Preliminary Foundation Investigation and Design Report
Whitson River Crossing

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

G.W.P. 5023-09-00

Geocres No. 41I-301

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Ministry of Transportation Ontario

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March 2014

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

Whitson River Crossing, 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 foundation 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 Whitson River Crossing along the preferred alignment, approximately 3 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
Project Location:	Highway 144, from 12 km north of Highway 17, northerly 27 km
Site Location:	Approximately 3 km south of the Town of Chelmsford, Greater Sudbury, Ontario

2.0 Site Description and Geology

Site Location

The proposed structure site location is shown on the Key Plan inset to Drawing No. 1 provided in Appendix A. The preferred route of Highway 144 is shown in Drawing No. 2. At the project site, the proposed Highway 144 alignment is oriented in approximately east-west direction.

The proposed final grade of Highway 144 at the Whitson River Crossing site is approximately 266.0 m, based on the preliminary General Arrangement drawing. The anticipated maximum height of embankment to achieve the proposed grade is approximately 3.2 m.

Chainage along Highway 144 increases from east to west at the site location. The approximate location of the proposed crossing is near the future Highway 144 Station 13+090 Balfour Twp.

General Site Description

Whitson River flows in a southwesterly direction. The Whitson River is deeply entrenched and has several meanders. At the proposed crossing, both sides of the river are covered with relatively thick bush and mature trees. Photographs 1 and 2 show the general site features at the proposed crossing site.

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 deposits (sands, gravels, silts, and boulder clays) 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.

In the vicinity of the proposed Whitson River Crossing, the surficial materials consist of glaciolacustrine deposits (silt and clay, minor sand, basin and quiet water deposits) of the Pleostocene Era.

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 in the vicinity of the abutment foundations as part of the geotechnical investigation for the preliminary design of the bridge foundations for the proposed

Whitson River Crossing. The boreholes are designated BH13-21A and BH13-22A, and their locations are shown on the Borehole Location Plan, Drawing No. 1 in Appendix A.

The originally proposed foundation boreholes are designated BH13-21 and BH13-22, and are located close to proposed foundation elements. Permission to access these borehole locations was not granted at the time of scheduled drilling and consequently, alternative boreholes BH13-21A and BH13-22A were drilled. These drilled alternative boreholes, BH13-21A and BH13-22A represent offsets of approximately 85 m and 185 m, respectively, with respect to BH13-21 and BH13-22.

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

The field drilling program was carried out on March 20 and March 21, 2013. The boreholes were advanced with a track-mounted CME 55 drill rig equipped hollow-stem augers and N-sized steel casings for soil and rock sampling.

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

After completion of drilling, boreholes were backfilled with auger cuttings mixed with bentonite. Road holes were sealed with cold asphalt patch where applicable.

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

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 1A 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	
	BH13-21A	**BH13-22A**
MTM Zone 12 Coordinates Northing Easting	 5157104 288344	 5157096 288587
Ground Surface Elevation, m	265.0	264.7
Total Depth Drilled, m	24.6	13.5
End of Borehole Elevation, m	240.4	251.2
Depth Augered, m	23.0	11.8
Depth Cored, m	1.6	1.7
Number of Soil Samples	23	15

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	Remarks
Moisture Content	39	By Stantec
Atterberg Limits	8	By Stantec
Grain Size Distribution	10	By Stantec
Consolidation (oedometer)	1	By Golder
Unconfined Compression (soil)	1	By Golder
Unconfined Compression (rock)	2	By Stantec
Specific Gravity	1	By Golder

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 granular fill material overlying silty sand and silt layers. The silt layer was underlain by a silty clay layer overlying 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 Fill

A granular surface layer consisting primarily of silty sand (with gravel at borehole BH13-21A) was observed at both borehole locations. The approximate thickness of the granular fill layer encountered in BH13-21A and BH13-22A was 690 mm and 460 mm respectively. The base elevation of the fill layer was approximately 264.3 m in both boreholes.

Moisture content tests carried out on two samples of the fill indicated moisture contents of 23% and 15%.

A grain size distribution analysis carried out on one sample of the fill indicated 4% gravel, 57% sand and 39% fines (silt & clay). The Unified Soil Classification System (USCS) group symbol for the fill is SM (silty sand). Representative grain size distribution plot for the fill layer is provided in Figure 1 of Appendix C.

4.1.2 Silty Sand

A silty sand layer was encountered in both boreholes immediately beneath the fill layer. The thickness of the silty sand layer was approximately 4.6 m in BH13-21A and 2.5 m in BH13-22A, and extended to respective bottom elevations of approximately 259.7 m and 261.7 m.

The silty sand layer was composed of predominantly sand with some fines (silt and clay). The Standard Penetration Test (SPT) blow count (N-value) for the silty sand layer ranged from 4 to 15 blows/0.3 m, indicating a loose to compact state.

Index tests carried out on representative samples from the silty sand layer yielded the following results:

Gravel:	0%
Sand:	74 and 80%
Fines (silt & clay):	20 to 26%
Moisture Content:	8 to 28%

The Unified Soil Classification System (USCS) group symbol for this layer was SM (silty sand). Representative grain size distribution plots for the silty sand layer are provided in Figure 2 of Appendix C.

4.1.3 Silt

A silt layer was encountered in both boreholes immediately beneath the silty sand layer. The thickness of the silt layer was approximately 15.6 m in BH13-21A and 3.1 m in BH-22A, and extended to respective bottom elevations of approximately 244.1 and 258.6 m.

The silt layer was composed of predominantly silt size materials and some clay size materials. The SPT N-value for the silt layer ranged between 2 and 16 blows/0.3 m, suggesting a very loose to compact state.

Index tests carried out on representative samples from the silt layer yielded the following results:

Gravel:	0%
Sand:	0 to 1%
Silt Size:	78 to 97%
Clay Size:	3 to 22%
Moisture Content:	23 to 31%

Atterberg limits tests carried out on five representative samples of this layer produced non-plastic results. The USCS group symbol for this layer is ML (silt). Representative grain size distribution plots for the silt layer are provided in Figure 3 of Appendix C. The plasticity chart is provided in Figure 5.

4.1.4 Silty Clay

A silty clay layer was encountered in both boreholes immediately beneath the silt layer. The approximate thickness of the silty clay layer was 2.1 m in BH13-21A and 5.7 m in BH13-22A, and extended to respective bottom elevations of approximately 242.0 m and 252.9 m.

The silty clay layer was composed of clay and silt size materials. The SPT N-value for the clayey silt layer ranged between 1 and 9 blows/0.3 m. Pocket penetrometer testing carried out on a selected split-spoon sample indicated an undrained shear strength of 110 kPa. In-situ field vane tests carried out within the silty clay layer indicated undrained shear strength range of 68 to 92 kPa. An unconfined compression test carried out on a thin-wall tube sample of the silty clay layer indicated an undrained shear strength of 90 kPa; the unit weight of this sample was

19.5 kN/m^3. Based on the undrained shear strength, the silty clay layer can be considered to be stiff to very stiff.

Index tests carried out on representative samples from the silty clay layer yielded the following results:

Gravel:	0%
Sand:	0%
Silt:	44 and 71%
Clay:	29 and 56%
Moisture Content:	29 to 50%

Atterberg limits tests carried out on two representative samples of the silty clay layer indicated plasticity indexes of 7 and 22. The USCS group symbol for this layer is CL (samples BH13-21A SS-22 and BH13-22A ST-9) and CI (sample BH13-21A SS-15). This deposit has a low to medium plasticity.

Representative grain size distribution plots for the silty clay layer are provided in Figure 4 of Appendix C. The plasticity chart is provided in Figure 5.

The results of one consolidation test carried out are included in Appendix C. The results of the consolidation test are summarized below:

Table 4.1: Consolidation Test Results (silty clay layer)

Sample ID	Sample Elev.	Moisture Content	Initial Void Ratio/Initial Unit Weight	Estimated Overburden Pressure P'o	Estimated Preconsolidation Pressure, P'c	Recompression Index, Cr	Compression Index, Cc
BH13-22A ST-9	258.3 m	27%	0.74/19.6 kN/m^3	75 kPa	116 kPa[1]	0.017[1]	0.06[1]

Note 1: The results of the consolidation test suggest the sample was disturbed or very silty and not representative of the clay layer.

4.2 BEDROCK

A black metasedimentary bedrock was encountered in both boreholes. The bedrock was confirmed by coring 1.6 m and 1.7 m with using N size coring equipment. Bedrock was encountered at approximate elevation of 242.0 m in BH13-21A and 252.9 m in BH13-22A. The metasedimentary bedrock consisted of laminated argillites and siltstones.

The Rock Quality Designation (RQD) values were 61% and 63%, indicating a fair rock quality. The Total Core Recovery (TCR) ranged between 97 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 one bedrock sample from each of boreholes BH13-21A and BH13-22A. The results of these tests are summarized in Table 4.2.

Table 4.2: Unconfined Compressive Strength of Rock Cores

Borehole No	Ground Surface Elevation (m)	Test Elevation (m)	Unconfined Compressive Strength (MPa)
BH13-21A	265.0	240.5	84
BH13-22A	264.7	252.7	85

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

4.3 CHEMICAL ANALYSIS

One representative sample retrieved from the silty sand from each of boreholes BH13-21A and BH13-22A were tested for pH, water soluble sulphates and chloride concentrations, and resistivity. The results of this chemical analysis are provided in Table 4.3.

Table 4.3: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH13-21A	SS-5	3.05 to 3.66	7.5	<5	92	312
BH13-22A	SS-4	2.30 to 2.90	7.5	15	119	63

4.4 GROUNDWATER

Groundwater level was estimated within the 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 4.4.

Table 4.4: Inferred Groundwater Levels (time of drilling)

Borehole No	Ground Surface Elevation (m)	Groundwater	
		Depth (m)	Elevation (m)
BH13-21A	265.0	1.5	263.5
BH13-22A	264.7	1.8	262.9

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 on behalf of MTO 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 by-pass the communities of Chelmsford and Dowling.

The preferred alignment includes 11 structure sites, including the Whitson River Crossing site.

Proposed Crossing Structures

The proposed Whitson River Crossing is planned to be constructed in two phases: initial phase and ultimate phase. The initial phase will involve a structure with approximate span of 28 m and a width of 15 m. The ultimate phase will include a second structure of identical size approximately 35 m south of the structure for the initial phase. The ultimate configuration of the new Highway 144 will include a four-lane divided highway.

The proposed new crossing location is approximately 1.7 km south of the existing Highway 144 crossing.

The preliminary General Arrangement (GA) drawing indicates that the proposed bridges will each have a single span with two integral abutments east and west of Whitson River.

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

Proposed Underside of Pile Cap Elevation (East Abutment):	260.9 m
Proposed Underside of Pile Cap Elevation (West Abutment):	261.2 m
Proposed Final Grade (Top of Road) at East Abutment:	266.0 m
Proposed Final Grade (Top of Road) at West Abutment:	265.9 m
Existing Grade (East Abutment):	263.0 m
Existing Grade (West Abutment):	263.3 m

5.1 GEOTECHNICAL DESIGN PARAMETERS

The subsurface conditions encountered at the two boreholes drilled either side of the proposed Whitson River Crossing (approximately 85 m northwest and 185 m northeast of the bridge location) consisted of granular fill material overlying silty sand and silt layers. The silt layer was underlain by a silty clay layer overlying bedrock. For preliminary design purposes, it is assumed that the subsurface conditions encountered in BH13-21A and BH13-22A are representative of the west and east abutments, respectively, of the bridge. Based on the information in these boreholes the estimated depth to bedrock at the proposed Whitson River Crossing is 10.1 m (263.0 m - 252.9 m) for the east abutment and 21.3 m (263.3 m- 242.0 m) for the west abutment.

The RQD of the bedrock was 61% and 63%, indicating a fair rock quality. The unconfined compressive strength was approximately 85 MPa (strong bedrock).

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 Standard Penetration Test, the field vane and pocket penetrometer measurements, and laboratory consolidation and unconfined compression and index test results (including moisture contents) of samples retrieved from the site. This profile is included in Figures 6a and 6b, and was developed based on the information obtained from boreholes BH13-21A and BH13-22A. The strata boundaries presented in Table 5.1 and Figures 6a & 6b were interpolated between boreholes BH13-21A and BH13-22A.

Table 5.1: Preliminary Subsurface Profile at Proposed Whitson River Crossing

Elevation (m)		Soil Type	Design Parameters				
From	To		γ (kN/m³)	ϕ (°)	S_u (kPa)	UCS (MPa)	E (MPa)
East of Whitson River							
263.0	261.8	Silty sand (loose to compact)	20	30	-	-	10
261.8	258.6	Silt (very loose to compact)	19.6	28	-	-	7
258.6	256	Silty clay (very stiff)	19.6	-	90	-	-
256	252.9	Silty Clay (stiff)	17.3	-	60	-	-
< 252.9		Metasedimentary bedrock (fair quality, strong bedrock)	24	-	-	85	-
West of Whitson River							
263.3	259.8	Silty sand (loose to compact)	20	30	-	-	10
259.8	256	Silt (very loose to compact)	19.6	28	-	-	7 to 15
256	259.8	Silt (compact)	19.6	28	-	-	101
244.2	242.0	Silty clay (stiff to very stiff)	19.6	-	75	-	-
< 242.0		Metasedimentary bedrock (fair quality, strong bedrock)	24	-	-	84	-

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

(2) Groundwater is assumed to be at an approximate elevation of 261.6 m (depth of 1.8 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 crossing location generally includes an approximately 3 m thick layer of loose to compact silty sand overlying 11 to 12.5 m thick very loose to compact silt layer followed by an approximately 3 m thick stiff to very stiff silty clay layer. A fair quality metasedimentary bedrock is found approximately 11.8 to 23.0 m below existing ground surface. It is recommended that a Soil Profile II, 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 (Figure J-2 of Commentary J to the 2006 National Building Code of Canada (NBC, 2006)). The assessment result is indicated in Figure 7 of Appendix D. This assessment indicates that liquefaction of the foundation soils will not be of concern for this site.

Even though not significant, seismically induced lateral earth pressures 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 Whitson River Crossing

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences	Rank*
Shallow foundation on soil	-	-	-	▪ The soil conditions below the proposed founding level are not suitable to support spread footing foundations for a bridge.	3
Piles End bearing on or socketed into bedrock	▪ Reduces risk of differential settlement ▪ Suitable for integral abutment bridge ▪ Large geotechnical resistances	▪ May attract drag loads	Medium	▪ Pile lengths may vary depending on actual bedrock elevation at abutment locations.	1
Drilled Caissons	▪ Can transmit very large axial and lateral loads	▪ Not suitable for integral bridge abutment ▪ Caisson holes will require steel liner	Medium to High	▪ Risk of cave-in, especially during drilling in sand below groundwater table; mitigated by use of a liner	2

*Based on qualitative criteria.

Based on the comparison presented above, the option of flexible piles consisting of steel H-piles driven to bedrock is the most viable option for the proposed integral abutments of the Whitson River Crossing.

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

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 east and west of Whitson River. Based on the preliminary GA drawing and the frost depth at the site, the underside of the pile caps (bottom of concrete abutments) are assumed to be at an approximate elevation of 260.8 m for both the east and west abutments.

The piles should be driven to refusal on bedrock. The anticipated pile lengths will be approximately 8 to 15 m for the east abutment and 16 to 19 m for the west abutment; due to site restrictions, the boreholes were not drilled within the abutment footprints.

A factored axial geotechnical 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 driven to refusal on 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 crossing structure will require an approximately 3.2 m high approach embankment fill to raise the final grade of the proposed Highway 144 at the crossing. 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 abutments is approximately 50 mm, most of which will be within the silt layer and would be complete within a few weeks after the embankment has reached their final grade.

Assuming that piles are driven prior to construction of the complete embankment, it should be anticipated that drag loads along the full length of the pile would develop as the deeper clay undergoes consolidation. The maximum unfactored drag loads is estimated to be 400 kN per pile based on a 15 m long pile length beneath the abutment. 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 at the site consisted of very loose to compact silty sand overlying a very loose to compact silt followed by a stiff to very stiff silty clay. No obstructions to pile driving are anticipated.

Piles should have Type 1 reinforced 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 firm to stiff cohesive material.

<u>ULS Resistance</u>

The passive earth pressure for the pile driven through layers consisting of silty sand, silt and silty clay was estimated using the procedure described in Section C6.8.7.1 of CHBDC (CHBDC, 2006). For an HP 310x110, the estimated factored lateral resistance at ULS_f is 110 kN for both the east and west abutments.

<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 Tables 5.3. These parameters were developed based on the synthesis of field and laboratory data.

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	
						k	ε_{50}
	From	To	kN/m³	Degrees	kPa	kN/m³	-
East Abutment							
Silty sand	263.0	261.8	20	30	-	5,400	-
Silt	261.8	258.6	19.6	28	-	5400	-
Silty clay	258.6	252.9	19.6	-	75	-	0.007
Bedrock	< 252.9		24	-	-	-	-
West Abutment							
Silty sand	263.3	259.8	20	30	-	5,400	-
Silt	259.8	244.2	19.6	28	-	5,400	-
Silty clay	244.2	242.0	17.3	-	75	-	0.007
Bedrock	< 242.0		24	-	-	-	-

Notes:

(1) Submerged unit weight will be used below groundwater level.
(2) k = p-y modulus; ε_{50} = strain corresponding to one-half the maximum principal stress difference.
(3) Groundwater level will be assumed to be at an elevation of 261.6 m.

Plots from LPILE for both the east and west abutments are presented in Figures 8 through 11 of Appendix D. Figures 8 & 10 show the deformed shape of the pile for lateral (shear) force ranging between 50 and 100 kN for both the east and west abutment piles. These plots indicate that the pile will undergo negligible lateral deflection below a depth of approximately 7 m from the underside of the pile caps for both the east and west abutments (approximate elevation 254 m).

Figures 8 and 10 in Appendix D illustrate the displacement of the pile in depth for different lateral loads. Based on these Figures, a lateral load of 70 kN corresponds to a pile head (top) displacement of approximately 10 mm for both the east and west abutment piles. Therefore, the SLS geotechnical resistance of an HP 310x 110 at this site is estimated as 70 kN for both the east and west abutment piles.

Figures 9 and 11 of Appendix D present the p-y plot that gives the non-linear response of the pile-soil interaction. The plots provide 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 5.4: 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 behind structures for the proposed crossing structures 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, Granular A or 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 5.5 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.

