Preliminary Foundation Investigation and Design Report
Gordon Lake Road Underpass

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

G.W.P. 5023-09-00

Geocres No. 41I-302

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Ministry of Transportation Ontario

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
For
G.W.P. 5023-09-00

Gordon Lake 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 Gordon Lake Road Underpass along the preferred alignment, approximately 8.5 km west of the town 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 8.5 km west of Chelmsford and 820 m south of 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, provided in Appendix A. At the project site, the proposed Highway 144 is oriented approximately in an east-west direction. The approximate location of the proposed structure is near the future Highway 144 Station 21+240 Balfour Township. An exhibit showing the preferred Highway 144 alignment along with the proposed Gordon Lake Road Underpass site is provided in Drawing No. 2 of Appendix A.

The proposed final grade of Highway 144 at the proposed Gordon Lake Road Overpass is site is approximately 267.3 m, based on the preliminary General Arrangement drawing. The proposed final grade of Gordon Lake Road is approximately 275.0 m. The anticipated height of embankment to achieve the proposed grade is approximately 8.0 m.

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

General Site Description

At the project site, the existing Gordon Lake Road is oriented approximately in a north-south direction and has a single lane in each direction. Both sides of the road are covered with dense shrubs, bushes, and mature trees. The surrounding area is generally flat to undulating. Photographs 1 through 4 in Appendix A show the general site features near the proposed structure site.

The site is located within the mid-Vermilion watershed and is located approximately 300 m east of the Vermilion River which flows south towards Vermilion Lake and ultimately further south to its mouth at the Spanish River just east of Espanola. Locally, drainage is provided by streams flowing westerly towards the Vermilion River. Surface drainage along Gordon Lake Road is controlled with a network of ditches and culverts.

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 route selection and planning assignment 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

The proposed foundation elements are located approximately 40 to 50 m east of the existing Gordon Lake Road. The proposed alignment passes through a heavily treed area on private property; consequently, two boreholes were advanced near the eastern edge of Gordon Lake Road within 50 m of the proposed location of the underpass. The boreholes are designated BH13-8 and BH13-9, 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 public utility authorities to clear the borehole locations of both private and public utilities.

A road occupancy permit was issued by the City of Greater Sudbury prior to drilling the boreholes.

The field drilling program was carried out on February 26, 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. Split spoon samples were collected every 760 mm interval up to the depth of bedrock. 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. Road holes were sealed with cold asphalt patch where applicable.

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 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	
	BH13-8	**BH13-9**
MTM Zone 12 Coordinates Northing Easting	 5159730 281190	 5159812 281202
Ground Surface Elevation, m	268.1	266.7
Total Depth Drilled, m	7.8	6.2
End of Borehole Elevation, m	260.3	260.5
Depth Augered, m	4.5	3.2
Depth Cored, m	3.3	3.0
Number of Soil Samples	6	5

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	12
Atterberg Limits	2
Grain Size Distribution	3
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 roadway/embankment fill materials over clayey silt 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 thickness of the topsoil layer encountered in BH13-8 and BH13-9 was 150 mm and 50 mm, respectively.

4.1.2 Fill

A granular fill material was encountered in both boreholes immediately beneath the topsoil. The thickness of the granular fill was approximately 650 mm in BH13-8 and 750 mm in BH13-9, extending to bottom elevations of 267.3 m and 265.9 m respectively.

The fill was predominantly composed of silty sand with gravel. Occasional cobbles were noted within the fill in BH13-9.

The moisture content of the fill was 18% in BH13-8 and 12% in BH13-9. The grain size analysis test carried out on one sample of the roadway fill material indicated the following results:

Gravel:	8%
Sand:	44%
Fines (silt & clay):	48%

Representative grain size distribution plot for the fill 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 roadway fill. The thickness of the clayey silt layer was approximately 3.7 m in BH13-8 and 2.3 m in BH13-9, and extended to approximate bottom elevation of 263.6 m in both boreholes.

The clayey silt layer was composed predominantly of silt with clay and trace amounts of sand. The Standard Penetration Test (SPT) blow count (N-value) for the silt layer ranged from 5 to 28 blows/0.3 m. Pocket penetrometer testing carried out on selected split-spoon samples indicated an undrained shear strength measurement of 25 to 55 kPa, suggesting a firm to stiff consistency.

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

Gravel:	0%
Sand:	2 and 3%
Silt:	71 and 77%
Clay:	21 and 25%
Moisture Content:	21 to 32%

Atterberg limits tests carried out on two representative samples from this layer indicated plasticity indexes of 11 and 13. The Unified Soil Classification System (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.

4.2 BEDROCK

Bedrock was encountered in both boreholes immediately beneath the clayey silt layer at an approximate elevation of 263.6 m. The bedrock consists of slightly metamorphosed interbedded layers of grey to dark grey mudstone, lithic wacke, and siltstone of sedimentary origin.

The Rock Quality Designation (RQD) values ranged between 86% and 100%, indicating a good to excellent rock quality. The Total Core Recovery (TCR) was 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)
BH13-8	268.1	262.7	72
		260.4	107
BH13-9	266.7	263.2	159
		260.7	146

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

4.3 CHEMICAL ANALYSIS

One representative sample retrieved from the clayey silt layer in BH13-8 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)
BH13-8	SS-5	3.05 to 3.66	7.4	23	25	60

4.4 GROUNDWATER

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)
BH13-8	268.1	2.1	266.0
BH13-9	266.7	1.0	265.7

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, 27 km. The new route includes a four-lane divided highway and will by-pass the towns of Chelmsford and Dowling.

The preferred alignment includes 11 structure sites, including the Gordon Lake Road Underpass site.

Proposed Underpass Structure

The proposed Gordon Lake Road Underpass will direct Gordon Lake Road over the new Highway 144. Gordon Lake Road has a two-lane rural road cross-section with one lane in each direction. The ultimate configuration of the new Highway 144 will include a four-lane divided highway.

The preliminary General Arrangement (GA) drawing indicates that the proposed underpass will have two spans with the centre pier within the median of the new Highway 144 and two integral abutments north and south of the new alignment supported on piles.

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

Proposed Underside of Pile Cap Elevation (North Abutment):	269.2 m
Proposed Underside of Pile Cap Elevation (South Abutment):	269.2 m
Proposed Final Grade (Top of Gordon Lake Road) at North Abutment:	274.9 m
Proposed Final Grade (Top of Gordon Lake Road) at South Abutment:	275.0 m
Proposed Final Grade of Highway 144:	267.3 m
Existing Ground Elevation at North Abutment:	267.7 m
Existing Grade Elevation at South Abutment:	268.8 m
Proposed Elevation of Highway 144 Median:	266.9 m
Proposed Underside of Footing (Centre Pier):	264.6 m

5.1 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered at this site generally consist of a thin layer of topsoil overlying fill overlying a clayey silt layer overlying metamorphosed sedimentary bedrock. The native soils at the site are generally firm to stiff. Bedrock was encountered at 4.5 m depth in BH13-8 and 3.2 m depth in BH13-9 below existing ground surface, corresponding to an approximate elevation of 263.6 m. The RQD of the bedrock ranged between 86% and 100%, indicating a good to excellent rock quality. The unconfined compressive strength ranged between 72 MPa and 159 MPa (strong to 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, pocket penetrometer measurements, and laboratory index test results (including moisture contents) of samples retrieved from the site. This profile is included in Figure 4 of Appendix D and was developed based on the information obtained from boreholes BH13-8 and BH13-9; however, the fill materials associated with the existing roadway were not included in the interpretaion.

Table 5.1: Preliminary Subsurface Profile at Proposed Underpass

Elevation (m)		Soil Type	Design Parameters				
From	To		γ (kN/m^3)	ϕ (°)	S_u (kPa)	USC (MPa)	E (MPa)
varies	265.5	Clayey silt (very stiff to stiff)	20.5	-	75	-	15
265.5	263.6	Clayey silt (firm)	20	-	45	-	10
< 263.6		Metasedimentary bedrock (good to excellent quality, strong to very strong bedrock)	24	-	-	72 to 159	22,000

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 266.0 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 an approximately 3.2 to 4.5 m thick layer of very stiff to firm clayey silt layer over a good to excellent quality bedrock; the clayey silt strength decreases with depth. 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 Sudbury, Ontario, which is approximately 17 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 soils under seismic loading conditions was assessed. The assessment indicated that liquefaction of the site soils is not of a concern due to:

(a) A very low ZAR,
(b) Shallow bedrock (less than 5 m deep), and
(c) Relatively high fraction of fines content within the shallow soils.

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

5.4 FOUNDATION OPTIONS

Tables 5.2a and 5.2b compare the foundation options from a foundation design and constructability perspective.

Table 5.2a: Comparison of Foundation Options for Gordon Lake Road Underpass (Abutments)

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences	Rank*
Shallow foundation on soil	▪ Limited excavation involved ▪ Generally suitable to support bridge piers	▪ May necessitate large footing area ▪ Not suitable for integral abutment bridge construction ▪ Clayey silt is easily disturbed, excavation and removal of unsuitable soil is required	Low to Medium	▪ Potential differential settlement	4
Shallow foundation on bedrock	▪ High geotechnical resistance ▪ Reduces risk of settlement	▪ Requires substantial excavation ▪ Not suitable for integral abutment bridge construction	Medium	▪ Excavation below groundwater level	3
Piles End bearing on or socketed into bedrock	▪ Reduces risk of differential settlement ▪ Suitable for integral abutment bridge	▪ Cobbles encountered in the fill may require pre-augering ▪ May not be practical for shallow bedrock	High	▪ Possible pile damage during installation; pre-drilling of bedrock for socketing the piles may be required	1
Drilled Caissons	▪ Can transmit very large axial and lateral loads ▪ Generally suitable if bedrock is relatively shallow	▪ Not suitable for integral bridge abutment	Medium to High	▪ Risk of cave-in, especially below groundwater table during drilling	2

*Based on qualitative assessment only.

