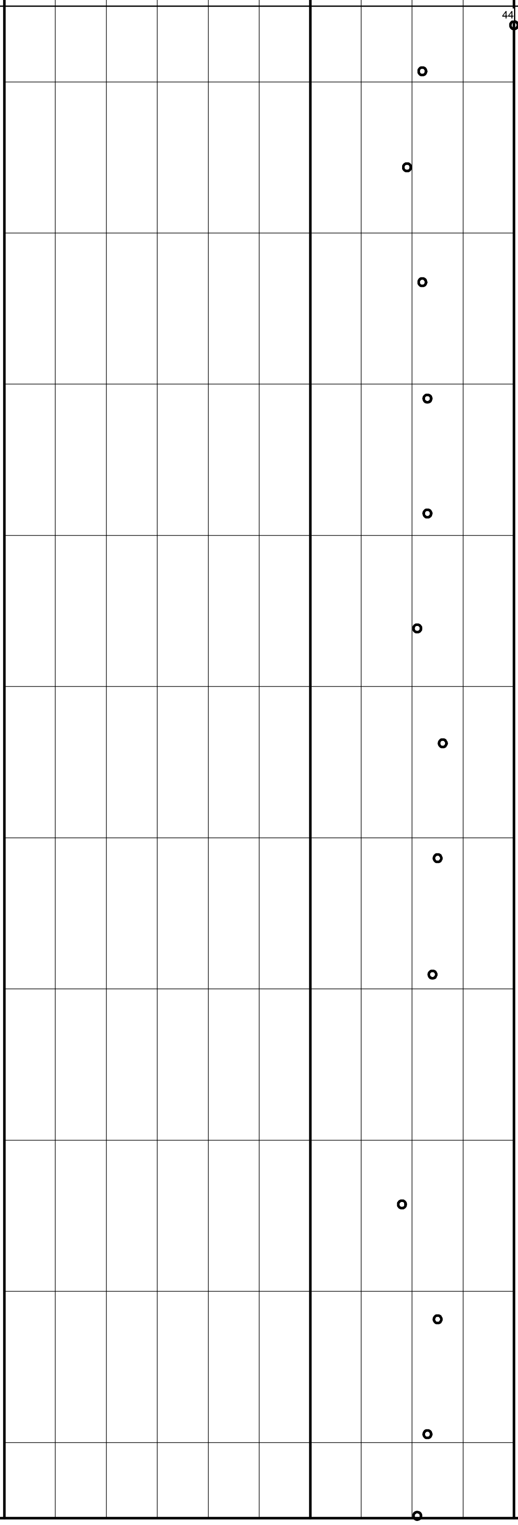
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3



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

O $^{3\%}$ STRAIN AT FAILURE

Stantec

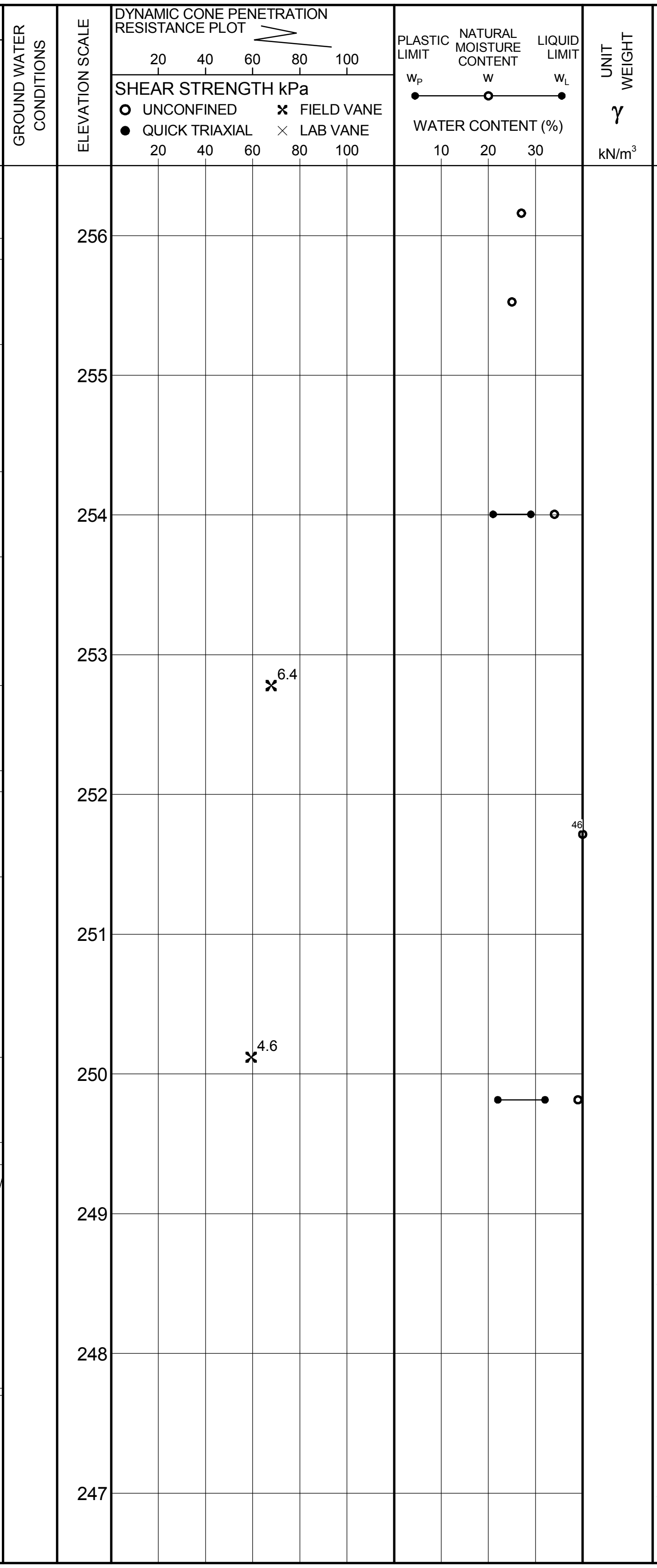
RECORD OF BOREHOLE No BH13-28

2 OF 3

METRIC

W.P. GWP 5023-09-00
LOCATION Old Hwy 144 to Forest Ridge Road, Greater Sudbury, ON N: 5 156 599 E: 290 408
ORIGINATED BY JHJ
DIST
HWY 144
BOREHOLE TYPE Hollow Stem Augers - Splitspoon Sampler, NQ Rock Core
COMPILED BY SH
DATUM Geodetic
DATE 2013 02 24 - 2013 02 25
CHECKED BY CM/SG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS	GR	SA	SI	CL
256.4 10.1	CLAYEY SILT (CL) Firm to very stiff Grey, wet	14	SS	16					
		15	SS	10	PP = 85 kPa				
		16	SS	4		0	0	61	39
		17	SS	8					
		18	SS	9					
		19	SS	4		0	2	54	44
249.3 17.2	Metasedimentary BEDROCK: Laminated argillites and siltstones - fair to good quality - very strong - black colour - unweathered - two joint sets with very close to moderate spacing (Refer to Field Bedrock Core Log)	20	SS	100/ 75mm					
		21	NQ	-	TCR = 93% RQD = 83% UCS = 104 MPa				
246.5		22	NQ	-	TCR = 69% RQD = 69% UCS = 148 MPa				



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× 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH13-28

3 OF 3

METRIC

W.P. GWP 5023-09-00
LOCATION Old Hwy 144 to Forest Ridge Road, Greater Sudbury, ON N: 5 156 599 E: 290 408
ORIGINATED BY JHJ
DIST
HWY 144
BOREHOLE TYPE Hollow Stem Augers - Splitspoon Sampler, NQ Rock Core
COMPILED BY SH
DATUM Geodetic
DATE 2013 02 24 - 2013 02 25
CHECKED BY CM/SG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED ✕ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
20.0	*(continued)* Metasedimentary BEDROCK: Laminated argillites and siltstones										
246.2 20.3	End of Borehole										

✕ 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

Field Bedrock Core Log

Stantec

Client: Ontario Ministry of Transportation
Project: Hwy 144 - Chelmsford Bypass
Contractor: Abraflex Drilling

Project No.: 165000734
Date: February 19, 2013
Borehole No.: BH13-27
Logger: SH

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		
19.16	NQ22	96%	96%	20.03	Metasedimentary bedrock: black laminated argillites and siltstones (Onwatin Formation)	S	U	1	J	D	C-M	SP	-	T	dipping calcite and pyrite lenses <1-1 mm thick	Mohs Hardness: H=3-5.5
20.03	NQ23	100%	100%	21.57	Metasedimentary bedrock: black laminated argillites and siltstones (Onwatin Formation)	-	U	1	J	D	C-W	SP	-	T	dipping calcite and pyrite lenses <1-1 mm thick	Mohs Hardness: H=3-5.5
21.57	NQ24	98%	98%	22.87	Metasedimentary bedrock: black laminated argillites and siltstones (Onwatin Formation)	MS	U	1	J	D	C-M	SP	-	T	dipping calcite and pyrite lenses <1-1 mm thick / subvertical calcite vein 3 mm thick	Mohs Hardness: H=3-5.5

STRENGTH (MPa)
EH = Extremely Strong = > 250
VS = Very Strong = 100-250
S = Strong = 50-100
MS = Medium Strong = 25-50
W = Weak = 5 - 25
VW = Very Weak = 1-5
EW = Extremely Weak = < 1