$P_A = ½ K_a \gamma H^2$
$P_P = ½ K_p \gamma H^2$
$P_O = ½ K_o \gamma H^2$

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 Silt
Bulk Unit Weight, γ (kN/m^3)	21.2	22.0	20	19.6
Effective Friction Angle (º)	32	35	30	28
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43	0.50	0.53
Coefficient of Active Earth Pressure (K_a)	0.31	0.27	0.33	0.36
Coefficient of Passive Earth Pressure (K_p)	3.2	3.7	3.0	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 5.6 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 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° backfill slope)
- Vertical Back of Wall — (0° angle of wall from vertical)

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 Silt	
Bulk Unit Weight, γ (kN/m^3)	21.2		22.0		20		19.6	
Effective Friction Angle (º)	32		35		30		28	
Wall type: (a) yielding; (b) non-yielding	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)
Active Earth Pressure (K_{AE})	0.32	0.35	0.28	0.31	0.35	0.38	0.38	0.41
Height of Application of PAE from base as a ratio of wall height, (H)	0.341	0.356	0.342	0.358	0.340	0.355	0.340	0.354
Passive Earth Pressure, (K_{PE})	3.21	N/A	3.64	N/A	2.96	N/A	2.73	N/A
Height of Application of PPE from base as a ratio of wall height, (H)	0.325	N/A	0.325	N/A	0.325	N/A	0.325	N/A

For non-yielding walls, the combined static and seismic earth pressure can be estimated using the above K_{AE} (non-yielding) values rather than k_o values typically used for static analysis.

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 river crossing requires approach embankments for future Highway 144 eastbound and westbound lanes to be built east and west of the structure. For preliminary design purposes, it is assumed that the embankment will be constructed using either a Select Subgrade Material or Earth Borrow material.

Based on the preliminary GA drawing, the anticipated maximum embankment height is approximately 3.2 m near the east abutment and 2.6 m near the west 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 3.2 m as discussed above. The evaluation was carried out using a commercial program, Slope/W (Geo-Slope, 2012). 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 12a through 12c and 13a through 13c of 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.

The natural slope of the Whitson River was also assessed. Preliminary evaluation indicates that these slopes should be flattened to be no steeper than 2.5H:1V unless slope stabilization methods are put in place to improve the stability of these slopes. As well, the flattened slope would require rip-rap and scour protection.

5.7.3 Embankment Settlement

The proposed embankment geometry (height, top width, and side slope) has not yet been established. A more 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:

- There is negligible superposition of settlements caused by the east and west approach embankments;
- 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 silt) and consolidation settlement (for silty clay) is considered;
- A Poisson's ratio of 0.35 will be used for all soil types;
- Groundwater is assumed at approximate elevation 261.6 m;
- The maximum embankment height will be approximately 3.2 m (in the immediate vicinity of the east abutment);
- The approach embankment will have 2V:1H side slopes;
- The embankment extends approximately 640 m east and west from the respective abutments of the proposed crossing structures;
- The top width of the embankment will be approximately 20 m (including shoulders and roundings); and
- The distance between the abutments will be approximately 28 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 Figures 14a and 14b of Appendix D, respectively, for the east and west approach embankments. The preliminary analysis results indicate that the maximum total vertical settlement of the existing materials for the conditions presented above is approximately 60 mm beneath the east embankment and 95 mm beneath the west embankment. The maximum settlement will take place approximately 15 m back from each bridge abutment. The estimated settlement at the abutments is 32 mm near the east abutment and 48 mm near the west abutment.

Most of the settlements anticipated are within the silt deposits for both the east and west abutments. Settlements anticipated within the underlying silty clay less than 15 mm at the east abutment and less than 3 mm at the west abutment.

Assuming 0.5% strain under self-weight, the estimated embankment self-weight settlement is approximately 16 mm. This settlement is anticipated to be completed by the end of embankment construction.

5.7.4 Settlement Mitigation

The majority of the estimated settlement given above will be due to the elastic compression of the silty sand and silt layers. Most of this settlement will take place in a relatively short period of time, in the order of a few weeks. Consolidation settlement of the lower lying silty clay is not considered significant for preliminary design purposes since the loading stress reaching this layer is small for the anticipated embankment height. Additional foundation investigations are required to better assess the settlement characteristics of the site soil within the footprint of the abutments and the embankments. It is recommended that these considerations be carried out during the final foundation investigation and design.

5.8 PRELIMINARY CONSTRUCTION CONSIDERATIONS

5.8.1 Excavation and Backfilling

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

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.

Side slopes for open cut excavations should conform to the Occupational Health and Safety Act (OHSA) regulations for Construction Projects. The loose to compact silty sand soil at the site should be considered a Type 3 Soil in accordance with the OHSA. The underlying very loose to compact silt should be considered a Type 4 Soil.

The abutment excavations may extend as much as 2.7 m below existing grade. Depending on the contractor's construction preference, temporary support systems may be used on this project.

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

The groundwater elevation for preliminary design is 261.6 m. Abutment excavations are likely to extend below the groundwater level. Unwatering is required to maintain dry working conditions desirable during excavation and construction of the pile caps and abutments.

The native soils within the anticipated depth of excavation are expected to have a low to medium hydraulic conductivity, in the order of 10-5 to 10-7 m/s. Unwatering of the structure excavation using conventional sump and pump techniques with cofferdams should be adequate.

5.8.3 Reuse of Excavated Material

The native material at the bridge site may include organic material and silts. These materials will not be suitable for use as backfill within and behind the structures for the proposed structures and embankments. Silty sand was encountered within both boreholes. This material is expected to be encountered within the footprints of the approach embankments and the abutments. This material will not be suitable for use backfill within and behind the proposed structures may be used for construction of the approach embankments.

5.9 CEMENT TYPE AND CORROSION POTENTIAL

Two sample of the native silty sand silt were tested for pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructures. The analysis results are summarized in Table 4.3.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate concentrations for both samples were 92 and 119 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 7.5, 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 4.3 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 crossing. The recommendations were made based on the interpretation of a limited number of test holes advanced in the general vicinity of each proposed foundation elements. 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 crossing structures 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 HP 310 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

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. 2012. Stability Modeling with SLOPE/W 2012©. Calgary, AB.

NBC. 2006. User's Guide – NBC 2005 Structural Commentaries. National Research Council of Canada, Ottawa.

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 Engineering Technologist, 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's Ottawa and Golder's Mississauga Laboratories. Chemical testing on soil samples was carried out by Paracel Laboratories in Ottawa.

This report was prepared by Simon Gudina, Ph.D., P.Eng., and reviewed by Chris McGrath, P.Eng. and Raymond Haché, M.Sc., 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,

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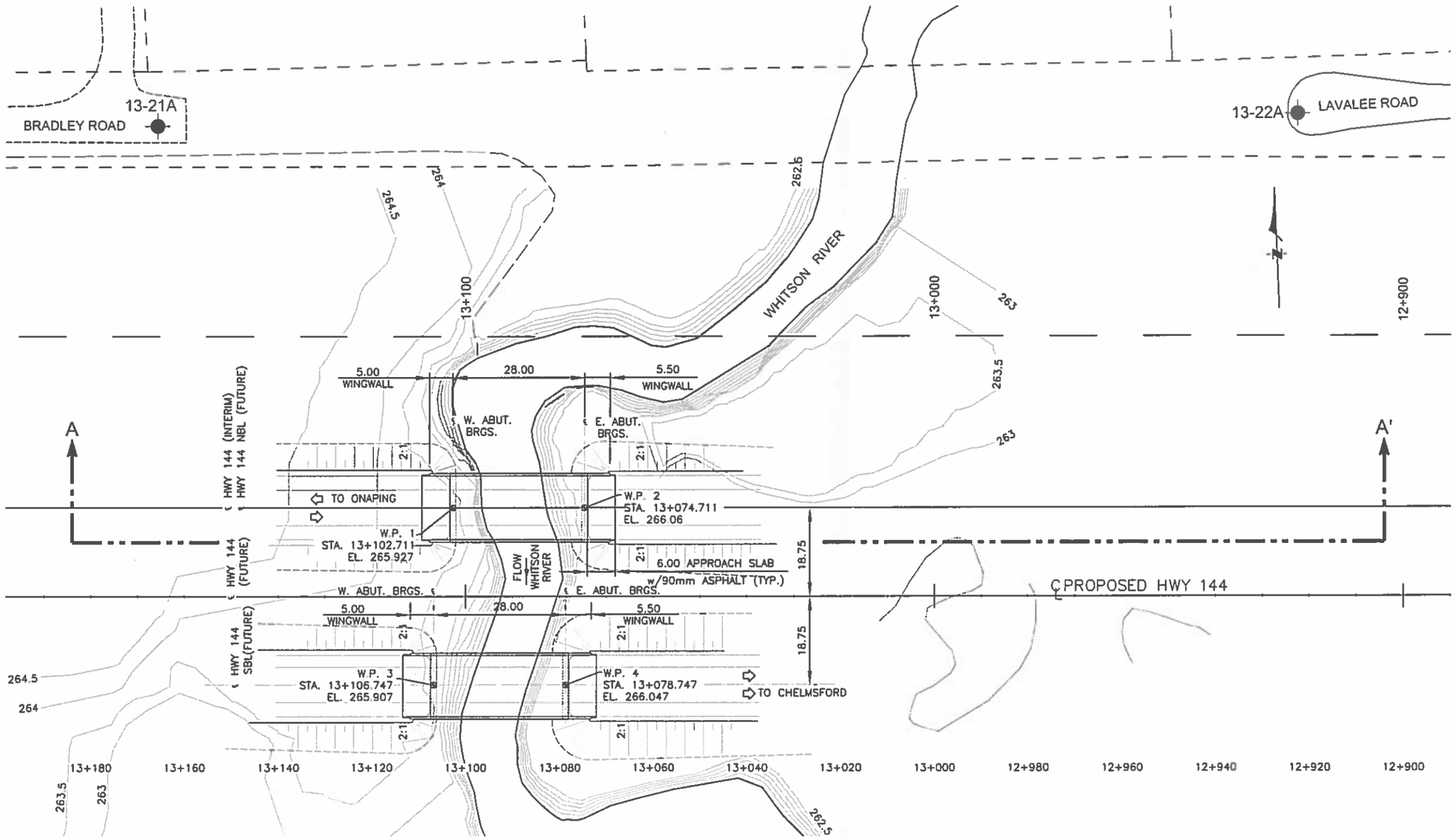
APPENDIX A

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



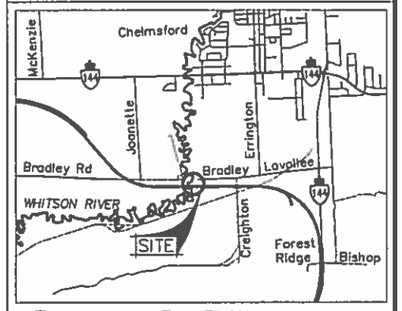
PLAN
SCALE
10 m 0 10 20 m

PLATE No
CONT
WP 5023-09-00

HWY 144 ROUTE SELECTION STUDY
WHITSON RIVER CROSSING
BORE HOLE LOCATIONS

SHEET

Stantec



KEY PLAN
1 km 0 1 2 km

LEGEND

Alternative Borehole (Drilled)

No	ELEVATION	MTM ZONE 12 COORDINATES NORTH	EAST
13-21A	265.0	5 157 103.7	288 343.5
13-22A	264.7	5 157 096.2	288 586.9

NOTES

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

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REVISIONS		
DATE	BY	DESCRIPTION

GEOCRES No 41I-301

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

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MINISTRY OF TRANSPORTATION, ONTARIO 88-25 PH-0-101

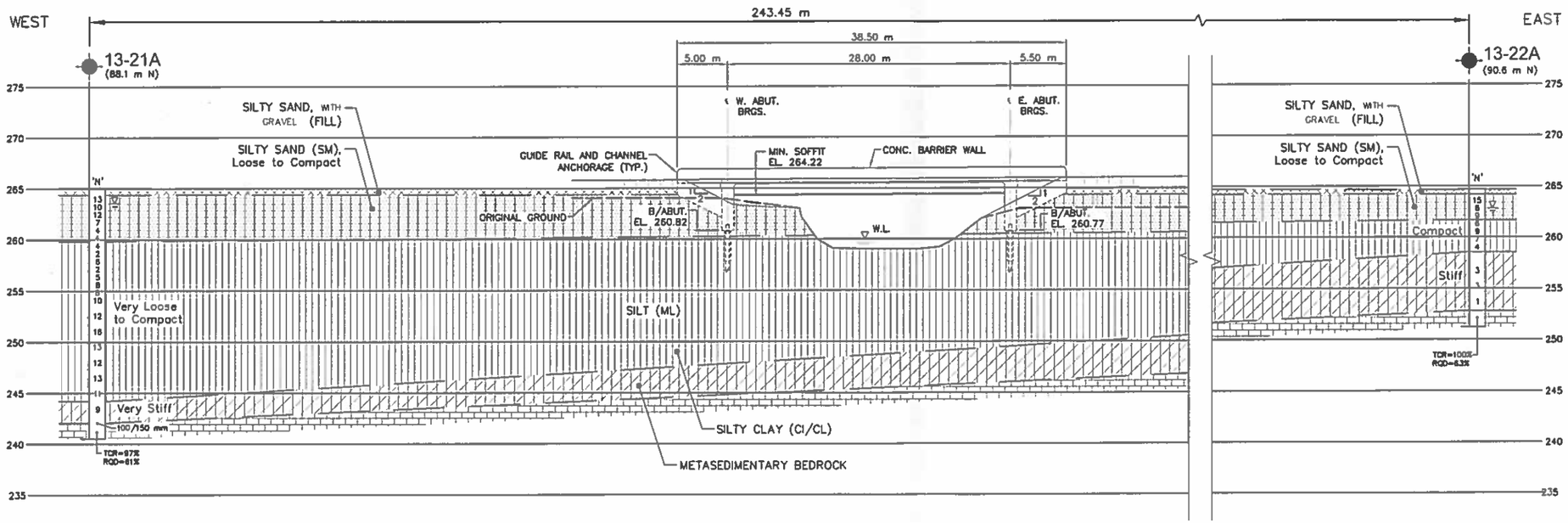
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No
CONT
WP 5023-09-00

HWY 144 ROUTE SELECTION STUDY
WHITSON RIVER CROSSING
SOIL STRATA

SHEET

Stantec



CROSS SECTION
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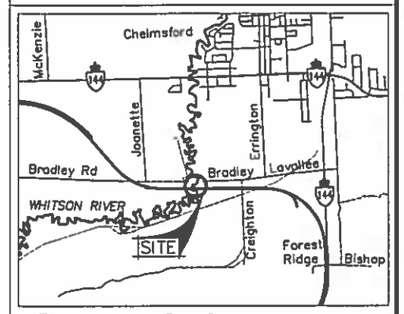

KEY PLAN
1 km 0 1 2 km

LEGEND

- Alternative Borehole (Drilled)
- N — Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at Time of investigation March 2013
- (x.x m N) — Offset in meters North of Cross Section Line A–A'

No	ELEVATION	MTM ZONE 12 COORDINATES NORTH	EAST
13-21A	265.0	5 157 103.7	288 343.5
13-22A	264.7	5 157 096.2	288 586.9

NOTES

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

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

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REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No 41I-301

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

DRAWING NAME: 165000734_58_Whitson River_GA_2013-11-01.dwg
CREATED BY: CBB MODIFIED:
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MINISTRY OF TRANSPORTATION, ONTARIO

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: January 22, 2013

Site Photo No.: 1	Looking east near proposed crossing (west of Whitson River)

Site Photo No.: 2	Shrubs and mature trees west of proposed Whitson River Crossing

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:

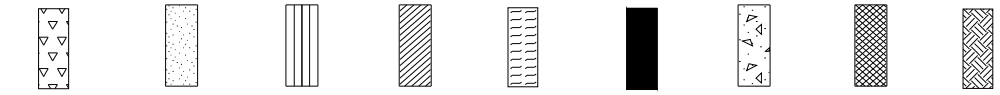
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

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 BH13-21A

METRIC

W.P. GWP 5023-09-00 | LOCATION Bradley Road, Greater Sudbury, ON N: 5 157 104 E: 288 344 | ORIGINATED BY JHJ

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

DATUM Geodetic | DATE 2013 03 20 - 2013 03 21 | CHECKED BY CM/SG

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL × FIELD VANE ⨯ 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
265.0	Roadbase Granulars										
0.0	FILL: brown silty sand with gravel		1	BS	-						
264.3											
0.7	SILTY SAND (SM) Loose to compact Brown to light brown, moist to wet		2	SS	13		264				
	- Groundwater inferred at a depth of 1.5 m (elevation 263.5 m)		3	SS	10	▽	263				0 80 (20)
			4	SS	12						
			5	SS	7		262				
			6	SS	4		261				
			7	SS	4						0 74 (26)
259.7							260				
5.3	SILT (ML) Very loose to compact Grey, wet		8	SS	4						
			9	SS	2		259				
			10	SS	6		258				
			11	SS	2		257				
			12	SS	5						0 0 97 3 Non-plastic
			13	SS	8		256				
255.0											

Continued Next Page

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

Stantec

RECORD OF BOREHOLE No BH13-21A

METRIC

W.P. GWP 5023-09-00 | LOCATION Bradley Road, Greater Sudbury, ON N: 5 157 104 E: 288 344 | ORIGINATED BY JHJ

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

DATUM Geodetic | DATE 2013 03 20 - 2013 03 21 | CHECKED BY CM/SG

SOIL PROFILE ELEV/DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, ● QUICK TRIAXIAL, × FIELD VANE, ⨯ 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
10.0	*(continued)* SILT (ML)		14	SS	8						
	Very loose to compact										
	Grey, wet										
	- Compact below el. 254 m		15	SS	10		254				
							253				
			16	SS	12						0 0 91 9 Non-plastic
							252				
			17	SS	16		251				
							250				
			18	SS	13						
							249				
			19	SS	12		248				
							247				
			20	SS	13						0 0 80 20 Non-plastic
							246				
245.0											

Continued Next Page

×³, ⨯³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH13-21A

METRIC

W.P. GWP 5023-09-00 | LOCATION Bradley Road, Greater Sudbury, ON N: 5 157 104 E: 288 344 | ORIGINATED BY JHJ

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

DATUM Geodetic | DATE 2013 03 20 - 2013 03 21 | CHECKED BY CM/SG

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL × FIELD VANE × 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
20.0	*(continued)* SILT (ML)		21	SS	11						
	Very loose to compact										
	Grey, wet										
244.1											
20.9	SILTY CLAY (CL)						244				
	Very stiff										
	Grey to black, wet		22	SS	9						0 0 71 29 PP = 110 kPa
							243				
	- bedrock fragments										
242.0			23	SS	100,/ 150 mm		242				
23.0	Metasedimentary BEDROCK: Laminated argillites and siltstones										
	-fair quality -unweathered -black colour -two dipping joint sets with very close to moderate spacing		24	NQ	-		241				TCR = 97% RQD = 61% UCS = 84 MPa
240.4											
24.6	End of Borehole										PP = Pocket Penetrometer