Table 5.3b: Comparison of Foundation Options for Gordon Lake Road Underpass (Piers)

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences	Rank*
Shallow foundation on soil	▪ Limited excavation involved ▪ Generally suitable to support bridge piers	▪ May necessitate large footing area ▪ Not suitable for integral abutment bridge construction ▪ Clayey silt is easily disturbed, excavation and removal of unsuitable soil is required	Low to Medium	▪ Potential differential settlement	4
Shallow foundation on bedrock	▪ High geotechnical resistance ▪ Reduces risk of settlement	▪ Requires substantial excavation ▪ Not suitable for integral abutment bridge construction	Medium	▪ Excavation below groundwater level	1
Piles End bearing on or socketed into bedrock	▪ Reduces risk of differential settlement ▪ Suitable for integral abutment bridge	▪ Cobbles encountered in the fill may require pre-augering ▪ May not be practical for shallow bedrock	High	▪ Possible pile damage during installation; pre-drilling of bedrock for socketing the piles may be required	3
Drilled Caissons	▪ Can transmit very large axial and lateral loads ▪ Generally suitable if bedrock is relatively shallow	▪ Not suitable for integral bridge abutment	Medium to High	▪ Risk of cave-in, especially below groundwater table during drilling	2

*Based on qualitative assessment only.

Based on the comparison presented in Tables 5.2a and 5.2b above, the following foundation options are recommended:

- For the proposed integral abutments: Flexible piles consisting of H-piles socketed into bedrock.
- For the piers: Shallow foundation on bedrock.

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 Abutment Foundations – Driven Piles

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

5.5.1.1 Geotechnical Axial Resistance

Anticipated 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 integral abutments to be located north and south of Highway 144. Based on the preliminary GA plan drawing, the underside of the pile caps (bottom of concrete abutments) will be at an approximate elevation of 269.2 m.

To provide the desired integral action, 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 will be 5.6 m.

A factored axial resistance in compression at ULS_f for an HP310x110 pile of 2,000 kN may be used for this site. This resistance at ULS_f assumes that the piles are socketed into 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 8.5 m high approach embankment fill to raise the final grade of the proposed Gordon Lake Road. 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 120 mm.

The thickness of the potentially compressible clayey silt layer between the bottom of the pile cap (elevation 269 m) and the top surface of the bedrock (elevation 263.6 m) is approximately 3.4 m (elevation 267 to 263.6 m). A 3 m long CSP filled with loose uniform sand will be installed between elevations 269 m and 266 m. The remaining thickness of the compressible layer between the underside of the CSP and the bottom of the potentially compressible layer is 2.4 m. The potential downdrag load was estimated over this compressible layer. The maximum unfactored downdrag load was estimated to be 120 kN. 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.

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 in the boreholes consisted of fill with occasional cobbles overlying a loose to compact silt; no fill is anticipated at the proposed structure location. No obstructions to pile driving are anticipated. However, this should be confirmed during the Detailed Design.

Piles should have reinforced OSLO Point tips according to Ontario Provincial Standard detail, OPSD 3000.201.

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 firm to stiff cohesive material.

<u>ULS Resistance</u>

The passive earth pressure for the pile driven though a loose uniform sand in CSP and 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 earth resistance by using a bearing width equal to the flange width of HP310x110. 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 ULSf was 130 KN.

<u>SLS Resistance</u>

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.

Table 5.4: 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/m³	Degrees	kPa	kN/m³	-
Loose to compact sand[1]	269.0	266.0	20	33	-	5,400	-
Clayey silt	266.0	263.6	21[2]	-	40		0.005
Bedrock	< 263.6		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 266.0 m.

Two plots from LPILE are presented in Figures 5 and 6 of Appendix D. Figure 5 shows the deformed shape of the pile for lateral (shear) force ranging between 50 and 100 kN. 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 the pile undergoes negligible lateral deflection below a depth of approximately 5 m from the underside of the pile cap (at approximate elevation of 264 m).

Figure 5 in Appendix D illustrates the displacement of the pile in depth for different lateral loads. Based on Figure 5, a lateral load of 85 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 85 kN.

Figure 6 in Appendix D presents the p-y plot that gives the non-linear response of the pile-soil interaction. It provides a series of curves obtained from LPILE and generated for selected depths 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 5.5: 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	-	-

5.5.2 Piers - Shallow Foundation

This section provides recommendations for the design of spread footings founded on bedrock. As indicated in Section 5.4, shallow foundations are recommended for the piers.

5.5.2.1 Geotechnical Vertical Resistance

The geotechnical resistances provided in Table 5.5 may be used in the design, provided the footings are placed on sound bedrock.

Table 5.6: Geotechnical Resistance for Shallow Foundation (Spread Footing)

Founding Element	Founding Elevation (m)	Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa)
Spread footing on unweathered bedrock	± 263.6	1 to 4	8000	N/A

Note: the above ULS_f values were calculated based on estimated Rock Mass Rating (RMR) of 64 resulting in an equivalent rock mass uniaxial compressive strength of 6 MPa.

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).

The axial reaction at SLS is not applicable for footings on competent bedrock.

5.5.2.2 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
0.65 between clean sound bedrock and cast-in-place concrete

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.

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

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II	Native Clayey Silt
Bulk Unit Weight, γ (kN/m3)	21.2	22.0	21
Effective Friction Angle (º)	32	35	26
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43	0.56
Coefficient of Active Earth Pressure (K_a)	0.31	0.27	0.39
Coefficient of Passive Earth Pressure (K_p)	3.2	3.7	2.6

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:

$$P_{AE} = ½ K_{AE} \gamma H^2 (1 - k_V)$$
$$P_{PE} = ½ K_{PE} \gamma H^2 (1 - k_V)$$

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 KAE and KPE 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 5.8: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B Type I		OPSS Granular A and Granular B Type II		Native Clayey Silt	
Bulk Unit Weight, γ (kN/m^3)	21.2		22.0		21	
Effective Friction Angle (º)	32		35		26	
Wall Type	Yielding	Non-yielding	Yielding	Non-yielding	Yielding	Non-yielding
Active Earth Pressure (K_{AE})	0.32	0.35	0.28	0.31	0.41	0.44
Height of Application of P_{AE} from base as a ratio of wall height, (H)	0.341	0.356	0.342	0.358	0.340	0.353
Passive Earth Pressure, (K_{PE})	3.21	-	3.64	-	2.52	-
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.325	-	0.325	-	0.324	-

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

Based on the preliminary GA drawing, the expected maximum embankment height at the proposed interchange is approximately 8.0 m near the abutment.

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 8.0 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 7a through 7c in Appendix D.

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

5.7.3 Embankment Settlement

It is noted that the proposed embankment geometry (height, top width, and side slope) has not yet been established. 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:

- For preliminary analysis purposes, a simplified soil profile at the site of an approximately 3 m thick firm clayey silt layer overlying unweathered bedrock 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;
- Consolidation and creep settlement of the clayey silt soil will be considered;
- The clayey silt soil is assumed to be overconsolidated with an estimated overconsolidation ratio of 6.
- Groundwater is assumed at 1 m below existing ground surface (at the bottom of the existing fill);
- The maximum embankment height will be approximately 8 m (in the immediate vicinity of the north abutment);
- The approach embankment will have a 5% longitudinal slope and 2V:1H side slopes;
- The embankment extends approximately 190 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 70 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 typical Settle3D preliminary analysis is presented in Figure 8 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 120 mm, under an 8 m high embankment. The maximum settlement will take place approximately 25 m back from each bridge abutment. The estimated settlement at the abutments is 75 mm.

Assuming 0.5% strain under self-weight, the estimated embankment self-weight settlement is approximately 40 mm. This settlement is anticipated to be completed by the end of embankment construction.

5.7.4 Settlement Mitigation

The above estimated settlement will take place over a period of time, given that the majority of the settlement (approximately 95%) will be due to consolidation of the clayey silt layer. Based on an initial estimate, approximately 50% of the consolidation settlement of the clayey silt layer is expected to occur within approximately 150 days (22 weeks), while 90% of the settlement is expected to occur within approximately 660 days (1.8 years).

In order to minimize the potential impacts of post-construction settlements, it is recommended that the embankments be constructed up to two years ahead of the construction of the structure. This should be considered at the time of the final design. The construction constraints identified at the time of the final design may necessitate the use of methods to accelerate settlements, such as the use of wick drains.

5.8 PRELIMINARY CONSTRUCTION CONSIDERATIONS

5.8.1 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 is 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 geotechnical investigation within both boreholes included granular fill over predominantly firm clayey silt overlying bedrock at depths of 3.2 m and 4.5 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).

The founding level for the center pier is expected to be approximately 5.0 m below existing grade, but only about 2.0 m below final grade. Should the foundation be constructed prior to general site grading, the contractor may choose to use a temporary support system for this work.

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.

5.8.2 Unwatering

Groundwater was encountered at an elevation of approximately 266.0 m, which is approximately 2.4 m above the bedrock surface elevation of 263.6 m (equal to the anticipated footing elevation of the center pier). 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 low to moderate hydraulic conductivity. Unwatering of the structure excavation using conventional sump and pump techniques should be adequate.

5.8.3 Reuse of Excavated Material

The native material at the site includes clayey silt. This material will not be suitable for use as backfill within and behind the structures for the proposed structures and embankments.

5.9 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 25 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 7.4, 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.

5.10 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; due to the foundation elements being within a heavily treed area within a private property, the current boreholes are up to 45 m west of the preferred alignment. 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 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 3000.201	Foundation Piles Steel HP 310 Oslo Point
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

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.

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 Bridgit Bocage, Geotechnical Engineering Intern, 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 Stantec.

