Foundation Investigation and Design Report

Highway 17
Culvert Replacements and Removal
Townships of Lorne and Nairn
G.W.P. 5182-08-00

McIntosh Perry Consulting Engineers Ltd.

Project No. 122410534
Geocres No. 41I-263

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For

G.W.P. 5182-08-00

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1.0 Introduction

This Foundation Investigation Report has been prepared specifically and solely in support of the detailed design for the replacement of three culverts and removal of one culvert on Highway 17 in the Townships of Lorne and Nairn, west of Sudbury, Ontario.

The proposed culvert work is to be carried out under Ministry of Transportation of Ontario (MTO) G.W.P. 5182-08-00 and is summarized in Table 1.1, below.

Table 1.1: Proposed Culvert Work under GWP 5182-08-00

Site No.	Name	Station	Township	Proposed Work
46-395/C	Blake Creek Culvert #1	13+506	Lorne	• Replacement
46-396/C	Blake Creek Culvert #2	13+631	Lorne	• Removal
46-397/C	Blake Creek Culvert #3	11+373	Lorne	• Replacement
46-398/C	Unknown Creek Culvert #4	18+667	Nairn	• Replacement

Stantec Consulting Ltd. was engaged to carry out the Foundation Investigation work as a sub-consultant by McIntosh Perry Consulting Engineers Ltd., the Prime Consultant for this project.

2.0 Site Description and Geology

Site Location

The four culvert sites are located along an approximately six kilometer stretch of Highway 17 within the Townships of Lorne and Nairn, west of Sudbury, Ontario. The culvert site locations are shown on the overall Key Plan, Drawing No. 1 in Appendix A, and on the individual key plan inset to Drawings 2 through 5 in Appendices B through E.

It is noted that for project orientation purposes, Highway 17 is assumed to run east-west with chainage increasing from west to east. There is one chainage equation within the project limits (20+907.021 Nairn Township = 10+000 Lorne Township).

General Site Description

Within the project limits, Highway 17 is classified as a two-lane rural arterial undivided highway that has a posted speed limit of 90 km/h.

The existing highway section typically includes two 3.75 m lanes with 3.0 m wide shoulders and 1.0 m shoulder roundings. Passing lanes and turn tapers are also present at some locations.

Physiographic Description

This section of Highway 17 site is located within the Canadian Shield and is characterized by frequent rock knobs and rock ridges.

Based on geological mapping of the area obtained from Ontario Geological Survey Map 5002, Northern Ontario Engineering Geology Terrain Study Espanola, the area of the site is characterized by two main deposits identified as glaciofluvial sands and gravels to the west and glaciolacustrine silts and sands to the east.

Based on geological mapping of the area obtained from Geological Atlas Map NL-16/17-G Geology of Lake Superior - Sudbury, Ontario and Ontario Geological Survey Open File Report 6243, A Field Guide to the Geology of Sudbury, Ontario, bedrock consists of diamictite from the Hough Lake Group of the Ramsay Lake formation.

Culvert Locations

The existing conditions at the four existing culverts are described as follows:

Site 46-398/C – 18+667 Nairn

The existing culvert consists of a triple cell, timber box culvert (no footings) with dimensions of 4.2 m x 1.3 m x 32.0 m.

At this site, Highway 17 consists of two lanes. The height of the existing highway embankment is approximately 2.0 m. The embankment side slopes consist of exposed granular base material within the upper portion and vegetation within the lower portion. The existing side slopes are approximately 3H:1V. Some erosion was noted on the side slopes.

The direction of the stream flow is from north to south. The depth of water within the un-named creek was shallow (< 500 mm) at the time of the investigation and sand was visible at surface within the creek bed.

Photo B1 in Appendix B shows the general condition of the culvert and the creek on the north side of the highway. Photo B2 in Appendix B shows the general condition of the embankment and the creek on the south side. Photos B3 and B4 document the erosion observed on site.

Site 46-397/C – 11+373 Lorne

The existing culvert consists of a triple cell, timber box culvert (no footings) with dimensions of 6.3 m x 1.8 m x 26.2 m. The width of the interior cells ranges from 2.1 m to 2.2 m.

This site is located near the start of a westbound passing lane and the existing Highway 17 platform supports two driving lanes, a passing lane and shoulders. The height of the existing highway embankment is approximately 1.6 m. The embankment side slopes consist of exposed granular base material within the upper portion and vegetation within the lower portion. The existing side slopes range from approximately 2.5H:1V to 3H:1V.

Blake Creek flows through the culvert from south to north. The depth of water within the creek was approximately 600 mm to 900 mm at the north end and 200 mm to 1.4 m at the south end at the time of the investigation.

Photo C1 in Appendix C shows the general condition of the culvert and the creek on the north side of the highway. Photo C2 in Appendix C shows the general condition of the culvert and the creek on the south side.

Site 46-395/C – 13+506 Lorne

The existing culvert consists of a triple cell, timber box culvert (no footing) with dimensions of 7.3 m x 2.0 m x 32.7 m.

At this site, Highway 17 consists of two lanes. The height of the existing highway embankment is approximately 2.0 m. The embankment side slopes consist of exposed granular base material within the upper portion and grassy vegetation within the lower portion. The existing side slopes range from approximately 2.5H:1V to 3H:1V.

Blake Creek flows through the culvert from south to north. The depth of water within the creek was approximately 750 mm at both ends at the time of the investigation.

Photo D1 in Appendix D shows the general condition of the culvert and the creek on the north side of the highway. Photo D2 in Appendix D shows the general condition of the culvert and the creek on the south side, including a rock outcropping.

Site 46-396/C – 13+631 Lorne

The existing culvert consists of a triple cell, timber box culvert (no footing) with dimensions of 4.2 m x 1.0 m x 31.6 m.

At this site, Highway 17 consists of two lanes. The height of the existing highway embankment is approximately 3.7 m above the bottom of ditch. The embankment side slopes consist of exposed granular base material within the upper portion and grassy vegetation within the lower portion. The existing side slopes range from approximately 2.5H:1V to 3H:1V.

Blake Creek flows through the culvert from north to south. The downstream and upstream invert elevations for the existing culvert are 221.83 m and 221.86 m respectively. The depth of water within the creek was approximately 400 mm at the south end and 600 mm at the north end at the time of the investigation.

Photo E1 in Appendix E shows the general condition of the culvert and the creek on the north side of the highway. Photo E2 in Appendix E shows the general condition of the culvert and the creek on the south side.

3.0 Method of Investigation

3.1 SURVEYING

Borehole locations were established in the field by Stantec personnel relative to the existing culverts and centerline chainage marked out by the McIntosh Perry survey crew.

The location (MTM Zone 12 northing and easting) and ground surface elevation at each borehole location was surveyed by Stantec personnel with reference to Geodetic Benchmarks provided by McIntosh Perry for each culvert location.

3.2 DRILLING INVESTIGATION

General Details

The field drilling program was carried out between July 5, 2010, and August 5, 2010.

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

All but one borehole (10-5) were advanced using either a truck-mounted CME 75 or track-mounted CME 55 drill equipped for soil and bedrock sampling. The CME drilling equipment was owned and operated by Abraflex Drilling from Lively, Ontario. Borehole 10-5, located in wet ground at the north end of Culvert 46-397/C, was not accessible by a truck or track-mount drill and was advanced using portable drilling equipment consisting of a tripod, full-weight hammer for advancing the split spoon sampler, and an electric core drill for advancing casing. The portable drilling equipment was owned and operated by Landcore Drilling from Chelmsford, Ontario.

The subsurface stratigraphy encountered in each borehole was recorded in the field by an experienced Stantec Field Technologist. Split spoon samples were collected at regularly spaced intervals ranging from 760 mm to 1500 mm during the course of Standard Penetration Testing (SPT). The SPT N values presented herein represent the number of blows required to advance the sampler 0.3 m, and have not been corrected. In-situ shear vane testing was conducted in cohesive soil deposits to determine the undrained shear strength of these deposits.

Undisturbed samples of soft to firm silty clay were collected at selected locations using Shelby tube samplers.

The depth to groundwater was observed and documented in the open boreholes at the time of drilling. Piezometers were not installed as the boreholes were drilled where water was present either at or slightly above ground surface. Artesian conditions were not observed in any of the boreholes.

The boreholes were completed in accordance with the Ministry of the Environment Regulation 903, including backfilling with a combination of auger cuttings and bentonite. For the boreholes advanced within the roadway, the surface was reinstated with 150 mm of cold patch asphalt. For the boreholes advanced into bedrock the cored hole was sealed with bentonite to 300 mm above the soil/bedrock interface.

All recovered samples were returned to our Ottawa laboratory for detailed classification and testing.

Further details regarding the drilling investigation carried out at each site are provided below.

<u>Site 46-398/C – 18+667 Nairn</u>

The drilling investigation at this site included three boreholes, located at the inlet, outlet and through the existing embankment beside the existing culvert, and one dynamic cone penetration test (identified as 10-1B). The locations of the boreholes are shown on the Borehole Location Plan in Appendix B.

A loose sand deposit was encountered at this site. Below a depth of approximately 6 m, sand and water came up inside the augers. The drilling operation was switched from augering to advancing casing using wash-bore techniques. A dynamic cone penetration test was carried out at this site to assess the depth of loose to compact sand since the borehole through the existing embankment was still within loose to compact sand at the planned investigation depth of 20 m.

The investigation at this site is summarized in Table 3.1.

Table 3.1: Summary of Drilling Investigation at Site 46-398/C (18+667)

	Boreholes			
	10-1	**10-1B**	**10-2**	**10-3**
MTM Zone 12 Coordinates Northing Easting	 5132211.8 259242.6	 5132212.2 259243.5	 5132217.6 259231.7	 5132189.3 259230.1
Station	18+672	18+673	18+665	18+652
Offset	2.2 m Lt	2.2 m Lt	11.0 m Lt	14.0 m Rt
Ground Surface Elevation, m	217.4	217.4	215.4	215.4
Total Depth Drilled, m	19.8	32.6	15.1	15.9
End of Borehole Elevation, m	197.6	184.8	200.3	199.5
Depth Augered/cased, m	19.8	32.6	15.1	15.9
Number of Soil Samples	22	0	15	15
Depth of Dynamic Cone Penetration Test, m	0	30.2	0	0
Depth Cored, m	0	0	0	0

<u>Site 46-397/C – 11+373 Lorne</u>

The drilling investigation at this site included three boreholes, located at the inlet, outlet and through the existing embankment beside the existing culvert. The locations of the boreholes are shown on the Borehole Location Plan in Appendix C.

Borehole 10-4 was drilled and sampled to a depth of 20.7 m below ground surface and then extended by carrying out a dynamic cone penetration test since the borehole was still within soft clay at the planned investigation depth of 20 m.

Thin walled tube samples were collected in Borehole 10-6 due to the presence of firm to soft silty clay at this site.

The investigation at this site is summarized in Table 3.2.

Table 3.2: Summary of Drilling Investigation at Site 46-397/C (11+373)

	Boreholes		
	10-4	**10-5**	**10-6**
MTM Zone 12 Coordinates Northing Easting	 5132055.1 262535.1	 5132068.9 262525.2	 5132042.8 262514.9
Station	11+384	11+370	11+369
Offset	2.3 m Lt	12.0 m Lt	16.0 m Rt
Ground Surface Elevation, m	208.1	206.5	206.5
Total Depth Drilled, m	22.2	15.9	15.9
End of Borehole Elevation, m	185.9	190.6	190.7
Depth Augered, m	20.7	15.9	15.9
Number of Soil Samples	14	9	10
Depth of Dynamic Cone Penetration Test, m	1.5	0	0
Depth Cored, m	0	0	0

Site 46-395/C – 13+506 Lorne

The drilling investigation at this site included four boreholes (10-7, 10-8, 10-9 and 10-12) and one probe hole (10-9B). The locations of the boreholes are shown on the Borehole Location Plan in Appendix D.

Borehole 10-12 was advanced approximately 3.2 m into bedrock by coring with NQ diamond wire coring equipment.

The investigation at this site is summarized in Table 3.3.

Table 3.3: Summary of Drilling Investigation at Site 46-395/C (13+506)

	Boreholes				
	10-7	**10-8**	**10-9**	**10-9B**	**10-12**
MTM Zone 12 Coordinates Northing Easting	 5132548.7 264429.7	 5132558.5 264416.1	 5132524.3 264428.6	 5132536.8 264440.6	 5132537.5 264421.6
Station	13+513	13+508	13+498	13+515	13+500
Offset	2.1 m Lt	18.0 m Lt	17.0 m Rt	14.0 m Rt	2.2 m Rt
Ground Surface Elevation, m	225.7	223.6	223.6	223.3	226.0
Total Depth Drilled, m	8.2	8.9	3.5	4.0	6.6
End of Borehole Elevation, m	217.5	214.7	220.1	219.3	219.4
Depth Augered, m	8.2	8.9	3.5	4.0	3.4
Number of Soil Samples	8	8	5	0	4
Depth Cored, m	0	0	0	0	3.2

<u>Site 46-396/C – 13+631 Lorne</u>

The drilling investigation at this site included two boreholes, located approximately 5 m east and west of the existing culvert. The locations of the boreholes are shown on the Borehole Location Plan in Appendix E.

It is noted that auger refusal was encountered within the embankment fill at a depth of 1.8 m in Borehole 10-10. Casing was advanced to penetrate through the boulders. Borehole 10-11 was initially advanced to split spoon refusal at a depth of 2.8 m on July 8, 2010. Cobbles and/or boulders within the fill resulted in bending of the split spoon sampler (see Photo No E3 in Appendix E). When the drill crew attempted to auger past the obstruction, they encountered an abrupt refusal that stopped the rotation of the augers, cracked the transmission case and sheared the bolts connecting the drill engine to the rig. Drilling was resumed and completed on August 4, 2010.

The boreholes at this site were drilled and sampled to depths of 12.8 m and 11.3 m. Since the boreholes were still within soft to firm silty clay at these depths, the boreholes were further advanced using dynamic cone penetration tests to assess the depth of the cohesive deposits.

The investigation at this site is summarized in Table 3.4.

Table 3.4: Summary of Drilling Investigation at Site 46-396/C (13+631)

	Boreholes	
	10-10	**10-11**
MTM Zone 12 Coordinates Northing Easting	 5132612.1 264520.3	 5132616.7 264535.2
Station	13+624	13+639
Offset	2.0 m Lt	2.1 m Rt
Ground Surface Elevation, m	225.7	225.4
Total Depth Drilled, m	22.8	21.2
End of Borehole Elevation, m	202.9	204.1
Depth Augered, m	12.8	11.3
Number of Soil Samples	10	8
Depth of Dynamic Cone Penetration Test, m	10.0	9.9
Depth Cored, m	0	0

3.3 LABORATORY TESTING

All samples were taken to our Ottawa laboratory where they were subjected to a detailed visual examination by a Geotechnical Engineer. Selected soil samples underwent a gradation analysis, Atterberg Limit testing of the cohesive material, and moisture content testing. Samples of the bedrock underwent unconfined compression testing to determine the strength characteristics of the rock.

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 unless we are directed otherwise by the client.

4.0 Subsurface Conditions

An explanation of the symbols and terms used to describe the Borehole Records is provided in Appendix A. All elevations referenced in this report are geodetic.

4.1 SITE NO. 46-398/C - STATION 18+667

In general, the soil stratigraphy at this site consisted of pavement structure and embankment fill over a deep sand layer. The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix B. A stratigraphic cross-section is provided in Drawing No. 2 in Appendix B.

4.1.1 Pavement Structure & Embankment Fill

Borehole 10-1 was advanced through the westbound driving lane. Boreholes 10-2 and 10-3 were advanced through the toe of the existing embankment fill on the north and south sides, respectively.

The pavement structure was observed to consist of 210 mm of asphalt over granular base/subbase over embankment fill. The embankment fill extended down to a depth of 3.9 m below ground surface (Elev. 213.5 m).