WEATHERING
U = Unweathered = No Signs
S = Slightly = Oxidized
M = Moderately = Discoloured
H = Highly = Friable
C = Completely = Soil-like

DISCONTINUITY TYPE
B = Bedding Joint
J = Cross Joint
F = Fault
S = Shear Plane

SPACING
VW = Very Wide = >3m
W = Wide = 1-3 m
M = Moderate = 0.3-1 m
C = Close = 5-30 cm
VC = Very Close = <5 cm

ORIENTATION
F = Flat = 0-20^{0}
D = Dipping = 20-50^{0}
V = n-Vertical = >50^{0}

ROUGHNESS
RU = Rough Undulating
RP = Rough Planar
SU = Smooth Undulating
SP = Smooth Planar
LU = Slickensided Undulating
LP = Slickensided Planar

FILLING
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

Field Bedrock Core Log

Stantec

Client: Ontario Ministry of Transportation
Project: Hwy 144 - Chelmsford Bypass
Contractor: Abraflex Drilling

Project No.: 165000734
Date: February 24, 2013
Borehole No.: BH13-28
Logger: SH

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		
17.22	NQ21	93%	83%	18.75	Metasedimentary bedrock: black laminated argillites and siltstones (Onwatin Formation)	VS	U	2	J	F	VC	RP	-	T	dipping calcite lenses <<1 mm thick	Mohs Hardness: H=3-5.5
									J	D	C-M	SP	-	T		
18.75	NQ22	69%	69%	20.25	Metasedimentary bedrock: black laminated argillites and siltstones (Onwatin Formation)	VS	U	0	-	-	W	-	-	-	dipping calcite lenses <<1 mm thick	Mohs Hardness: H=3-5.5

STRENGTH (MPa)
EH = Extremely Strong = > 250
VS = Very Strong = 100-250
S = Strong = 50-100
MS = Medium Strong = 25-50
W = Weak = 5 - 25
VW = Very Weak = 1-5
EW = Extremely Weak = < 1

WEATHERING
U = Unweathered = No Signs
S = Slightly = Oxidized
M = Moderately = Discoloured
H = Highly = Friable
C = Completely = Soil-like

DISCONTINUITY TYPE
B = Bedding Joint
J = Cross Joint
F = Fault
S = Shear Plane

SPACING
VW = Very Wide = >3m
W = Wide = 1-3 m
M = Moderate = 0.3-1 m
C = Close = 5-30 cm
VC = Very Close = <5 cm

ORIENTATION
F = Flat = 0-20^0
D = Dipping = 20-50^0
V = n-Vertical = >50^0

ROUGHNESS
RU = Rough Undulating
RP = Rough Planar
SU = Smooth Undulating
SP = Smooth Planar
LU = Slickensided Undulating
LP = Slickensided Planar

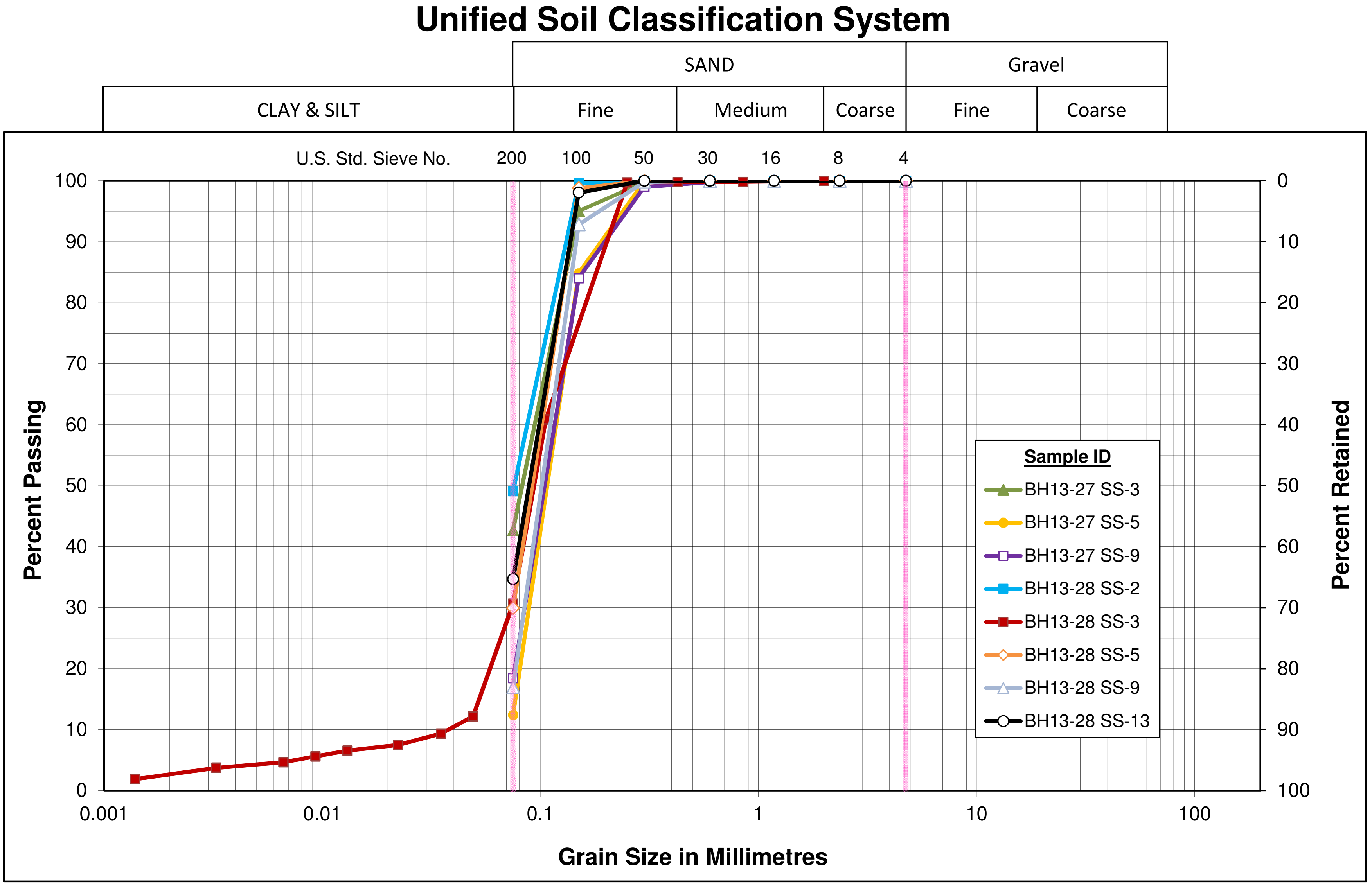
FILLING
T = Tight, Hard
O = Oxidized
SA = Slightly Altered, Clay Free
S = Sandy, Clay Free
Si = Sandy, Silty, Minor Clay
NC = Non-softening Clay
SC = Swelling, Soft Clay

APPENDIX C

Laboratory Test Results

Figures 1 and 2: Grain Size Distribution Plots

Figure 3: Plasticity Chart



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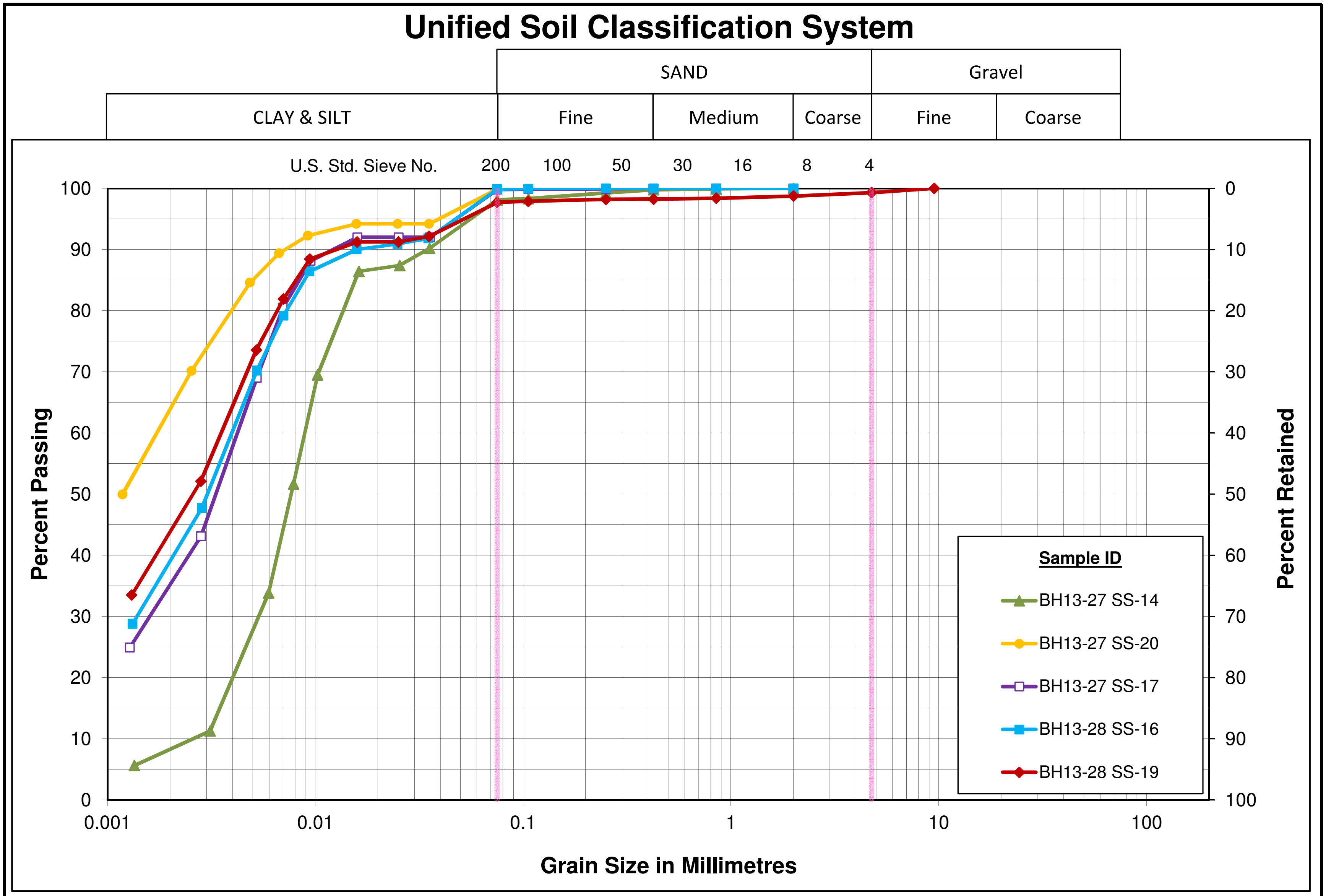
GRAIN SIZE DISTRIBUTION