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 Simon Gudina, and reviewed by Chris McGrath and Raymond Haché, 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,

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APPENDIX A

Drawing No. 1 – Borehole Location Plan and Soil Strata

Drawing No. 2 – Preferred Route

Site Photographs

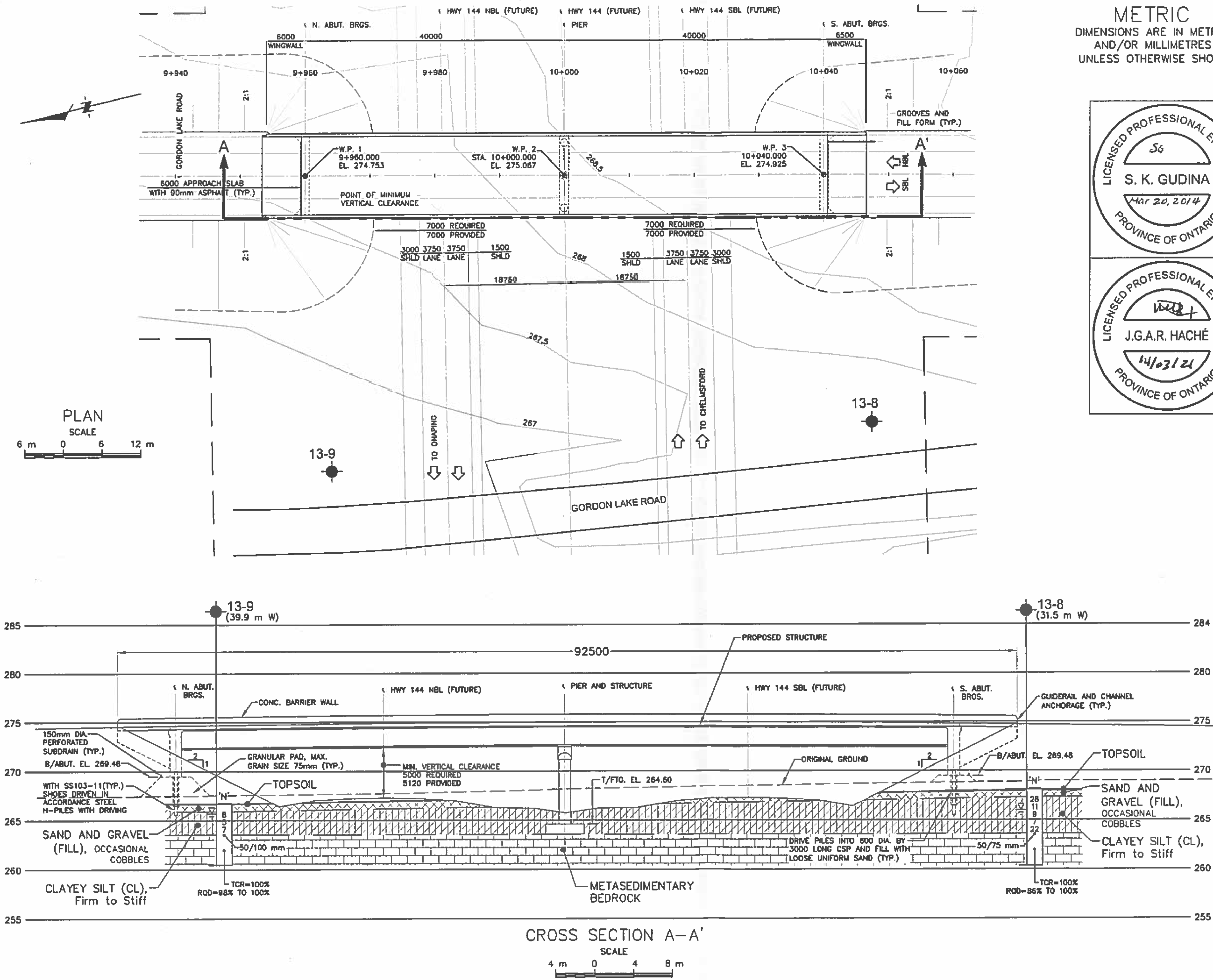

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PLATE No
CONT
WP 5023-09-00

HWY 144 ROUTE SELECTION STUDY
GORDON LAKE ROAD UNDERPASS
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Stantec

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 W) Offset in meters West of Cross Section Line A-A'

No	ELEVATION	MTM ZONE 12 COORDINATES NORTH	EAST
13-8	268.1	5 159 729.6	281 189.9
13-9	266.7	5 159 812.2	281 202.3

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 41I-302

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

DRAWING NAME: 165000734_S4_Gordon Lake_GA_2013-11-11
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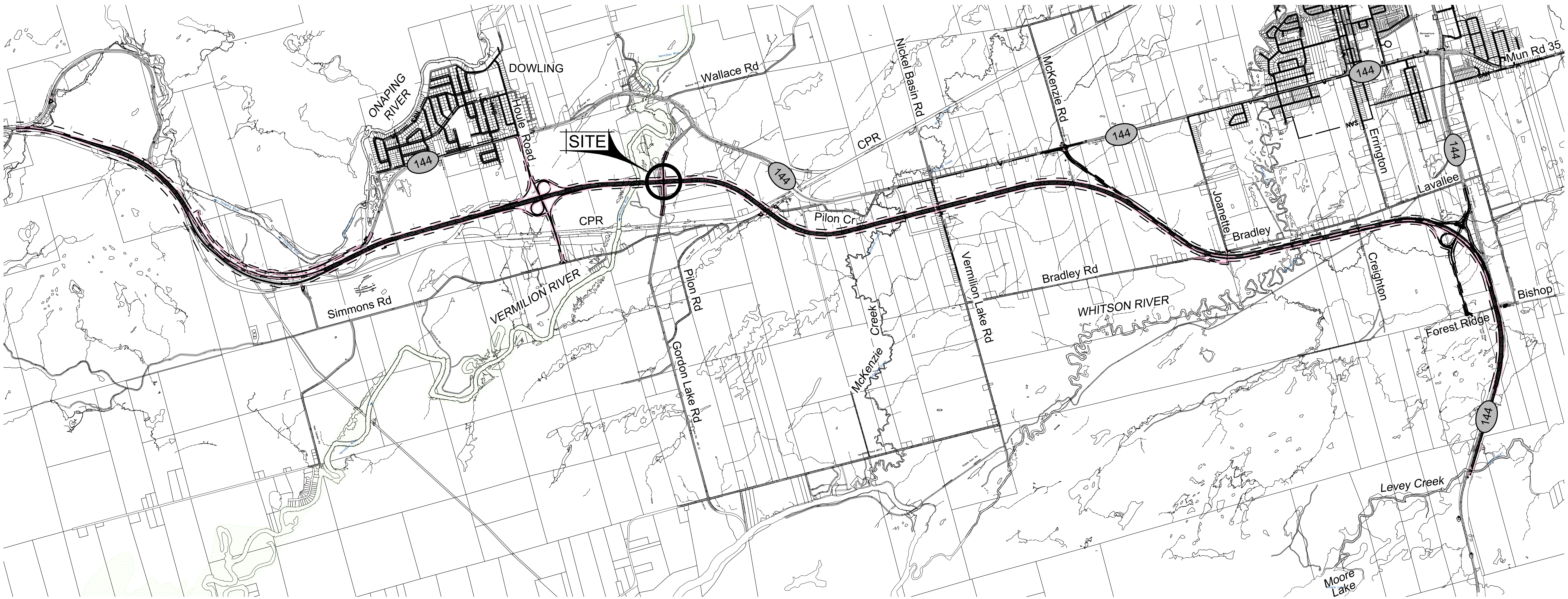
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No
CONT
WP 5023–09–00

HWY 144 ROUTE SELECTION STUDY
GORDON LAKE ROAD UNDERPASS
PREFERRED ROUTE

SHEET

Stantec



PLAN

SCALE

500 m 0 500 1000 m

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 411–302

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

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

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: Feb 26, 2013

Site Photo No.: 1	Looking east near BH13-9 at proposed Gordon Lake Road Underpass site

Site Photo No.: 2	Looking west near BH13-9

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: Jan 22, 2013

Site Photo No.: 3	Looking north near BH13-9

Site Photo No.: 4	Looking south towards BH13-8

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:

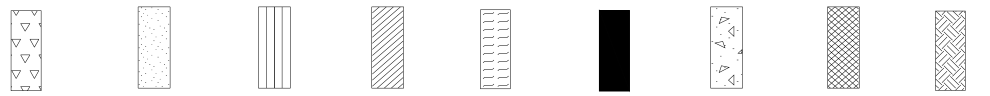
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

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.