Gradation analyses on three samples of the embankment fill indicated that it contained 2% to 8% gravel, 66% to 83% sand and 15% to 26% fines. The results of the gradation analyses are provided on Figure No. 1 in Appendix B. The material is a silty sand (SM) in accordance with the MTO soil classification system. The moisture content of the samples tested ranged from 6% to 15%, with an average of 11%.

SPT ‘N’ values ranged from 5 to 30 indicating that the fill varied from a loose to compact state.

4.1.2 Silt (ML) with Sand

A brown silt deposit was identified directly beneath the existing fill in Boreholes 10-1 and 10-3. The base of this deposit ranged in elevation from 214.6 m to 213.0 m, with an observed thickness ranging from 300 to 500 mm.

Sieve analysis on one sample of the silt material indicated that it contained 3% gravel, 14% sand and 83% fines. The results of the sieve analysis are provided on Figure No. 2 in Appendix B. The material is classified as silt (ML) with sand in accordance with the MTO soil classification system. The moisture content of the sample tested was 18%.

4.1.3 Silty Sand (SM)

A silty sand deposit was observed below the silt deposit in Borehole 10-1. The thickness of this layer was 1.2 m, with a base elevation of 212.1 m.

The moisture content of the tested sample was 10%. The results of the sieve analysis indicated that this material contained 10% gravel, 47% sand, and 43% fines. The results of the sieve analyses are provided on Figure No. 3 in Appendix B. The material is classified as silty sand (SM) in accordance with the MTO soil classification system.

The SPT ‘N’ value within this deposit was 16 indicating a compact state.

4.1.4 Poorly-graded Sand (SP)

A deep sand deposit that ranged from grey to brown was identified in all boreholes advanced at this site. Boreholes 10-1, 10-2 and 10-3 were terminated within the sand deposit at depths of 19.8 m, 15.1 m and 15.9 m, respectively. A dynamic cone penetration test (identified as Borehole 10-1B) was advanced to a depth of 32.6 m (elevation 184.8 m).

Sieve analysis on eleven samples of the sand material indicated that it contained between 0% and 17% gravel, 72% and 98% sand, and 1% and 11% fines. The results of the sieve analysis are provided on Figures No. 4 and 5 in Appendix C. Ten of the eleven samples are classified as poorly graded sand (SP) in accordance with the MTO soil classification system. One sample is classified as well-graded sand with silt and gravel (SW-SM).

The SPT ‘N’ values ranged from 2 to 38 with an average of 15, indicating generally a loose to compact state. It is noted that difficulty was encountered balancing the hydrostatic pressure inside the borehole during drilling and some disturbance of the sand which would lead to artificially low ‘N’ values was noted. The SPT ‘N’ values were typically greater than 20 below elevation 200 m, indicating compact to dense conditions. A distinct increase in resistance was also noted in the DCPT in 10-1B below elevation 200 m.

The moisture content for the samples tested ranged from 15% to 29% with an average of 22%.

4.1.5 Bedrock

Bedrock was not encountered within the depth of investigation (32.6 m) at this site.

4.1.6 Groundwater

Groundwater was measured in the open boreholes at the time of drilling to be 2.7 m below ground surface in Boreholes 10-1 and 2.8 m in Borehole 10-3. The groundwater depths correspond to elevation 214.7 m and 213.1 m, respectively. The water level in the creek at this location was surveyed to be at elevation 214.4 m on June 24, 2010.

4.2 SITE NO. 46-397/C - STATION 11+373

In general, the soil stratigraphy at this site consisted of pavement structure and embankment fill over a deep clay deposit. The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix C. A stratigraphic cross-section is provided in Drawing No. 3 in Appendix C.

4.2.1 Pavement Structure & Embankment Fill

Borehole 10-4 was advanced through the westbound driving lane adjacent to the existing culvert. Borehole 10-5 was advanced through the toe of the existing embankment fill on the north side.

The pavement structure was observed to consist of 280 mm of asphalt over granular base over a buried asphalt layer (100 mm thick) over granular base/subbase. The embankment fill beneath the pavement structure extended down to a depth of 4.1 m below ground surface (Elev. 204.0 m). The fill in Borehole 10-5 was 0.9 m thick and extended down to elevation 205.6 m.

The composition of the embankment fill was variable and ranged from silty sand with gravel to sandy silty clay with gravel.

Sieve analysis on two samples of the fill materials indicated that it contained 30% and 25% gravel, 50% and 61% sand and 20% and 14% fines. The results of the sieve analysis are provided on Figure No. 6 in Appendix C. The material is classified as silty sand (SM) with gravel in accordance with the MTO soil classification system. The moisture contents of the samples tested were 4% and 11%.

SPT ‘N’ values ranged from 7 and 84 indicating that the fill varied from a loose to very dense state.

Poor sample recovery (50 mm to 140 mm) was encountered in samples SS4 and SS5. The recovered material consisted of sandy silty clay with gravel based on visual classification. Full gradation analysis of this layer was not possible due to the minimal sample recovery. The moisture content of the sample tested was 26%. SPT ‘N’ values were 14 and 27 within this zone.

4.2.2 Varved Silty Clay (CL to CH)

A deposit of varved silty clay was observed below the surficial materials in all boreholes advanced at this site. Boreholes 10-5 and 10-6 were terminated within the silty clay deposit at depths of 15.9 m below ground surface (elevation 190.6 m and 190.7 m). The base of the silty clay deposit was inferred at elevation 185.9 m in Borehole 10-4 based on refusal to a dynamic cone penetration test.

The silty clay was observed to be varved with alternating layers 3 to 5 mm thick (See Photos C3 and C4 in Appendix C. The layers could not be cleanly separated to allow for individual laboratory testing. It was noted that some layers felt stiffer and exhibited some dilatency, suggesting that these layers have a higher silt content.

Gradation analyses carried out on seven samples of the silty clay deposit indicated that it contained 0% gravel, 0 to 1% sand, 14 to 39% silt size particles and 60 to 86% clay size particles. The results of the gradation analyses are provided on Figure No. 7 in Appendix D.

The moisture content of the samples tested ranged from 37% to 71% with an average of 58%. Results of Atterberg Limit testing indicated Plastic Limits of 16% to 23%, Liquid Limits of 33% to 64% and Plasticity Indices of 11 to 41. Results of the Atterberg Limit testing are provided on Figure No. 8 in Appendix C. The material ranges from low plasticity (CL) to high plasticity (CH) in accordance with the MTO soil classification system.

The consistency of the silty clay was very soft to firm as indicated by the measured in-situ shear strength ranging from 11 kPa to 46 kPa with an average of 27 kPa. The sensitivity of the silty clay ranged from 2.4 to 10.1 with an average of 4.5.

4.2.3 Bedrock

Bedrock was not proven by coring at this site. The abrupt refusal of the DCPT in Borehole 10-4 may be due to bedrock.

4.2.4 Groundwater

Groundwater was measured in the open boreholes at the time of drilling to be 2.6 m below ground surface in Borehole 10-4. The groundwater depth corresponds to an elevation of 205.5 m. The water level in the creek at this location was surveyed to be 205.8 m on June 24, 2010.

4.3 SITE NO. 46-395/C - STATION 13+506

In general, the soil stratigraphy at this site consisted of pavement structure and embankment fill over silty clay over glacial till over bedrock. It is noted that the depth to bedrock is highly variable. The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix D. A stratigraphic cross-section is provided in Drawing No. 4 in Appendix D. Bedrock outcropping was noted on the south side of the highway near the culvert inlet.

4.3.1 Pavement Structure & Embankment Fill

Boreholes 10-7 and 10-12 were drilled through the west and east bound lanes near the existing culvert. The pavement structure in the east bound lane consisted of 420 mm of asphalt while in the west bound lane 220 mm of asphalt was observed over approximately 100 mm of silty sand with gravel over a buried asphalt layer (100 mm thick). A sieve analysis on one sample of the base material from Borehole 10-12 indicated that it contained 16% gravel, 70% sand, and 14% fines. The results of the sieve analysis are provided on Figure No. 9 in Appendix D. The material is classified as silty sand with gravel (SM) with in accordance with the MTO soil classification system.

The embankment fill beneath the pavement structure extended to depths of 1.9 m and 2.3 m below top of pavement in Boreholes 10-12 and 10-7, respectively (elevation 224.1 m and 223.3 m, respectively).

Sieve analyses on two samples of the embankment fill material indicate that it contained 53% and 84% gravel, 13 % and 28% sand, and 2% and 19% fines. The results of the sieve analyses are provided on Figure No. 9 in Appendix D. The material is classified as silty gravel (GM) with sand to poorly-graded gravel (GP) in accordance with the MTO soil classification system. It is noted that cobbles and boulders were encountered within the embankment fill. The moisture contents of the samples tested were 2% and 7%.

SPT 'N' values were from 11 to greater than 50 indicating that the fill varied from a compact to very dense state. 'N' values greater than 50 are likely attributable to the coarse nature of the material.

4.3.2 Topsoil

A 900 mm thick layer of silt and sand with organic matter was observed at ground surface in Borehole 10-8. A 120 mm thick layer of topsoil was observed at ground surface in Borehole 10-9.

4.3.3 Silty Clay (CI to CH)

A brownish grey to grey silty clay deposit was observed below the surficial materials in all boreholes advanced at this site. The thickness of the silty clay deposit ranged from 1.5 m to 8.1 m. The base of the silty clay deposit ranged from elevation 222.7 m to 214.7 m.

Gradation analyses carried out on six samples of the silty clay deposit indicated that it contained 0% gravel, 0 to 14% sand, 17 to 49% silt size particles and 37 to 83% clay size particles. The results of the gradation analyses are provided on Figure No. 10 in Appendix D.

The moisture content of the samples tested ranged from 24% to 61% with an average of 38%. Results of Atterberg Limit testing indicated Plastic Limits of 18% to 23%, Liquid Limits of 43% to 53% and Plasticity Indices of 20 to 30. Results of the Atterberg Limit testing are provided on Figure No. 11 in Appendix D. The material ranges from intermediate plasticity (CI) to high plasticity (CH) in accordance with the MTO soil classification system.

The consistency of the silty clay was very soft to firm as indicated by the measured in-situ shear strength ranging from 12 kPa to 47 kPa with an average of 21 kPa. The sensitivity of the silty clay ranged from 3 to 9 with an average of 5.

4.3.4 Glacial Till

A thin glacial till deposit was observed beneath the silty clay in Boreholes 10-7 and 10-9. The thickness of the till layer ranged from 300 mm to 1.3 m. The base of the till varied from elevation 217.5 m to 220.1 m.

Sieve analyses on two samples of the till material indicated that it contained 20 and 22% gravel, 24 and 46% sand, and 34% and 54% fines. The results of the sieve analyses are provided on Figure No. 12 in Appendix D. The material is classified as silty sand (SM) with gravel in accordance with the MTO soil classification system. The moisture content of the samples tested were 10% and 14%.

An SPT 'N' value of 22 was obtained in the only test carried out entirely within the till deposit, indicating that the till was generally in a compact state.

4.3.5 Bedrock

Bedrock was encountered or inferred at depths of 3.4 m to 8.9 m below existing ground surface. The surface of the bedrock varied from elevation 214.7 m to 222.7 m. A bedrock outcropping was also noted on the south side of the highway near the culvert inlet (see Photo No. 6).

Borehole 10-12 was advanced approximately 3 m into bedrock by coring with NQ-size diamond coring equipment. The core recovery ranged from 85% to 100% with an average of 90%. The rock quality designation (RQD) ranged from 7% to 100% with an average of 58%, indicating very poor to excellent quality rock mass. A photograph of the recovered bedrock cores is provided in Appendix D.

The recovered rock core consisted of unweathered grey to dark grey granitic igneous bedrock. Joint spacing ranged from close to wide with dipping orientation typically 20° to 50° from horizontal. Vertical cracking was observed from 3.4 m to 3.9 m. A detailed description of the rock cores is provided in the Field Core Log in Appendix D.

Unconfined compressive strength tests were carried out on two samples of the recovered bedrock core. The tests results are presented in Table 4.1.

Table 4.1: Unconfined Compressive Strength of Bedrock

Borehole No.	Ground Surface Elevation (m)	Test Elevation (m)	Unconfined Compressive Strength (MPa)	Rock Strength Classification
10-12	226.0	221.9	129	Very Strong
10-12	226.0	219.0	236	Very Strong

4.3.6 Groundwater

Groundwater measured in the open boreholes at the time of drilling was 4.3 m and 5.3 m below ground surface in Boreholes 10-7 and 10-8, respectively. The groundwater depths correspond to elevations of 221.4 m and 218.3. The water elevation in the creek at this culvert location was surveyed to be 222.8 m on June 23, 2010.

4.4 SITE NO. 46-396/C - STATION 13+631

Both boreholes at this site (10-10 and 10-11) were advanced within the existing Highway 17 driving lanes. In general, the soil stratigraphy at this site consisted of pavement structure over fill underlain by a deep clay layer. The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix E. A stratigraphic cross-section is provided in Drawing No. 5 in Appendix E.

4.4.1 Pavement Structure & Embankment Fill

Boreholes 10-10 and 10-11 were drilled through the existing driving lanes approximately 5 m west and east of the existing culvert, respectively. The pavement structure was observed to consist of 320 to 350 mm of asphalt over a thin granular base (approximately 100 mm) over a buried asphalt layer (110 to 150 mm thick) over a granular base. The bottom of the pavement structure was approximately 800 mm to 1.0 m below top of pavement, which corresponds to elevation 224.7 m and 224.6 m in Boreholes 10-10 and 10-11, respectively.

The embankment fill beneath the pavement structure extended down to depths of 2.1 m and 3.1 m below ground surface in Boreholes 10-10 and 10-11, respectively. These depths correspond to elevations 223.6 m and 222.3 m, respectively.

Sieve analyses on three samples of the fill materials indicated that it contained between 23% and 35% gravel, 31% and 56% sand, and 9% to 46% fines. The results of the sieve analysis are provided on Figure No. 13 in Appendix E. The material is classified as silty sand with gravel (SM) in accordance with the MTO soil classification system. The moisture content of the samples tested ranged from 3% to 9%.

SPT ‘N’ values ranged from 31 to greater than 100, indicating that the fill varied from a dense to very dense state. It is noted that frequent cobbles and boulders were encountered during drilling.

The bottom 400 mm of the fill in Borehole 10-11 consisted of sandy clay (between elevation 222.3 m and 222.7 m). A gradation analysis indicated that this portion of the fill consisted of 9% gravel, 22% sand, 41 % silt and 28% clay. The results of the sieve analysis are provided on Figure No. 14 in Appendix E. The plastic limit was 18 and the liquid limit was 28, which indicated that the fines consist of low plasticity clay. The moisture content of the sample tested was 15%. The SPT N-value within this zone was 10.

4.4.2 Clay

A brownish grey to grey silty clay deposit was observed below the fill in both boreholes advanced at this site. Boreholes 10-10 and 10-11 were sampled to depths of 12.8 m and 11.3 m, respectively. The silty clay deposit extended to beyond these depths. The base of the silty clay was inferred based on increased resistance during dynamic cone penetration tests at depths of approximately 22 m and 20.5 m (elevation 204 m and 205.5 m) in Boreholes 10-10 and 10-11, respectively.

Gradation analyses was carried out on five samples of the silty clay deposit indicated that it contained 0% gravel, 0 to 5% sand, 20 to 46% silt size particles and 49 to 79% clay size particles. The results of the gradation analyses are provided on Figure No. 15 in Appendix D.

The moisture content of the samples tested ranged from 32% to 62% with an average of 49%. Results of Atterberg Limit testing indicated Plastic Limits of 18% to 23%, Liquid Limits of 28% to 55% and Plasticity Indices of 10 to 33. Results of the Atterberg Limit testing are provided on Figure No. 16 in Appendix E. The material ranges from low plasticity (CL) to high plasticity (CH) in accordance with the MTO soil classification system.