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

Stantec

RECORD OF BOREHOLE No BH13-22A

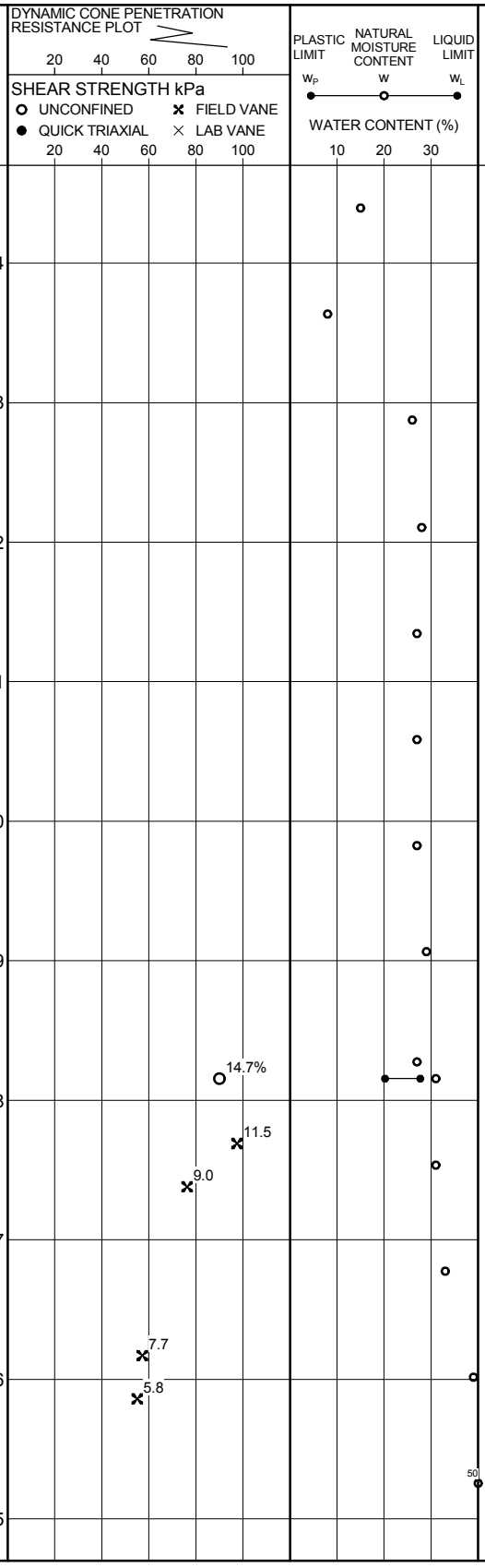
METRIC

W.P. GWP 5023-09-00 | LOCATION Lavallee Road, Greater Sudbury, ON N: 5 157 096 E: 288 587 | ORIGINATED BY JHJ

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

DATUM Geodetic | DATE 2013 03 21 - 2013 03 21 | CHECKED BY CM/SG

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
264.7	Roadbase Granulars					
0.0	FILL: brown silty sand (SM)	1	BS	-		4 57 (39)
264.2						
0.5	SILTY SAND (SM) Loose to compact Light brown to grey, moist to wet	2	SS	15		
	- Groundwater inferred at a depth of 1.8 m (elevation 262.9 m)	3	SS	8		
		4	SS	9		
261.7						
3.0	SILT (ML) Compact Light brown to grey, wet	5	SS	6		0 1 89 10 Non-plastic
		6	SS	9		
		7	SS	7		
		8	SS	4		0 0 78 22 Non-plastic
258.6						
6.1	SILTY CLAY (CI; CL) Stiff Grey, wet	9	ST	-	19.6 19.5	
		10	SS	-		
		11	SS	3		
		12	SS	-		
		13	SS	3		
254.7						



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×³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH13-22A

METRIC

W.P. GWP 5023-09-00 | LOCATION Lavallee Road, Greater Sudbury, ON N: 5 157 096 E: 288 587 | ORIGINATED BY JHJ

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

DATUM Geodetic | DATE 2013 03 21 - 2013 03 21 | CHECKED BY CM/SG

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, ● QUICK TRIAXIAL, × FIELD VANE, ⨯ LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L (WATER CONTENT (%))	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
10.0	*(continued)* SILTY CLAY (CI)		14	SS	-			8.0, 8.7	47		
	Stiff										
	Grey, wet						254				
			15	SS	1				46, 42		0 0 44 56
	- bedrock fragments						253				
252.9 / 11.8	Metasedimentary BEDROCK: Laminated argillites and siltstones -fair quality -unweathered -black colour -two dipping joint sets with close to moderate spacing		16	NQ	-		252				TCR = 100% RQD = 63% UCS = 85 MPa
251.2 / 13.5	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 No.:**	165000734
Project:	Hwy 144 - Chelmsford Bypass	**Date:**	March 20, 2013
Contractor:	Abarflex Drilling	**Borehole No.:**	BH13-21A
		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		
23.01	NQ24	97%	61%	24.59	Metasedimentary bedrock: black laminated argillites and siltstones (Onwatin Formation)	S	U	2	J	D	VC-M	SP	-	T	dipping pyrite lenses >1-10 mm in thickness	Mohs Hardness: H=3-5.5
									J	D	M	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 No.:**	165000734
Project:	Hwy 144 - Chelmsford Bypass	**Date:**	March 21, 2013
Contractor:	Abarflex Drilling	**Borehole No.:**	BH13-22A
		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		
11.84	NQ16	100%	63%	13.46	Metasedimentary bedrock: black laminated argillites and siltstones (Onwatin Formation)	S	U	2	J	D	C-M	SP	-	T	dipping pyrite lenses >1-10 mm in thickness	Mohs Hardness: H=3-5.5
									J	D	C-M	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

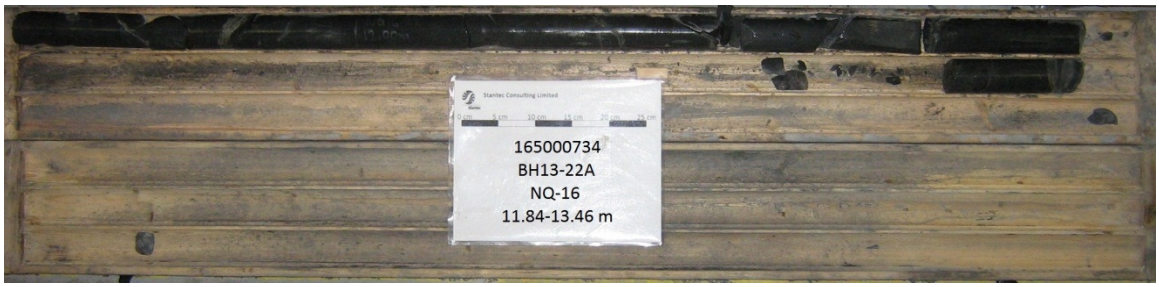
RU = Rough Undulating
RP = Rough Planar
SU = Smooth Undulating
SP = Smooth Planar
LU = Slickensided Undulating
LP = Slickensided Planar

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

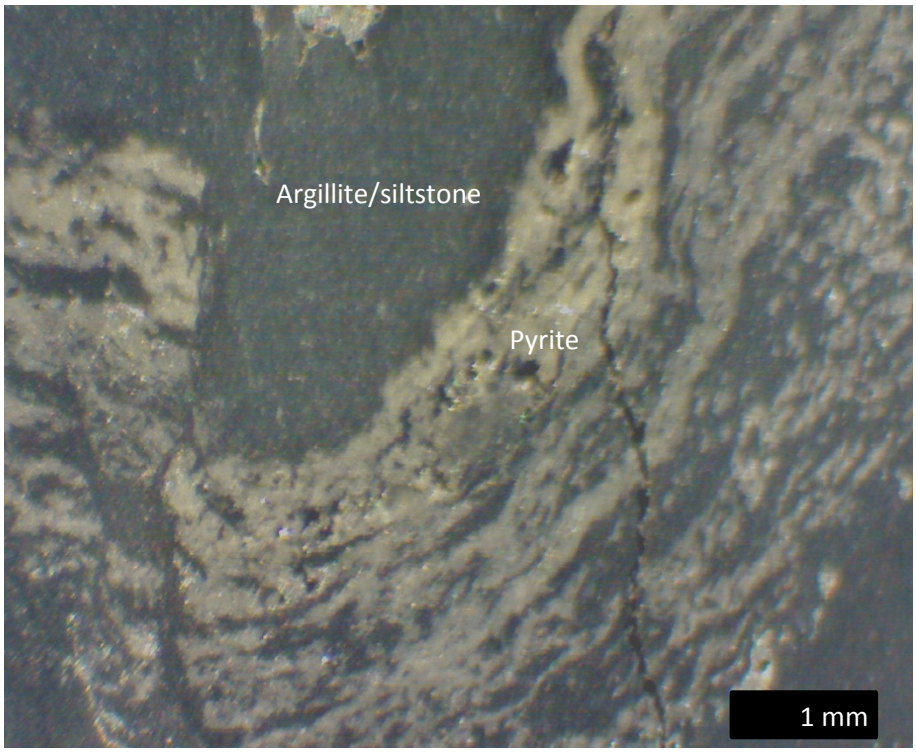
Stantec	Project No.: 165000734	GWP: 5023-09-00	Rockcore Photographs
	Project Name: Highway 144 Route Planning and Preliminary Design Study, Chelmsford to Dowling, ON		Date: March 21, 2013

Rock Core Photo No.: 1	Borehole:BH13-21A	Depth: 23.01 to 24.59 m

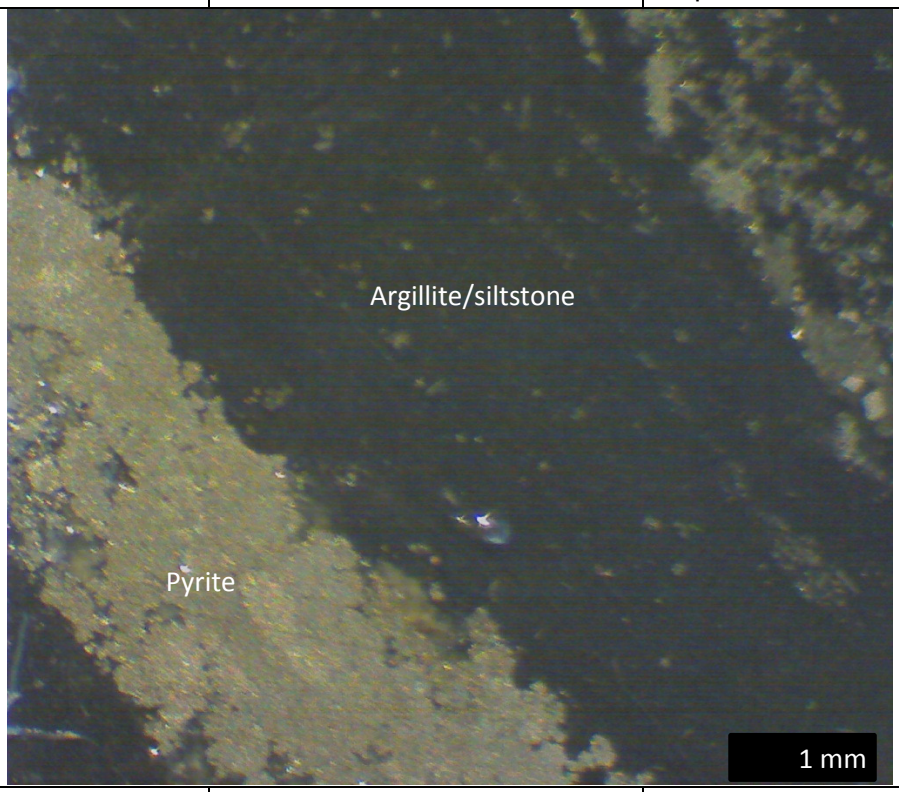


Rock Core Photo No.: 2	Borehole: BH13-22A	Depth: 11.84 to 13.46 m

Stantec	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.: 1	Borehole: BH13-21A	Depth: 23.25 m



Rock Core Photo No.: 2	Borehole: BH13-21A	Depth: 23.25 m

APPENDIX C

Laboratory Test Results

Figures 1 through 4: Grain Size Distribution Plots

Figure 5: Plasticity Chart

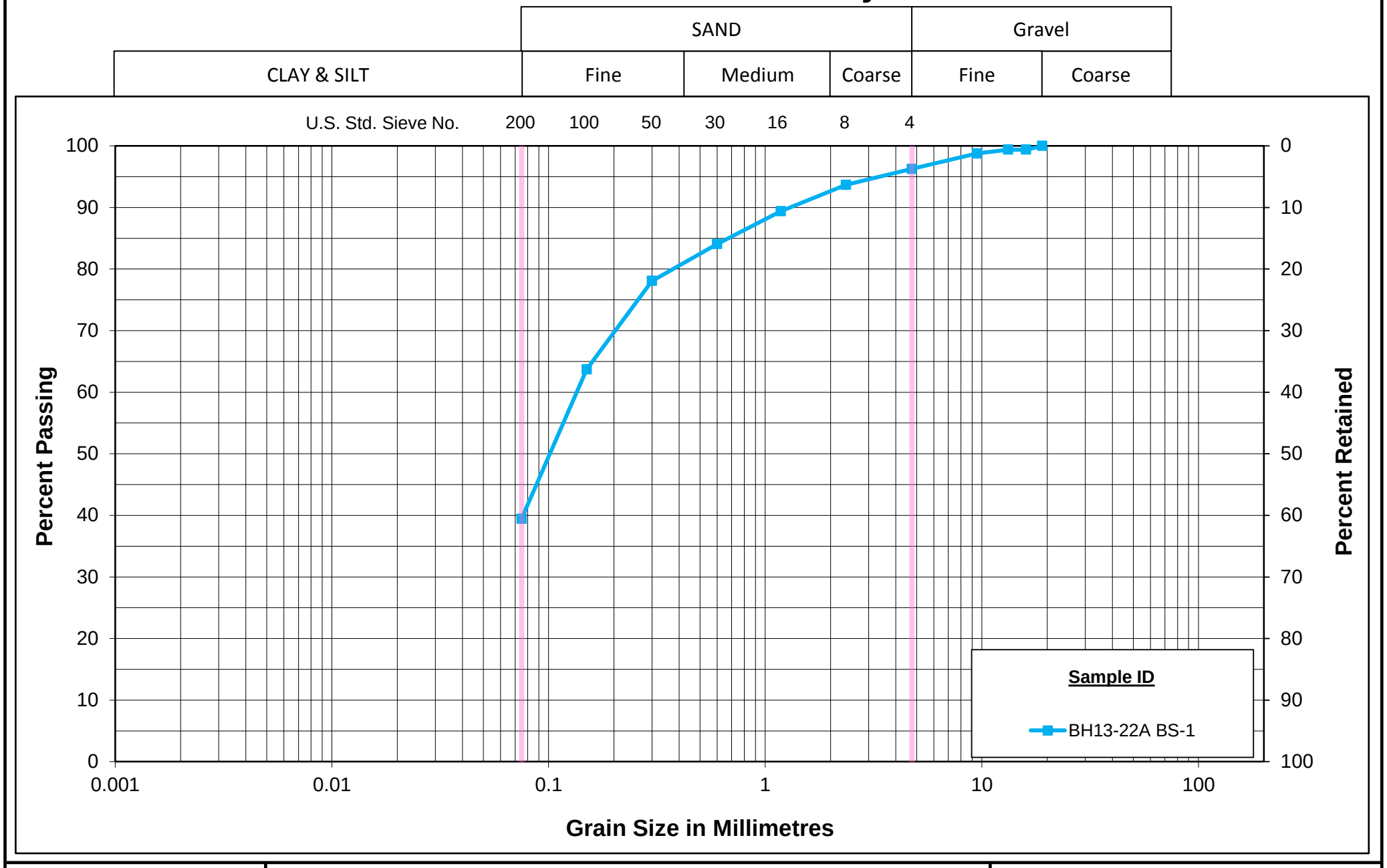
Laboratory Testing by Golder Associates

Unconfined Compression (soil)

Specific Gravity

Consolidation

Unified Soil Classification System



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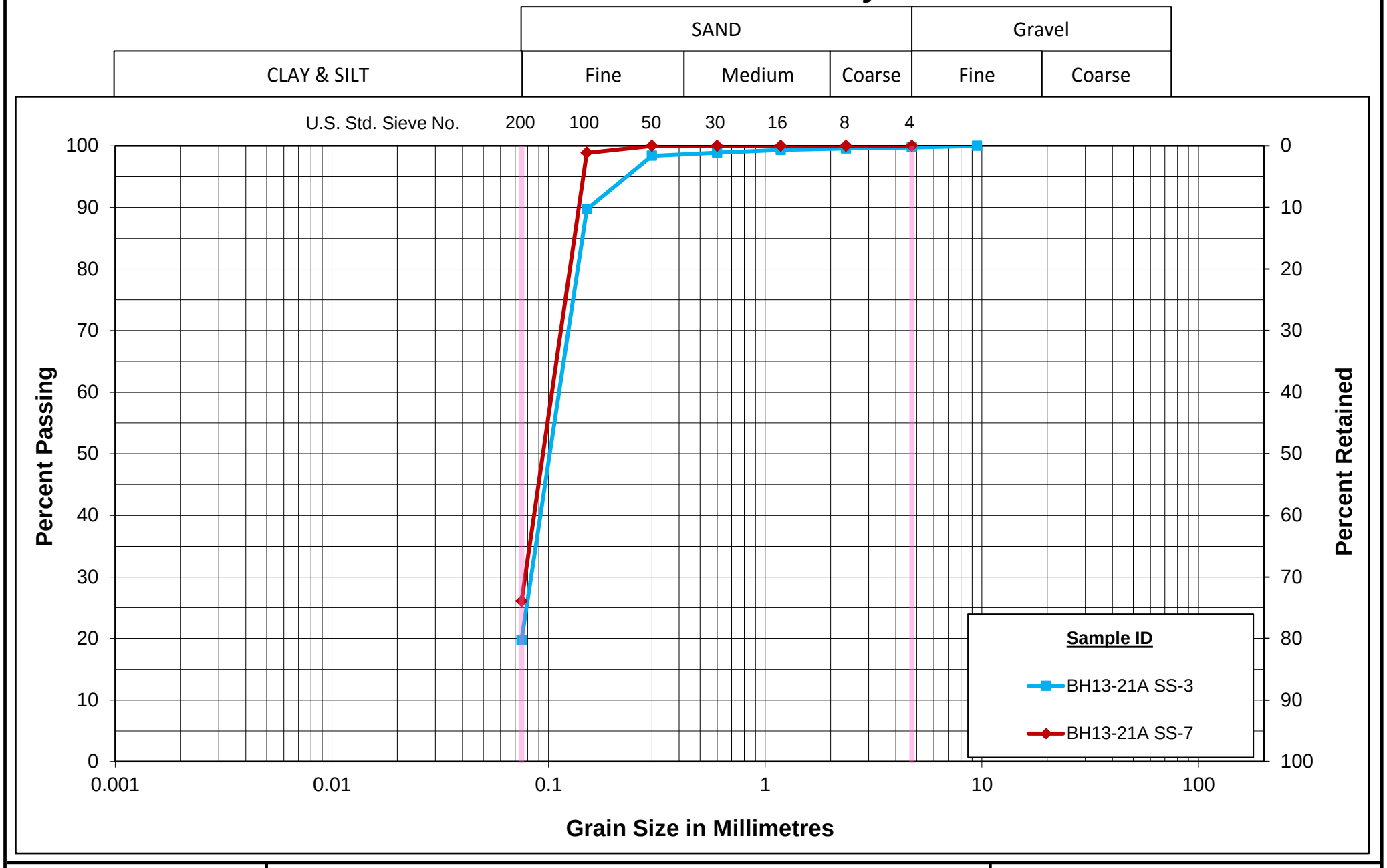
GRAIN SIZE DISTRIBUTION