Silty Sand (SM)

Figure No. 1

Project No. 165000734
GWP 5023-09-00



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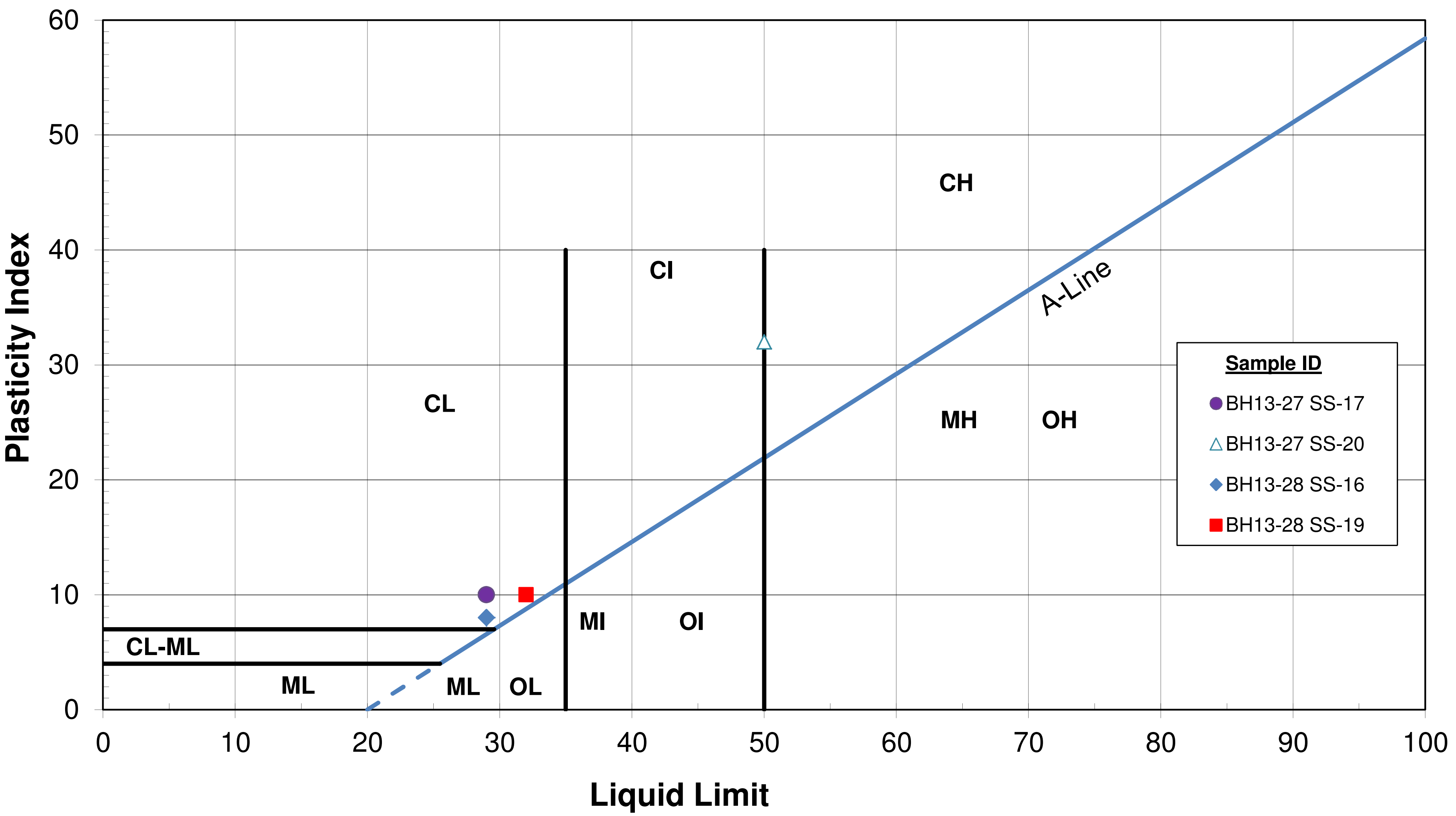
GRAIN SIZE DISTRIBUTION

Clayey Silt (CL) & Silty Clay (CI)

Figure No. 2

Project No. 165000734

GWP 5023-09-00



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PLASTICITY CHART

Figure No. 3

Project No. 165000734
GWP 5023-09-00

APPENDIX D

Figure 4: Preliminary Design Parameters

Figure 5: Assessment of Liquefaction Resistance

Preliminary LPILE Analysis Results

Figure 6: Lateral Deflection of HP310x110

Figure 7: P-y Curves for HP310x310

Preliminary Slope Stability Evaluation Results

Figure 8a: Static (long-term)

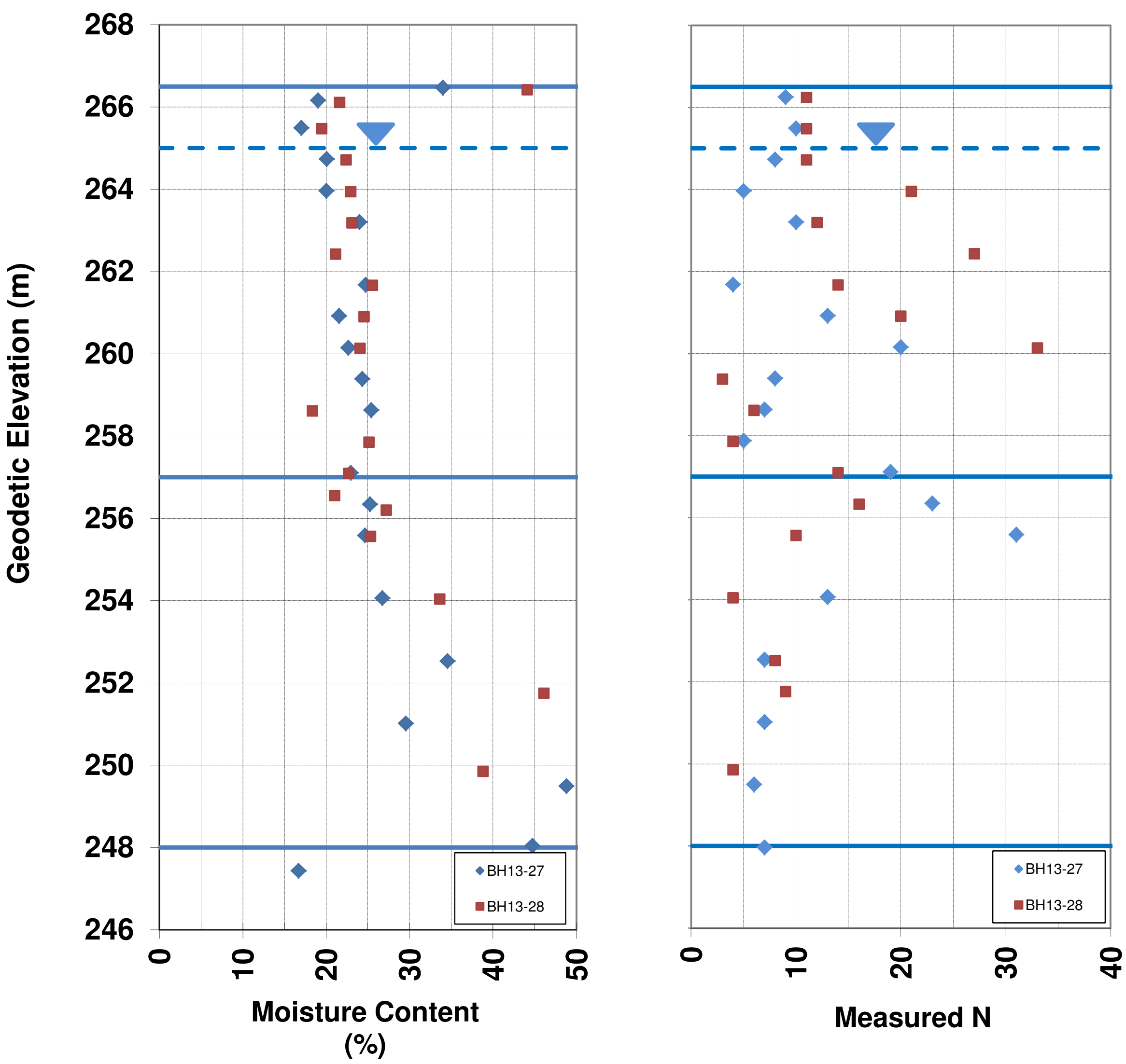
Figure 8b: Static (short-term)