The consistency of the silty clay was stiff to soft as indicated by the measured in-situ shear strength ranging from 63 kPa to 18 kPa. The sensitivity of the silty clay ranged from approximately 3 to 7.

4.4.3 Bedrock

Bedrock was not proven by coring at this site. The abrupt refusal of the DCPT in Boreholes 10-10 and 10-11 may be due to bedrock.

4.4.4 Groundwater

Groundwater was measured in the open boreholes at the time of drilling to be 4.3 m below ground surface in Borehole 10-11. The groundwater depth corresponds to elevations of 221.1 m. The water level in the creek at this culvert location was surveyed to be 222.8 m on June 23, 2010.

5.0 Closure

A subsurface investigation is a limited sampling of a site. The subsurface conditions 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.

This report has been prepared by Paul Carnaffan and Raymond Haché. A technical review was carried out by Fred Griffiths.

Respectfully submitted,

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FOUNDATION DESIGN REPORT
For

G.W.P. 5182-08-00

Culvert Replacements and Removals
Highway 17
Townships of Lorne and Nairn

6.0 Discussion

6.1 GENERAL

The scope of work for this assignment identified four, triple cell, timber box culverts whose deteriorated conditions necessitated replacement or removal.

The culvert locations and proposed work are summarized in Table 6.1, below. Design considerations that are based on the project location rather than site specific conditions and are therefore common to all four sites are discussed in Sections 6.2. Site specific design considerations and design recommendations are presented in Sections 7 through 10.

Table 6.1: Culvert Locations

Site No.	Culvert Location	Township	Proposed Work	Report Section
46-398/C	18+667	Nairn	Replace Culvert	7
46-397/C	11+375	Lorne	Replace Culvert	8
46-395/C	13+506	Lorne	Replace Culvert	9
46-396/C	13+631	Lorne	Culvert Removal & Backfill	10

6.2 COMMON DESIGN CONSIDERATIONS

6.2.1 Seismic Design Considerations

CHBDC Design Parameters

Table A3.1.1 of the CHBDC indicates that the Zonal Acceleration Ratio (ZAR) for both Sudbury and Espanola is 0.05. A seismic hazard calculation for the site was obtained from the National Resources Canada (copy attached in Appendix F). It indicates that for this site, the peak ground acceleration (PGA) value corresponding to a 10% probability of exceedance in 50 years is 0.024 which is slightly less than the ZAR for Sudbury and Espanola.

Liquefaction of Foundation Soils

Seismic liquefaction refers to a situation where a sudden loss of stiffness and strength of soil occurs due to cyclic loading effects of earthquake. Liquefaction can cause loss of bearing resistance and/or excessive settlement.

An assessment for seismically induced liquefaction has been carried out for Site No. 46-398/C where loose, saturated, poorly-graded sand was identified. The assessment was carried out using the Seed and Idriss simplified method. The assessment indicates that liquefaction of the foundation soils is not a concern at this site due to low peak horizontal acceleration at the site.

Although the low to intermediate plasticity clay at the other three culverts sites would be classified as moderately susceptible to liquefaction based on the criteria proposed by Bray et. al. (2004), observations of past performance indicate that liquefaction is not a concern in areas with low Zonal Acceleration Ratios, such as this site.

Seismic Forces on Buried Structures

The walls of buried structures should be designed to resist earth pressures produced under earthquake conditions. For routine design purposes CHBDC (2006) Clause 4.6.4 recommends the use of the combined coefficients of static and seismic earth pressure, referred to as K_{AE} for active conditions and K_{PE} for passive conditions. The seismic earth pressures may be calculated using the parameters provided in Table 6.2.

The total active and passive thrusts under earthquake 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; and

γ = total unit weight.

For the site under consideration, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values:

Description	Yielding	Non-Yielding
Zonal Acceleration Ratio, A	0.05	0.05
Horizontal Acceleration Coefficient, k_h	0.025	0.075
Vertical Acceleration Coefficient, k_v	0.017	0.05

The back of wall was assumed to be vertical. The angle of friction between the soil and the wall has been set to 0° to provide a conservative estimate. The parameters corresponding to "Non-Yielding" condition were provided for situations where no lateral movements are allowed such as the walls of concrete box culverts.

The Mononobe-Okabe (M-O) method should be used in calculating the design lateral earth pressures (Section C4.6.4 of the Canadian Highway Bridge Design Code).

Table 6.2: Combined Static and Seismic Earth Pressure Parameters (non-yielding)

Parameter	OPSS Granular A or Granular B Type II	OPSS Granular B, Type I or III
Bulk Unit Weight, γ (kN/m^3)	22.8	21.2
Effective Friction Angle, ϕ' (°)	35	32
Friction Angle between Wall and Backfill Soil, δ (°)	0	0
Active Earth Pressure Coefficient, K_{AE}	0.31	0.35
Height of Application of P_{AE} above Base as Ratio of Wall Height (H)	0.358	0.356
Passive Earth Pressure Coefficient, K_{PE}	3.54	3.11
Height of Application of P_{PE} above Base as Ratio of Wall Height (H)	0.307	0.306

6.2.2 Frost Depth

The design frost penetration depth for foundations, f, within the project limits is 2.1 m based on OPSD 3090.100. Spread footings should be provided with 2.1 m of earth cover or equivalent insulation for frost protection. This depth of frost penetration should also be used in the design of frost tapers for the culvert backfill.

6.2.3 Lateral Earth Pressures

Earth pressures will need to be considered in the design of the box culverts as well as for roadway protection systems.

The box culvert should be backfilled in accordance with OPSD 803.010.

Computation of earth pressures should be 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, such as the box culvert, the at-rest earth pressure should be used for design. The unfactored soil parameters provided in Table 7.4 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) and passive (P_P) thrusts can be calculated using the following equations

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

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

Where H is the height of the wall. Values for Ka, Kp, K_o and γ are provided in Table 6.3 below. The thrust acts at a point one third up the height of the wall.

Table 6.3: Recommended Earth Pressure Parameters

Parameter	OPSS Gran A and Gran B Type II	OPSS Gran B Type I	Existing Road Embankment Fill
Bulk Unit Weight, γ (kN/m^3)	22.8	21.2	20.0
Effective Friction Angle	35º	32º	30º
Coefficient of Earth Pressure at Rest (K_o)	0.43	0.47	0.5
Coefficient of Active Earth Pressure (K_a)	0.27	0.31	0.33
Coefficient of Passive Earth Pressure (K_p)	3.7	3.2	3.0

7.0 Site No. 46-398/C - Station 18+667, Twp. Of Nairn

7.1 PROPOSED WORK

It is proposed to replace the existing three-cell timber box culvert with a new concrete culvert.

Performance of Existing Foundations

An inspection of the existing culvert from a geotechnical perspective identified the following issues:

- Erosion of the embankment slopes immediately adjacent to the edges of the culvert on the south side (see Photo B4 in Appendix B).

The geotechnical inspection did not reveal any indications of problems associated with bearing capacity, settlement or scour of the existing culvert foundations.

Proposed Structure

It is understood that the proposed culvert will consist of a 4.5 m x 1.75 m x 31.8 m concrete box culvert. The new culvert will be located at the same location and along the same alignment (21.8° skew) as the existing culvert.

It is understood that both a pre-cast concrete box and a rigid frame open footing culvert are options for this site.

Key elevations associated with the proposed culvert replacement are as follows:

Pavement Elevation	217.45 m (approximate near C/L)
Invert Elevation:	213.80 m North End (approx) – inlet 213.68 m South End (approx) – outlet
Creek Water Elevation:	214.4 m at time of Foundation Investigation (summer 2010)
Founding Elevation	≥ 213.4 m Pre-cast Concrete Box Culvert ≥ 212.15 m Cast-in-place Open Footing Culvert

Construction Staging & Detours

The existing platform width is approximately 15.5 m from shoulder rounding to shoulder rounding in the area of the culvert.

An assessment of the staging options by McIntosh Perry has identified that the work can be completed using two stages by providing one 3.5 m wide lane with traffic controlled by temporary signals. This approach will necessitate the use of a temporary protection system to support the open lane of Highway 17 and widening of the pavement surface across the existing granular shoulder. A platform widening is not required. A copy of the Draft Staging Drawings is provided in Appendix F.

7.2 SOIL SUMMARY

The subsurface conditions observed at this site are presented in detail on the Borehole Records provided in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is provided Appendix A.

The soil conditions at this site generally consist of fill over a deep sand deposit.

For design purposes, the following soils profile will be used:

Table 7.1: Geotechnical Model

Elevation (m)		Soil Type	Design Properties
From	To		
217.4	214.5	FILL: poorly-graded sand with silt and gravel, loose to compact	Total Unit Weight = 20.0 kN/m^3 Friction Angle, ϕ = 30°
214.5	200.0	Poorly-graded Sand (SP), Very loose to compact	Total Unit Weight = 19.5 kN/m^3 Friction Angle, ϕ = 29º E' = 15 MPa
200.0	185.0	Inferred: Sand, compact to dense	Total Unit Weight = 21.0 kN/m^3 Friction Angle, ϕ = 33º E' = 30 MPa

The 2010 creek water elevation of 214.4 m will be used as the design groundwater elevation.

7.3 STRUCTURE/FOUNDATION OPTIONS

Both a concrete Rigid Frame Open Footing culvert and a precast concrete box culvert are being considered by the design team for replacement of the existing structure. Both of these structures would be founded below groundwater level within the very loose to compact poorly-graded sand deposit.

The following table compares the structure options from a foundations design and constructability perspective:

Table 7.2: Foundation Comparison for Replacement Culvert

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Rigid Frame Open Footing		▪ Slower construction process ▪ Deeper excavation required ▪ More extensive (deeper and longer duration) unwatering required	Medium	▪ increased risk of dewatering problems/construction delays
Precast Concrete Box	▪ Use of precast sections minimizes construction period ▪ Wide bottom increases the ultimate bearing resistance and distributes load over a wider area resulting in a more conservative foundation design.		Low	▪ If not properly installed, leakage and loss of backfill could occur at joints/settlement of roadway platform

Although both options are technically feasible, the use of pre-cast concrete box culvert sections will allow for a shorter construction period which offers the following benefits:

- Minimized impacts to traffic.
- Reduced efforts for flow diversion and excavation unwatering. The volume of water to be pumped will be greatly reduced by the shorter construction period.
- Lower cost

Based on the advantages presented above, the use of a closed box culvert supported by the native soils is the recommended foundation approach.

7.4 SEISMIC DESIGN CONSIDERATIONS – SOIL PROFILE TYPE

The site soil is composed of a deep deposit of loose to compact sand. The depth to bedrock is greater than 30 m below existing ground surface. It is recommended that Soil Profile III as defined in Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC) be used in the seismic design of this site.

7.5 FOUNDATION RECOMMENDATIONS

7.5.1 Bearing Resistance

It is recommended that the new culvert consist of a precast concrete box culvert founded on undisturbed native sand or structural fill overlying undisturbed native sand. Based on these founding conditions, the geotechnical resistances provided in Table 7.3 may be used for design.

Table 7.3: Recommended Box Culvert Design Parameters

Founding Element	Founding Elev. (m)	Footing Size (m x m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Box culvert on sand (SP)	213.4 (approximate)	5.15 x 31.8	330	50

In accordance with Section 6.6.1 of the CHBDC, a resistance factor of 0.5 has been applied to calculate the factored geotechnical resistance at ULS.

The geotechnical reaction at SLS corresponds to a maximum settlement of 25 mm. It is noted that the proposed replacement has approximately the same dimensions and is on the same alignment as the existing culvert, therefore settlement of the underlying soils is not expected to be a concern as there is no net increase in loading.

Bedding beneath the culvert should consist of a 75 mm thick uncompacted OPSS Granular A leveling course over 200 mm of OPSS Granular A, compacted to at least 95% standard Proctor maximum dry density (SPMDD).

7.5.2 Sliding Resistance

The unfactored horizontal resistance of spread footings may be calculated using an unfactored coefficient of friction of 0.35 between OPSS Granular A and the pre-cast concrete.

7.6 CONSTRUCTION CONSIDERATIONS

7.6.1 Construction Staging

The proposed staging concept involves two stage construction as shown on the Draft Staging Plans provided in Appendix F. This option would require the use of temporary roadway protection near the centerline of the highway. Further discussion regarding temporary protection systems is provided in Section 7.6.3.

7.6.2 Excavation and Backfilling

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

All vegetation, fill, organic soils and other deleterious materials must be removed from beneath the proposed box culvert foundation. Where deleterious materials are encountered, the material should be excavated, wasted and replaced. The lateral extent of such excavation should include all deleterious material within the influence zone of the foundations.

Side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects. The existing highway embankment fill and native sand are considered Type 3 soil above the water level. Above the stream and groundwater level, temporary cut slopes should be no steeper than 1 horizontal to 1 vertical from the base of the excavation. For excavations below stream and/or groundwater levels, the soils should be considered as Type 4 soils and slopes no steeper than 3H:1V will be required. Flatter side slopes or supported excavations may be required.

Grading work for reinstatement of the highway embankment along the existing culvert alignment should be carried out in accordance with OPSS-206 Construction Specification for Grading and SP 206S03 using OPSS Select Subgrade Material.

7.6.3 Temporary Protection Systems

Two options for holding back the existing Highway 17 embankment during the staging of the culvert replacement were considered – a steel sheet pile (SSP) wall and a soldier pile with timber lagging wall. Due to the limited depth of excavation, cantilevering is likely feasible if a steel sheet pile system is used.

The culvert replacement will necessitate excavation below the waterline. As such, the type of protection system selected will play a significant role in helping to control or limit water inflow and in protecting and base instability due to upward seepage. The need for dewatering to allow for placement of the protection system must also be considered. Excavation to approximately 1 m below the creek water level is anticipated. Sheet piles would need to extend to approximately 2 m below the base of the excavation to avoid instability due to upward seepage, however, deeper penetration may be required to resist lateral earth pressures.

Table F-1 in Appendix F compares the advantages, disadvantages, relative cost and risk/consequences of available roadway protection options considered for the culvert replacement.

Given that the roadway protection system is required to support the roadway during both stages of the culvert replacement and the benefits with respect to simplifying dewatering requirements, the use of a cantilevered steel sheet pile system is recommended. A boxed approach, fully enclosing the work area, is considered most feasible as it provides the most benefit to groundwater control. The contractor will ultimately be responsible to develop and implement a roadway protection system meeting the requirements of OPSS 539, including establishing appropriate geotechnical design parameters.

Shoring design should meet the requirements of Performance Level 2 as per OPSS 539 and should consider traffic loading. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. Horizontal movement should be monitored throughout the culvert replacement process as described in OPSS 539. The monitoring requirements outlined in OPSS 539 are considered to be appropriate for this project.

7.6.4 Unwatering

The underside of the proposed box culvert is approximately 1 m lower than the observed groundwater and creek water levels.

Control of the water flow in the creek will require a cofferdam to prevent stream flow into the excavations. It is anticipated that creek flow will be diverted using pumps to allow construction of the replacement culvert.

The native soils within the anticipated depth of excavation have a high hydraulic conductivity. The roadway protection system design needs to consider groundwater control. Multiple sumps within the excavation will likely be required in order to lower the water level below the base of the excavation in order to allow for a dry, stable base.

7.6.5 Erosion and Scour Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes and adjacent stream banks. All slopes within 3 m of the culvert inlet and outlet should be surfaced with rip-rap at least 300 mm thick placed on a Class II non-woven filter fabric. Where embankment construction includes earth fill, normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surficial erosion.

The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site.

7.6.6 Cement Type and Corrosion Protection

Two samples of the native soil were submitted to Paracel Laboratories in Ottawa, Ontario, for analysis of 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 buried infrastructure. The analysis results are summarized in the Table 7.4.