FILL: Silty sand (SM)

Figure No. 1

Project No. 165000734
GWP 5023-09-00

Unified Soil Classification System



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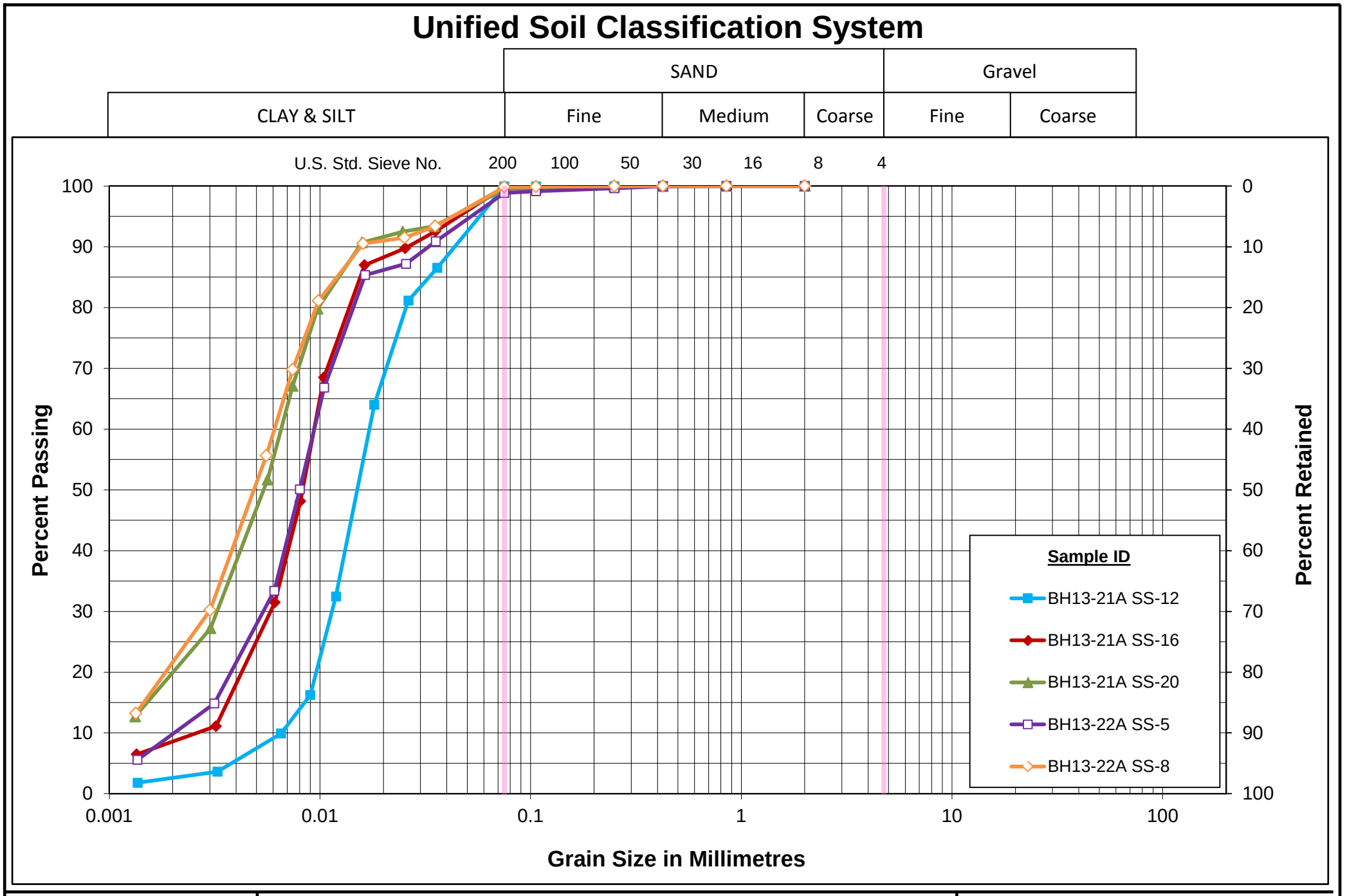
GRAIN SIZE DISTRIBUTION

Silty sand (SM)

Figure No. 2

Project No. 165000734
GWP 5023-09-00



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GRAIN SIZE DISTRIBUTION

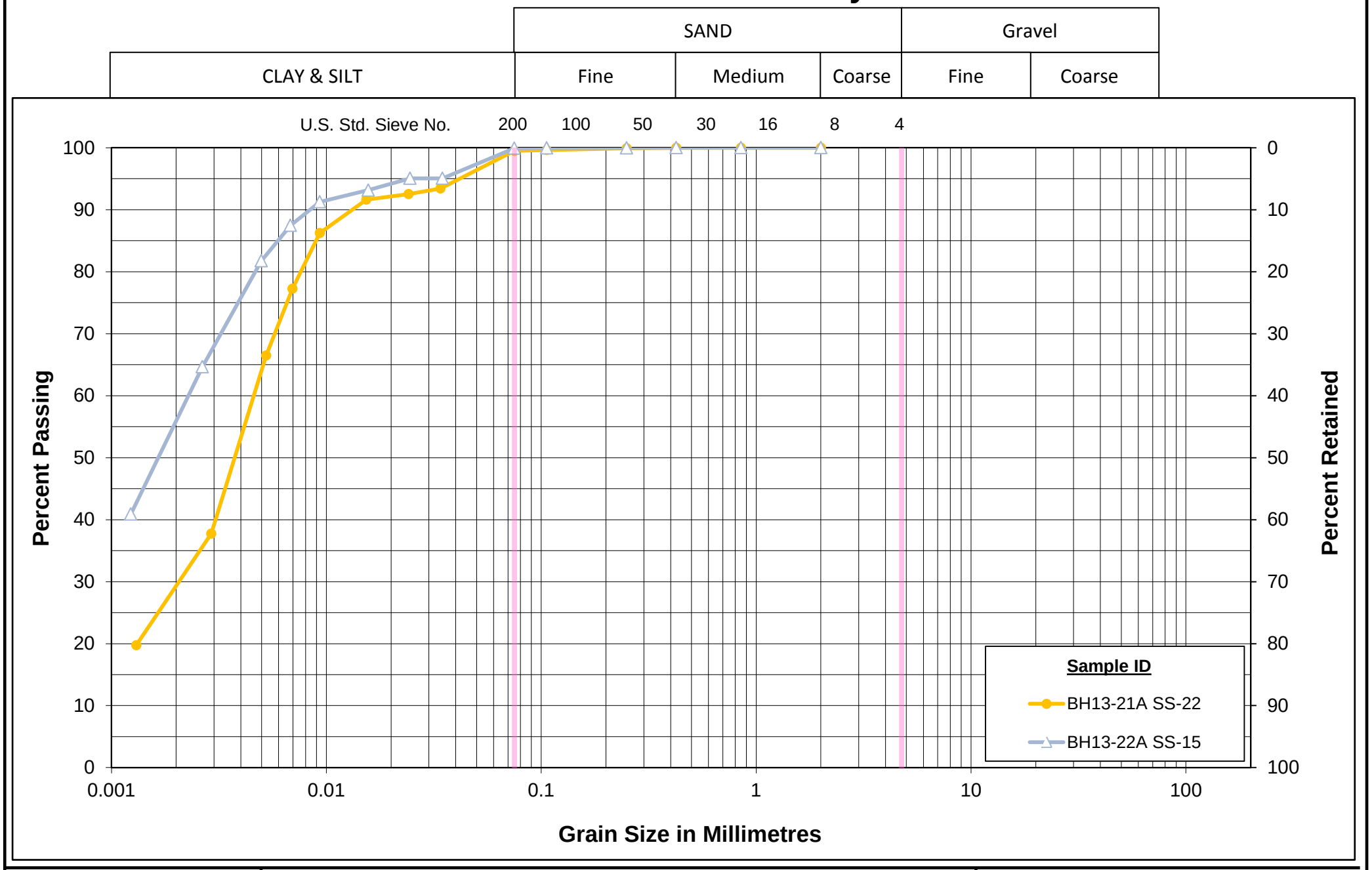
Silt (ML)

Figure No. 3

Project No. 165000734

GWP 5023-09-00

Unified Soil Classification System



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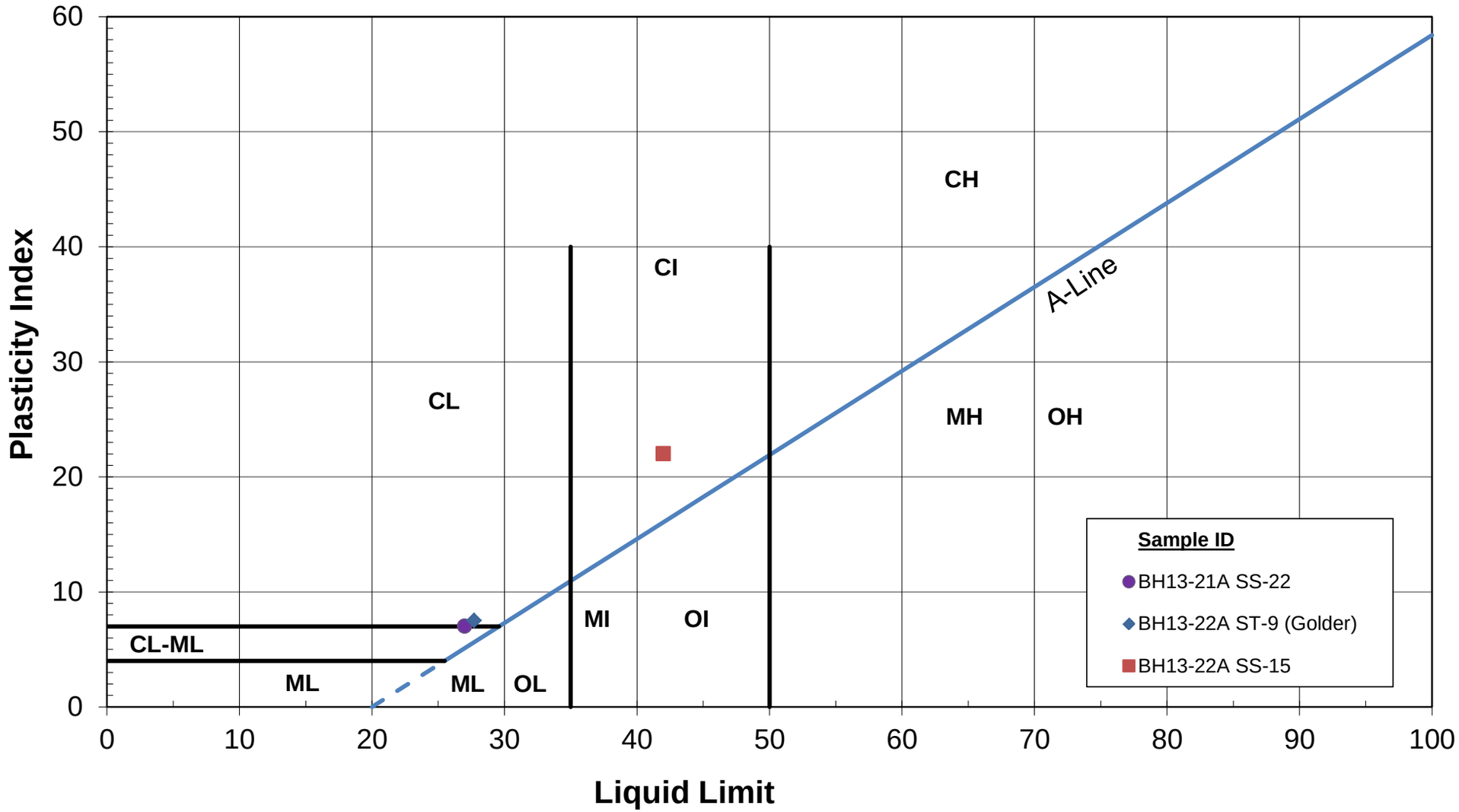
GRAIN SIZE DISTRIBUTION

Silty Clay (CI)

Figure No. 4

Project No. 165000734

GWP 5023-09-00



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PLASTICITY CHART

Figure No. 5

Project No. 165000734
GWP 5023-09-00

June 7, 2013

Project No. 13-1183-0043

165000734-307

Mr. Simon Gudina
Stantec Consulting
200-2781 Lancaster Road
Ottawa, Ontario
K1B 1A7

GEOTECHNICAL LABORATORY TESTING

Dear Sir

This letter reports the results of laboratory testing carried out on the samples received at our office in Mississauga. The results of the tests are summarized in the attached tables and figures.

The testing services reported herein have been performed in accordance with the indicated recognized standard, unless noted otherwise. This report is for the sole use of the designated client. This report constitutes a testing service only and does not represent any results interpretation or opinion regarding specification compliance or material suitability.

We trust that the results are sufficient for your current requirements. If you have any questions, please do not hesitate to call us.

GOLDER ASSOCIATES LTD.

Marijana Manojlovic
Laboratory Manager

MM/lg

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UNCONFINED COMPRESSION TEST (UC)

ASTM D 2166 - 06

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0043	SAMPLE NUMBER	ST-9
BOREHOLE NUMBER	13-22A	SAMPLE DEPTH, m	6.98-6.71

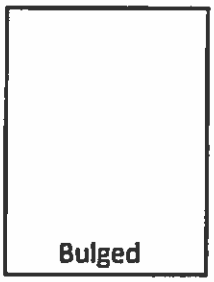
TEST CONDITIONS

MACHINE SPEED, mm/min	1.50	TYPE OF SPECIMEN	Thin wall tube sample
RATE OF AXIAL STRAIN, %/min	1.06	L/D	2.06

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	14.14	WATER CONTENT, (specimen) %	30.71
SAMPLE DIAMETER, cm	6.86	UNIT WEIGHT, kN/m^3	19.47
SAMPLE AREA, cm^2	36.95	DRY UNIT WT., kN/m^3	14.90
SAMPLE VOLUME, cm^3	522.47	SPECIFIC GRAVITY, measured	2.72
WET WEIGHT, g	1037.90	VOID RATIO	0.79
DRY WEIGHT, g	794.06		

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	14.7	COMPRESSIVE STRESS, kPa	180

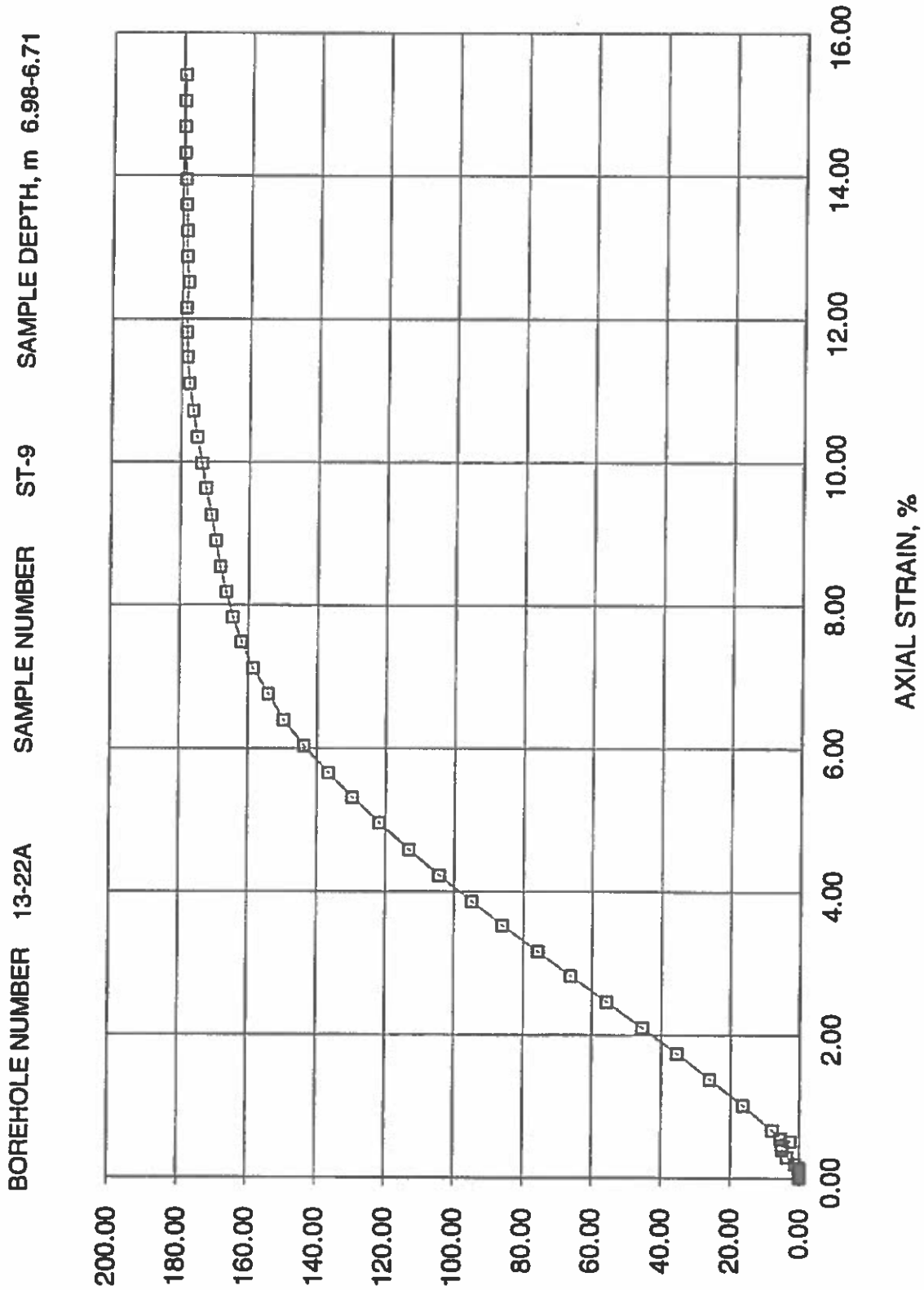
REMARKS: Sample taken 9-24 cm from the bottom of the tube

DATE: April 4, 2013

Checked By: MM

Golder Associates

UNCONFINED COMPRESSION TEST (UC)	FIGURE



Project No.13-1183-0043

Checked By:

SPECIFIC GRAVITY TEST RESULTS

ASTM D 854-06 TEST METHOD A

PROJECT NUMBER	13-1183-0043
PROJECT NAME	Stantec / Testing / 165000734-307
DATE TESTED	April, 2013

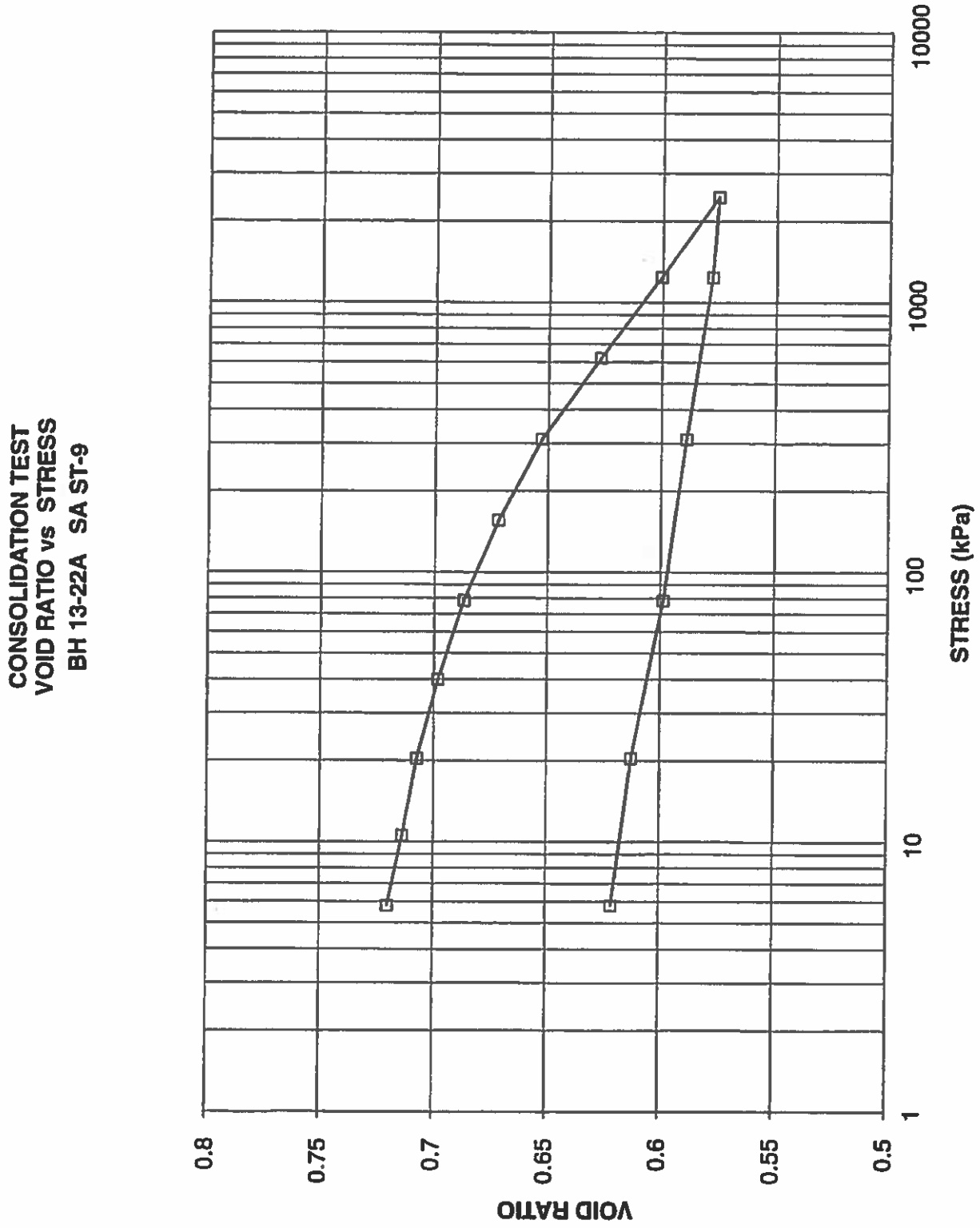
Borehole No.	Sample No.	Specific Gravity
13-22A	ST-9	2.72

Note: Test carried out on soil particles <2.00mm using distilled water.

Checked By: MH

Golder Associates

CONSOLIDATION TEST VOID RATIO VS LOG STRESS	FIGURE



Project No. 13-1183-0043

Prepared By: RD

Golder Associates

Checked By: MU

CONSOLIDATION TEST SUMMARY

FIGURE

SAMPLE IDENTIFICATION

Project Number	13-1183-0043	Sample Number	ST-9
Borehole Number	13-22A	Sample Depth, m	6.10-6.71

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	11		
Date Started	3/28/2013		
Date Completed	4/10/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m^3	19.62
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.43
Area, cm^2	31.70	Specific Gravity, measured	2.72
Volume, cm^3	80.23	Solids Height, cm	1.464
Water Content, %	27.13	Volume of Solids, cm^3	46.42
Wet Mass, g	160.53	Volume of Voids, cm^3	33.81
Dry Mass, g	126.27	Degree of Saturation, %	101.3

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.531	0.728	2.531				
5.81	2.519	0.720	2.525	105	1.29E-02	8.16E-04	1.03E-06
10.56	2.510	0.714	2.514	540	2.48E-03	7.90E-04	1.92E-07
20.38	2.500	0.707	2.505	265	5.02E-03	3.74E-04	1.84E-07
39.96	2.487	0.698	2.494	290	4.55E-03	2.66E-04	1.19E-07
78.43	2.470	0.687	2.479	327	3.98E-03	1.72E-04	6.69E-08
155.62	2.448	0.672	2.459	240	5.34E-03	1.13E-04	5.90E-08
310.37	2.421	0.653	2.434	217	5.79E-03	7.07E-05	4.01E-08
619.47	2.383	0.627	2.402	427	2.86E-03	4.79E-05	1.35E-08
1236.34	2.344	0.601	2.364	277	4.28E-03	2.50E-05	1.05E-08
2456.30	2.307	0.575	2.326	437	2.62E-03	1.19E-05	3.06E-09
1236.34	2.311	0.578	2.309				
310.37	2.327	0.589	2.319				
78.43	2.341	0.599	2.334				
20.38	2.362	0.613	2.352				
5.81	2.374	0.621	2.368				

Note:
Consolidation loading and unloading schedule assigned by the client.
Specimen taken 24-33cm from bottom of the tube
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

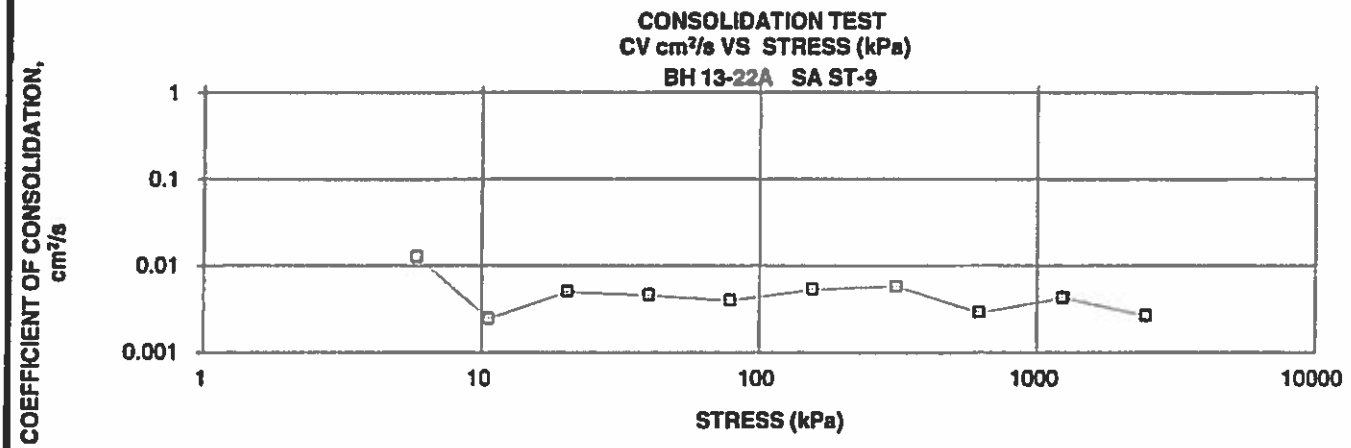
Sample Height, cm	2.37	Unit Weight, kN/m^3	20.21
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	16.45
Area, cm^2	31.70	Specific Gravity, measured	2.72
Volume, cm^3	75.26	Solids Height, cm	1.464
Water Content, %	22.85	Volume of Solids, cm^3	46.42
Wet Mass, g	155.12	Volume of Voids, cm^3	28.84
Dry Mass, g	126.27		

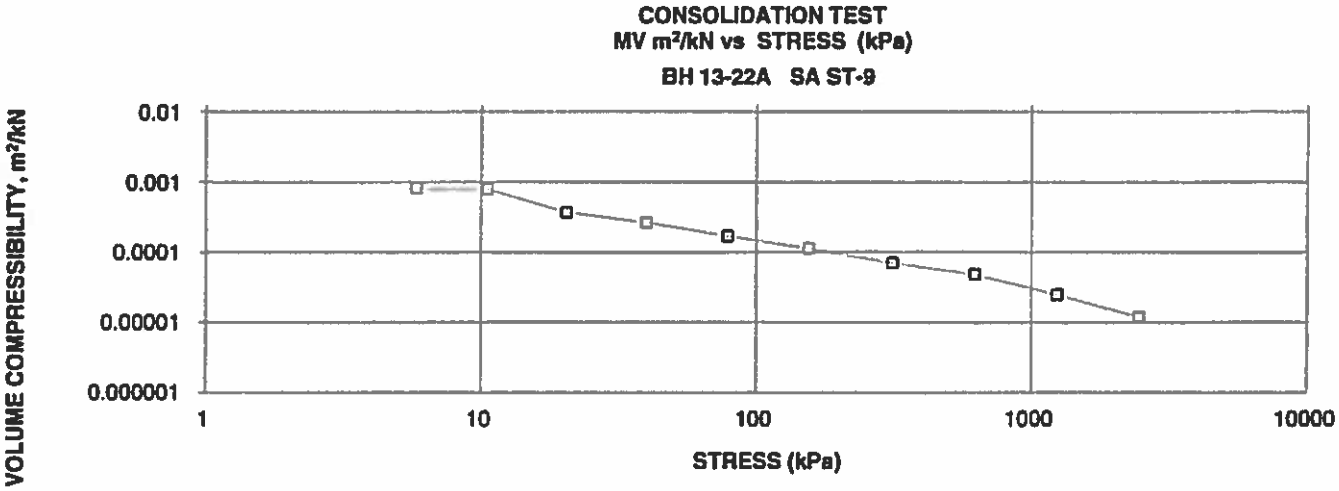
Prepared By: RD

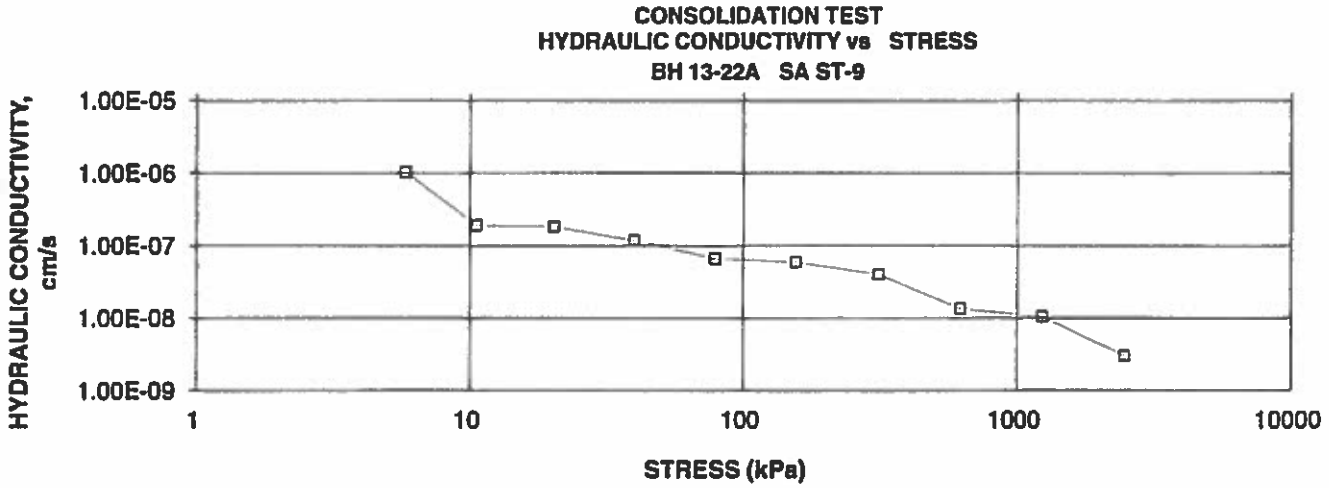
Golder Associates

Checked By: [illegible]

CONSOLIDATION TEST SUMMARY	FIGURE







Project No.
Prepared By: RD

Golder Associates

Checked By:

APPENDIX D

Figures 6a & 6b: Preliminary Design Parameters

Figure 7: Liquefaction Assessment

Preliminary LPILE Analysis Results for East and West Abutments

Figure 8: Lateral Deflection of HP310x110 – East Abutment

Figure 9: P-y Curves for HP310x310 – East Abutment

Figure 10: Lateral Deflection of HP310x110 – West Abutment

Figure 11: P-y Curves for HP310x310 – West Abutment

Preliminary Slope Stability Results for East and West Abutments

Figure 12a & 13a: Static (long-term)

Figure 12b & 13b: Static (short-term)

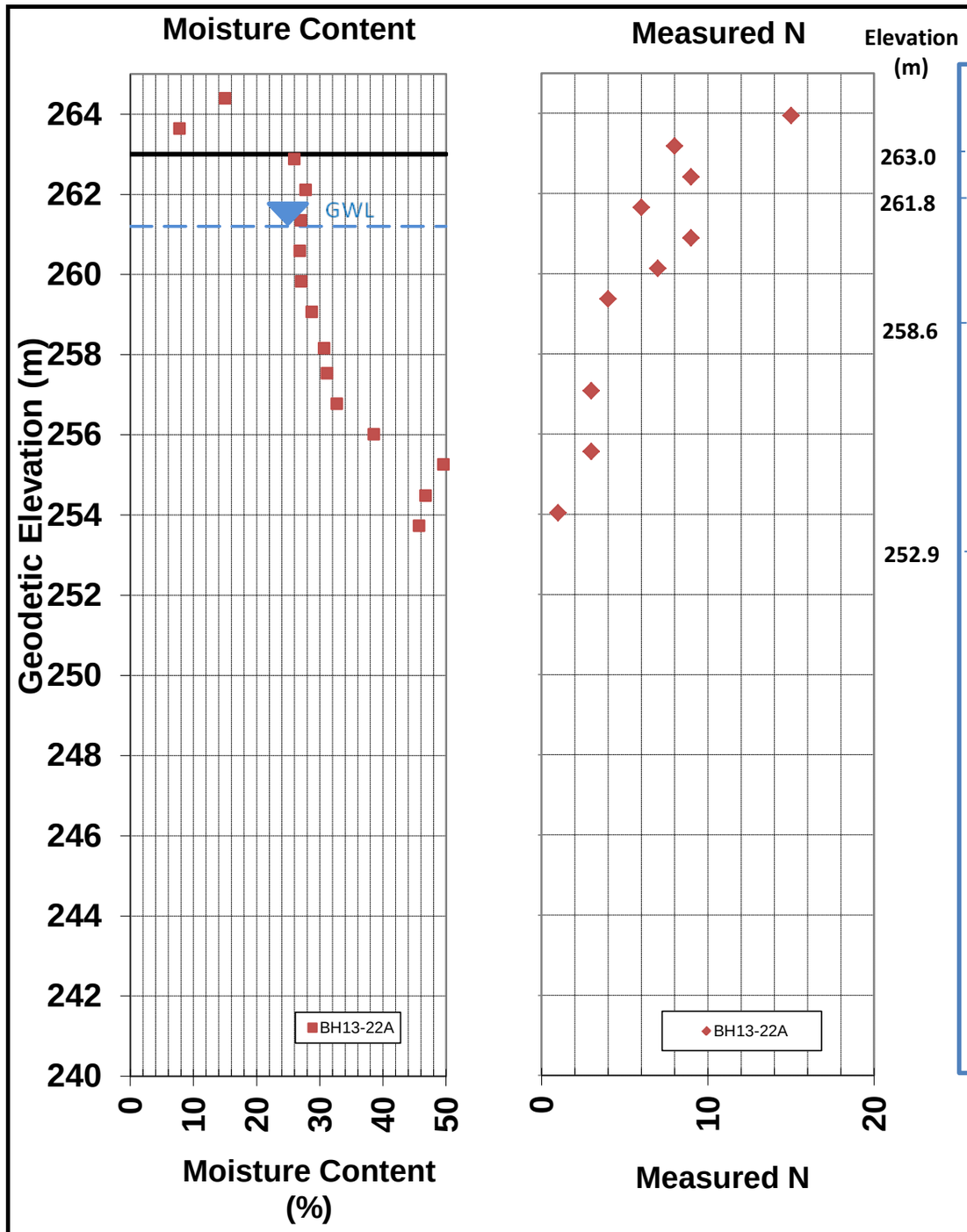
Figure 12c & 13c: Seismic

Preliminary Settlement Analysis

Figure 14a: Preliminary Settlement Results for East Approach Embankment

Figure 14b: Preliminary Settlement Results for West Approach Embankment

Table D-1: Spring Stiffnesses for HP310x110



Design Parameters

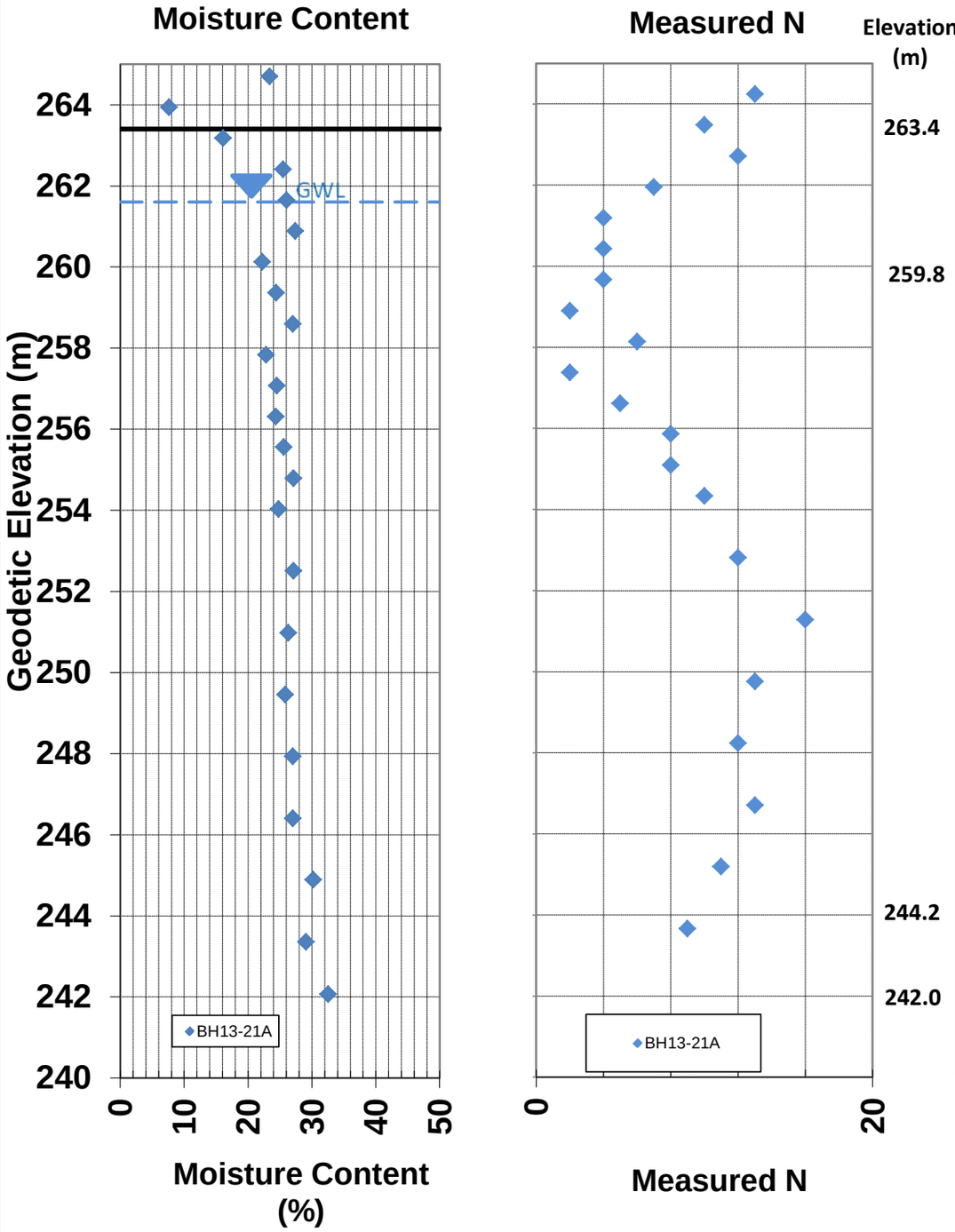
Approximate ground surface elevation at east abutment = 263 m