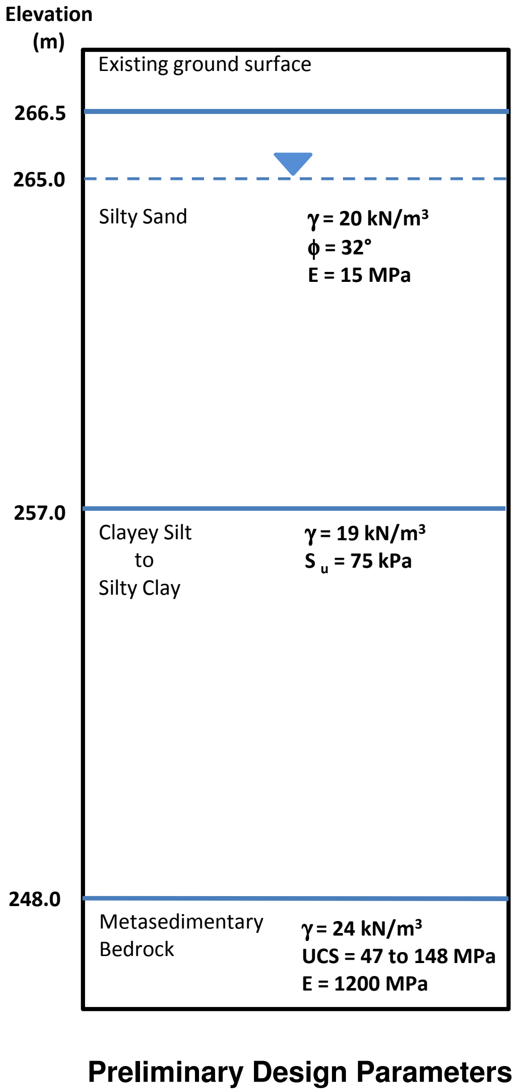
Figure 8c: Seismic

Preliminary Settlement Analysis

Figure 9: Preliminary Settlement Results

Table D-1: Spring Stiffnesses for HP310x110





Existing ground surface

Silty Sand
$\gamma = 20\ kN/m^3$
$\phi = 32°$
E = 15 MPa

Clayey Silt to Silty Clay
$\gamma = 19\ kN/m^3$
$S_u = 75\ kPa$

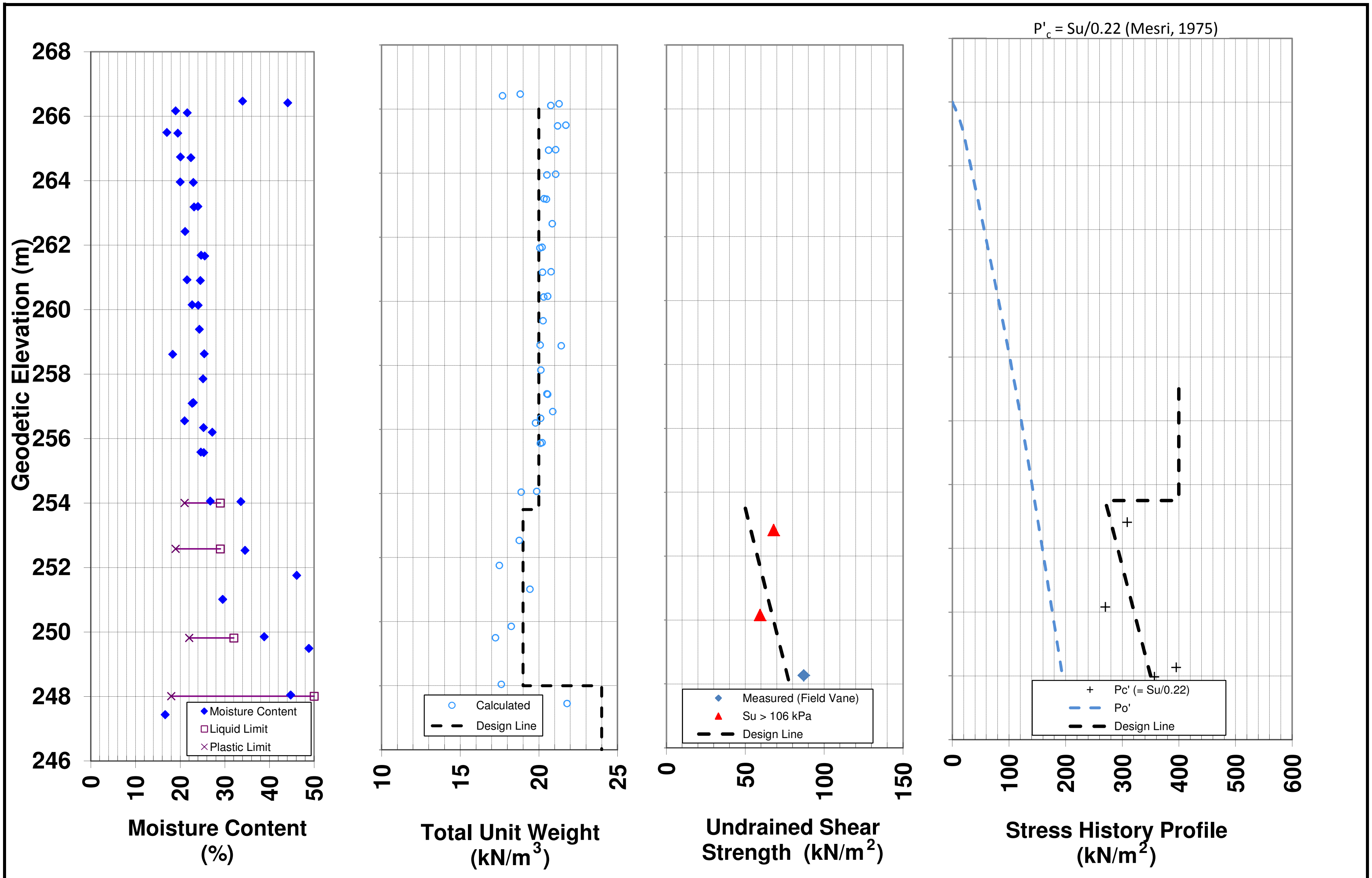
Metasedimentary Bedrock
$\gamma = 24\ kN/m^3$
UCS = 47 to 148 MPa
E = 1200 MPa

Preliminary Design Parameters

Stantec Consulting Ltd.

Project No. 16500734 - Old Highway 144
to Forest Ridge Road Underpass
GWP No. 5023-09-00

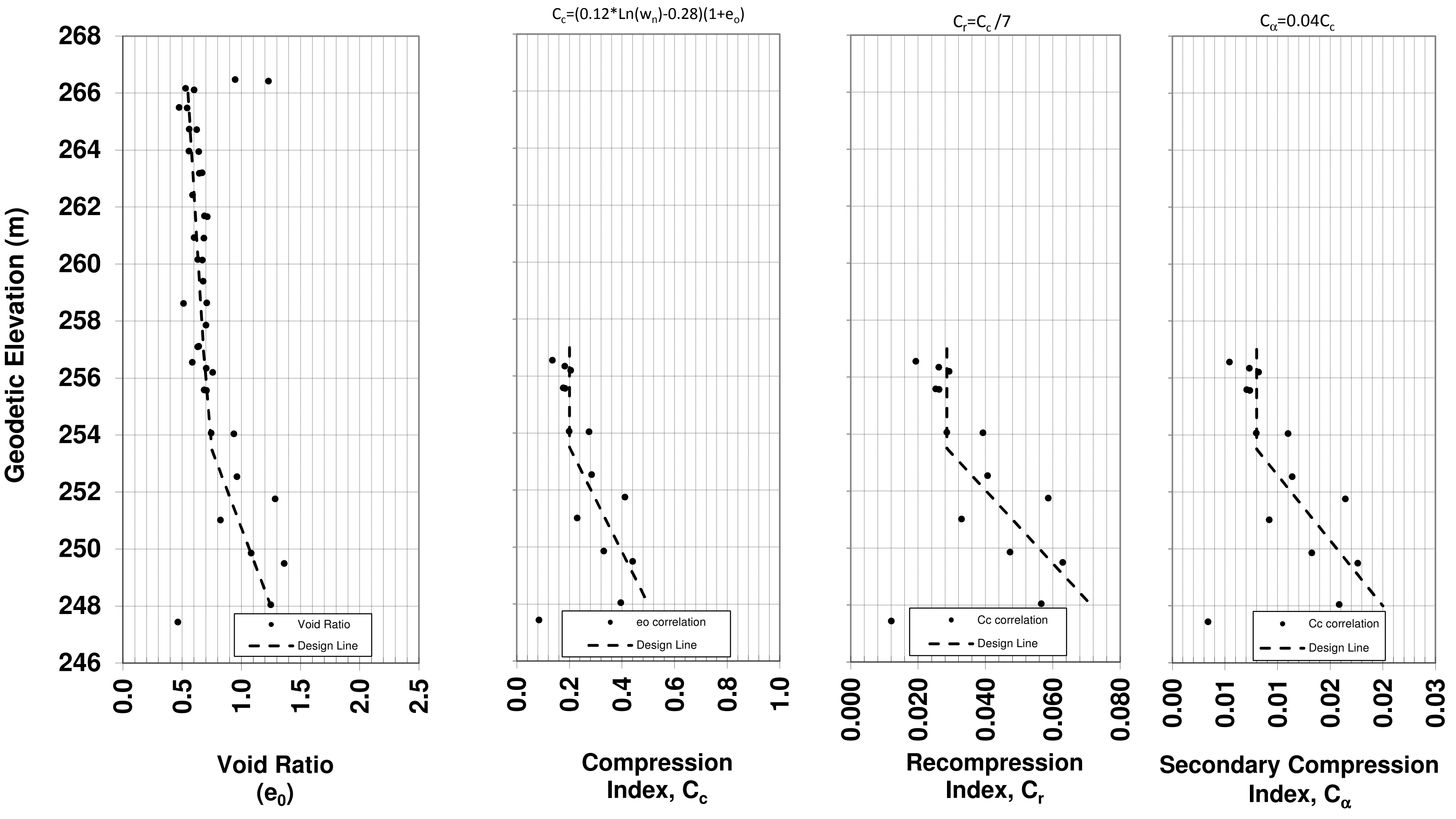
Figure 4a



Stantec Consulting Ltd.