Table 7.4: Results of Chemical Analysis

Borehole	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
10-2	SS3	1.8	7.34	38	5	174
10-3	SS4	2.6	6.95	76	14	111

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Both test results are significantly below the criteria, 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 within what is considered the normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soil do not indicate a highly corrosive environment, however, the test results should be considered in conjunction with other environmental factors such as road deicing practices when selecting coatings and corrosion protection systems for buried steel objects.

8.0 Site No. 46-397/C – Station 11+373, Twp. Of Lorne

8.1 PROPOSED WORK

It is proposed to replace the existing three-cell timber box culvert with a new concrete culvert.

Performance of Existing Foundations

An inspection of the existing culvert from a geotechnical perspective did not reveal any indications of current problems associated with slope stability, bearing capacity, settlement or scour of the existing culvert foundations.

It was noted that the asphalt surface was unusually thick (280 mm) and that a buried layer of asphalt was identified between a depth of 400 mm and 500 mm below top of pavement. These observations combined with the fact that the site is underlain by approximately 18 m of firm to soft clay suggest that the highway embankment in the vicinity of this culvert may have undergone significant settlement in the past. The additional lifts of asphalt may have been placed to correct the grades after settlement had reached unacceptable levels.

Proposed Structure

It is understood that the proposed culvert will consist of a 6.0 m x 2.0 m x 28.0 m concrete box culvert. The new culvert will be located at approximately the same location as the existing culvert.

It is understood that both a pre-cast concrete box and a rigid frame open footing culvert are options for this site.

Key elevations associated with the proposed culvert replacement are as follows:

Pavement Elevation	208.03 m (approximate near C/L)
Invert Elevation:	204.71 m South End (approx) – inlet 204.85 m North End (approx) – outlet
Creek Water Elevation:	205.8 m at time of Foundation Investigation (summer 2010)

Founding Elevation ≥ 204.4 m Pre-cast Concrete Box Culvert
≥ 202.7 m Cast-in-place Open Footing Culvert

Construction Staging & Detours

The existing platform width is approximately 19 m from shoulder rounding to shoulder rounding in the area of the culvert and includes two driving lanes, a passing lane and 3 m wide shoulders.

An assessment of the staging options by McIntosh Perry has identified that the work can be completed using two stages by providing one 3.5 m wide lane with traffic controlled by temporary signals. A platform widening is not required. A copy of the Draft Staging Drawings is provided in Appendix F.

8.2 SOIL SUMMARY

The subsurface conditions observed at this site are presented in detail on the Borehole Records provided in Appendix C. An explanation of the symbols and terms used to describe the Borehole Records is provided Appendix A.

The soil conditions at this site generally consist of fill over a deep deposit of varved silty clay. A summary plot is provided as Figure F1 in Appendix F.

For design purposes, the following soils profile will be used:

Table 8.1: Geotechnical Model

Elevation (m)		Soil Type	Design Properties
From	To		
208.1	206.1	FILL: silty sand (SM) with gravel, loose to compact	Total Unit Weight = 20.0 kN/m^3 Friction Angle, ϕ = 32°
206.1	204.0	Varved Silty Clay, firm	Total Unit Weight = 17.0 kN/m^3 Undrained Shear Strength, Su = 30 kPa
204.0	201.0	Varved Silty Clay, soft	Total Unit Weight = 16.0 kN/m^3 Undrained Shear Strength, Su =19 kPa
201.0	195.0	Varved Silty Clay, firm	Total Unit Weight = 16.5 kN/m^3 Undrained Shear Strength, Su = 28 kPa
195.0	185.9	Varved Silty Clay, firm	Total Unit Weight = 16.5 kN/m^3 Undrained Shear Strength, Su = 35 kPa
185.9		Inferred Bedrock	

The 2010 creek water elevation of 205.8 m will be used as the design groundwater elevation.

8.3 STRUCTURE/FOUNDATION OPTIONS

Both a concrete Rigid Frame Open Footing culvert and a precast concrete box culvert are being considered by the design team for replacement of the existing structure. Both of these structures would be founded within the firm to soft silty clay and at a depth below the groundwater level.

The following table compares the structure options from a foundations design and constructability perspective:

Table 8.2: Foundation Comparison for Replacement Culvert

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Rigid Frame Open Footing		▪ Slower construction process ▪ Deeper excavation required ▪ More extensive (deeper and longer duration) unwatering required ▪ Low ULS bearing resistance	Medium	▪ increased risk of dewatering problems/construction delays
Precast Concrete Box	▪ Use of precast sections minimizes construction period ▪ Wide bottom increases the ultimate bearing resistance and distributes load over a wider area resulting in a more conservative foundation design.		Low	▪ If not properly installed, leakage and loss of backfill could occur at joints/settlement of roadway platform

Due to the low bearing resistance offered by the soft silty clay, the precast concrete box option is recommended since the foundation loads are spread over a wider area. This option also has a lower cost and shorter construction period which minimizes impacts to traffic.

8.4 SEISMIC DESIGN CONSIDERATIONS – SOIL PROFILE TYPE

The site soil is composed of a deep deposit of soft to firm silty clay. The depth to bedrock is inferred to be approximately 22 m below existing ground surface. It is recommended that Soil Profile III as defined in Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC) be used in the seismic design of this site.

8.5 FOUNDATION RECOMMENDATIONS

8.5.1 Bearing Resistance

It is recommended that the new culvert consist of a precast concrete box culvert founded on undisturbed native soil or structural fill overlying undisturbed native soil. Based on these founding conditions, the geotechnical resistances provided in Table 8.3 may be used for design.

Table 8.3: Recommended Box Culvert Design Parameters

Founding Element	Founding Elev. (m)	Footing Size (m x m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Box culvert on clay	204.5 (approximate)	6.5 x 28.0	70	N/A

In accordance with Section 6.6.1 of the CHBDC, a resistance factor of 0.5 has been applied to calculate the factored geotechnical resistance at ULS. The low bearing resistance at ULS reflects the presence of the weak clay zone at a shallow depth below the founding elevation.

The geotechnical reaction at SLS is identified as non applicable (N/A). It is noted that the proposed replacement has approximately the same dimensions and is on nearly the same alignment as the existing culvert, therefore settlement of the underlying soils is not expected to be a concern as there is no net increase in anticipated load.

Bedding beneath the culvert should consist of a 75 mm thick uncompacted OPSS Granular A leveling course over a pad of OPSS Granular A, compacted to at least 95% standard Proctor maximum dry density (SPMDD). Given the consistency of the underlying clay, the pad should be 0.5 m thick (one lift) placed on a nonwoven geotextile (Type II, FOS of 50 to 150 µm).

8.5.2 Sliding Resistance

The unfactored horizontal resistance of spread footings may be calculated using an unfactored coefficient of friction of 0.35 between OPSS Granular A and the pre-cast concrete.

8.6 CONSTRUCTION CONSIDERATIONS

8.6.1 Construction Staging

The proposed staging concept involves two stage construction as shown on the Draft Staging Plans provided in Appendix F. The existing roadway platform is 19 m in width at this location. The depth of the proposed excavation is approximately 4 m to the underside of the Granular A pad. We have carried out slope stability analysis with the soil parameters provided in Table 8.1.

Traffic loads have been modeled using a 17.6 kN surcharge. The results of the analysis (Figure F3 in Appendix F) indicate that the temporary excavation for the culvert replacement is not stable with a 1H:1V side slope and traffic loads 1 m away from the crest of the slope. There is insufficient room on the existing roadway platform to incorporate a flatter side slope, therefore the excavation will require temporary roadway protection. Please refer to Section 8.6.3 below.

8.6.2 Excavation and backfilling

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

All vegetation, fill, organic soils and other deleterious materials must be removed from beneath the proposed box culvert foundation. Where deleterious materials are encountered, the material should be excavated, wasted and replaced. The lateral extent of such excavation should include all deleterious material within the influence zone of the foundations.

Side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects. The existing highway embankment fill is considered Type 3 soil above the water level. Above the stream and groundwater level, temporary cut slopes should be no steeper than 1 horizontal to 1 vertical from the base of the excavation. For excavations below stream and/or groundwater levels, the soils should be considered as Type 4 soils and slopes no steeper than 3H:1V will be required. Flatter side slopes or supported excavations may be required.

Grading work for reinstatement of the highway embankment along the existing culvert alignment should be carried out in accordance with OPSS-206 Construction Specification for Grading and SP 206S03 using OPSS Select Subgrade Material.

Reinstatement of the roadway embankment should include side slopes no steeper than 2.5H:1V.

8.6.3 Temporary Protection Systems

Two options for holding back the existing Highway 17 embankment during the staging of the culvert replacement were considered – a steel sheet pile (SSP) wall and a soldier pile with timber lagging wall. Due to the limited depth of excavation, cantilevering is likely feasible if a steel sheet pile system is used.

Table F-2 in Appendix F compares the advantages, disadvantages, relative cost and risk/consequences of available roadway protection options considered for the culvert replacement.

Given that the roadway protection system is required to support the roadway during both stages of the culvert replacement, the use of a cantilevered steel sheet pile system is recommended. The contractor will ultimately be responsible to develop and implement a roadway protection system meeting the requirements of OPSS 539, including establishing appropriate geotechnical design parameters.

The roadway protection system, parallel to the centerline of the highway, will need to extend approximately 12 to 15 m to each side of the culvert.

Shoring design should meet the requirements of Performance Level 2 as per OPSS 539 and should consider traffic loading. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. Pile and raker spacing must be designed not to exceed these limits. Horizontal movement should be monitored throughout the culvert replacement process as described in OPSS 539. The monitoring requirements outlined in OPSS 539 are considered to be appropriate for this project.

8.6.4 Unwatering

The underside of the proposed box culvert is approximately 1.5 m lower than the observed groundwater and creek water levels.

Control of the water flow in the creek will require a cofferdam to prevent stream flow into the excavations. It is anticipated that flow will be diverted using pumps to allow construction of the replacement culvert.

The native soils within the anticipated depth of excavation have a low hydraulic conductivity. Pumping from sumps within the excavation should be sufficient to unwater the excavation.

8.6.5 Erosion and Scour Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes and adjacent stream banks. All slopes within 3 m of the culvert inlet and outlet should be surfaced with rip-rap at least 300 mm thick placed on a Class II non-woven filter fabric. Where embankment construction includes earth fill, normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surficial erosion.

The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site.

8.6.6 Cement Type and Corrosion Protection

Two samples of the native soil were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of 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 buried infrastructure. The analysis results are summarized in the Table 8.4.

Table 8.4: Results of Chemical Analysis

Borehole	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
10-4	SS7	4.2	7.52	1620	46	8.21
10-5	SS2	1.2	7.13	1300	188	5.45

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Both test results are significantly below that criteria, 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 within what is considered the normal range for soil pH of 5.5 to 9.0, however, the resistivity is relatively low and the soluble chloride levels are relatively high, suggesting a potentially corrosive environment. The test results should be considered in conjunction with other environmental factors such as road deicing practices when selecting coatings and corrosion protection systems for buried steel objects.

9.0 Site No. 46-395/C – Station 13+506, Twp. Of Lorne

9.1 PROPOSED WORK

It is proposed to replace the existing three-cell timber box culvert with a new concrete culvert.

Performance of Existing Foundations

An inspection of the existing culvert from a geotechnical perspective did not reveal any indications of current problems associated with slope stability, bearing capacity, settlement or scour of the existing culvert foundations.

It was noted that the asphalt surface was unusually thick (220 to 420 mm) and that a buried layer of asphalt was identified between a depth of 400 mm and 500 mm below top of pavement in one borehole. These observations combined with the fact that the site is underlain by firm to soft clay suggest that the highway embankment in the vicinity of this culvert may have undergone settlement in the past. The additional lifts of asphalt may have been placed to correct the grades after settlement had reached unacceptable levels.

Proposed Structure

It is understood that the proposed culvert will consist of a 6.0 m x 2.5 m x 33.25 m concrete box culvert. The new culvert will be located at the same location and along the same alignment (27.1° skew) as the existing culvert.

It is understood that both a pre-cast concrete box and a rigid frame open footing culvert are options for this site.

Key elevations associated with the proposed culvert replacement are as follows:

Pavement Elevation	225.84 m (approximate near C/L)
Invert Elevation:	221.52 m South End (approx) 221.52 m North End (approx)
Creek Water Elevation:	222.8 m at time of Foundation Investigation (summer 2010)
Founding Elevation	≥ 221.1 m Pre-cast Concrete Box Culvert ≥ 219.8 m Cast-in-place Open Footing Culvert

Construction Staging & Detours

The existing platform width is approximately 15.5 m from shoulder rounding to shoulder rounding in the area of the culvert.

An assessment of the staging options by McIntosh Perry has identified that the work can be completed using two stages by providing one 3.5 m wide lane with traffic controlled by temporary signals. This approach will necessitate the use of a temporary protection system to support the open lane of Highway 17 and widening of the pavement surface across the existing granular shoulders. A platform widening is not required. A copy of the Draft Staging Drawings is provided in Appendix F.

It is noted that this staging would be carried out in conjunction with the proposed culvert removal at 13+631 (Site 46-396/C).

9.2 SOIL SUMMARY

The subsurface conditions observed at this site are presented in detail on the Borehole Records provided in Appendix D. An explanation of the symbols and terms used to describe the Borehole Records is provided Appendix A.

The soil conditions beneath the culvert alignment are highly variable and include over 7 m of soft to firm silty clay beneath the outlet and bedrock above the invert elevation within the eastbound driving lane.

The geotechnical design (bearing resistance and total settlement) will be governed by the weaker soil profile (soft clay) beneath the outlet.

For design purposes, the following soils profile will be used to assess bearing resistance and settlement at the north end of the culvert (outlet):

Table 9.1: Geotechnical Model at Outlet

Elevation (m)		Soil Type	Design Properties
From	To		
223.6	222.8	Silt and sand with organic matter	N/A
222.8	221.5	Silty clay, firm	Total Unit Weight = 17.0 kN/m^3 Undrained Shear Strength, Su = 50 kPa
221.5	214.7	Silty clay, soft	Total Unit Weight = 16.0 kN/m^3 Undrained Shear Strength, Su =18 kPa
214.7		Inferred Bedrock	Treated as unyielding surface

The 2010 creek water elevation of 222.8 m will be used as the design groundwater elevation.

9.3 STRUCTURE/FOUNDATION OPTIONS

Both a concrete Rigid Frame Open Footing culvert and a precast concrete box culvert are being considered by the design team for replacement of the existing structure.

The following table compares the structure options from a foundations design and constructability perspective:

Table 9.2: Foundation Comparison for Replacement Culvert

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Rigid Frame Open Footing		▪ Slower construction process ▪ Deeper excavation required ▪ More extensive (deeper and longer duration) unwatering required	Medium	▪ increased risk of dewatering problems/construction delays ▪ increased risk of bedrock excavation/construction delays
Precast Concrete Box	▪ Use of precast sections minimizes construction period ▪ Wide bottom increases the ultimate bearing resistance and distributes load over a wider area, thereby reducing settlement.		Low	▪ If not properly installed, leakage and loss of backfill could occur at joints/settlement of roadway platform

Although both options are technically feasible, the use of pre-cast concrete box culvert sections is better suited to minimizing potential differential settlement associated with the variable ground conditions, is less expensive and requires a shorter construction period which limits impacts to traffic. Therefore, the use of a closed box culvert supported by the native soil and rock is the recommended foundation approach.

9.4 SEISMIC DESIGN CONSIDERATIONS- SOIL PROFILE TYPE

The soil profile varies significantly beneath the proposed culvert alignment. The critical soil profile with respect to seismic design is the soft to firm silty clay deposit beneath the north end. It is recommended that Soil Profile III as defined in Section 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC) be used in the seismic design of this site.