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
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

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RECORD OF BOREHOLE No BH13-8

METRIC

W.P. GWP 5023-09-00 LOCATION Hwy 144, Gordon Lake Road, Greater Sudbury, ON N: 5 159 730 E: 281 190 ORIGINATED BY BB

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

DATUM Geodetic DATE 2013 02 26 - 2013 02 26 CHECKED BY CM/SG

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
268.1	Topsoil
268.0 / 0.0	150 mm TOPSOIL
0.2	FILL: Sand with gravel (SM), brown
267.3 / 0.8	Clayey SILT (CL) Firm to stiff Grey, moist to wet - oxidized seams from 0.8 to 1.4 m and 2.3 to 2.9 m - Groundwater inferred at a depth of 2.1 m (elevation 265.2 m)
263.6 / 4.5	Metasedimentary BEDROCK: Interbedded mudstone, lithic wacke, and siltstone - good to excellent quality - strong to very strong - dark grey to black colour - unweathered - two joint sets with very close to wide spacing (Refer to Field Bedrock Core Log)
260.3 / 7.8	End of Borehole

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	BS	-	8 44 (48)
2	SS	28	
3	SS	11	
4	SS	9	
5	SS	7	PP = 55 kPa
6	SS	22	0 2 77 21 PP = 25 kPa
7	SS	50/75mm	
8	NQ	-	TCR = 100% RQD = 100% UCS = 72 MPa
9	NQ	-	TCR = 100% RQD = 96%
10	NQ	-	TCR = 100% RQD = 86% UCS = 107 MPa

GROUND WATER CONDITIONS: water level symbol at elevation approx. 265.2 m

ELEVATION SCALE: 268, 267, 266, 265, 264, 263, 262, 261

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

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

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RECORD OF BOREHOLE No BH13-9

METRIC

W.P. GWP 5023-09-00	LOCATION Hwy 144, Gordon Lake Road, Greater Sudbury, ON N: 5 159 812 E: 281 202	ORIGINATED BY BB
DIST HWY 144	BOREHOLE TYPE 8" & Hollow Stem Augers, Splitspoon Sampler, NQ Rock Core	COMPILED BY SH
DATUM Geodetic	DATE 2013 02 26 - 2013 02 26	CHECKED BY CM/SG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
266.7	Topsoil							
266.6 / 0.1	50 mm TOPSOIL							
	FILL: Sand and gravel, brown - occasional cobbles	1	BS	-				
265.9 / 0.8	Clayey SILT (CL) Firm Grey to brownish-grey, moist - Groundwater inferred at a depth of 1.0 m (elevation 264.9 m)	2	SS	8	∇	266		
		3	SS	5		265		0 3 71 25
		4	SS	7		264		PP = 40 kPa
263.5	- Splitspoon Refusal on Bedrock	5	SS	50/ 100mm				
3.2	Metasedimentary BEDROCK: Interbedded mudstone, lithic wacke, and siltstone - excellent quality - very strong - dark grey to black colour - unweathered - one joint set with close to wide spacing (Refer to Field Bedrock Core Log)	6	NQ	-		263		TCR = 100% RQD = 100% UCS = 159 MPa
		7	NQ	-		262 261		TCR = 100% RQD = 98% UCS = 146 MPa
260.5 / 6.2	End of Borehole							

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; 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

Field Bedrock Core Log

Stantec

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

Project No.: 165000734
Date: February 26, 2013
Borehole No.: BH13-8
Logger: SH

DEPTH FROM	RUN NO.	% CORE RECOVERY	% RQD	DEPTH TO	GENERAL DESCRIPTION (Rock Type/s, %, Colour, Texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES: NO. OF SETS	TYPE/S	ORIENTATION	SPACING	ROUGHNESS	APERTURE	FILLING	OCCASIONAL FEATURES	DRILLING OBSERVATIONS
4.57	NQ8	100%	100%	5.54	Metasedimentary bedrock: Interbedded dark grey to black mudstone, lithic wacke, and siltstone (Chelmsford Formation)	S	U	2	J	D	VC-C	RP	-	T	-	Mohs Hardness: H=3-5-5
									J	V	-	RP	-	T		
5.54	NQ9	100%	96%	7.06	Metasedimentary bedrock: Interbedded dark grey to black mudstone, lithic wacke, and siltstone (Chelmsford Formation)		U	2	J	D	C-W	RP	-	T	-	Mohs Hardness: H=3-5-5
									J	V	-	RP	-	T		
7.06	NQ10	100%	86%	7.77	Metasedimentary bedrock: Interbedded dark grey to black mudstone, lithic wacke, and siltstone (Chelmsford Formation)	VS	U	2	J	V	-	RP	-	T	-	Mohs Hardness: H=3-5-5
									J	V	-	RP	-	T		

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: Landcore Drilling

Project No.: 165000734
Date: February 26, 2013
Borehole No.: BH13-9
Logger: SH

DEPTH FROM	RUN NO.	% CORE RECOVERY	% RQD	DEPTH TO	GENERAL DESCRIPTION (Rock Type/s, %, Colour, Texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES: NO. OF SETS	TYPE/S	ORIENTATION	SPACING	ROUGHNESS	APERTURE	FILLING	OCCASIONAL FEATURES	DRILLING OBSERVATIONS
3.15	NQ6	100%	100%	4.75	Metasedimentary bedrock: Interbedded dark grey to black mudstone, lithic wacke, and siltstone (Chelmsford Formation)	VS	U	1	J	D	C-M	RP	-	T	-	Mohs Hardness: H=3-5.5
4.75	NQ7	100%	98%	6.22	Metasedimentary bedrock: Interbedded dark grey to black mudstone, lithic wacke, and siltstone (Chelmsford Formation)	VS	U	1	J	F	M-W	RP	-	T	-	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

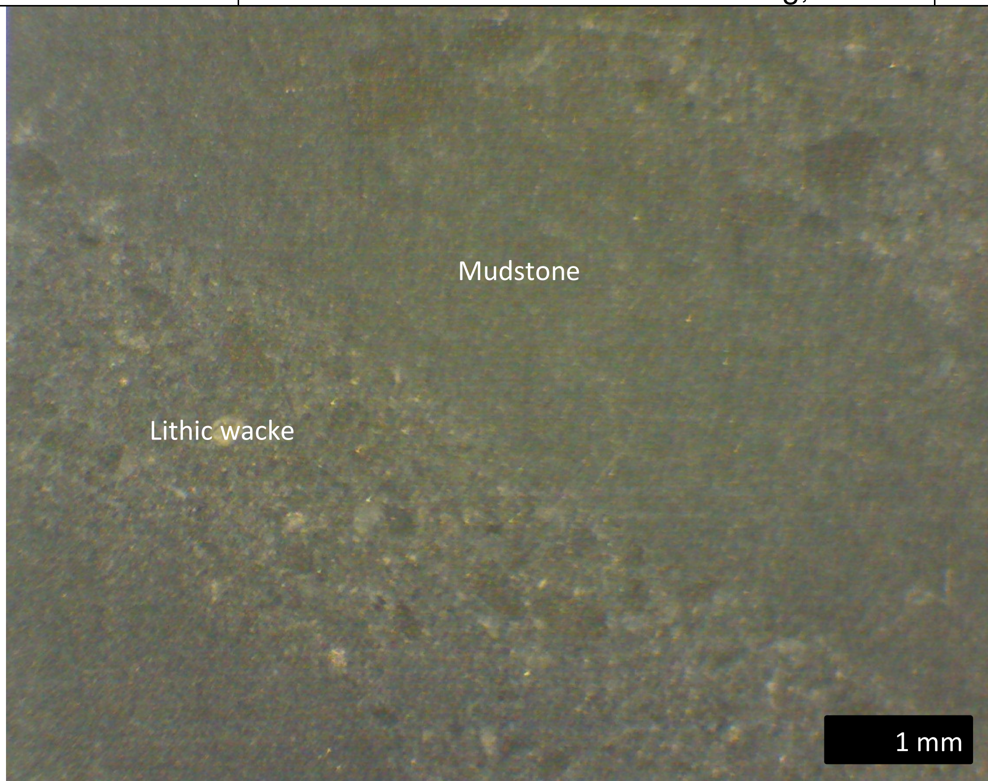
FILLING
T = Tight, Hard
O = Oxidized
SA = Slightly Altered, Clay Free
S = Sandy, Clay Free
Si = Sandy, Silty, Minor Clay
NC = Non-softening Clay
SC = Swelling, Soft Clay

Stantec	Project No.: 165000734	GWP: 5023-09-00	Rockcore Photographs
	Project Name: Highway 144 Route Planning and Preliminary Design Study, Chelmsford to Dowling, ON		Date: March 4, 2013

Rock Core Photo No.: 1	Borehole: BH13-8	Depth: 4.57 to 7.77 m

Rock Core Photo No.: 2	Borehole: BH13-9	Depth: 3.15 to 6.22 m

	Project No.: 165000734	GWP: 5023-09-00	Rock Core Microscopic Photographs
	Project Name: Highway 144 Route Planning and Preliminary Design Study, Chelmsford to Dowling, ON		



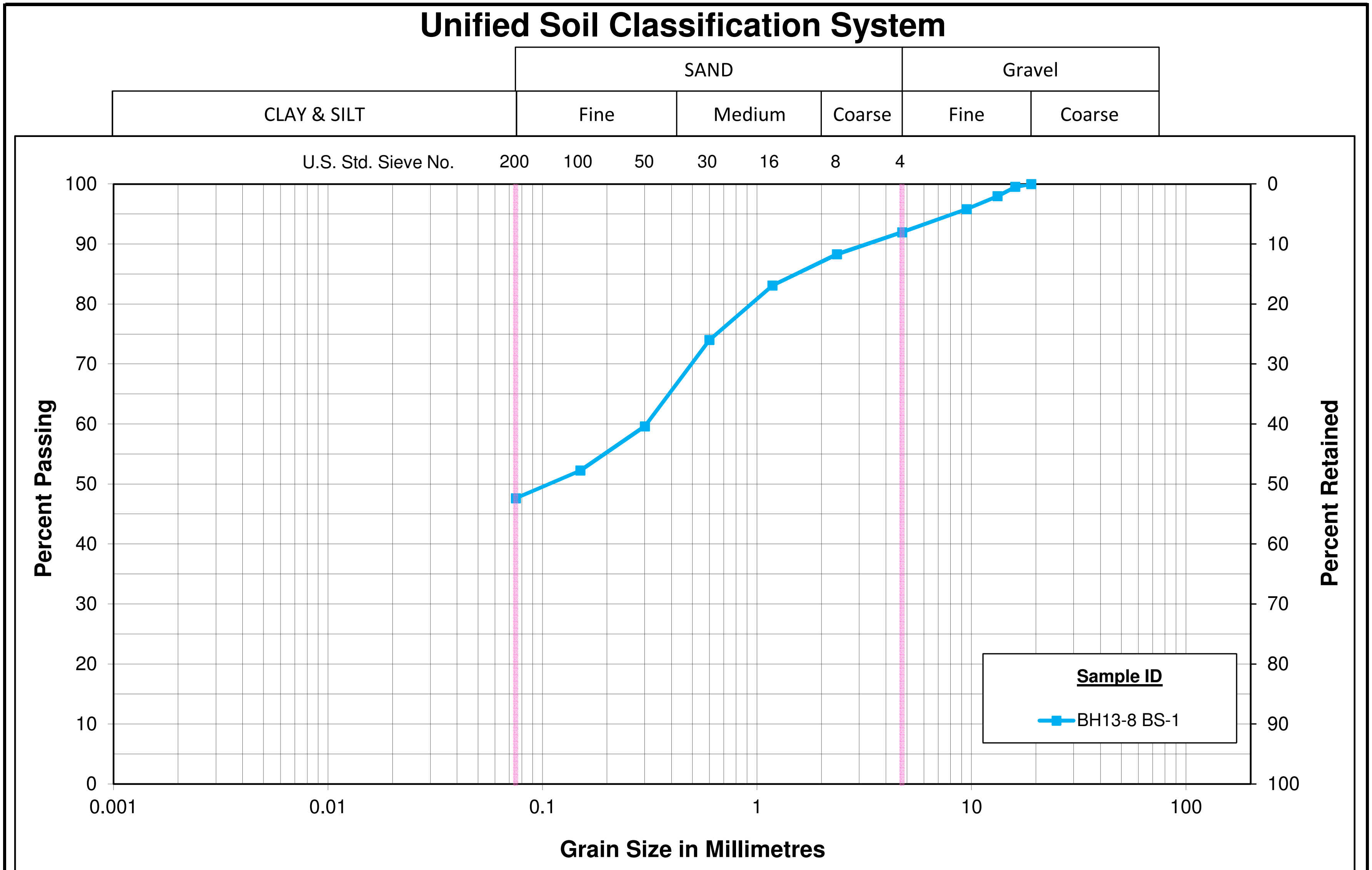
Rock Core Photo No.: 3	Borehole: BH13-9	Depth: 4.04 m