Project No. 16500734 - Old Highway 144
to Forest Ridge Road Underpass
GWP No. 5023-09-00

Figure 4b

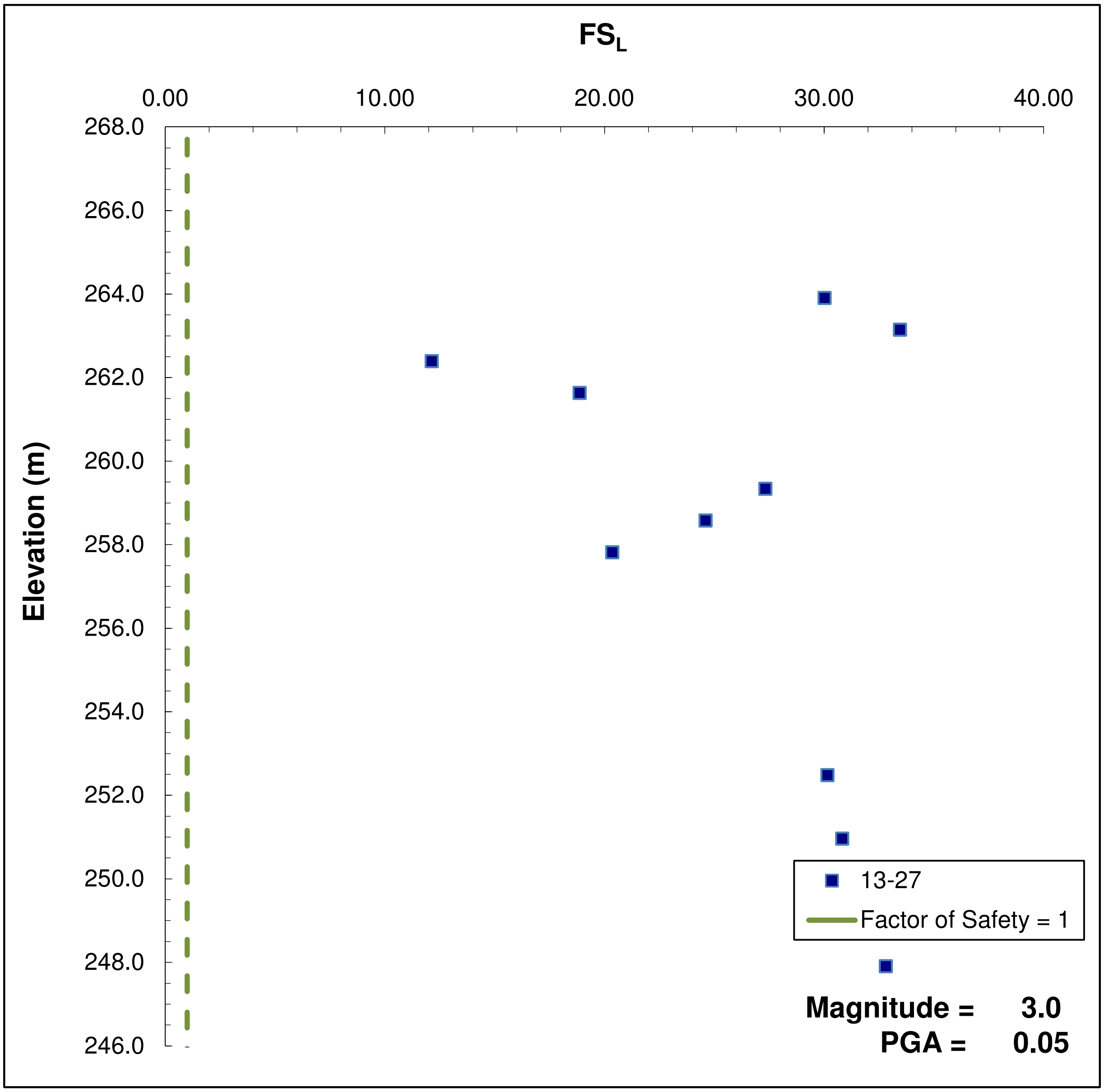


Stantec Consulting Ltd.

Project No. 165000734 - Old Highway
144 to Forest Ridge Road Underpass
GWP No. 5023-09-00