9.5 FOUNDATION RECOMMENDATIONS

9.5.1 Bearing Resistance

It is recommended that the new culvert consist of a precast concrete box culvert founded on undisturbed native soil or rock or structural fill overlying undisturbed native soil or rock. Based on these founding conditions, the geotechnical resistances provided in Table 9.3 may be used for design.

Table 9.3: Recommended Box Culvert Design Parameters

Founding Element	Founding Elev. (m)	Footing Size (m x m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Box culvert on silty clay	221.1 (approximate)	6.8 x 33.25	55	N/A

In accordance with Section 6.6.1 of the CHBDC, a resistance factor of 0.5 has been applied to calculate the factored geotechnical resistance at ULS. The low bearing resistance at ULS reflects the presence of the weak clay zone at a shallow depth below the founding elevation.

The geotechnical reaction at SLS is identified as non applicable (N/A). It is noted that the proposed replacement has approximately the same dimensions and is on the same alignment as the existing culvert, therefore, settlement of the underlying soils is not expected to be a concern as there is no net increase in anticipated load.

Bedding beneath the culvert should consist of a 75 mm thick uncompacted OPSS Granular A leveling course over 200 mm of OPSS Granular A, compacted to at least 95% standard Proctor maximum dry density (SPMDD). The leveling course in the northern two thirds of the culvert should be increased to a single 500 mm thick lift of OPSS Granular A, place on a non-woven geotextile (type II, FOS 50-150 µm).

9.5.2 Sliding Resistance

The unfactored horizontal resistance of spread footings may be calculated using an unfactored coefficients of friction of 0.35 between OPSS Granular A and the pre-cast concrete.

9.6 CONSTRUCTION CONSIDERATIONS

9.6.1 Construction Staging

The proposed staging concept involves two stage construction as shown on the Draft Staging Plans provided in Appendix F. This option would require the use of temporary roadway protection near the centerline of the highway. Further discussion regarding temporary protection systems is provided in Section 9.6.3.

9.6.2 Excavation and backfilling

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

All vegetation, fill, organic soils and other deleterious materials must be removed from beneath the proposed box culvert foundation. Where deleterious materials are encountered, the material should be excavated, wasted and replaced. The lateral extent of such excavation should include all deleterious material within the influence zone of the foundations.

Side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects. The existing highway embankment fill are considered Type 3 soil above the water level. Above the stream and groundwater level, temporary cut slopes should be no steeper than 1 horizontal to 1 vertical from the base of the excavation. For excavations below stream and/or groundwater levels, the soils should be considered as Type 4 soils and slopes no steeper than 3H:1V will be required. Flatter side slopes or supported excavations may be required.

The bedrock observed in Borehole 10-12 would likely require blasting for excavation.

Grading work for reinstatement of the highway embankment along the existing culvert alignment should be carried out in accordance with OPSS-206 Construction Specification for Grading and SP 206S03 using OPSS Select Subgrade Material.

Reinstatement of the roadway embankment should include side slopes no steeper than 2.5H:1V.

9.6.3 Temporary Protection Systems

Two options for holding back the existing Highway 17 embankment during the staging of the culvert replacement were considered – a steel sheet pile (SSP) wall and a soldier pile with timber lagging wall. Due to the presence of shallow bedrock beneath the southern half of the alignment, cantilevering is likely not feasible if a steel sheet pile system is used.

Table F-3 in Appendix F compares the advantages, disadvantages, relative cost and risk/consequences of available roadway protection options considered for the culvert replacement.

Given that the roadway protection system is required to support the roadway during both stages of the culvert replacement, and the variability in the depth to bedrock, H-piles with timber lagging and rakers are likely the most suitable protection system for this site. The contractor will ultimately be responsible to develop and implement a roadway protection system meeting the requirements of OPSS 539, including establishing appropriate geotechnical design parameters.

The roadway protection system, parallel to the centerline of the highway, will need to extend approximately 15 m to each side of the culvert.

Shoring design should meet the requirements of Performance Level 2 as per OPSS 539 and should consider traffic loading. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. Pile and raker spacing must be designed not to exceed these limits. Horizontal movement should be monitored throughout the culvert replacement process as described in OPSS 539. The monitoring requirements outlined in OPSS 539 are considered to be appropriate for this project.

Note that shoring design must include consideration of the blasting vibrations for bedrock excavation.

9.6.4 Unwatering

The underside of the proposed box culvert is approximately 1.2 m lower than the observed groundwater and creek water levels.

Control of the water flow in the creek will require a cofferdam to prevent stream flow into the excavations. It is anticipated that creek flow will be diverted using pumps to allow construction of the replacement culvert.

The native soils within the anticipated depth of excavation generally have a low to moderate hydraulic conductivity. The use of conventional pumps within sumps within the excavation is likely suitable for the excavation unwatering. It is noted that higher inflow volumes may be encountered if fractured bedrock is exposed.

9.6.5 Erosion and Scour Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes and adjacent stream banks. All slopes within 3 m of the culvert inlet and outlet should be surfaced with rip-rap at least 300 mm thick placed on a Class II non-woven filter fabric. Where embankment construction includes earth fill, normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surficial erosion.

The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site.

9.6.6 Cement Type and Corrosion Protection

Two samples of the native soil were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of 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 buried infrastructure. The analysis results are summarized in the Table 9.4.

Table 9.4: Results of Chemical Analysis

Borehole	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
10-8	SS6	5.6	8.00	172	20	29.6
10-9	SS3	1.4	7.33	8	15	142

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Both test results are significantly below the criteria, 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 within what is considered the normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soil do not indicate a highly corrosive environment, however, the test results should be considered in conjunction with other environmental factors such as road deicing practices when selecting coatings and corrosion protection systems for buried steel objects.

10.0 Site No. 46-396/C – Station 13+631, Twp. Of Lorne

10.1 PROPOSED WORK

It is proposed to remove the existing three-cell timber box culvert and to permanently backfill the space and reinstate the highway embankment and pavement structure.

The proposed removal and backfilling will require excavation down to approximately elevation 221.8 m (3.9 m below top of pavement).

Key elevations associated with the proposed culvert removal are as follows:

Pavement Elevation	225.4 m (approximate near C/L)
Invert Elevation:	221.86 m North End (approx) – inlet 221.83 m South End (approx) – outlet
Creek Water Elevation:	222.8 m at time of Foundation Investigation (summer 2010)

Performance of Existing Foundations

An inspection of the existing culvert from a geotechnical perspective did not reveal any indications of current problems associated with slope stability, bearing capacity, settlement or scour of the existing culvert foundations.

It was noted that the asphalt surface was unusually thick (320 to 350 mm) and that a buried layer of asphalt was identified between a depth of approximately 500 mm and 600 mm below top of pavement in both boreholes. These observations combined with the fact that the site is underlain by firm to soft clay suggest that the highway embankment in the vicinity of this culvert has undergone settlement in the past. The additional lifts of asphalt may have been placed to correct the grades after settlement had reached unacceptable levels.

Construction Staging & Detours

It is proposed to carry out the culvert removal and backfilling at this site in conjunction with the culvert replacement at Site 46-395/C, which is located approximately 125 m further west.

The existing platform width is approximately 17.5 m from shoulder rounding to shoulder rounding in the area of the culvert. An assessment of the staging options by McIntosh Perry has identified that the work can be completed using two stages by providing one 3.5 m wide lane with traffic controlled by temporary signals. This approach will necessitate the use of a temporary protection system to support the open lane of Highway 17 and widening of the pavement surface across the existing granular shoulder. A platform widening is not required. A copy of the Draft Staging Drawings is provided in Appendix F.

10.2 SOIL SUMMARY

The subsurface conditions observed at this site are presented in detail on the Borehole Records provided in Appendix E. An explanation of the symbols and terms used to describe the Borehole Records is provided Appendix A.

The soil conditions at this site generally consist of fill over a deep silty clay deposit. A summary plot is provided as Figure F-2 in Appendix F.

For design purposes, the following soils profile will be used:

Table 10.1: Geotechnical Model

Elevation (m)		Soil Type	Design Properties
From	To		
225.7	223.6	FILL: silty sand with gravel, cobbles and boulders	Total Unit Weight = 20.5 kN/m^3 Friction Angle, ϕ = 32º
223.6	220.5	Silty clay, firm	Total Unit Weight = 17.0 kN/m^3 Undrained Shear Strength, Su = 50 kPa Cc=0.356 Cr=0.1 Cα=0.014 e_o=1.38
220.5	218.0	Silty clay, soft	Total Unit Weight = 16.0 kN/m^3 Undrained Shear Strength, Su = 21 kPa Cc=0.544 Cr=0.1 Cα=0.022 e_o=1.788
218.0	202.9	Silty clay, firm	Total Unit Weight = 16.5 kN/m^3 Undrained Shear Strength, Su = 35 kPa Cc=0.427 Cr=0.1 Cα=0.017 e_o=1.54
202.9		Inferred Bedrock	Treated as unyielding surface

The 2010 creek water elevation of 222.8 m will be used as the design groundwater elevation.

The preconsolidation pressure in the silty clay has been estimated based on correlations with plasticity index and undrained shear strength. It is concluded that the silty clay can be considered to be normally consolidated. The time dependent deformation parameters were estimated using the following approaches:

$Cc = CR \cdot (1+e_o)$

Where the compression ratio (CR) is defined as $CR = 0.12 \ln (w_n) - 0.28$ (Lambe and Whitman, 1969)

Cr = 20% of Cc

Representing a conservative estimate according to the FHWA manual "Evaluation of Soil and Rock Properties"

$C\alpha = 0.04\ Cc$ (Mesri, 1994)

10.3 FOUNDATION RECOMMENDATIONS

10.3.1 Settlement

The existing timber cell box culvert has an opening height of 1.0 m and a total height of 1.5 m. This 1.5 m height of wood and open space will be replaced with compacted granular fill (assumed unit weight of 20.5 kN/m^3).

The observed water level was near the top of the culvert opening (see Photo No. E1 in Appendix E). Most of the new fill material will be placed below the water level, minimizing the increase in effective stress on the underlying clay. In addition, the top of the clay was encountered at elevation 223.6 m and 222.3 m in Boreholes 10-10 and 10-11, respectively, both of which are higher than the bottom of the existing culvert (elevation 221.6 m), which suggests that some clay was previously removed in order to embed the base of the existing culvert. Taking these factors into consideration, the net increase in effective stress on the underlying clay is in the range of 10 to 21 kPa.

The settlement due to this new embankment loading was estimated using three dimensional settlement analysis software (Settle 3D by RocScience) and a staged loading approach to model the initial embankment construction and the future culvert in-filling. The soil properties presented in Table 10.1 were used in the analysis.

The first stage of the analysis considered the initial highway embankment construction with loading representative of 3.0 m of granular fill adjacent to the culvert location and 1.5 m of granular fill within the footprint of the culvert. The primary and secondary consolidation profile were calculated for a time period of 20 years after completion of construction. Although the existing culvert is more than 20 years old, primary consolidation due to the original construction would be essentially complete at this time stage and it has been approximately 20 years since the last major rehabilitation that would have triggered new settlement.

The second stage considered removal and backfilling of the culvert with conventional granular fill, and was modeled by application of a second load equal to 1.5 m of granular fill above the footprint of the existing culvert (1 m of which would be submerged). The primary and secondary consolidation versus time was then calculated for the ground within approximately 15 m east and west of the centerline of the former culvert. The results of the settlement analysis are presented in Appendix F. The estimated combined primary and secondary settlement beneath the centerline of Highway 17 is as follows:

Table 10.2: Settlement Estimates

Location	Settlement Estimates at Time Periods Following Culvert Removal		
	1 years	**5 years**	**20 years**
C/L of culvert	45 mm	60 mm	100 mm
5 m offset from C/L of culvert	10 mm	30 mm	70 mm
Differential Settlement within 5 m of culvert C/L	35 mm	30 mm	30 mm
Differential Settlement Ratio	140:1	160:1	160:1

The estimated total and differential settlement are within the acceptable settlement tolerances outlined in MTO document: Embankment Settlement Criteria for Design (March 2, 2010) for new embankments, non-freeways on compressible soils (200 mm total settlement and 100:1 differential settlement rate during pavement design life (in this case 20 years)).

The estimated differential settlement will exceed the post-construction settlement criteria for transitions (maximum 25 mm within 0-20 m from the culvert location) and will therefore require pavement maintenance activities to maintain a desirable pavement performance. It is anticipated that asphalt patching will be required during the year following the culvert the removal and likely once more within the following 10 years.

10.3.2 Embankment Stability

The existing embankment slopes are approximately 3H:1V and do not exhibit evidence of global instability. It is understood that the backfilling will be carried out to match the existing embankment geometry immediately east and west of the culvert location.

Stability of the embankment in its final configuration for Site 46-396/C was analyzed using commercially available slope stability software (Slope/W) and the soil parameters provided in Table 10.1. The geometry consists of a 3.6 m high highway embankment slope with 3H:1V slopes. The analysis included a uniformly distributed load of 17.6 kPa to represent traffic loading as per Section 6.9.5 of the CHBDC.

The proposed final embankment geometry and materials were determined to be stable under long term conditions and under seismic loading conditions. Factors of safety of 2.1 and 1.8 were obtained under static and seismic loading conditions, respectively. A copy of the slope stability modeling results is provided in Appendix F as Figures F-4 and F-5.

10.4 CONSTRUCTION CONSIDERATIONS

10.4.1 Construction Staging

The proposed staging concept involves two stage construction as shown on the Draft Staging Plans provided in Appendix F. This option would require the use of temporary roadway protection near the centerline of the highway. Further discussion regarding temporary protection systems is provided in Section 10.4.3.

10.4.2 Excavation and backfilling

All vegetation, fill, organic soils and other deleterious materials must be removed along with the existing timber box culvert. Where deleterious materials are encountered, the material should be excavated, wasted and replaced.

Side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects. The existing highway embankment fill is considered Type 3 soil above the water level. Above the stream and groundwater level, temporary cut slopes should be no steeper than 1 horizontal to 1 vertical from the base of the excavation. For excavations below stream and/or groundwater levels, the soils should be considered as Type 4 soils and slopes no steeper than 3H:1V will be required. Flatter side slopes or supported excavations may be required.

Grading work for reinstatement of the highway embankment along the existing culvert alignment should be carried out in accordance with OPSS-206 Construction Specification for Grading and SP 206S03 using OPSS Select Subgrade Material.

10.4.3 Temporary Protection Systems

Two options for holding back the existing Highway 17 embankment during the staging of the culvert removal and backfilling were considered – a steel sheet pile (SSP) wall and a soldier pile with timber lagging wall.

The selection of a protection system for this site will need to consider the limited resistance offered by the soft clay and the presence of obstructions within the existing embankment fill.

Table F-4 in Appendix F compares the advantages, disadvantages, relative cost and risk/consequences of available roadway protection options considered for the culvert replacement.

Due to the high risk of damage or inability for sheet piles to penetrate through obstructions in the existing embankment fill, H-piles with timber lagging are likely the most suitable protection system for this site. Given the soft clay conditions, it is anticipated that the H-piles will need to penetrate to the base of the clay layer and that rakers will be used to provide lateral resistance.

Alternatively, the obstruction could be removed by partial excavation. In this case a cantilevered sheet pile would become more favourable.

The contractor will ultimately be responsible to develop and implement a roadway protection system meeting the requirements of OPSS 539, including establishing appropriate geotechnical design parameters.

The roadway protection system, parallel to the centerline of the highway, will need to extend approximately 12 to 15 m to each side of the culvert.

Shoring design should meet the requirements of Performance Level 2 as per OPSS 539 and should consider traffic loading. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. Pile and raker spacing must be designed not to exceed these limits. Horizontal movement should be monitored throughout the culvert replacement process as described in OPSS 539. The monitoring requirements outlined in OPSS 539 are considered to be appropriate for this project.

10.4.4 Unwatering

The depth of excavation for removal of the existing culvert is approximately 1 m lower than the observed groundwater and creek water levels.