APPENDIX C

Laboratory Test Results

Figures 1 and 2: Grain Size Distribution Plots

Figure 3: Plasticity Chart

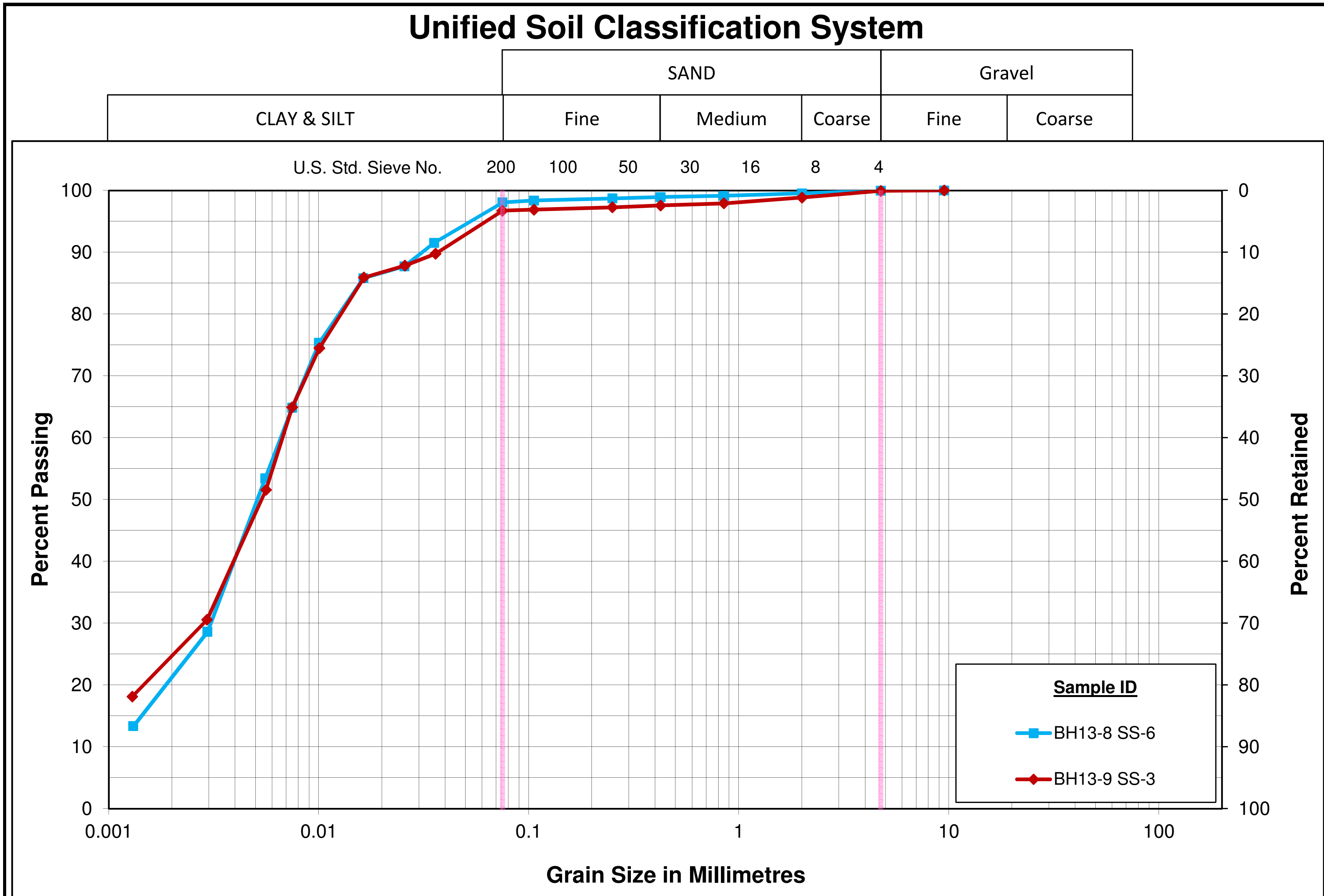


GRAIN SIZE DISTRIBUTION

Fill: Silty sand (SM)

Figure No. 1

Project No. 165000734
GWP 5023-09-00



GRAIN SIZE DISTRIBUTION

Clayey silt (CL)

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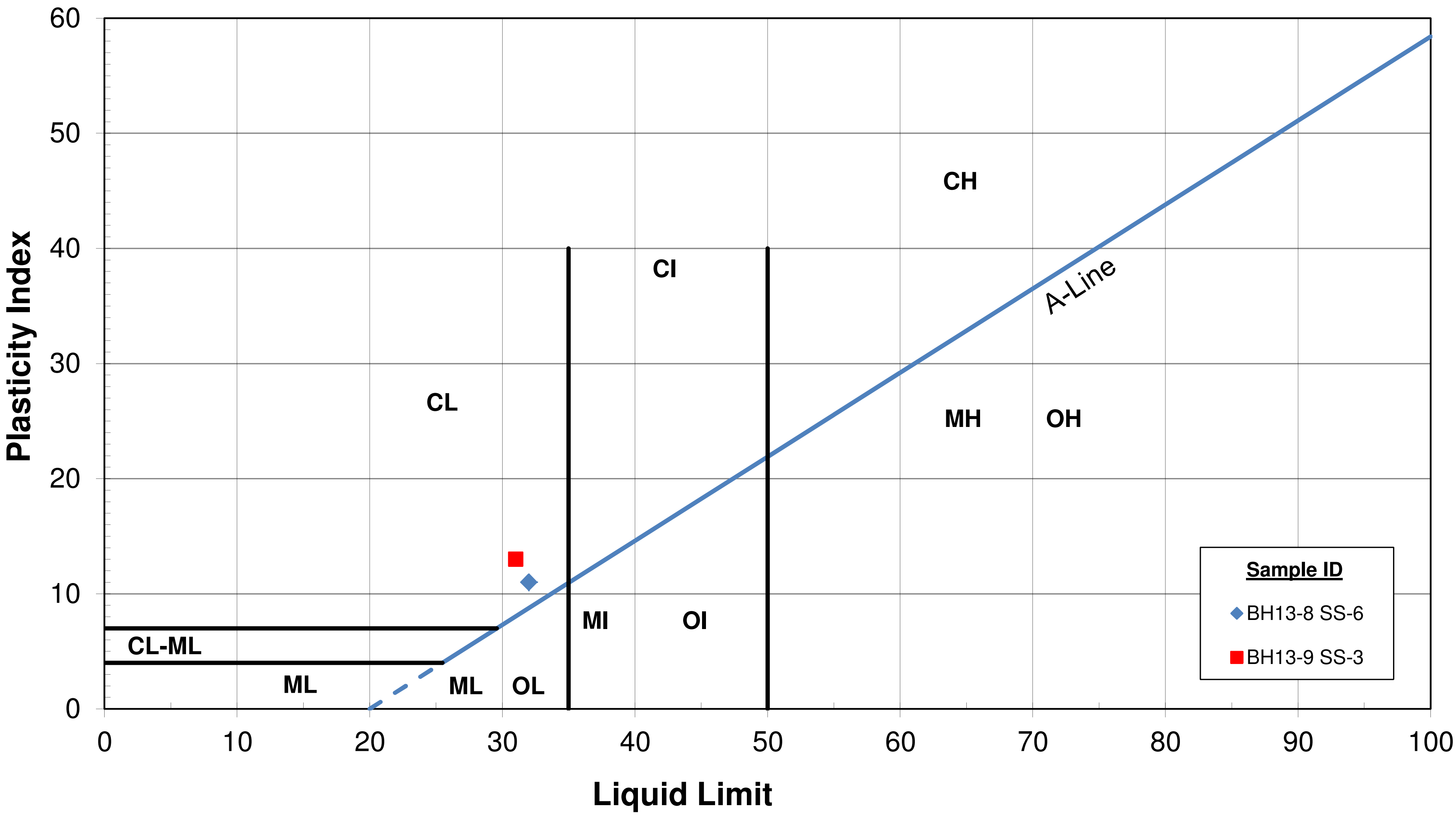
Figure No. 2