Silty sand	γ = 20 kN/m³, ϕ = 30°; E = 15 MPa
Silt	γ = 19.6 kN/m³ ϕ = 28° E = 7 MPa
Silty clay	γ = 19.6 kN/m³, C_c= 0.2, C_r= 0.03 S_u = 90 kPa, P'c = 400 kPa
	γ = 17.3 kN/m³, C_c= 0.4, C_r= 0.06 S_u = 60 kPa, P'c = 270 kPa
Bedrock	

Note: The strata boundaries at the east abutment were based on the information obtained from BH13-22A.

- P'c estimated as S_u/0.22
- C_c and C_r estimated based on moisture content

Stantec Consulting Ltd.	Project No. 165000734 Whitson River Crossing East Abutment GWP No. 5023-09-00	Figure 6a



Design Parameters

Approximate ground surface elevation at west abutment = 263.3 m

Silty sand	γ = 20 kN/m^3 ϕ = 30° E = 15 MPa
Silt	γ = 19.6 kN/m^3 ϕ = 28° E = 5 MPa
Silt	γ = 19.6 kN/m^3 ϕ = 28° E = 10 MPa
Silty clay	γ = 19.6 kN/m^3, C_c= 0.26, C_r= 0.03 S_u = 65 kPa, P'c = 270 kPa
Bedrock	

Note: The strata boundaries at the west abutment were basd on the information obtained from BH13-21A.

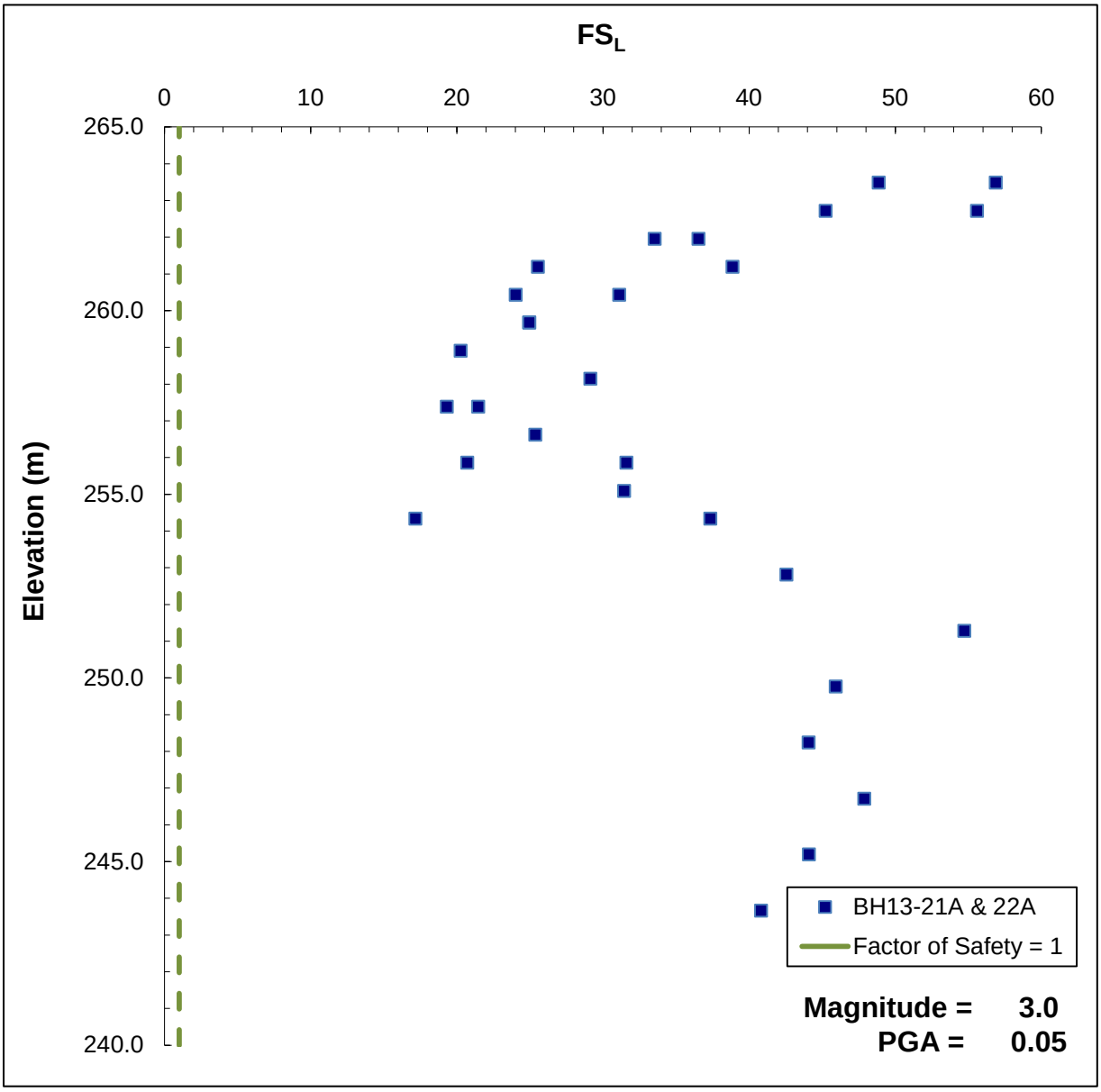
Stantec Stantec Consulting Ltd.	Project No. 165000734 Whitson River Crossing West Abutment GWP No. 3070-09-00	Figure 6b

Stantec Consulting Ltd.

Figure 7

CHARACTERIZATION OF LIQUEFACTION RESISTANCE

Whitson River Crossing



GWP 5023-09-00

FS_L = Factor of Safety against Liquefaction

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

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

Job No. 165000734

V:\01224\active\other_pc_projects\165000734\Reports\Site 8 - Whitson River Crossing\Figures\Fig07 - N-Value Liquifaction CRR - CSR (NCEER 1996)11.xlsx