Control of the water flow in the creek will require a cofferdam to prevent stream flow into the excavations. It is anticipated that flow will be diverted using pumps to allow construction of the replacement culvert.

The native soils within the anticipated depth of excavation generally have a low to moderate hydraulic conductivity. The use of conventional pumps within sumps within the excavation is likely suitable for the excavation unwatering.

10.4.5 Erosion and Scour Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes and adjacent stream banks. Where embankment construction includes earth fill, normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surficial erosion.

The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site.

11.0 Specifications

The following specifications are referenced in this report:

Table 11.1: Specifications Referenced in Report

Document	Title
OPSD 3090.100	Foundation Frost Depths for Northern Ontario
OPSD 803.010	Backfill and Cover for Concrete Culverts
OPSS902	Construction Specification for Excavation and Backfilling - Structures
OPSS 206	Construction Specification for Grading
SP 206S03	Earth Excavation, Grading
OPSS 539	Construction Specification for Temporary Protection System

12.0 References

ASTM 4.08. Standard D1586-99: Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

ASTM 4.08. Standard D2216-98: Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.

ASTM 4.08. Standard D2487-00: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

ASTM 4.08. Standard D422-63: Standard Test Method for Particle-Size Analysis of Soils.

Canadian Geotechnical Society. Canadian Foundation Engineering Manual, 4th Edition. Richmond: BiTech Publisher Ltd, 2006.

Canadian Standards Association. Concrete Materials and Methods of Concrete Construction.: CSA Standards A23.1-04. Mississauga, Ontario: Canadian Standards Association, 2004.

Chapman, L.J., and Putnam, D.F. The physiography of southern Ontario; Ontario Geological Survey, Special Volume 2. Toronto: Ontario Research Foundation, Ontario Geological Survey, 1984.

CHBDC, 2006. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.

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

Ministry of Labour. Occupational Health and Safety Act and Regulations for Construction Projects. Toronto, Ontario: Publications Ontario, 2002.

Ministry of Transportation. Ontario Provincial Standards for Roads and Municipal Services. Downsview, Ontario: Ministry of Transportation, 1998.

Mononobe, N. “Earthquake-Proof Construction of Masonry Dams,” Proceedings of the World Engineering Conference, 9: 1929.

NAVFAC DM-7.2. Foundation and Earth Structures. Department of the Navy Naval Facilities Engineering Command, Alexandria, VA, 1982.

Okabe, S. “General Theory of Earth Pressure, ”Journal of the Japanese Society of Civil Engineers. 12(1): 1926.

13.0 Closure

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

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

This report has been prepared by Paul Carnaffan and Raymond Haché. Technical review was carried out by Fred Griffiths.

Respectfully submitted,

STANTEC CONSULTING LTD.

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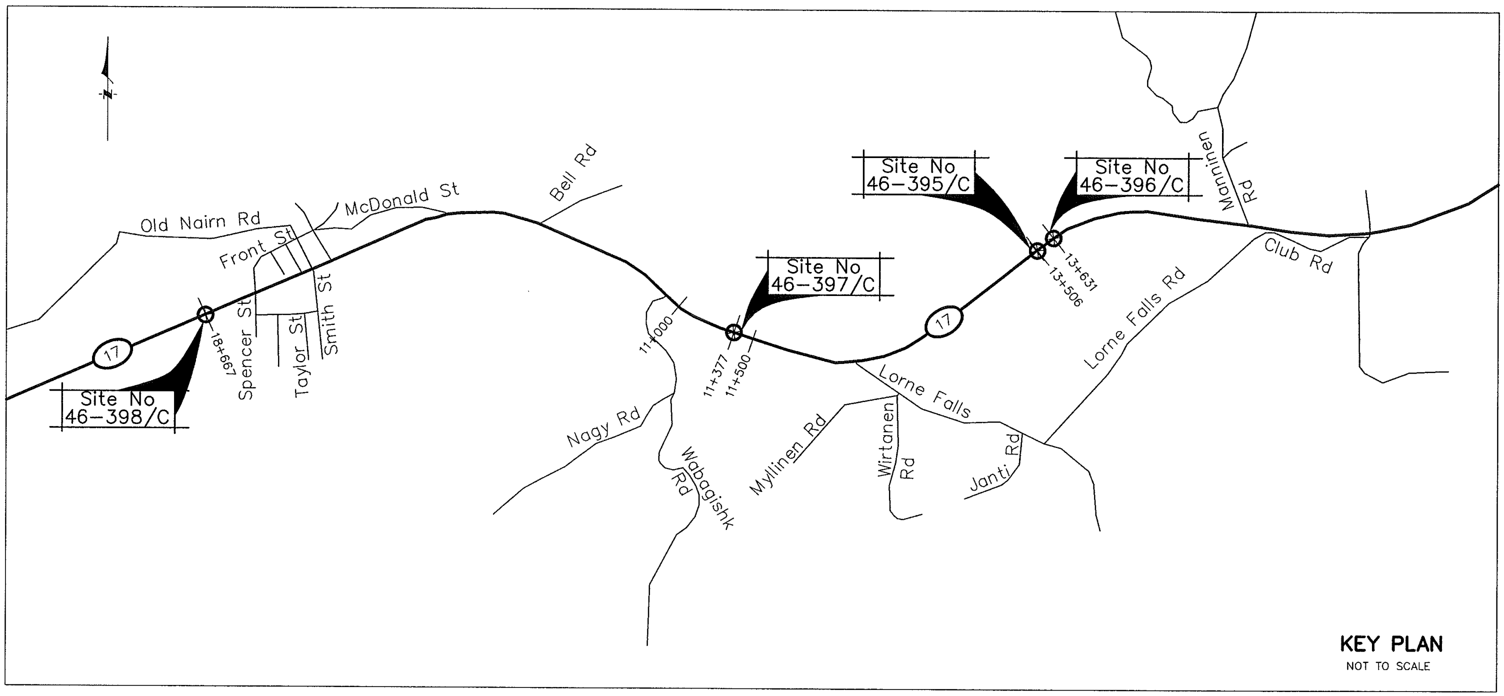
APPENDIX A

Drawing No. 1 – Culvert Location Plan
Symbols and Terms Used on Borehole and Test Pit Records

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No CONT GWP 5182-08-00		
KEY PLAN		SHEET



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

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No 411-263

HWY No 17			DIST
SUBM'D KP	CHECKED	DATE 2010-09-07	SITE
DRAWN GBB	CHECKED	APPROVED	DWG 1

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:

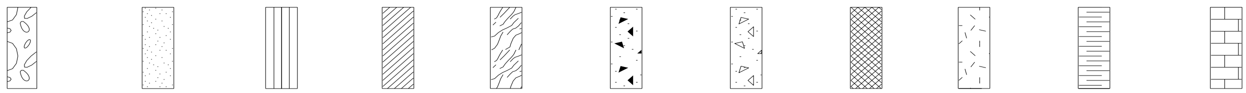
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

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 | Igneous Bedrock | Meta-morphic Bedrock | Sedi-mentary 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

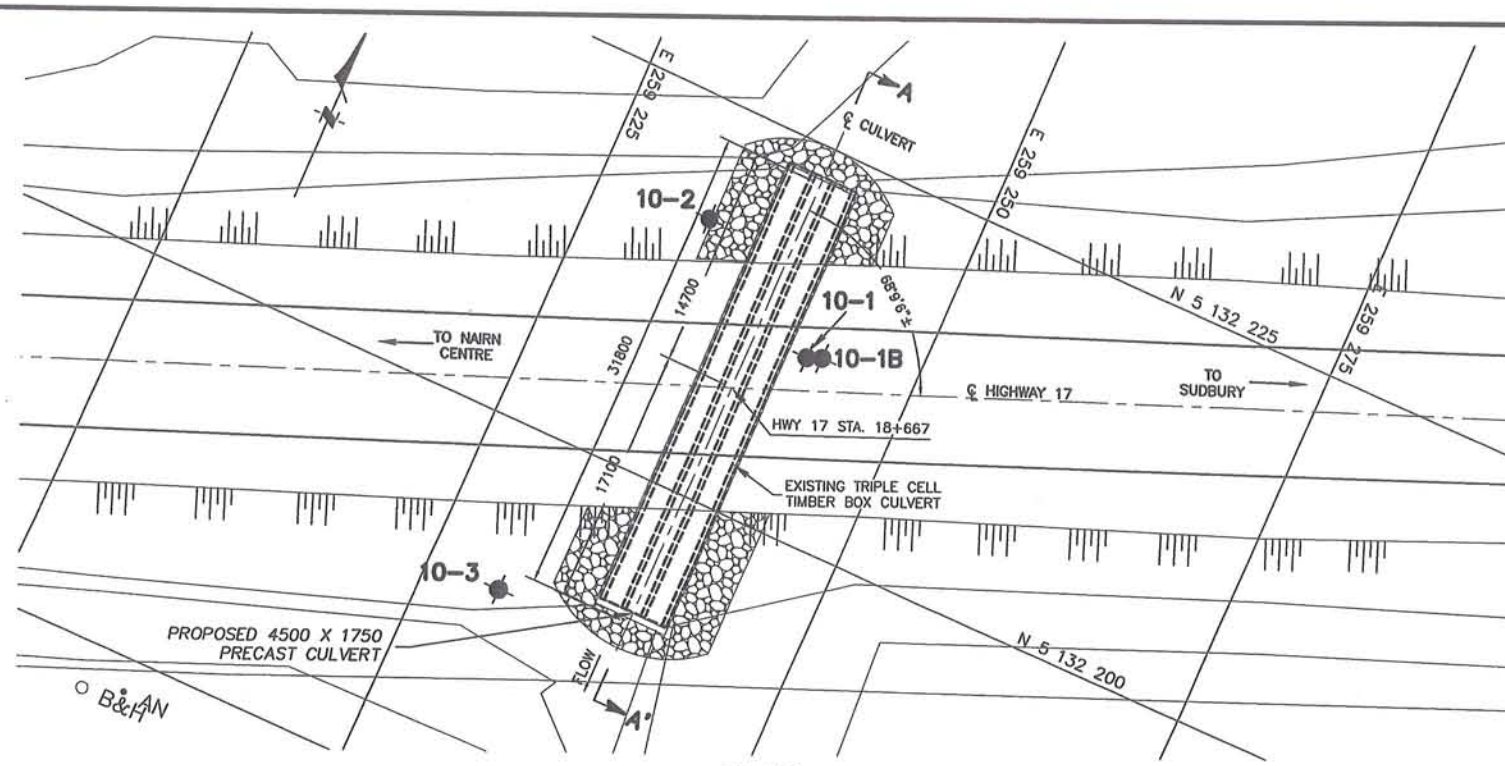
S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined 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

APPENDIX B

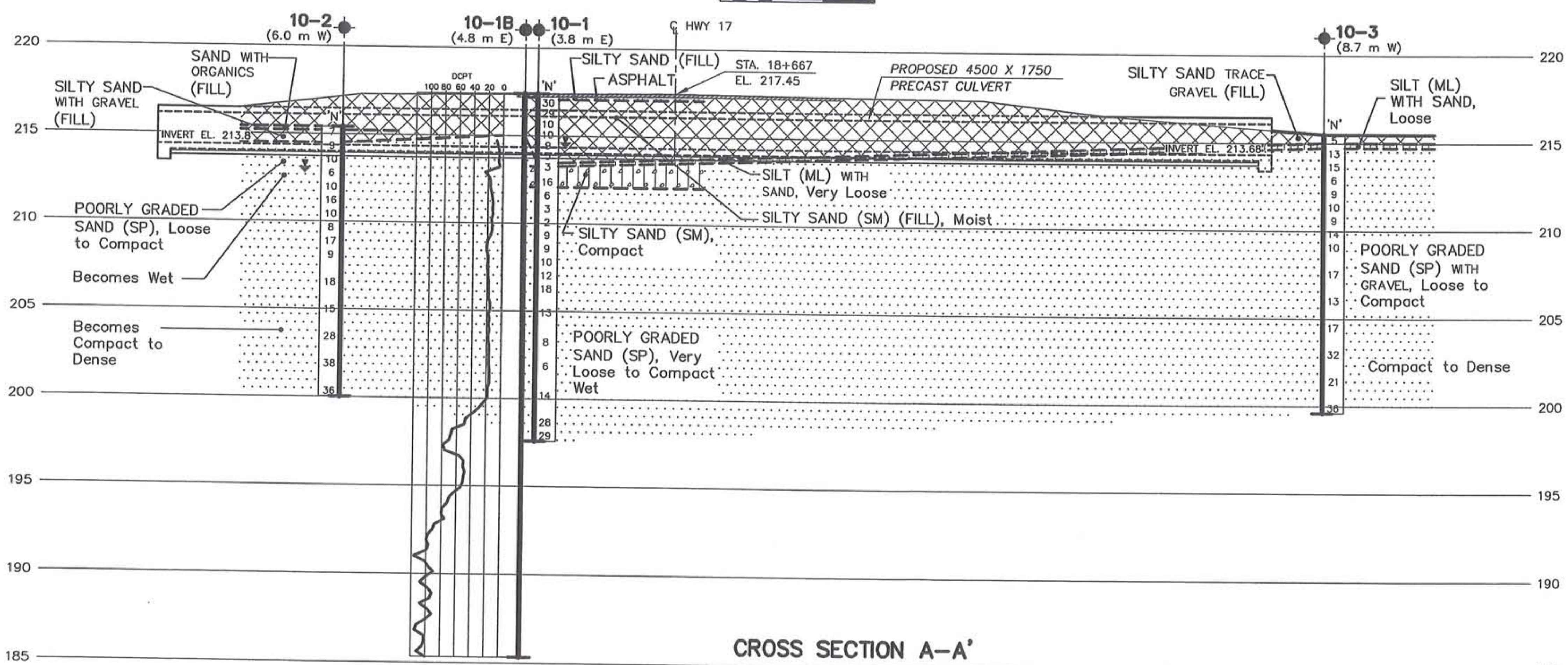
Site No. 46-398/C - Station 18+667

Borehole Location Plan and Stratigraphic Section
Borehole Records
Laboratory Test Results



PLAN

SCALE 5 m 0 5 10 m



CROSS SECTION A-A'

SCALE 1.5 m 0 1.5 3 m HORIZONTAL

3 m 0 3 6 m VERTICAL

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

PLATE No

CONT GWP 5182-08-00

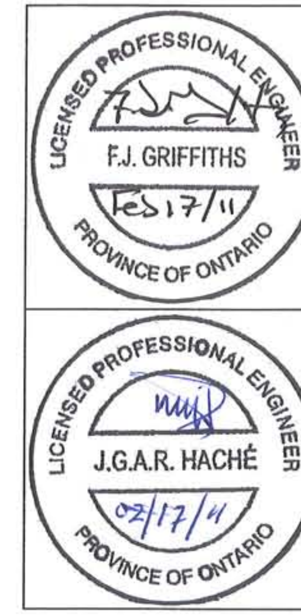
SHEET

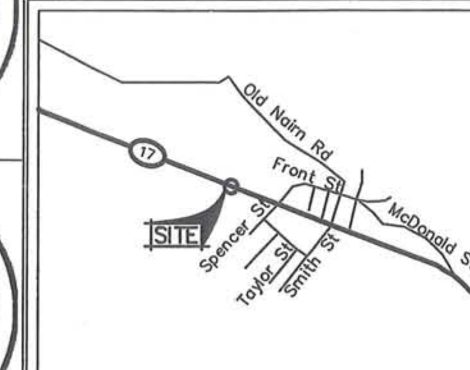
CULVERT AT STA 18+667

STA TO STA

BOREHOLE LOCATIONS & SOIL STRATA

Stantec





KEY PLAN

1 km 0 1 2 km

LEGEND

- Bore Hole
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at time of investigation July 2010