Project No. 165000734

GWP 5023-09-00



Stantec	PLASTICITY CHART	Figure No. 3
		Project No. 165000734 GWP 5023-09-00

APPENDIX D

Figure 4: Preliminary Design Parameters

Preliminary LPILE Analysis Results

Figure 5: Lateral Deflection of HP310x110

Figure 6: P-y Curves for HP310x310

Preliminary Slope Stability Results

Figure 7a: Static (long-term)

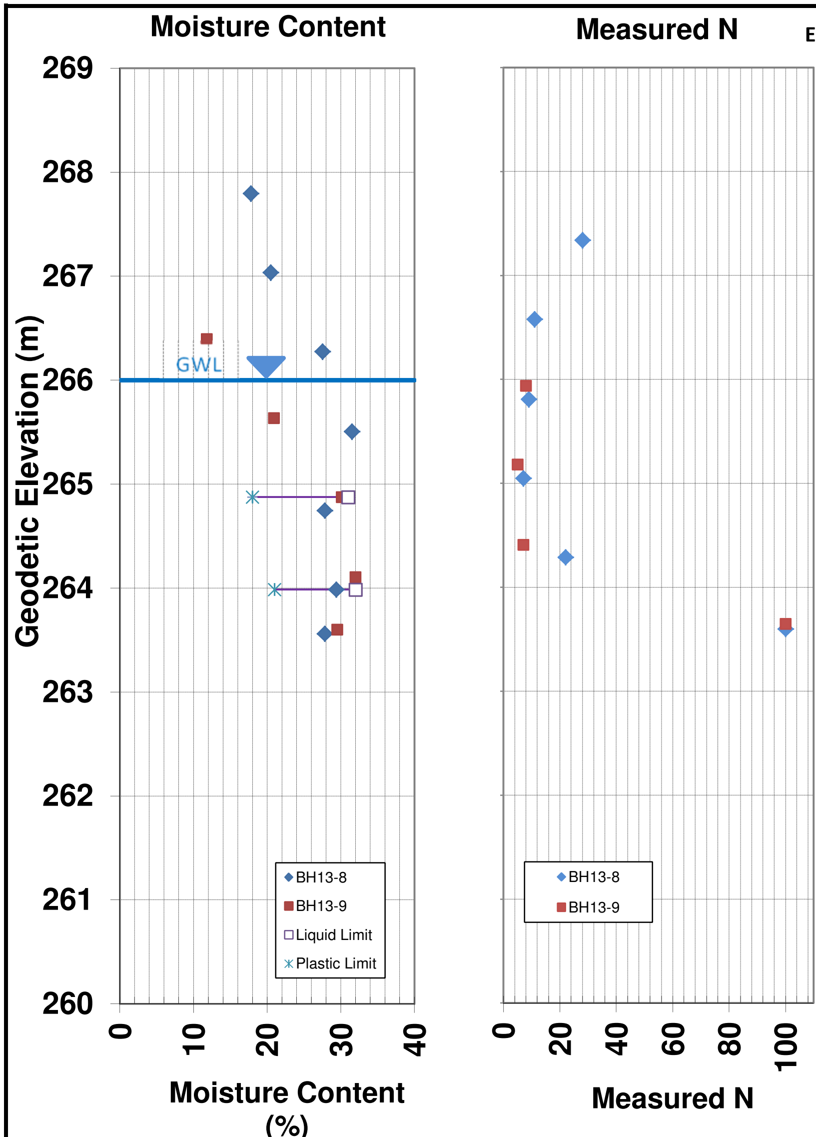
Figure 7b: Static (short-term)

Figure 7c: Seismic

Preliminary Settlement Analysis

Figure 8: Preliminary Settlement Results

Table D-1: Spring Stiffnesses for HP310x110



Elevation (m)

Preliminary Design Parameters

Existing ground surface elevation at:
South abutment = 268.8 m
North abutment = 266.7 m

268.1

Clayey Silt (very stiff to stiff)

γ = 20.5 kN/m³
S_u = 75 kPa
E = 15 MPa

266.5

Clayey Silt (firm)