Figure 4c

CHARACTERIZATION OF LIQUEFACTION RESISTANCE



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

Project No. 165000734
Figure No. 5

LPile Results - Lateral Deflection

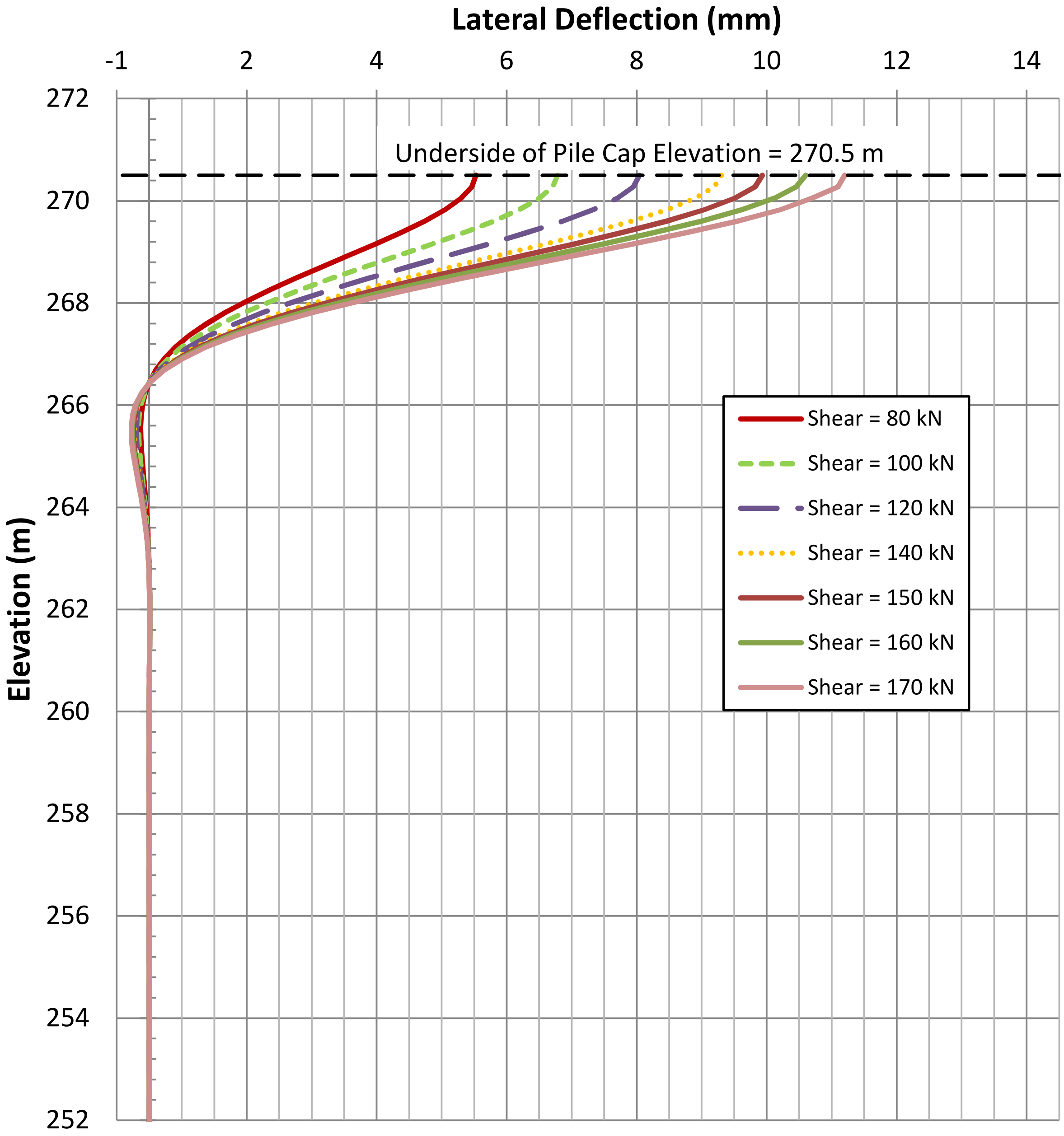


Figure 6
Lateral Deflection of HP 310x110 Piles

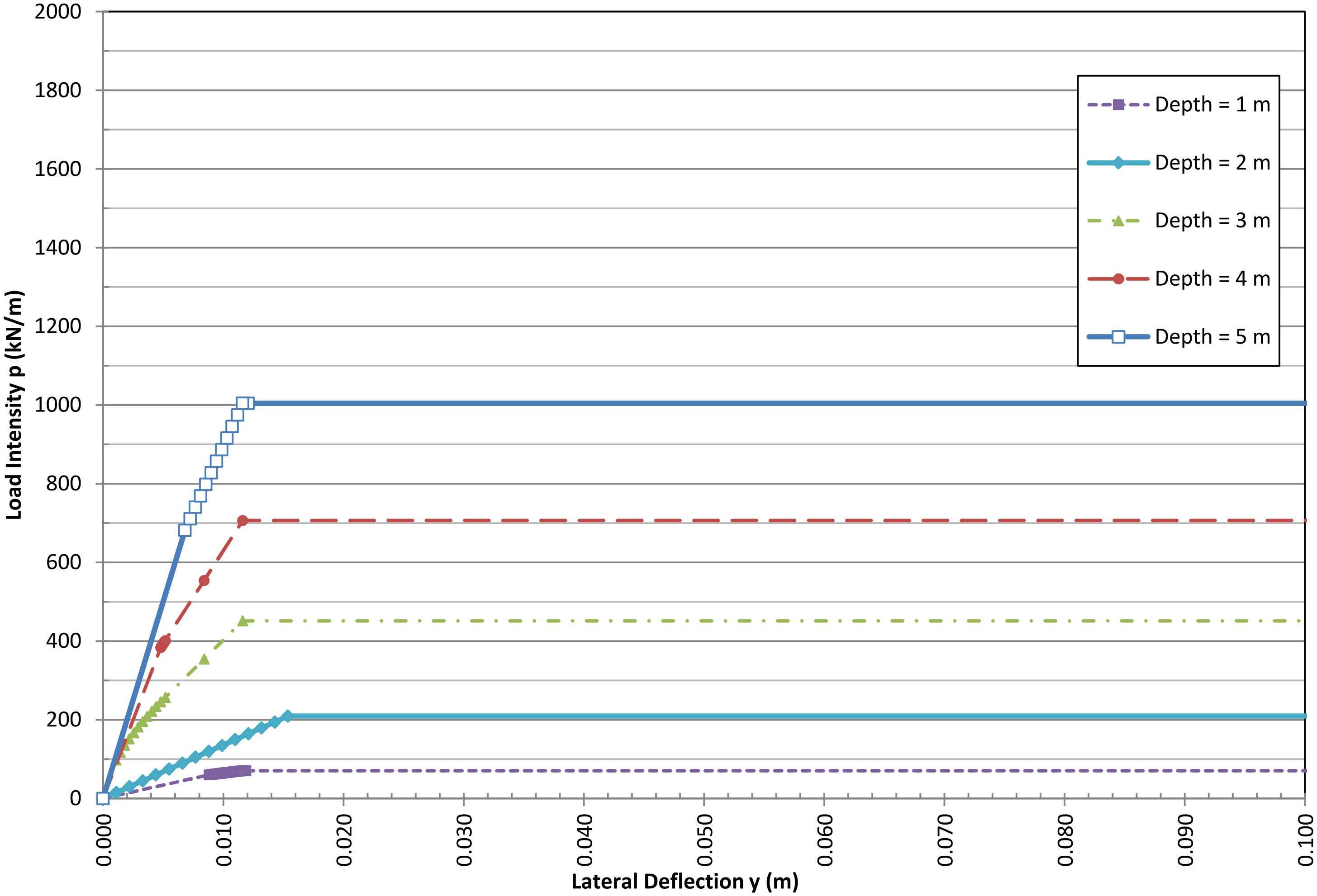


Figure 7a