No	ELEVATION	MTM ZONE 12 COORDINATES NORTH	EAST
10-1	217.4	5 132 211.8	259 242.6
10-1B	217.4	5 132 212.2	259 243.5
10-2	215.4	5 132 217.6	259 231.7
10-3	215.4	5 132 189.3	259 230.1

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

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

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No 41I-263

HWY No 17			DIST
SUBM'D KP	CHECKED	DATE 2010-09-07	SITE 46-398/C
DRAWN GBB	CHECKED	APPROVED	DWG 2

Printed: Feb 18, 2011

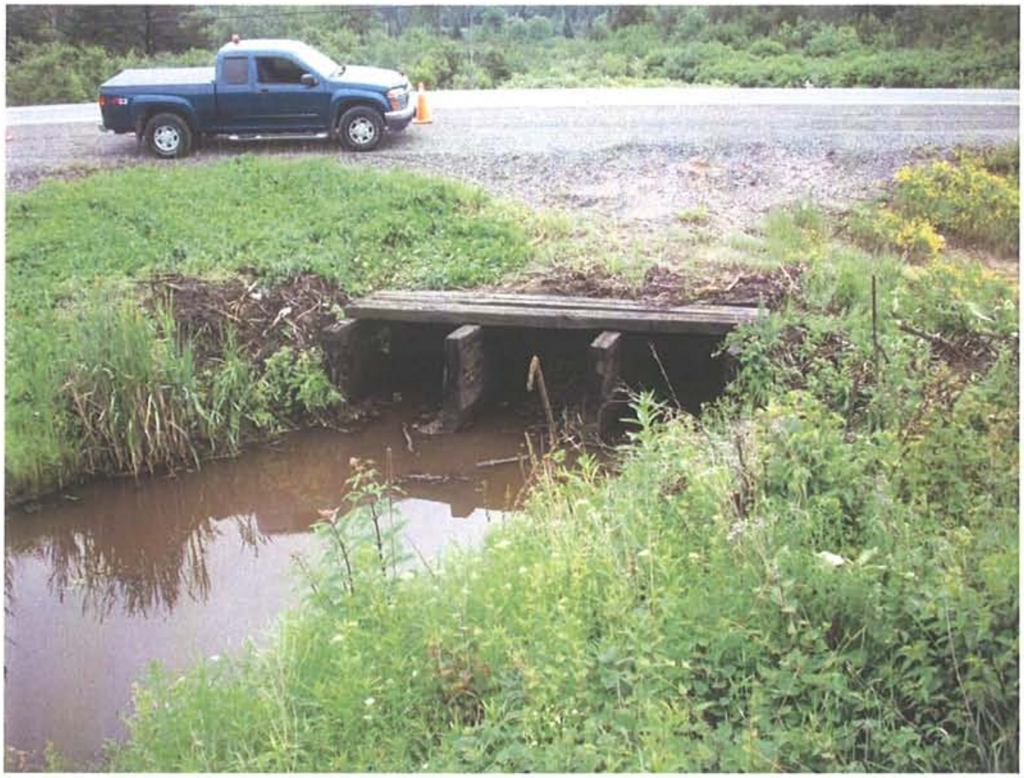


Photo No. B1: Site No. 46-398/c. North side of culvert of 18+667

Photo No. B2: Site No. 46-398/c. South side of culvert at 18+667.

Photo No. B3: Site No. 46-398/c. Erosion of ditch slopes beside culvert.

Photo No. B4: Site No. 46-398/c. Erosion at edge of culvert on south side.

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RECORD OF BOREHOLE No 10-1

METRIC

W.P. 5182-08-00	LOCATION 18+672 2.20 m Lt CL	N: 5 132 212 E: 259 243	ORIGINATED BY AS
DIST	HWY 17	BOREHOLE TYPE Splitspoons, Hollow Stem Augers	COMPILED BY AS
DATUM Geodetic	DATE 2010 07 05 - 2010 07 05		CHECKED BY FG

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
217.4	Ground Surface
217.2 / 0.2	210 mm ASPHALT
217.1 / 0.3	Silty sand (SM), brown, FILL
	Silty sand (SM), brown, FILL, moist - becomes wet
213.5 / 3.9	Silt (ML) with sand, very loose, dark brown, wet
213.3 / 4.1	Silty sand (SM), compact, grey, wet
212.1 / 5.3	Poorly graded sand (SP), very loose to compact, brownish grey, wet
197.6 / 19.8	End of Borehole Groundwater was observed in open borehole at depth of 2.7 m

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	30	8 69 (23)
2	SS	29	
3	SS	10	
4	SS	10	
5	SS	9	
6	SS	5	2 83 (15)
7	SS	3	3 14 (83)
8	SS	16	10 47 (43)
9	SS	6	0 97 (3)
10	SS	3	
11	SS	2	
12	SS	9	
13	SS	9	
14	SS	10	
15	SS	12	1 95 (4)
16	SS	18	
17	SS	13	
18	SS	8	
19	SS	6	
20	SS	14	
21	SS	28	3 92 (5)
22	SS	29	

GROUND WATER CONDITIONS ∇

ELEVATION SCALE: 216, 214, 212, 210, 208, 206, 204, 202, 200, 198

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

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

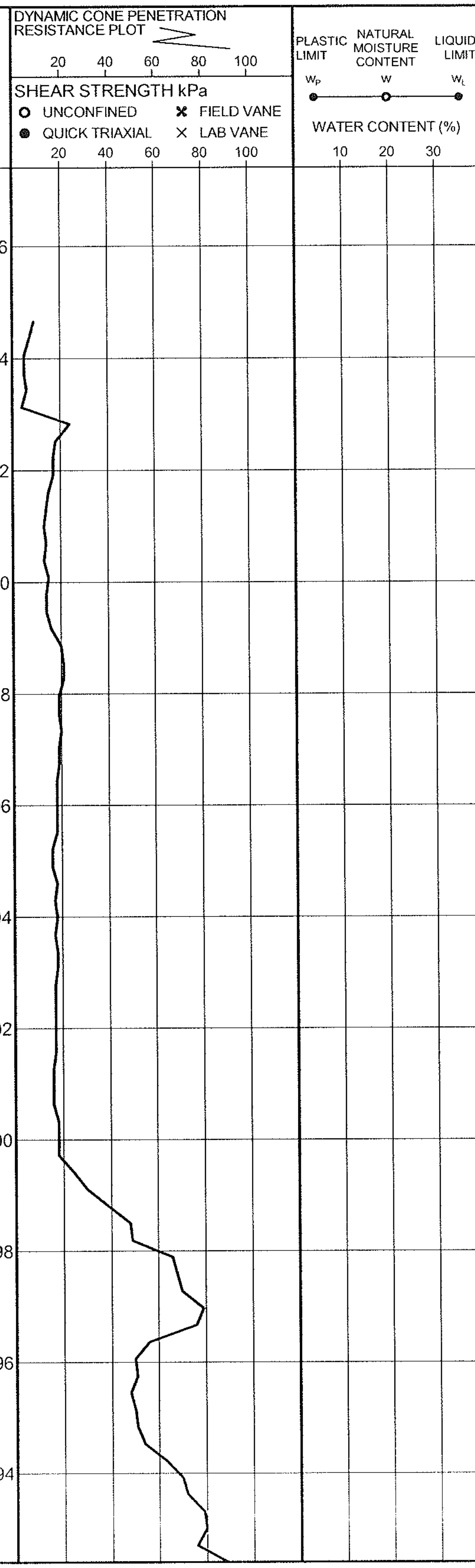
Stantec

RECORD OF BOREHOLE No 10-1B

METRIC

W.P.	5182-08-00	LOCATION	18+673 2.20 m Lt CL N: 5 132 212 E: 259 244	ORIGINATED BY	AS
DIST	HWY 17	BOREHOLE TYPE	Hollow Stem Augers, Dynamic Cone Test	COMPILED BY	AS
DATUM	Geodetic	DATE	2010 07 06 - 2010 07 06	CHECKED BY	FG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
217.4	Ground Surface							
217.2 / 0.2	210 mm ASPHALT							
	Auger to 2.4 m						216	
215.0 / 2.4	Dynamic Cone Penetration Test						214	
							212	
							210	
							208	
							206	
							204	
							202	
							200	
							198	
							196	
							194	



Continued Next Page

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

Stantec

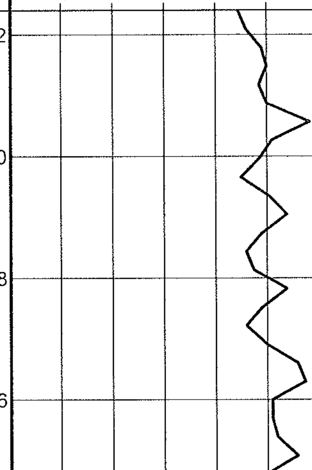
RECORD OF BOREHOLE No 10-1B

METRIC

W.P. 5182-08-00 LOCATION 18+673 2.20 m Lt CL N: 5 132 212 E: 259 244 ORIGINATED BY AS

DIST ____ HWY 17 BOREHOLE TYPE Hollow Stem Augers, Dynamic Cone Test COMPILED BY AS

DATUM Geodetic DATE 2010 07 06 - 2010 07 06 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Dynamic Cone Penetration Test *(continued)*						192 190 188 186				
184.8 32.6	Dynamic Cone Refusal at 32.6 m										

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

Stantec

RECORD OF BOREHOLE No 10-2

METRIC

W.P. 5182-08-00 LOCATION 18+665 11.0 m Lt CL N: 5 132 218 E: 259 232 ORIGINATED BY AS

DIST ____ HWY 17 BOREHOLE TYPE Splitspoons, Hollow Stem Augers, NQ Casing COMPILED BY AS

DATUM Geodetic DATE 2010 07 19 - 2010 07 20 CHECKED BY FG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
215.4	Ground Surface											
215.2 / 0.2	Sand with organics, dark brown, FILL		1	SS	7							
214.5 / 0.9	Silty sand with gravel, brown, FILL											
	Poorly-graded sand (SP), loose to compact, brown to grey		2	SS	9		214					
			3	SS	10	∇						
	- becomes wet		4	SS	6			~25	0	92	(8)	
			5	SS	10		212					
			6	SS	16							
			7	SS	10		210					
			8	SS	8			~22	0	95	(5)	
			9	SS	17							
			10	SS	9		208					
			11	SS	18		206					
			12	SS	15			~25	10	89	(1)	
	- becomes compact to dense						204					
			13	SS	28							
			14	SS	38		202					
200.3			15	SS	36			~17	0	93	(7)	
15.1	End of Borehole											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Stantec

RECORD OF BOREHOLE No 10-3

METRIC

W.P. 5182-08-00 LOCATION 18+652 14.0 m Rt CL N: 5 132 189 E: 259 230 ORIGINATED BY AS

DIST HWY 17 BOREHOLE TYPE Splitspoons, NQ Casing COMPILED BY AS

DATUM Geodetic DATE 2010 07 20 - 1010 07 22 CHECKED BY FG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.4	Ground Surface						
0.0	Silty sand, brown, FILL	1	SS	5			8 66 (26)
214.9 0.5	Silt (ML) with sand, loose, dark brown to black						
214.6 0.8	Poorly graded sand (SP) with gravel to poorly graded sand (SP), loose to compact, brown to grey	2	SS	13		214	
		3	SS	15			17 72 (11)
		4	SS	6	▽		
		5	SS	9		212	
		6	SS	10			1 96 (3)
		7	SS	9			
		8	SS	14		210	
		9	SS	10			
		10	SS	17		208	9 87 (4)
		11	SS	13		206	
		12	SS	17		204	
	- becomes compact to dense	13	SS	32			
		14	SS	21		202	1 98 (2)
199.5		15	SS	36		200	
15.9	End of Borehole Groundwater was observed in open borehole at depth of 2.8 m						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

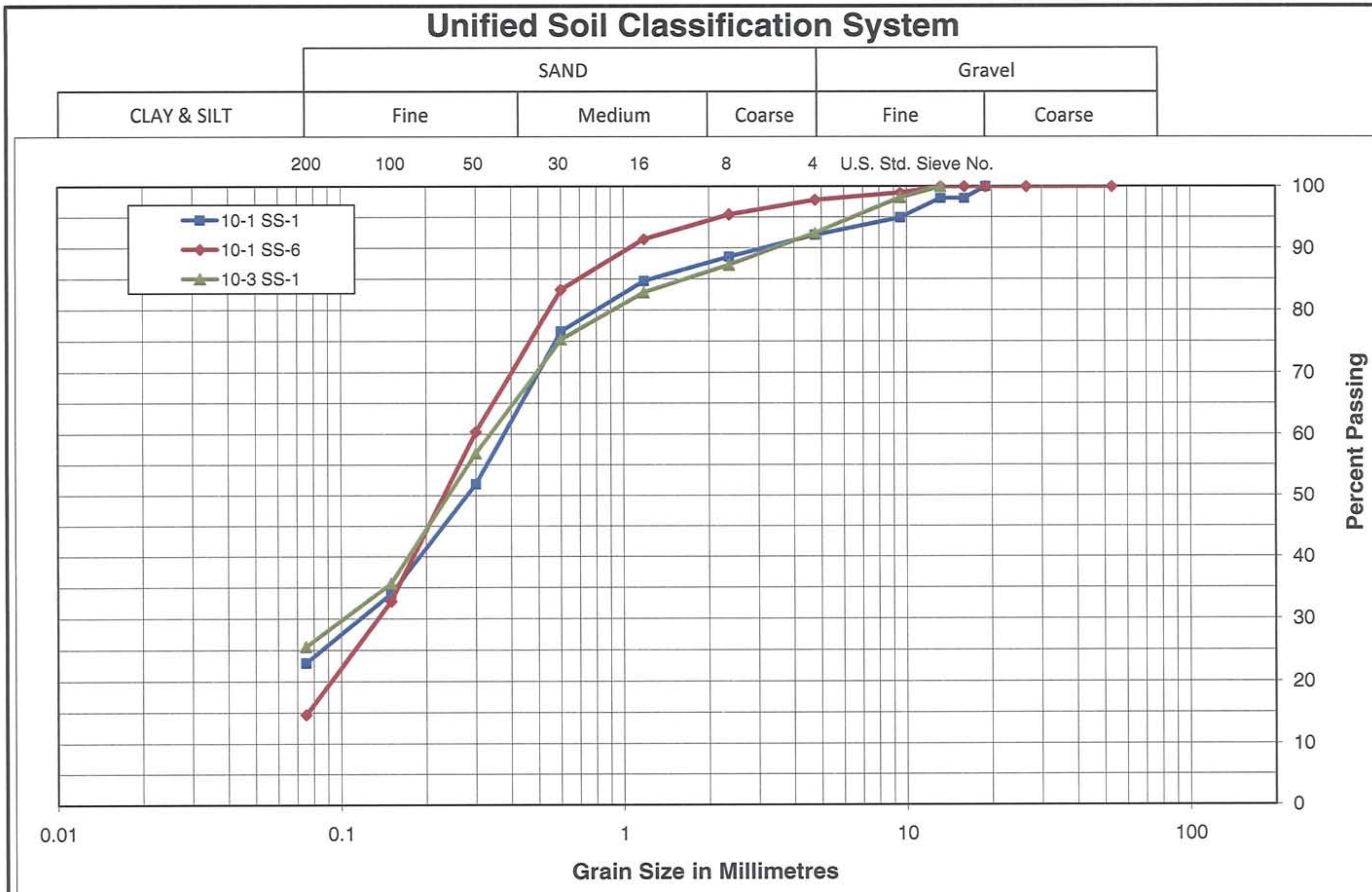
SHEAR STRENGTH kPa: O UNCONFINED, ● QUICK TRIAXIAL, ✖ FIELD VANE, × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