γ = 20 kN/m³
S_u = 45 kPa
E = 10 MPa

263.6

Bedrock
Good to excellent quality
little to no weathering
strong to very strong

γ = 24 kN/m³
UCS = 6000 kPa
E = 22 GPa

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Stantec Consulting Ltd

Project No. 16500734 Gordon Lake Road Underpass
GWP No. 5023-09-00

Figure 4

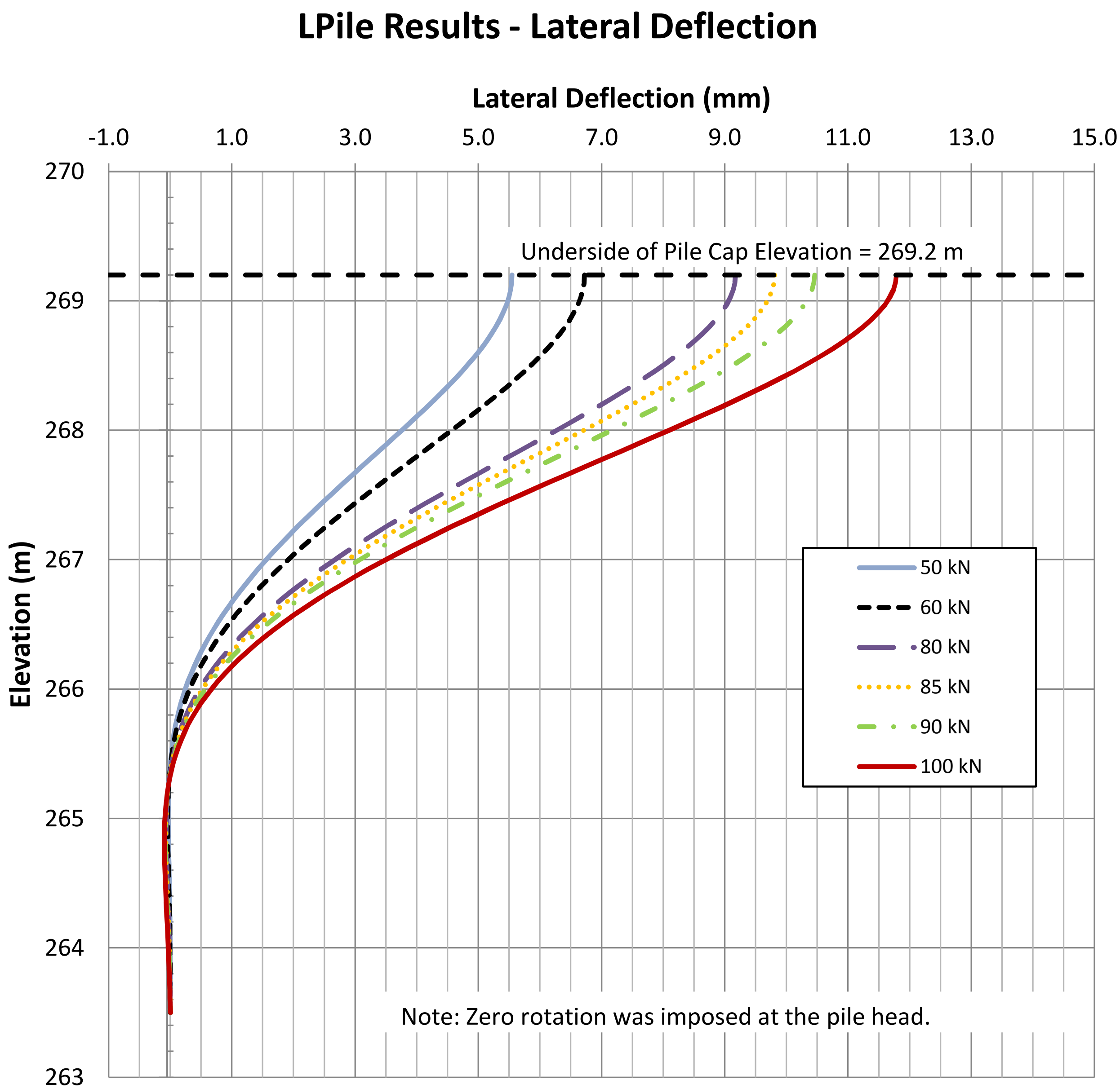


Figure 5
Lateral Deflection of HP 310x110 Piles

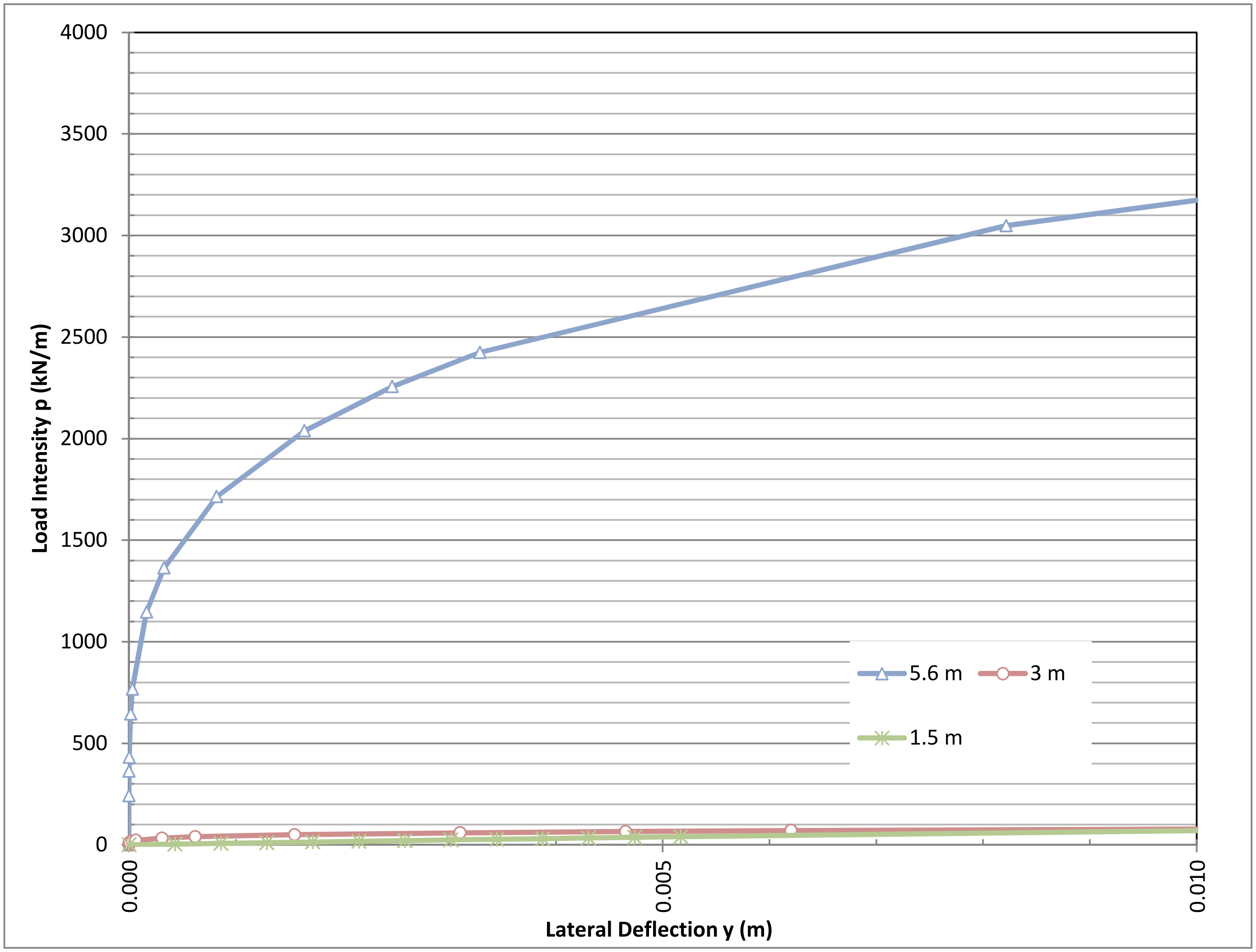
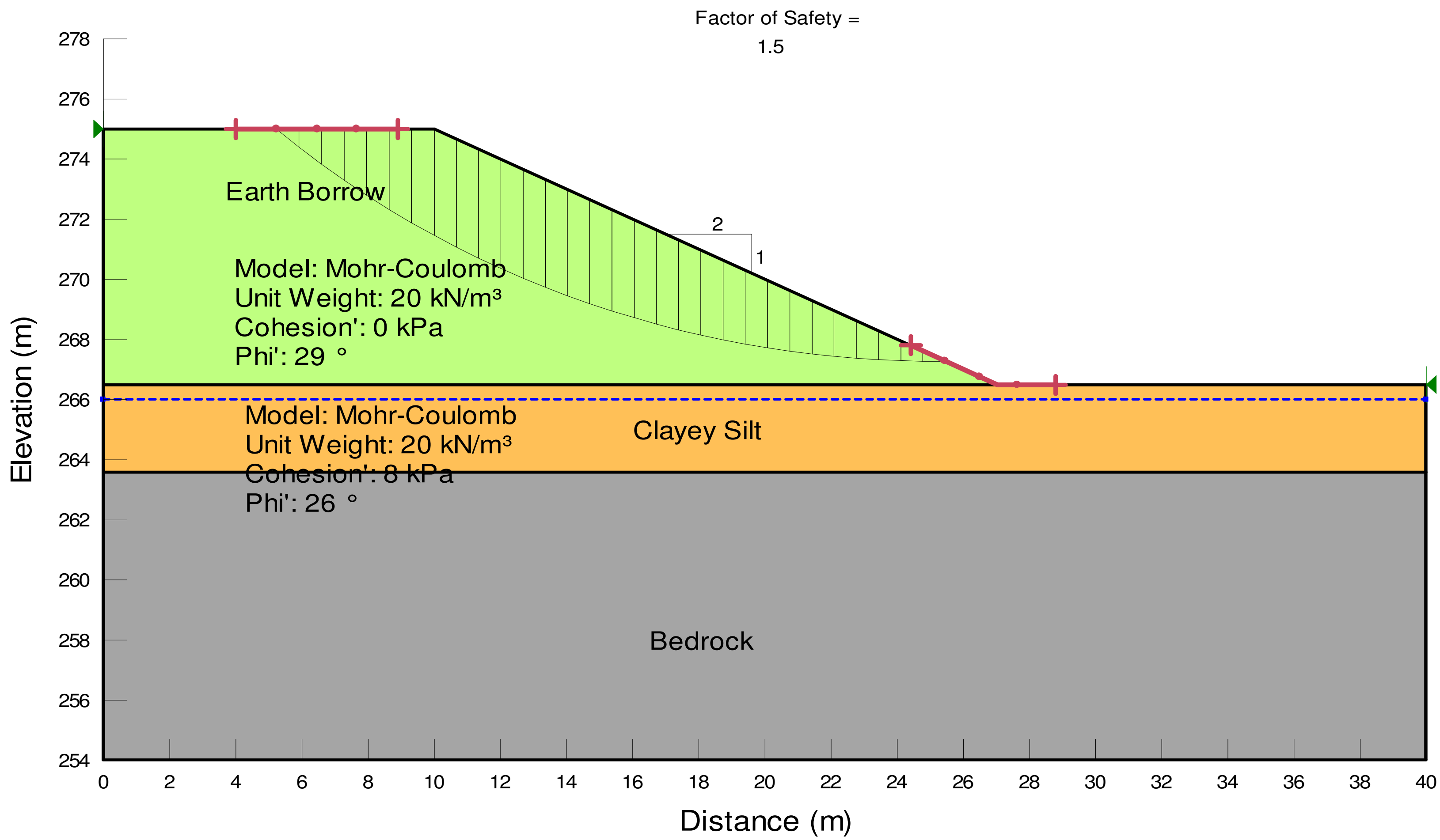
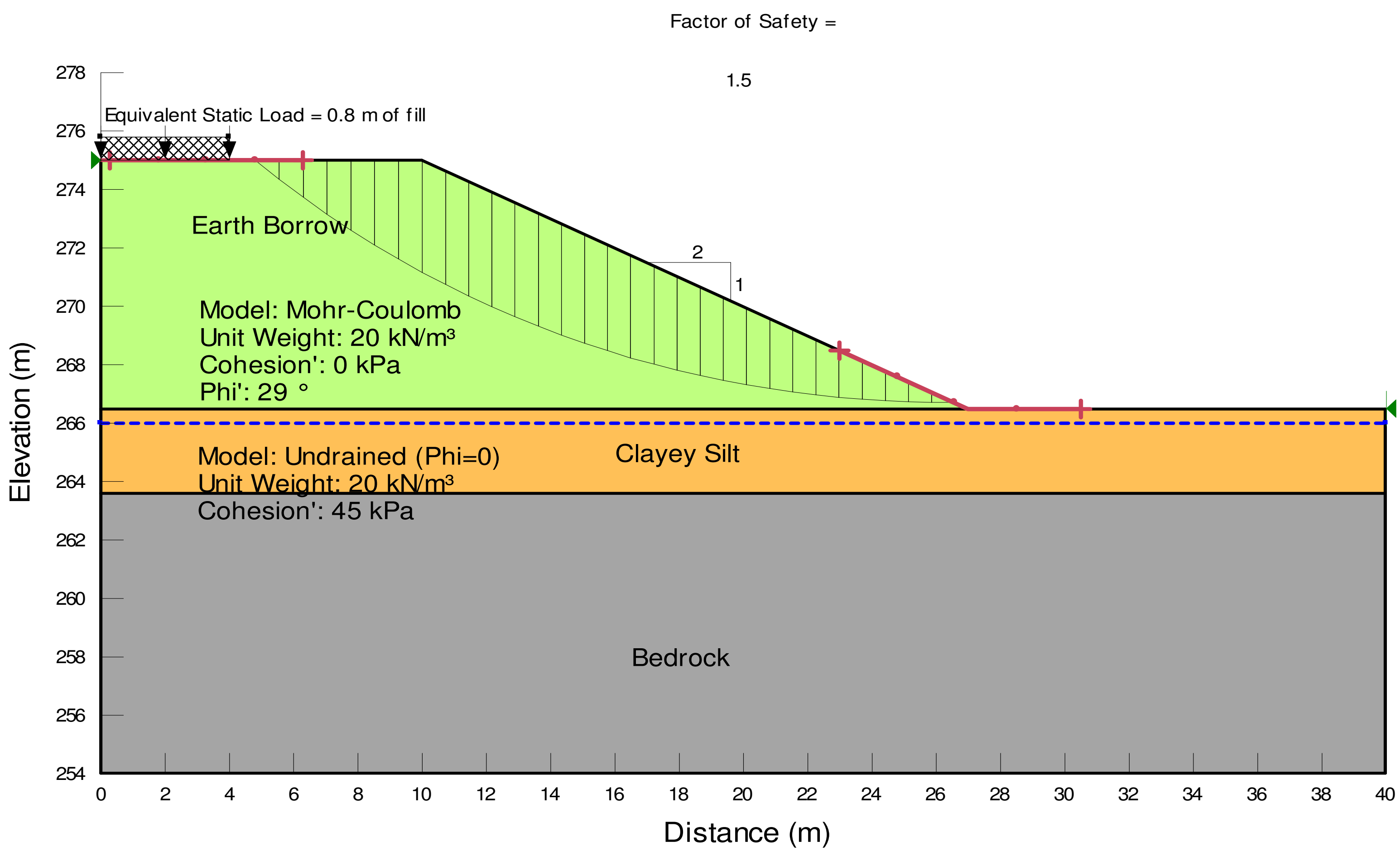