p-y Curves for Proposed HP 310x110 Piles

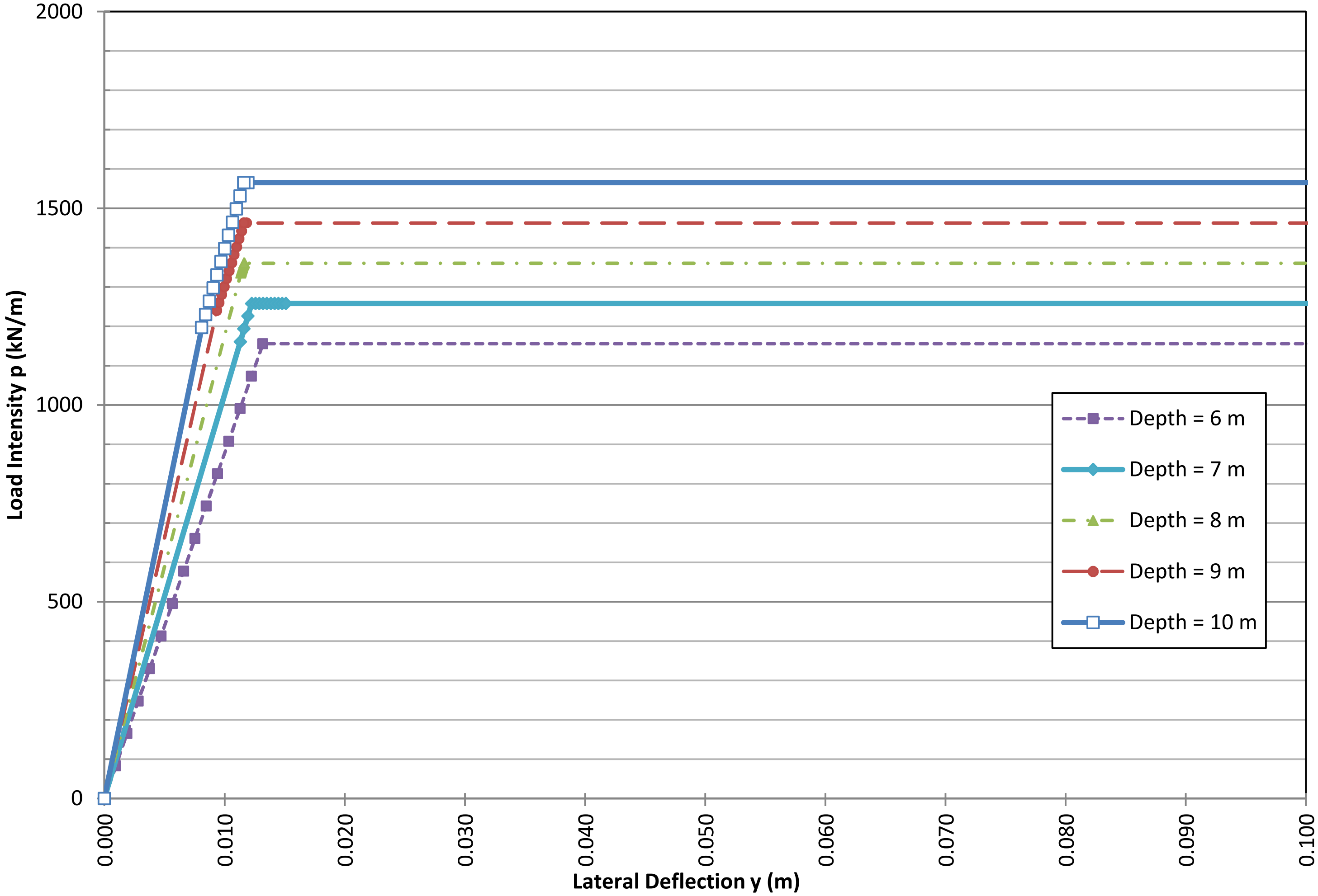


Figure 7b
p-y Curves for Proposed HP 310x110 Piles

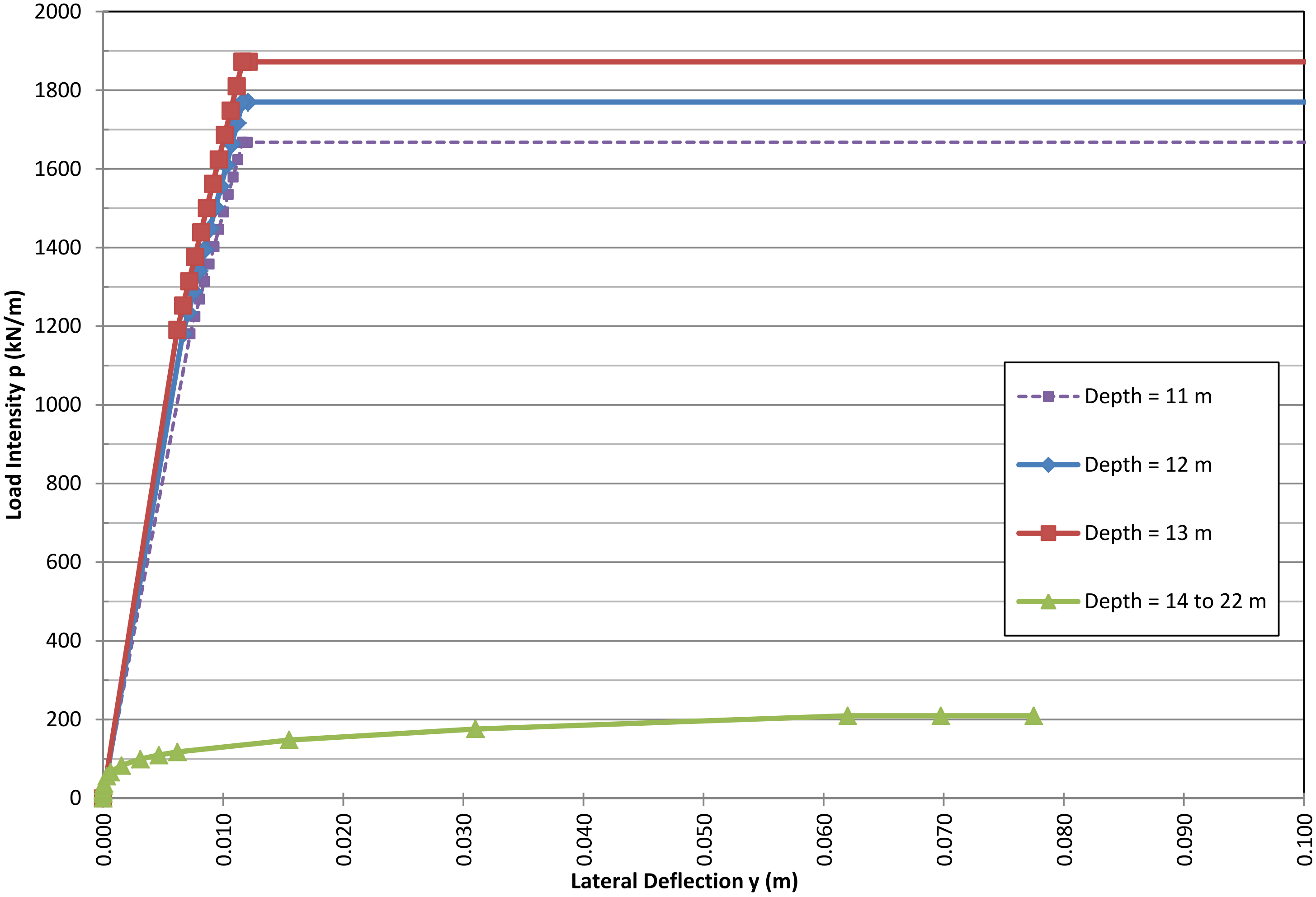
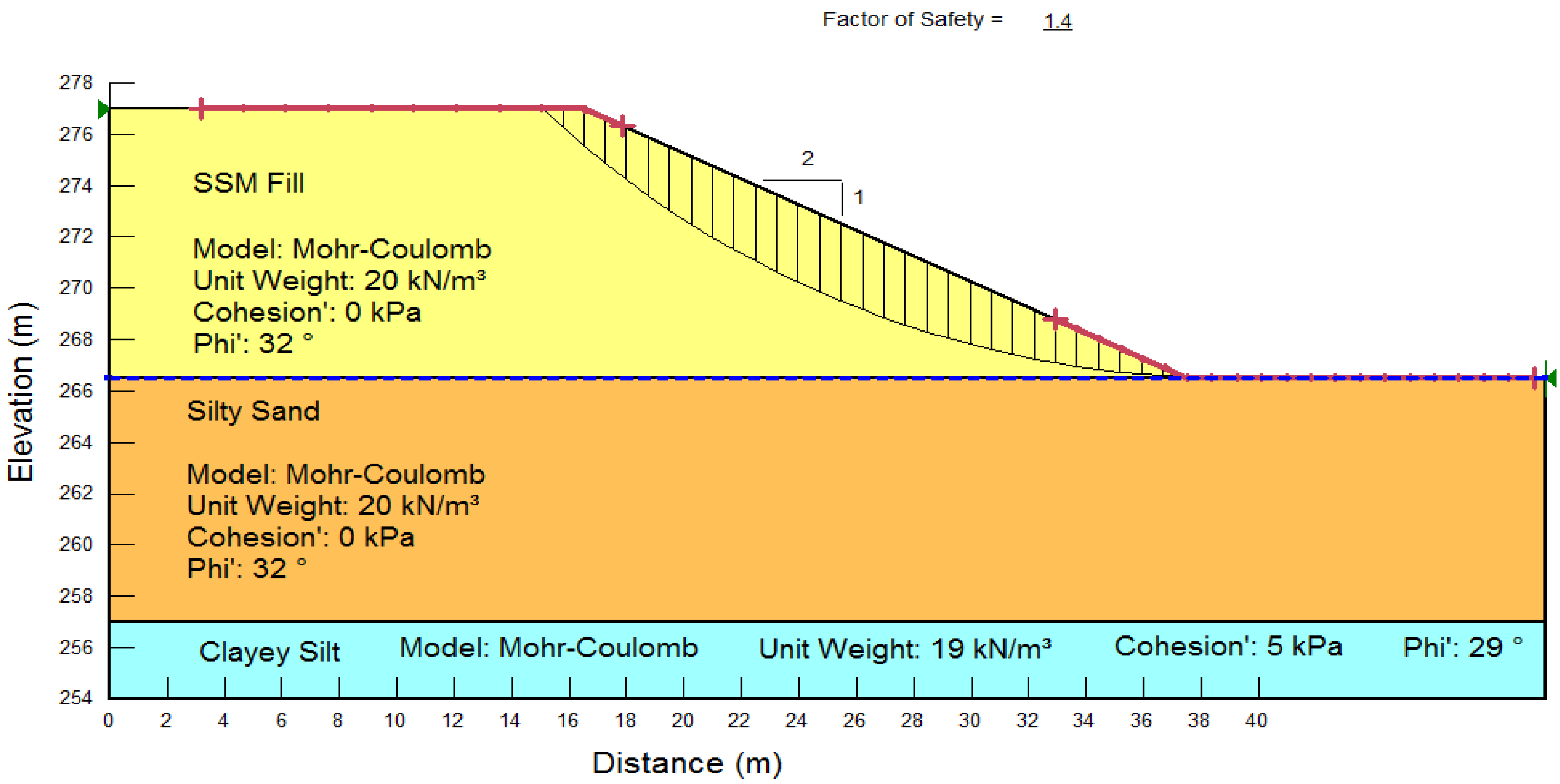


Figure 7c
p-y Curves for Proposed HP 310x110 Piles



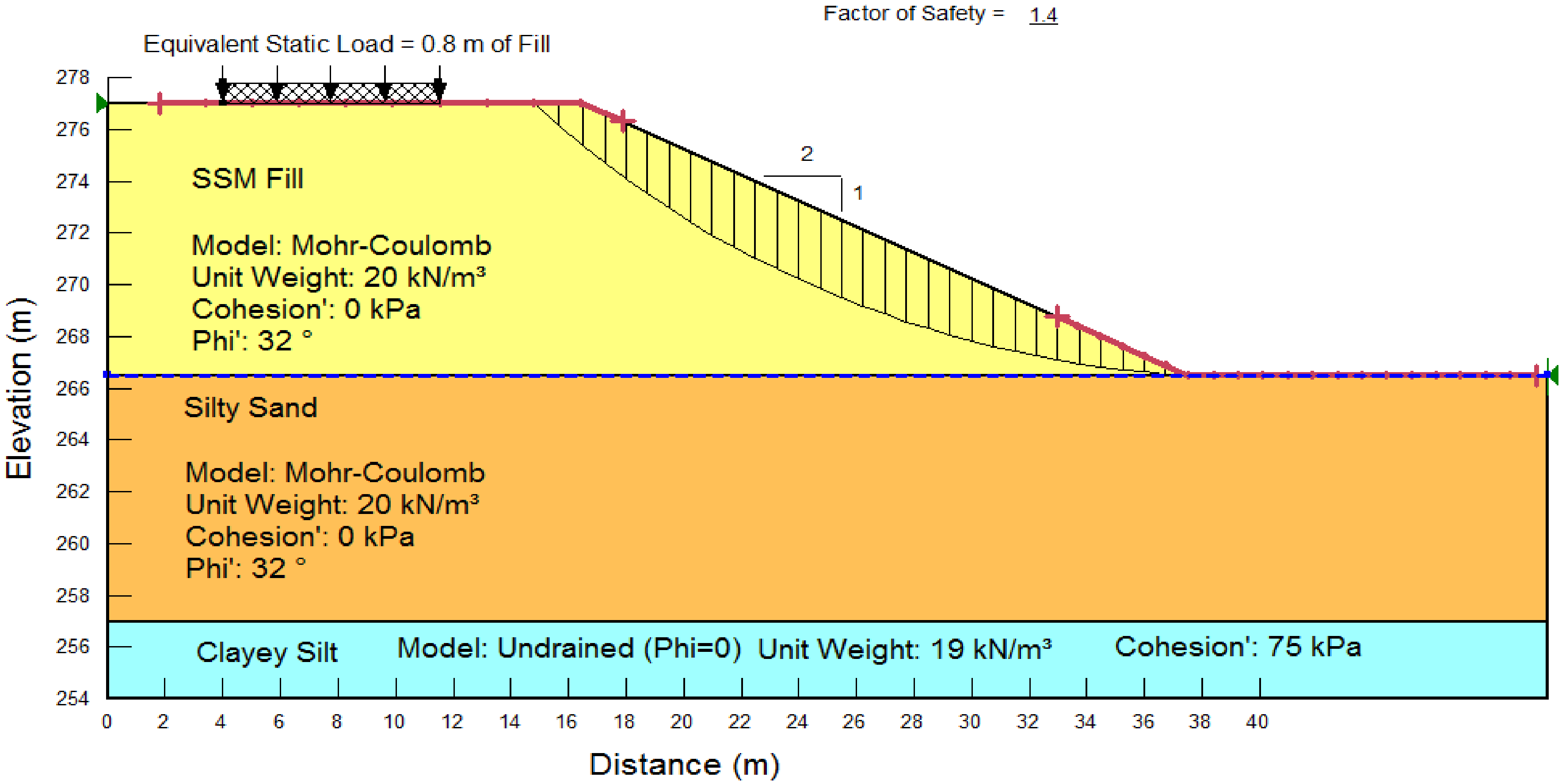
Stantec

Static Slope Stability Analysis (Drained)