LPile Results - Lateral Deflection

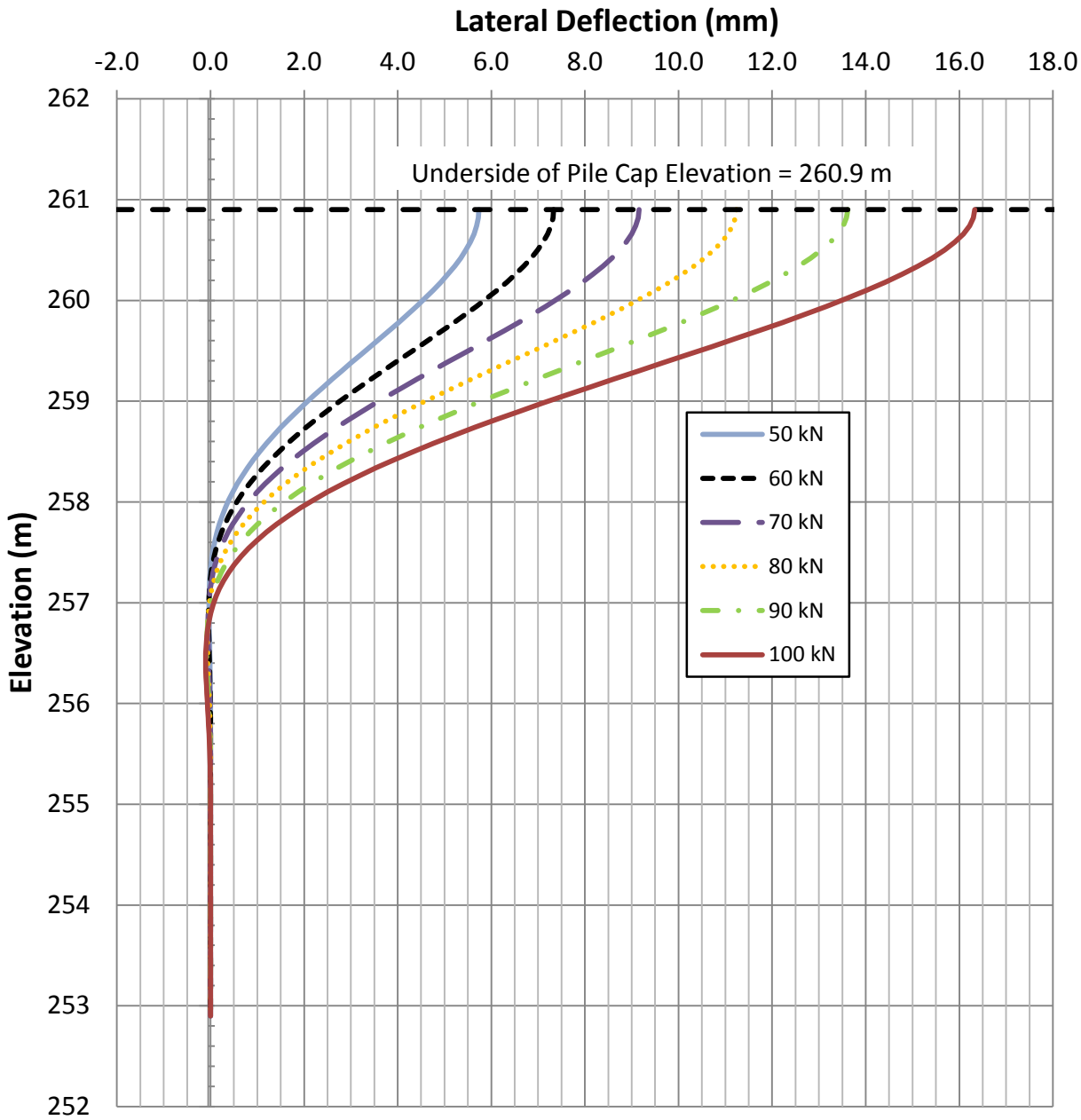


Figure 8
Lateral Deflection of HP 310x110 Piles

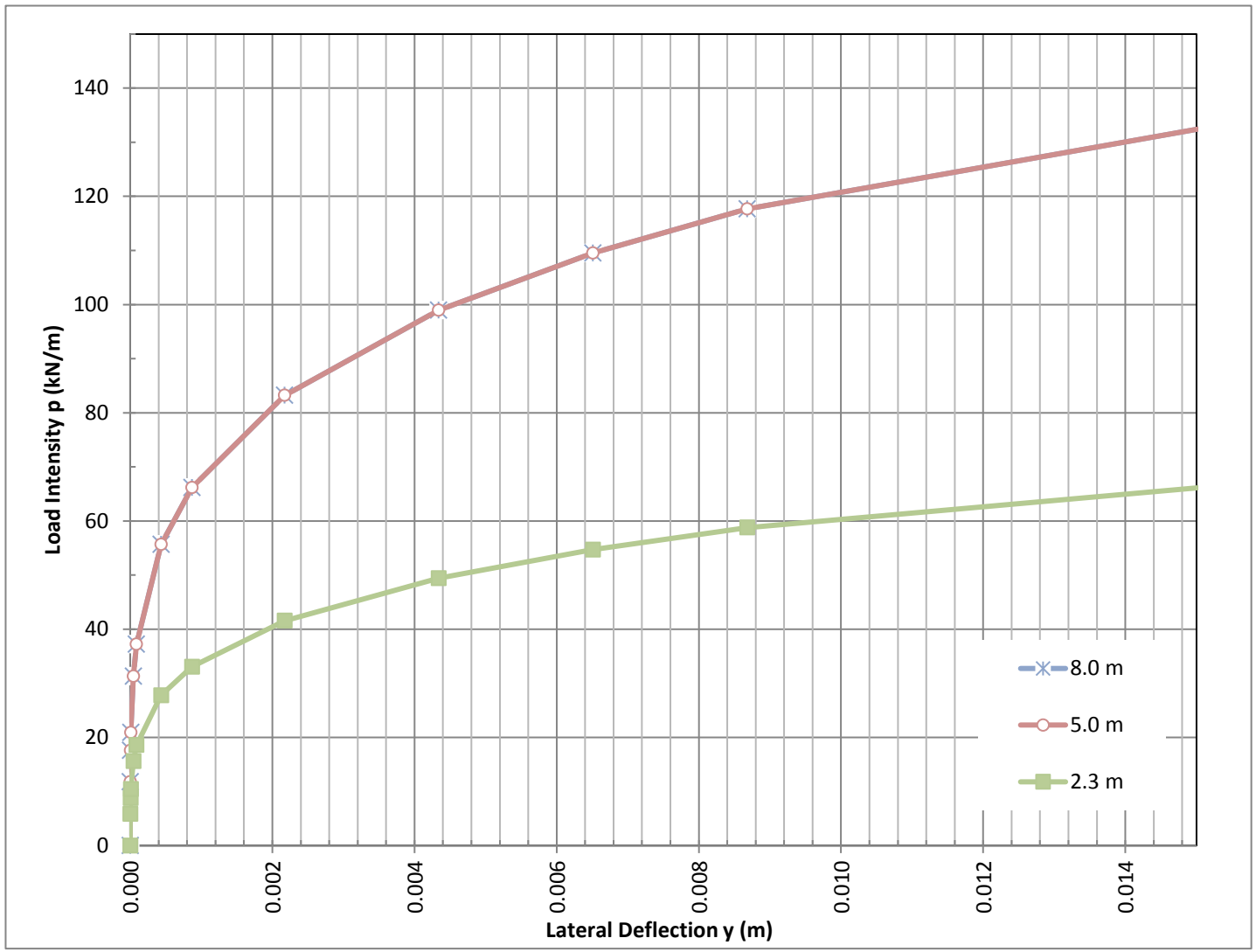


Figure 9
p-y Curves for Proposed HP 310x110 Piles

LPile Results - Lateral Deflection

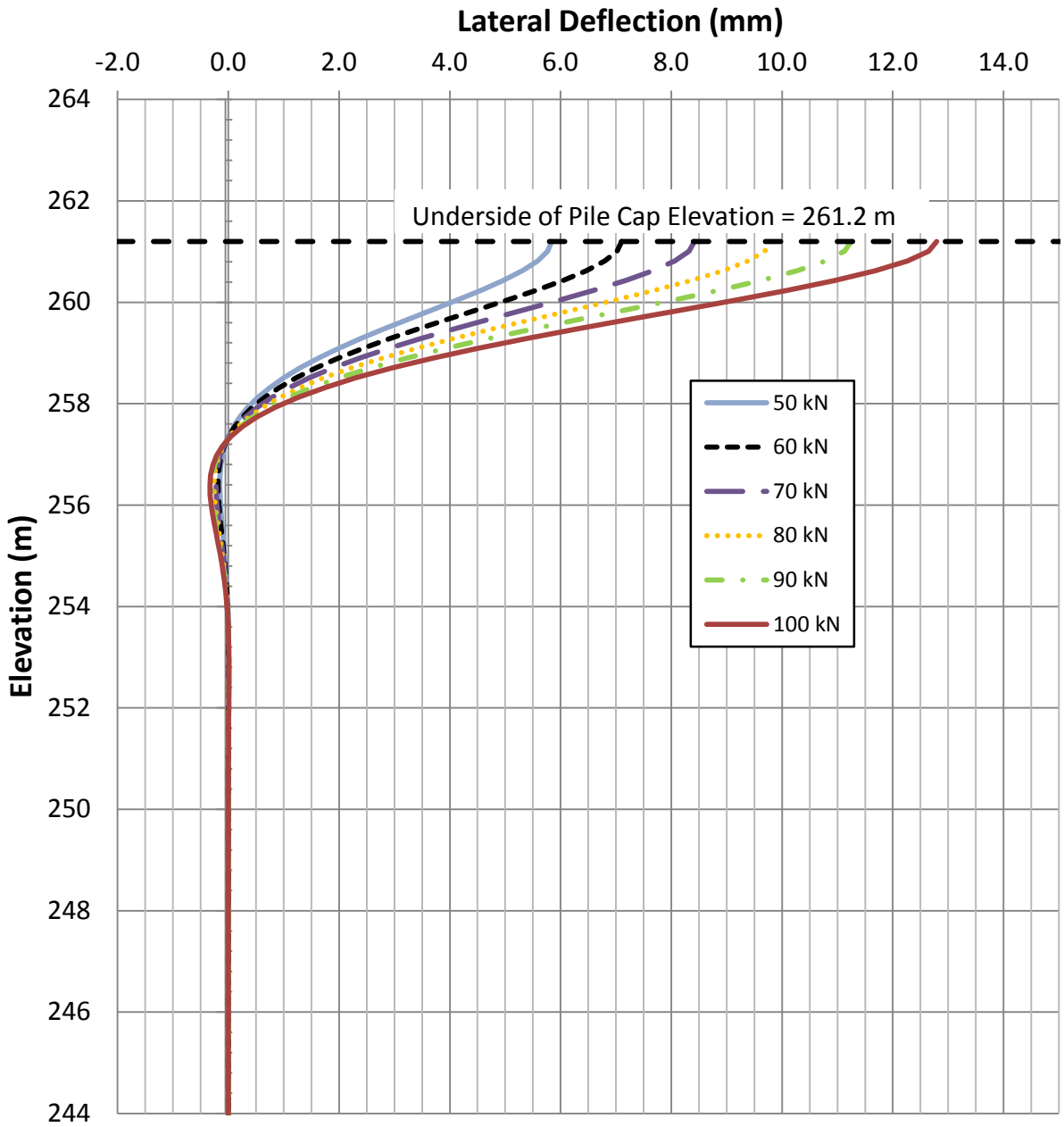


Figure 10
Lateral Deflection of HP 310x110 Piles

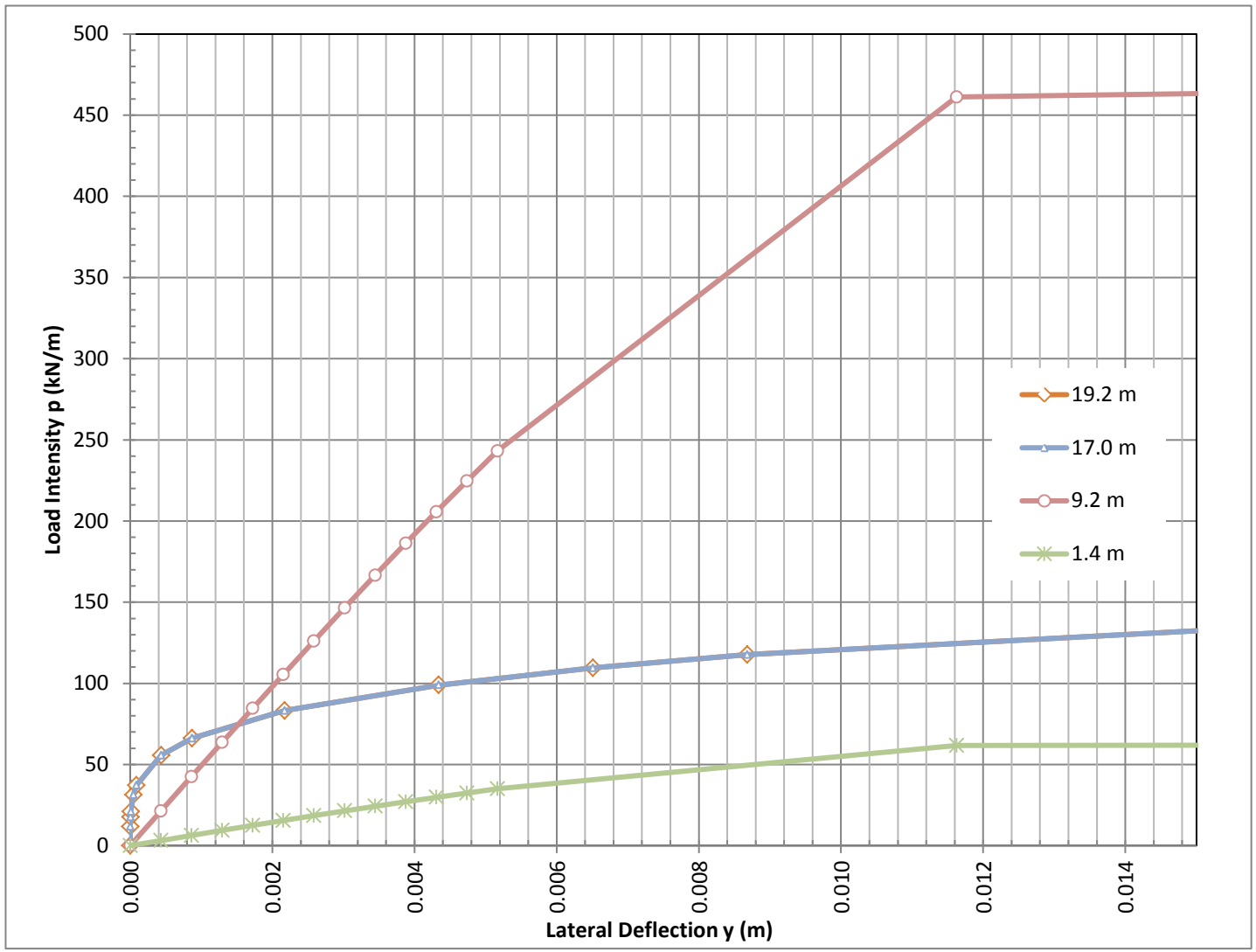
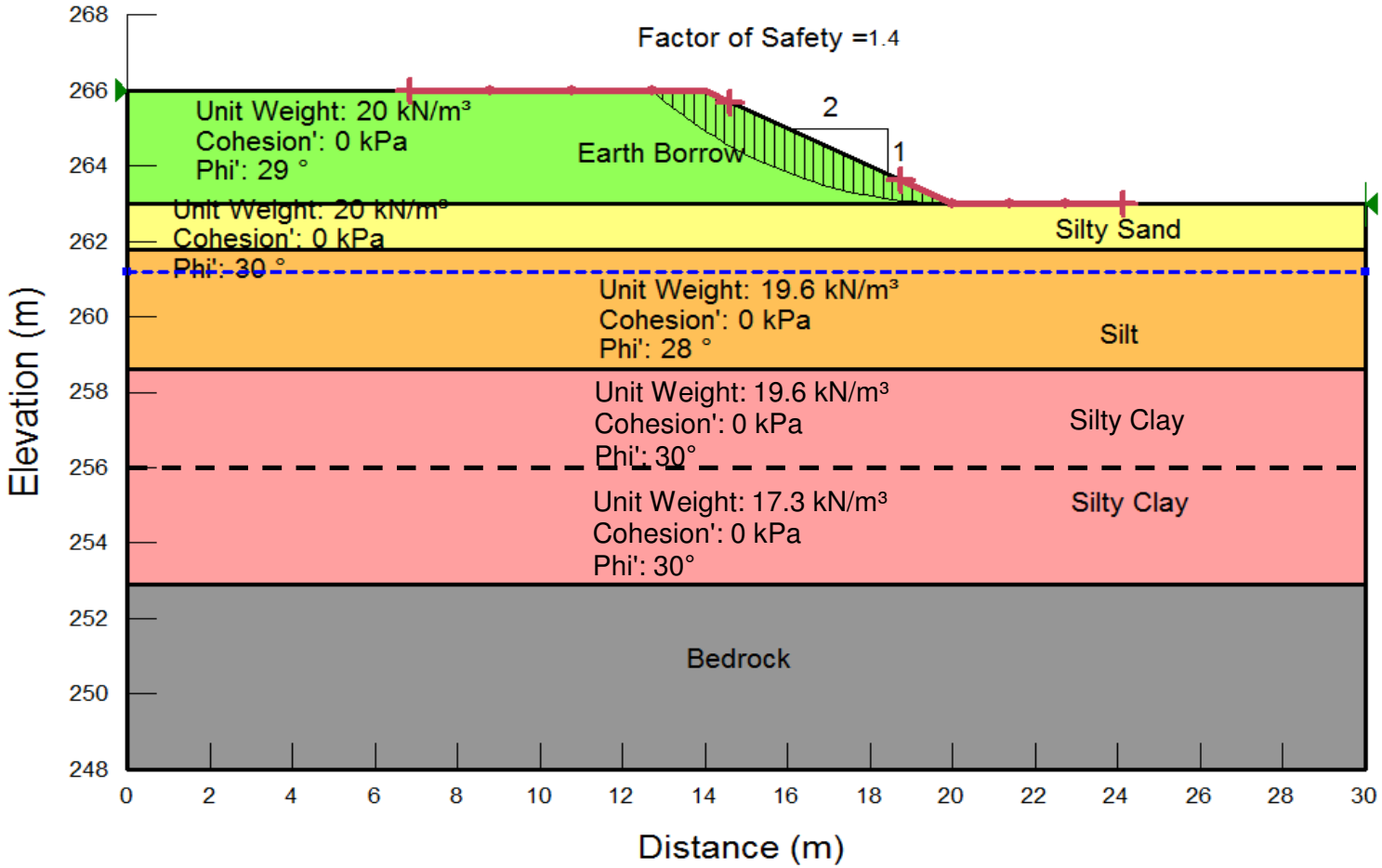


Figure 11
p-y Curves for Proposed HP 310x110 Piles



Stantec

Static Slope Stability Analysis (Drained)

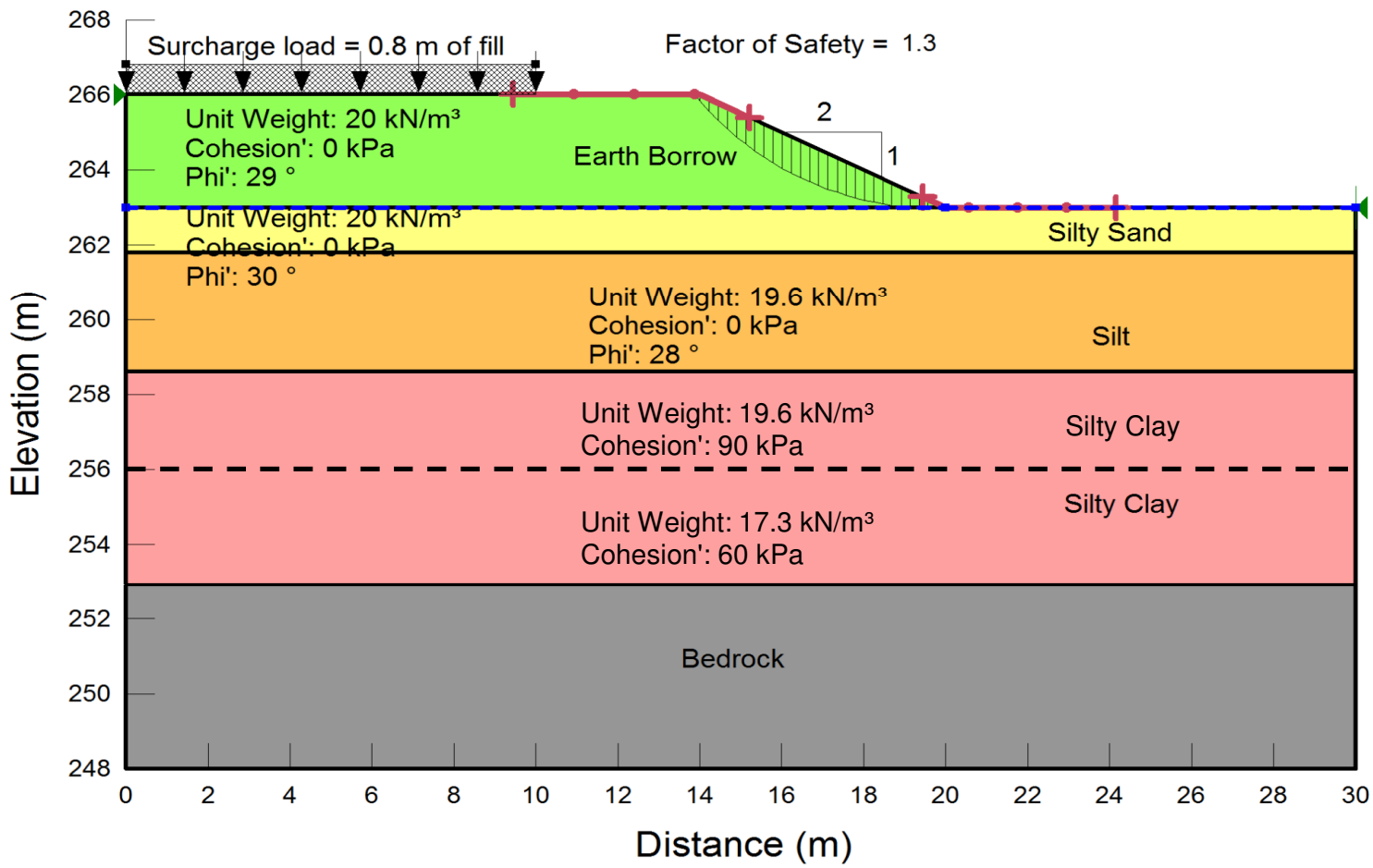
Highway 144 Chelmsford Bypass

Whitson River Crossing - East Abutment

Figure 12a

Project No. 165000734

GWP No. 5023-09-00



Stantec

Static Slope Stability Analysis (Undrained)

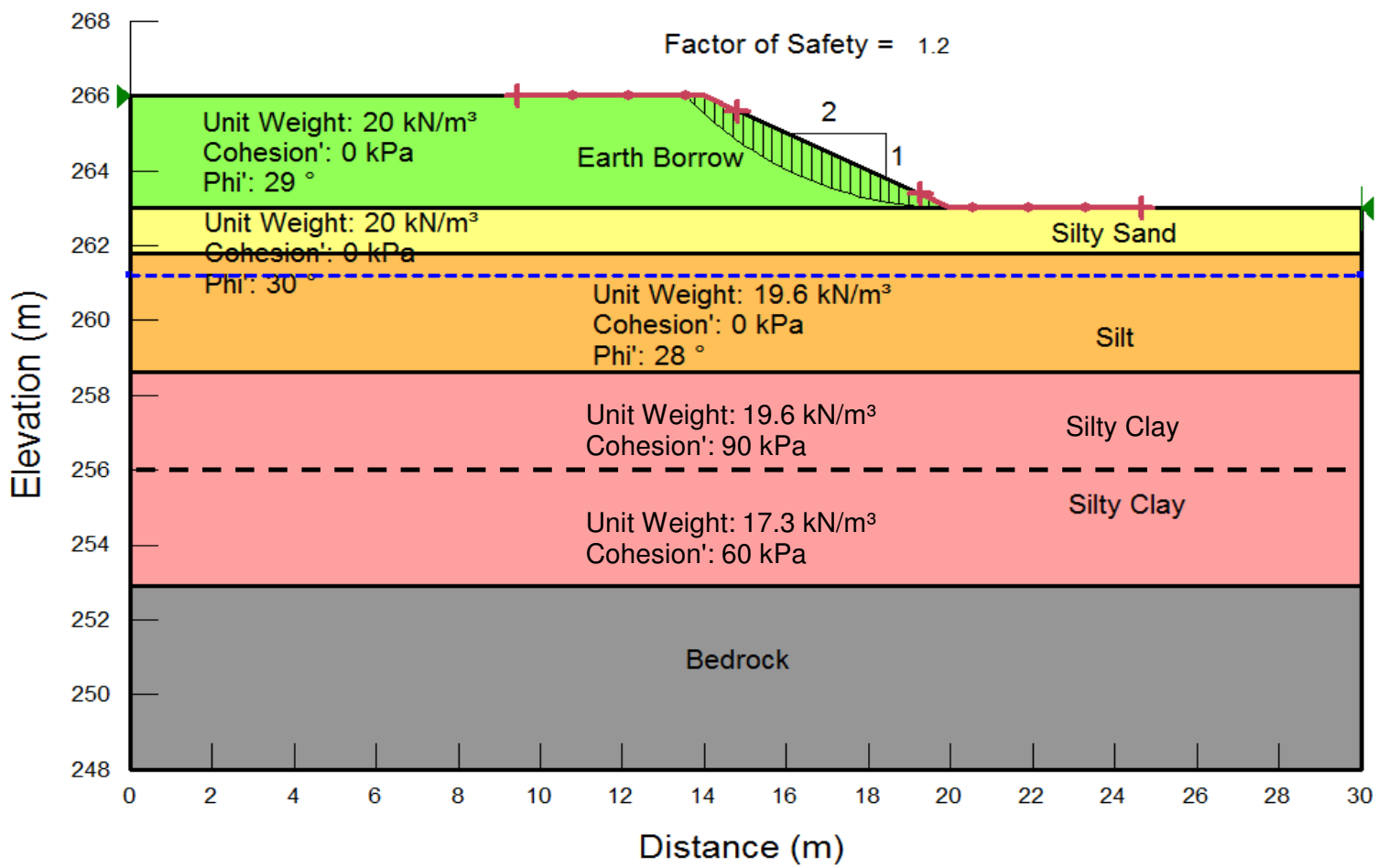
Highway 144 Chelmsford Bypass

Whitson River Crossing - East Abutment

Figure 12b

Project No. 165000734

GWP No. 5023-09-00



Stantec

Seismic Slope Stability Analysis

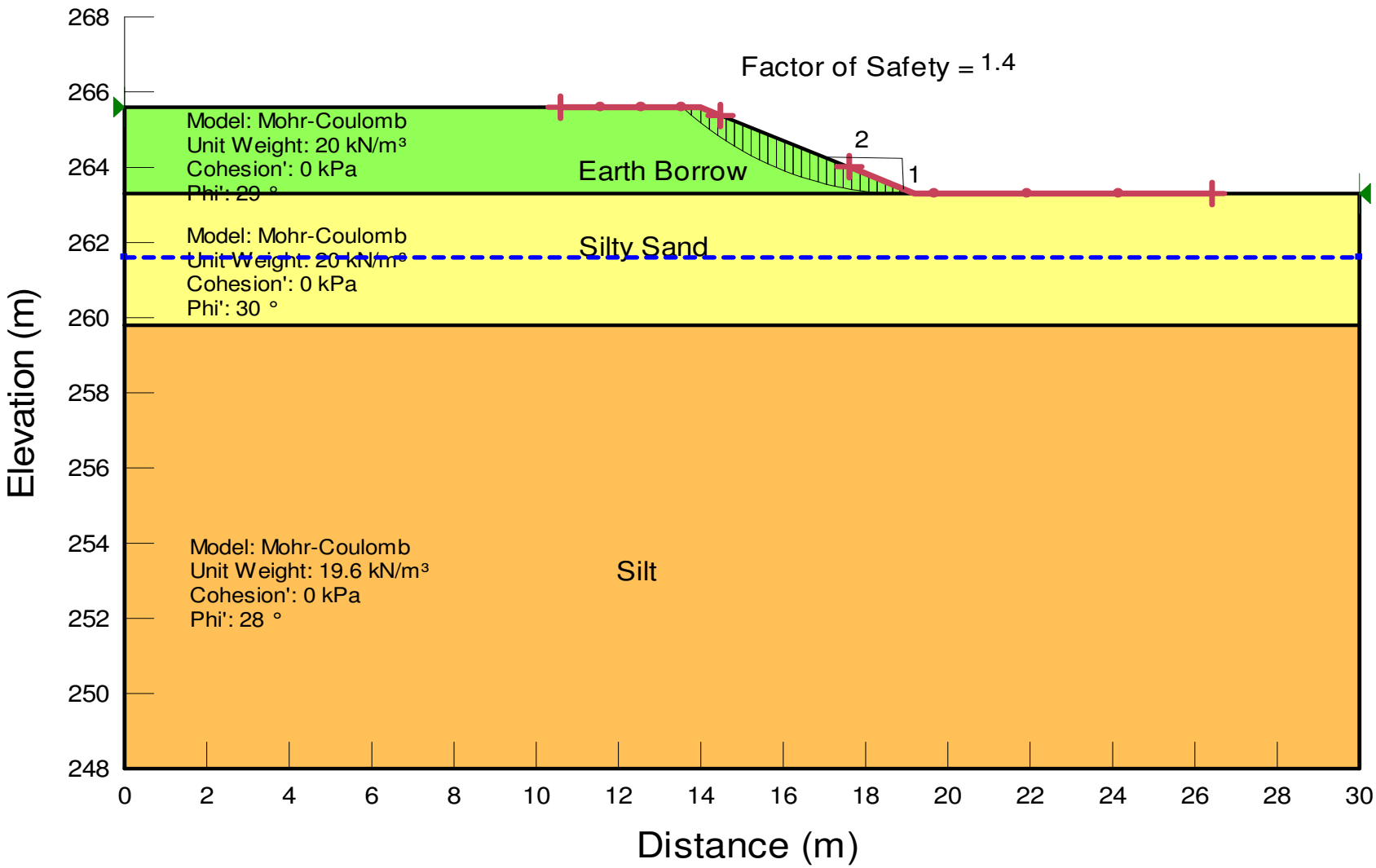
Highway 144 Chelmsford Bypass

Whitson River Crossing - East Abutment

Figure 12c

Project No. 165000734

GWP No. 5023-09-00



Stantec

Static Slope Stability Analysis (Drained)