Figure 6
p-y Curves for Proposed HP 310x110 Piles



Stantec	Static Slope Stability Analysis (Drained) Highway 144 Chelmsford Bypass Gordon Lake Road Underpass	Figure 7a Project No. 165000734 GWP No. 5023-09-00



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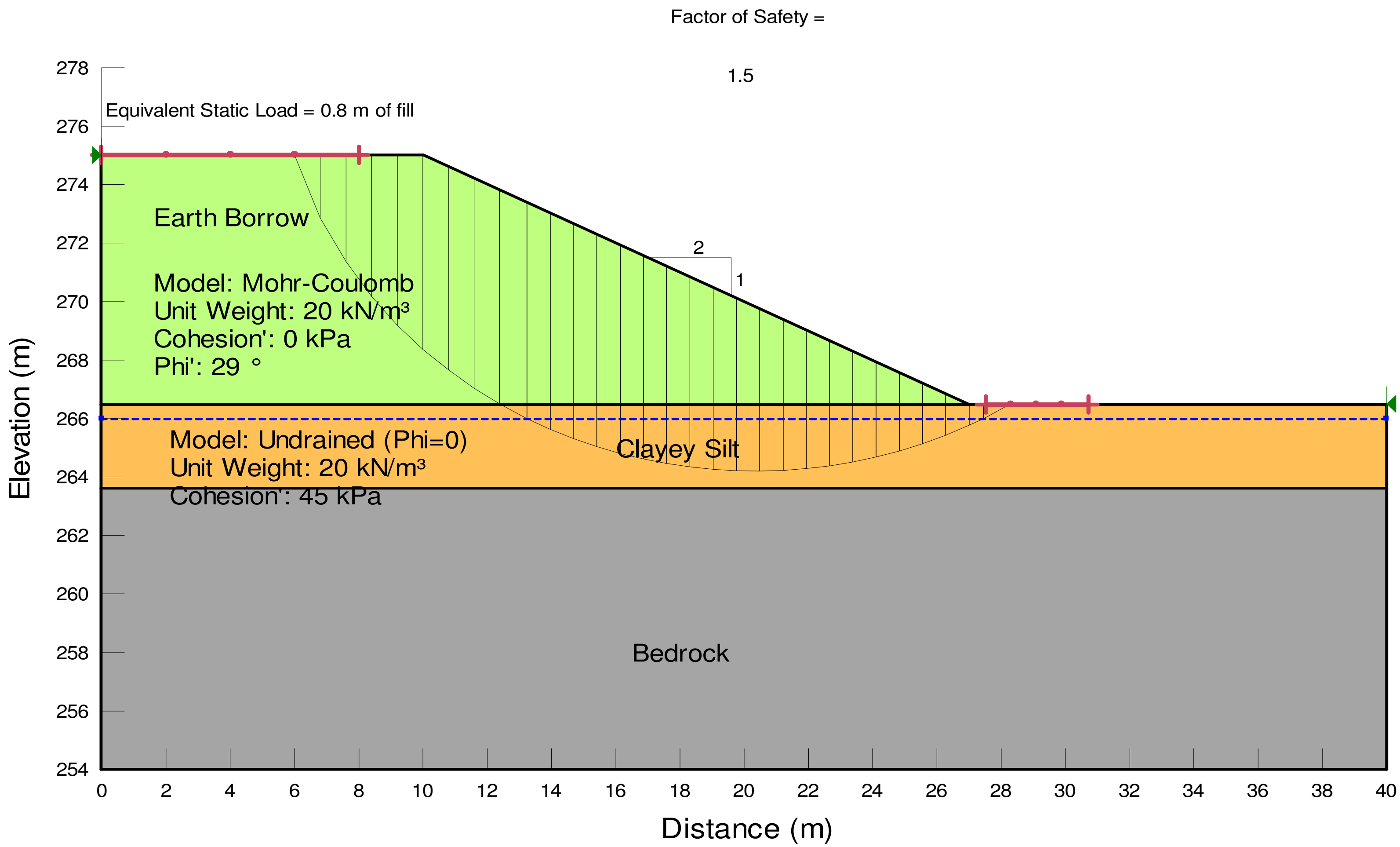
Static Slope Stability Analysis (Undrained)

Highway 144 Chelmsford Bypass
Gordon Lake Road Underpass

Figure 7b

Project No. 165000734

GWP No. 5023-09-00



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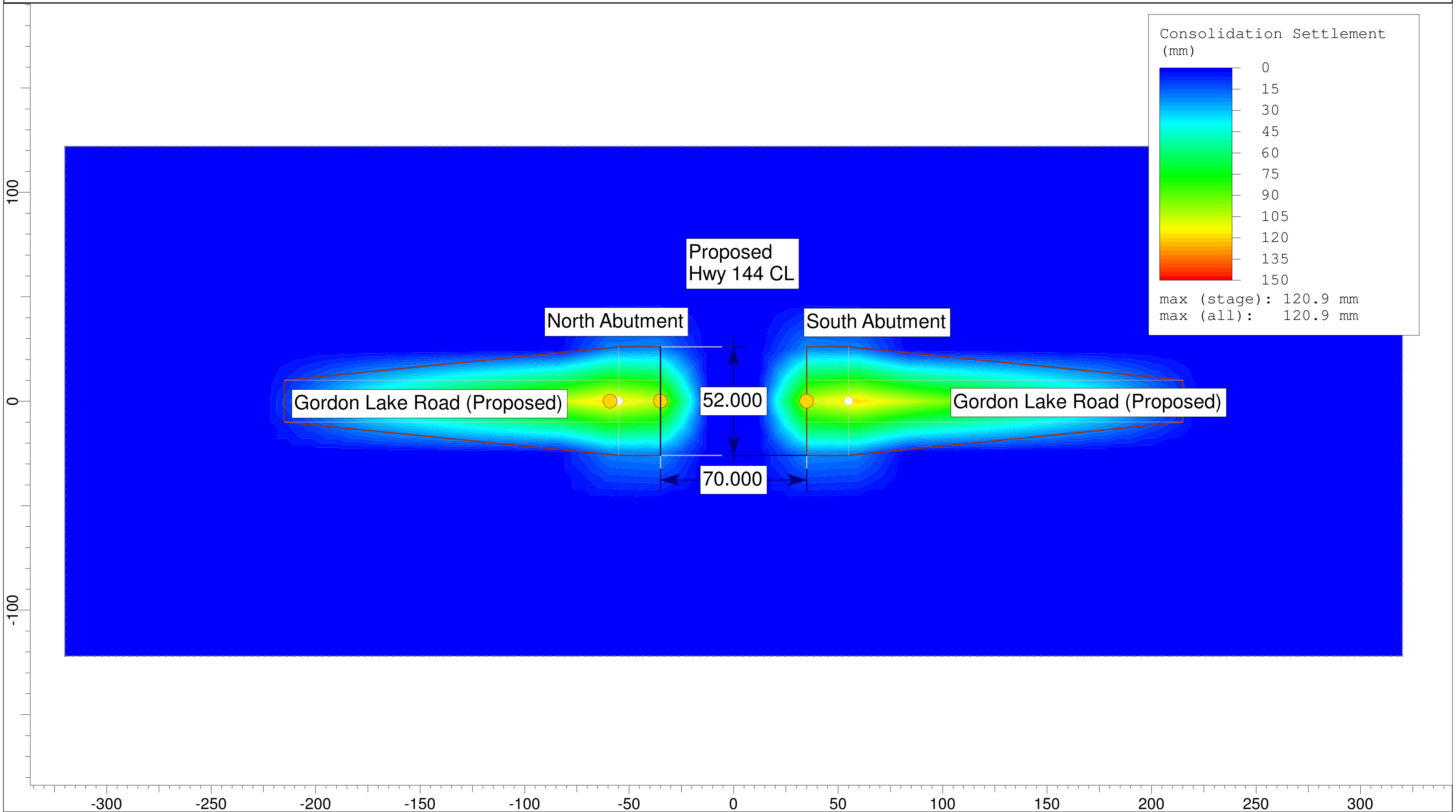
Seismic Slope Stability Analysis

Highway 144 Chelmsford Bypass
Gordon Lake Road Underpass

Figure 7c	
Project No. 16500734	
GWP No. 5023-09-00	

Figure 8



Stantec	*Project* Gordon Lake Road Underpass		
	Analysis Description Evaluation of Settlement due to Approach Embankment		
	Drawn By SG	*Company* Stantec	
SETTLE3D 2.011	*Date* 3/20/2013, 4:26:22 PM	*File Name* Gordon Lake Road Underpass-rev2.s3z	

Lateral Loading on Piles – Soil Springs

A common method of soil-structure modeling is to replace the soil medium with a series of linear-elastic springs. Ideally, the stiffness of the linear-elastic springs is selected such that the calculated deformation of the spring resistance to a lateral force is the same as would be experienced within the soil medium.

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.

Upon completion of the p-y method of analysis, the calculated pile deformation profile was used to develop appropriate linear-elastic springs to be used in structural engineering software where this type of spring is a required input to model the soil response.

The table below presents the spring stiffnesses for an HP310x110. Representative spring stiffnesses have been given for the top and bottom of each soil layer; values at specific elevations should be interpolated from these values. All stiffnesses are based on a spring spacing of 0.25 m and a lateral movement of 10 mm.

Table D-1: Spring Stiffnesses for HP310x110

Elevation (m)		Soil Layer	Spring Stiffness, k (kN/m) – 0.25 m spacing	
From	To		Top of Layer	Bottom of Layer
269.2	266.2	Loose Uniform Sand in CSP	0	1,950
266.2	263.6	Clayey Silt	1,950	79,500