Highway 144 Chelmsford Bypass
Old Highway 144 to Forest Ridge Road Underpass

Figure 8a

Project No. 165000734
GWP No. 5023-09-00



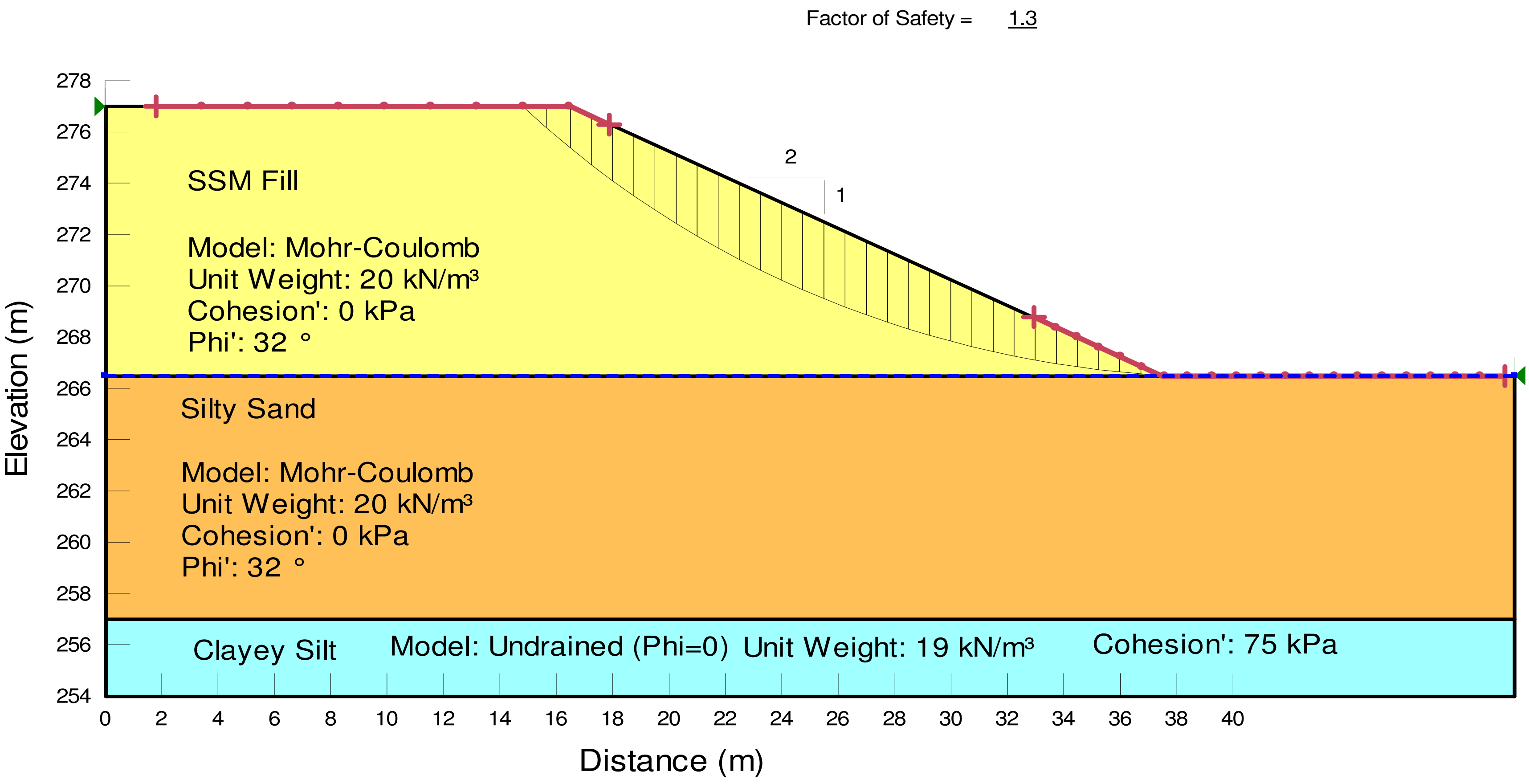
Stantec

Static Slope Stability Analysis (Undrained)

Highway 144 Chelmsford Bypass
Old Highway 144 to Forest Ridge Road Underpass

Figure 8b

Project No. 165000734
GWP No. 5023-09-00



Stantec

Seismic Slope Stability Analysis

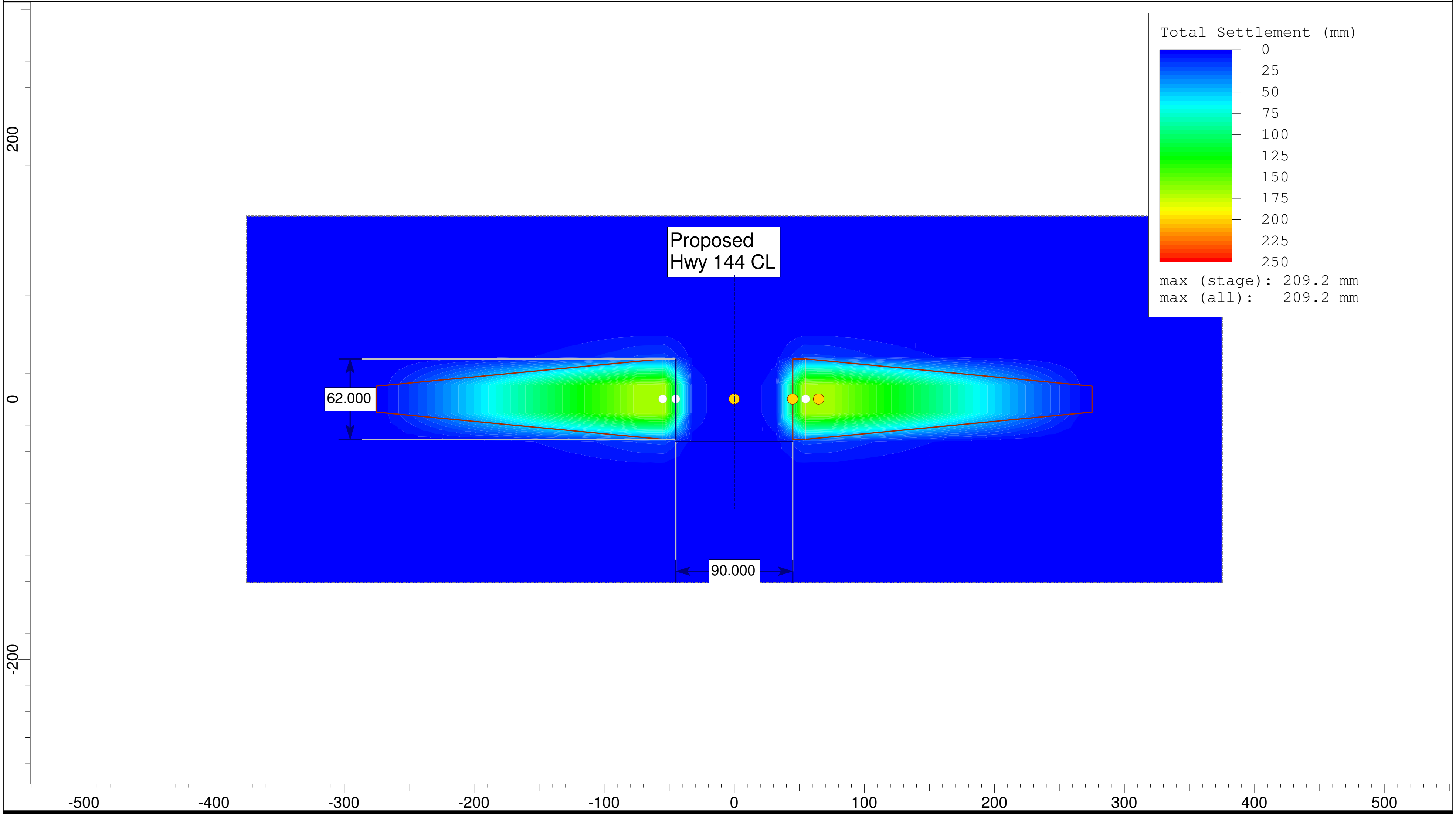
Highway 144 Chelmsford Bypass
Old Highway 144 to Forest Ridge Road Underpass

Figure 8c

Project No. 16500734

GWP No. 5023-09-00

Figure 9



Project
Highway 144 - Old Highway 144 to Forest Ridge Road Underpass, Sudbury, ON

Analysis Description
165000734

Drawn By
LB

Company
Stantec

Date
4/4/2013, 12:09:56 PM

File Name
Old Hwy 144 to Forest Ridge Rd.s3z

SETTLE3D 2.011