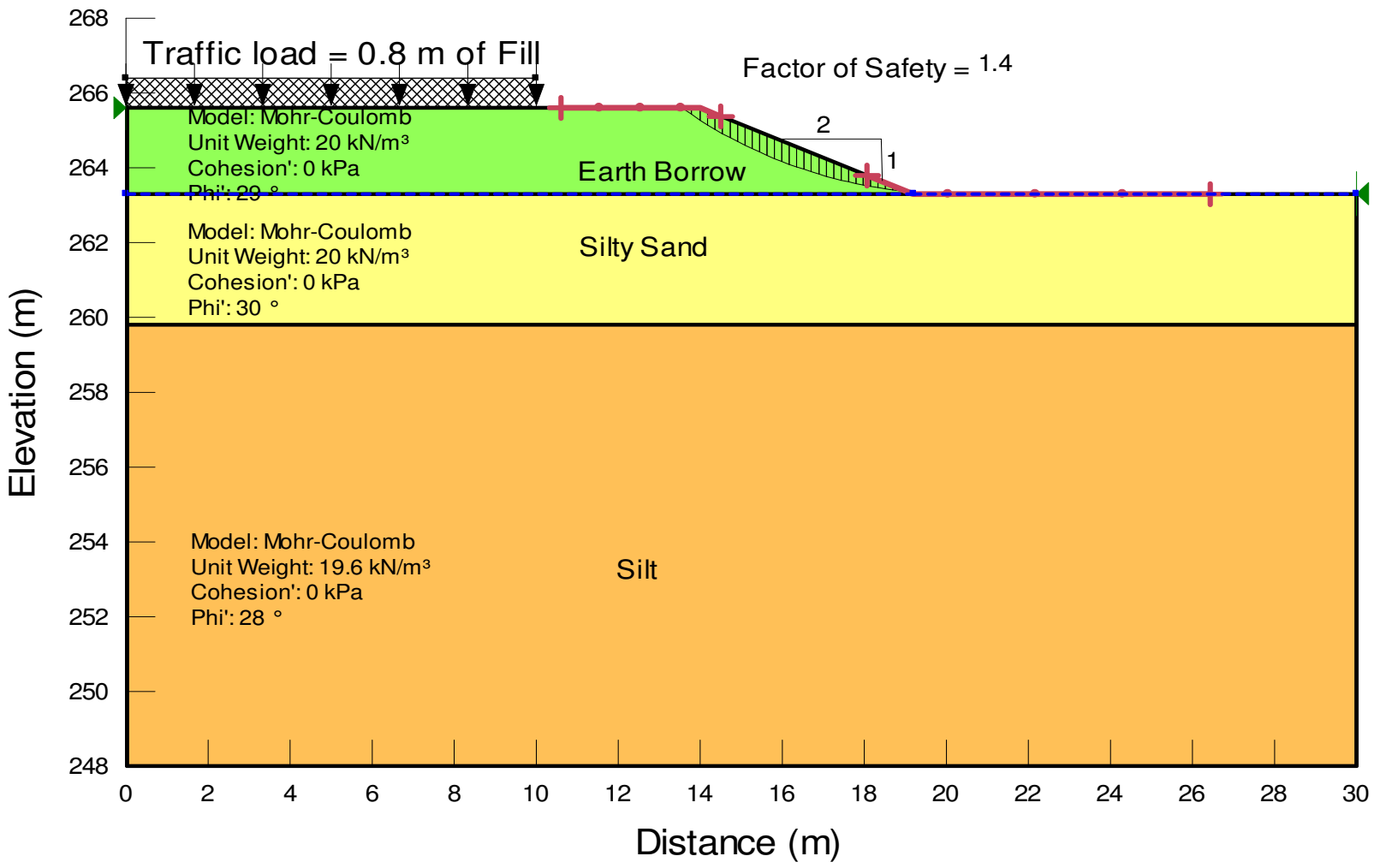
Highway 144 Chelmsford Bypass

Whitson River Crossing - West Abutment

Figure 13a

Project No. 165000734

GWP No. 5023-09-00



Stantec

Static Slope Stability Analysis (Undrained)

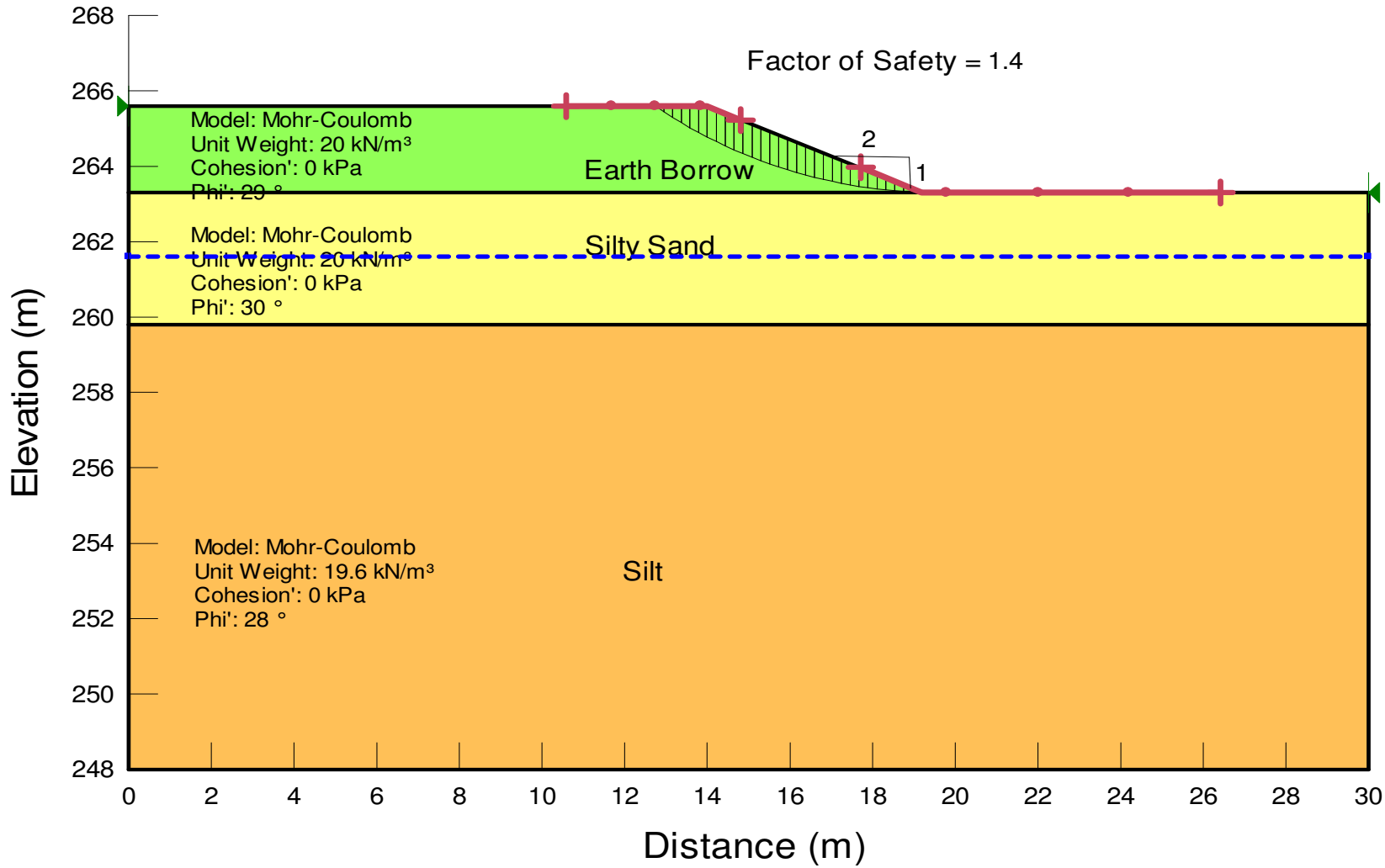
Highway 144 Chelmsford Bypass

Whitson River Crossing - West Abutment

Figure 13b

Project No. 165000734

GWP No. 5023-09-00



Stantec

Seismic Slope Stability Analysis

Highway 144 Chelmsford Bypass

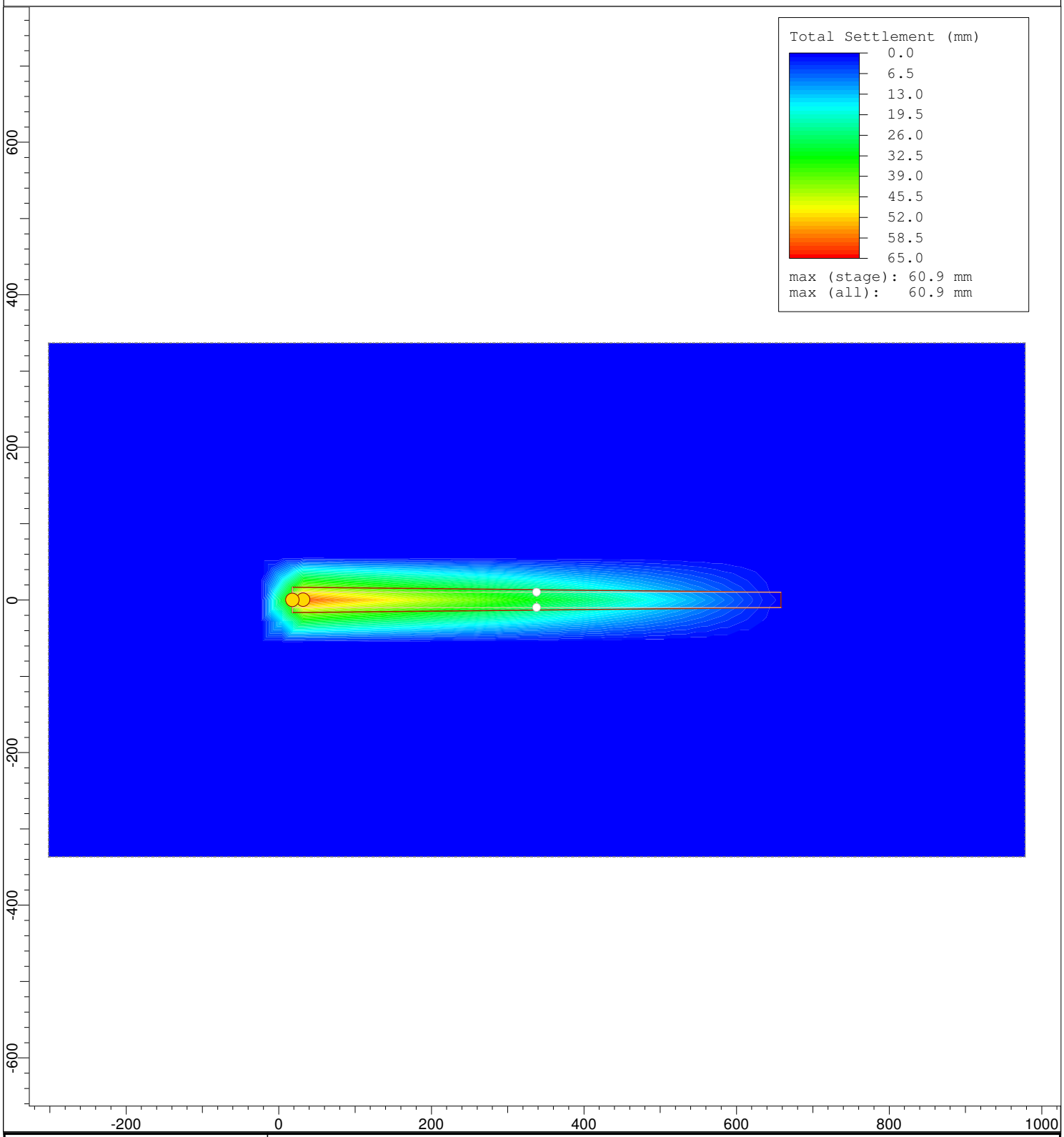
Whitson River Crossing - West Abutment

Figure 13c

Project No. 165000734

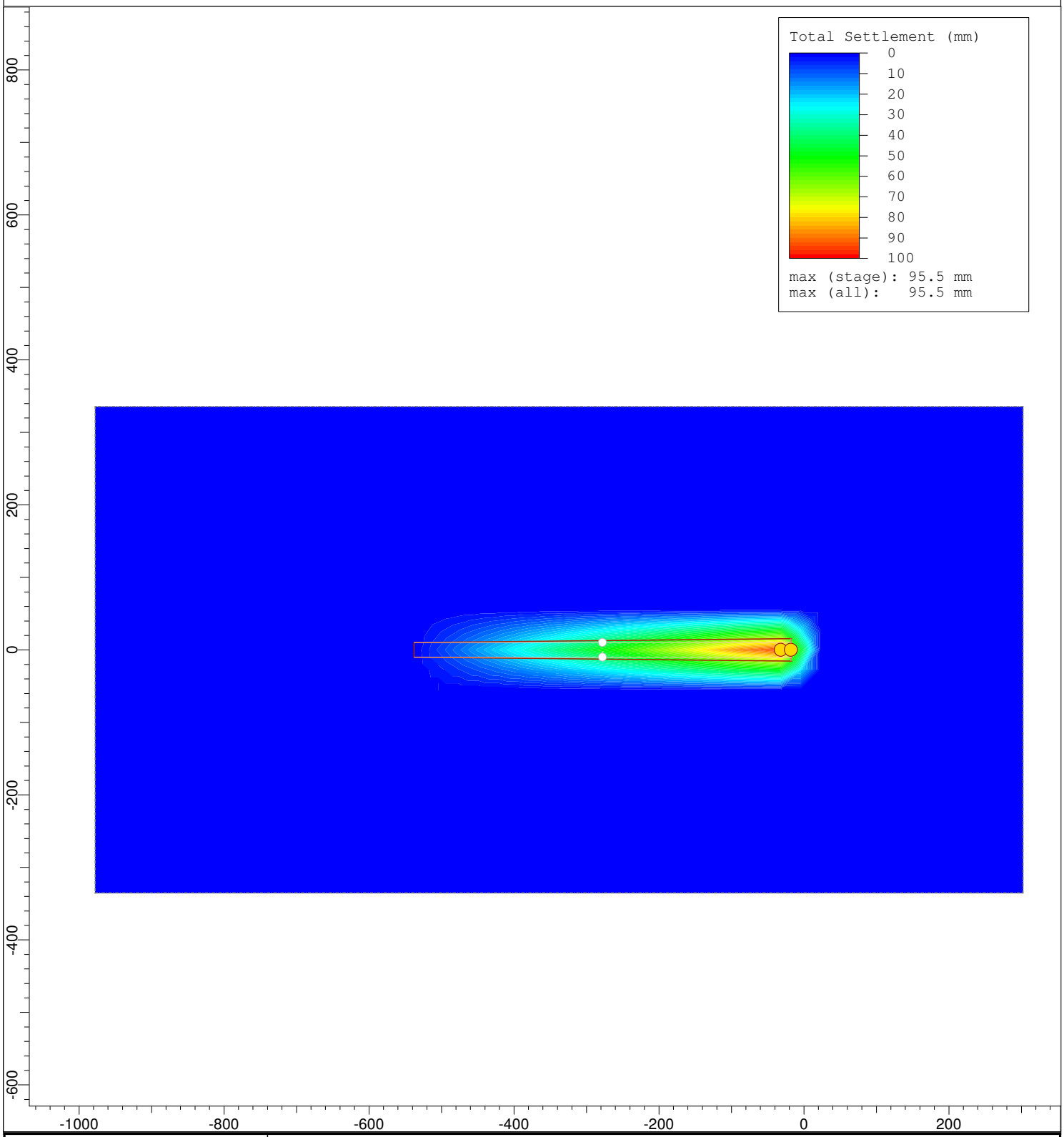
GWP No. 5023-09-00

Figure 14a



Stantec	Project: Hwy 144, Whitson River Crossing, East Embankment, Greater Sudbury, ON	
	Analysis Description: Evaluation of Settlement due to Approach Embankment for Preliminary Design	
	Drawn By: BB	Company:
SETTLE3D 2.011	Date: 5/7/2013, 11:23:15 AM	File Name: Whitson R Crossing_East Abutment.s3z

Figure 14b



Stantec	*Project* Hwy 144, Whitson River Crossing, West Embankment, Greater Sudbury, ON	
	Analysis Description Evaluation of Settlement due to Approach Embankment for Preliminary Design	
	Drawn By BB	*Company*
SETTLE3D 2.011	*Date* 5/7/2013, 11:23:15 AM	*File Name* Whitson R Crossing_West Abutment.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 for both the east and west abutment piles. 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.

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
East Abutment Piles				
260.9	258.6	Silt	0	1,500
258.6	252.9	Silty clay	1,500	3,025
< 252.9		Bedrock	-	-
West Abutment Piles				
261.2	259.8	Silty Sand	0	1,375
259.8	244.2	Silt	1,375	3,025
244.2	242.0	Silty clay	3,025	3,025
< 242.0		Bedrock	-	-