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

ONTARIO MTO STANTEC 122410534 - HWY 17 CULVERT REPLACEMENTS, NAIRN, ON.GPJ ONTARIO MOT.GDT 11-01-12



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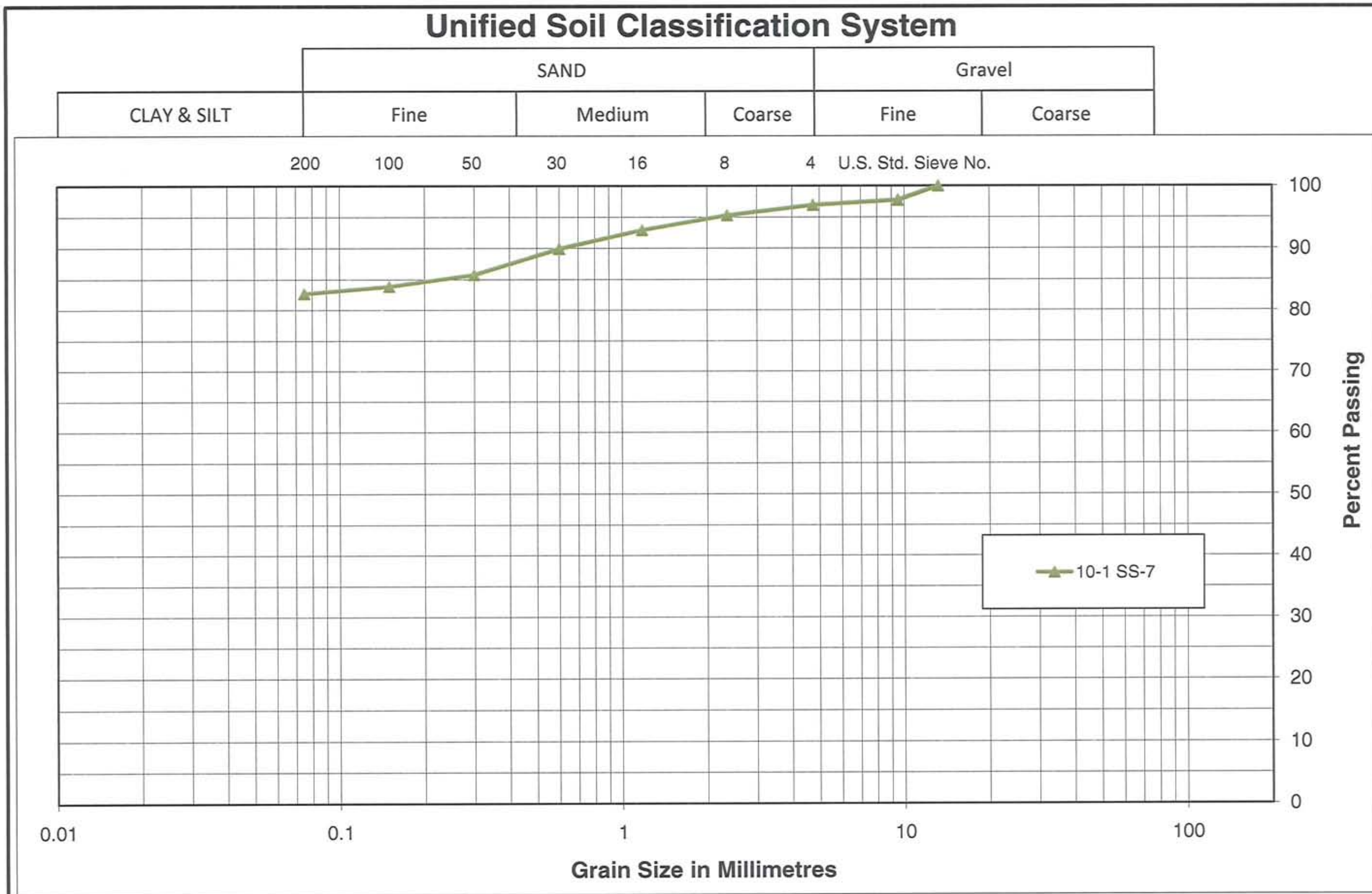
GRAIN SIZE DISTRIBUTION

FILL: silty sand (SM)

Figure No. 1

Project No. 122410534



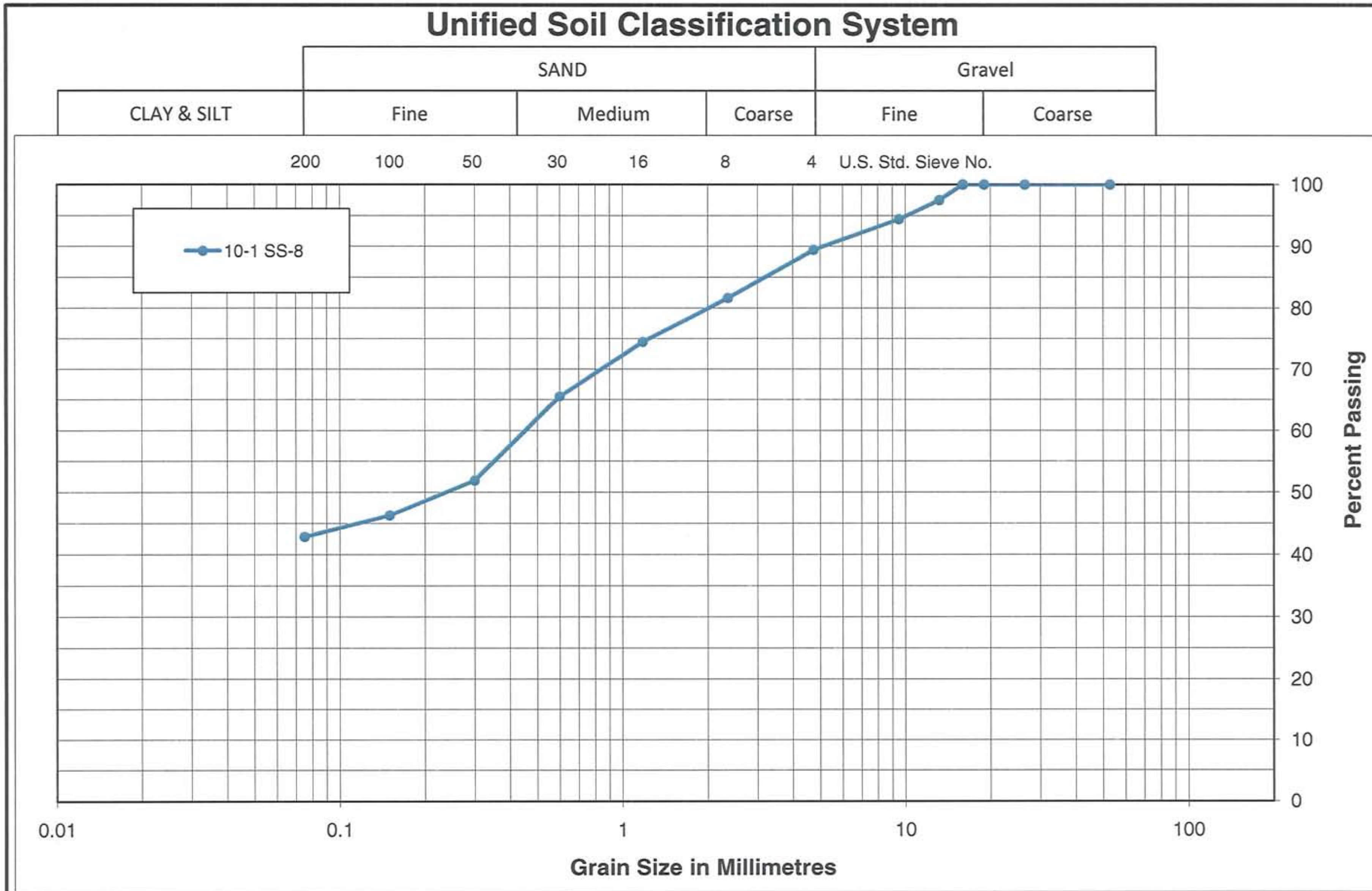
GRAIN SIZE DISTRIBUTION

Silt (ML) with sand

Figure No. 2

Project No. 122410534

Stantec

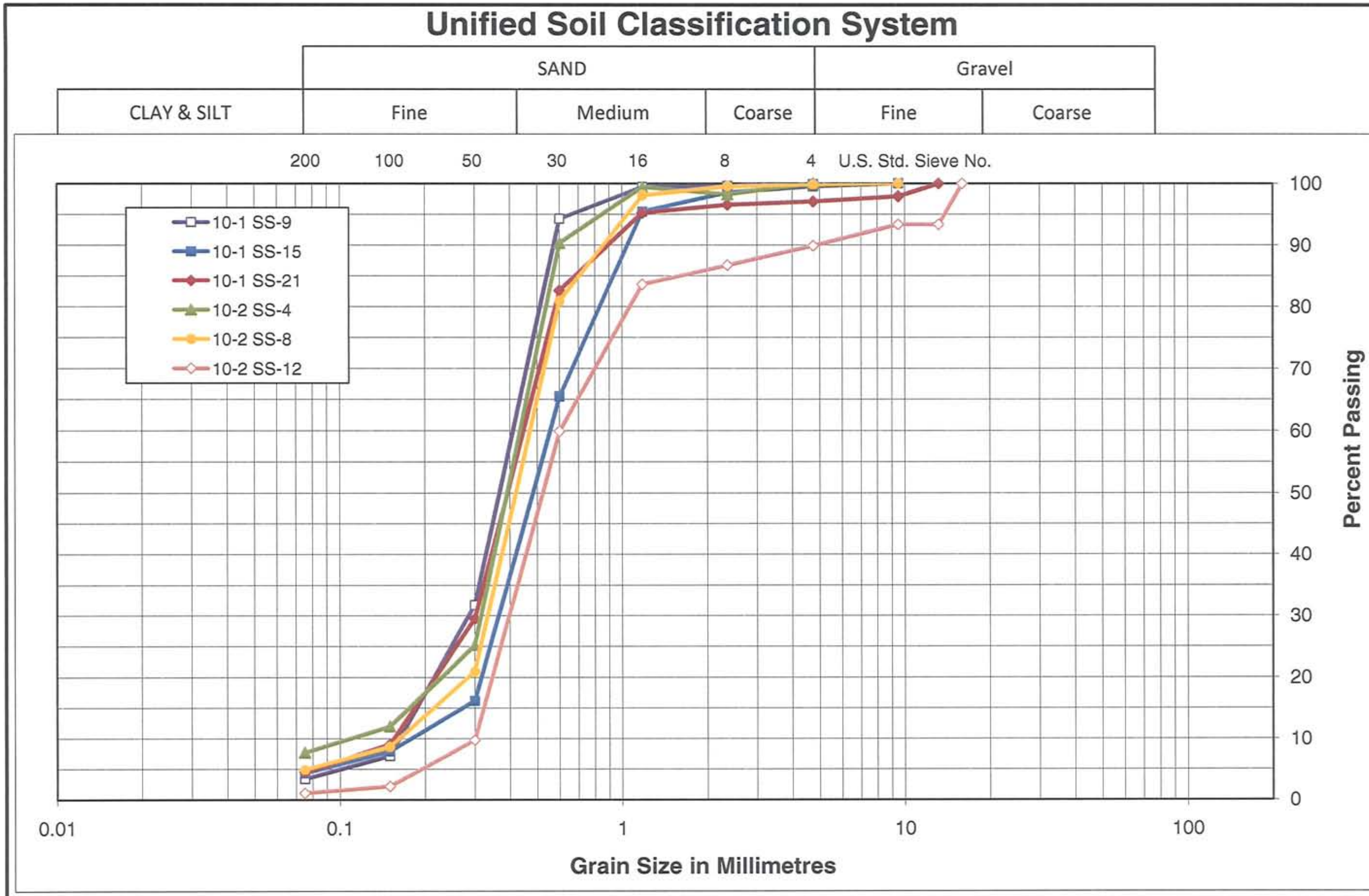


GRAIN SIZE DISTRIBUTION

Silty sand (SM)

Figure No. 3

Project No. 122410534

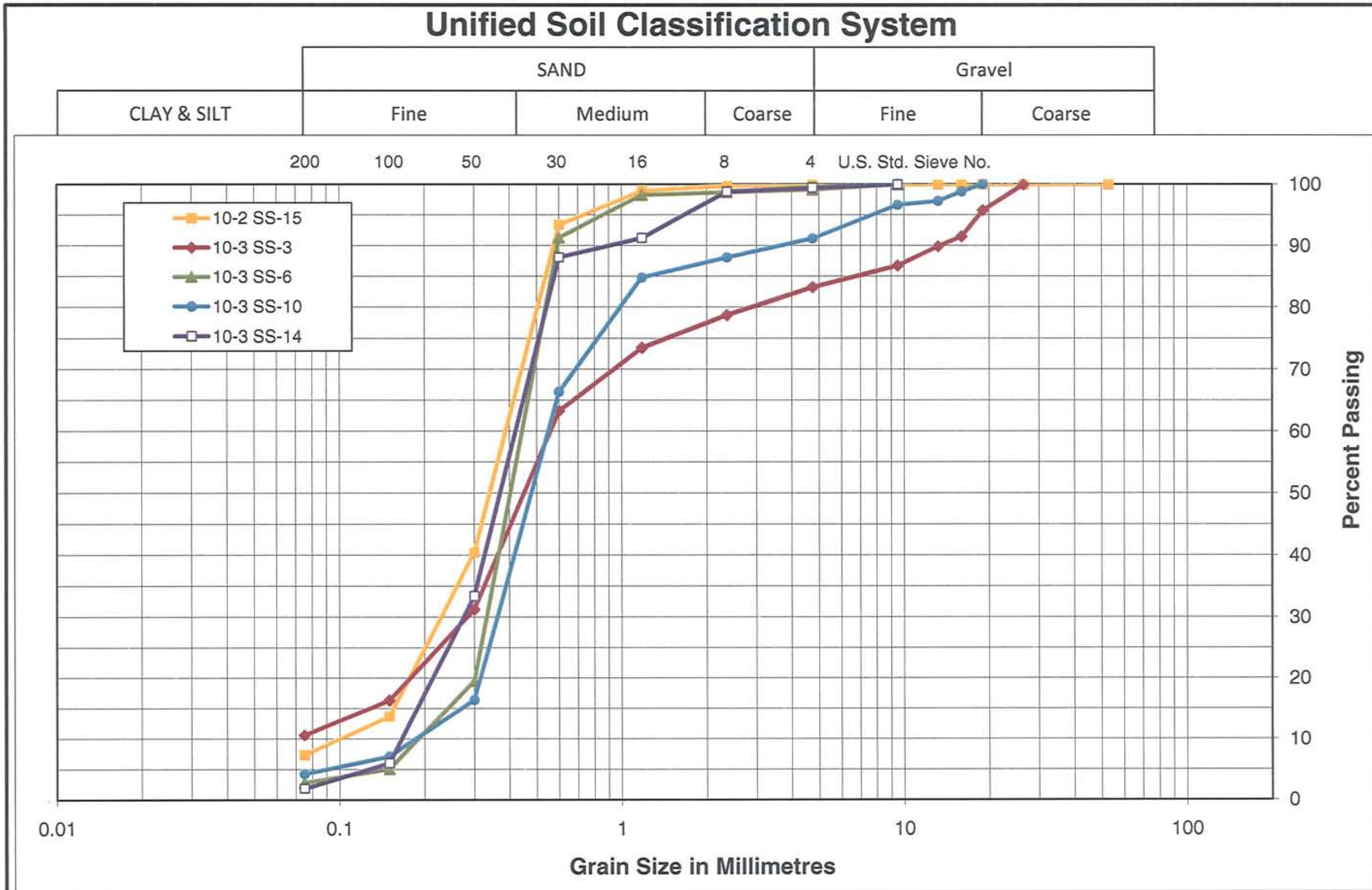


GRAIN SIZE DISTRIBUTION

Poorly graded sand (SP)

Figure No. 4

Project No. 122410534



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GRAIN SIZE DISTRIBUTION

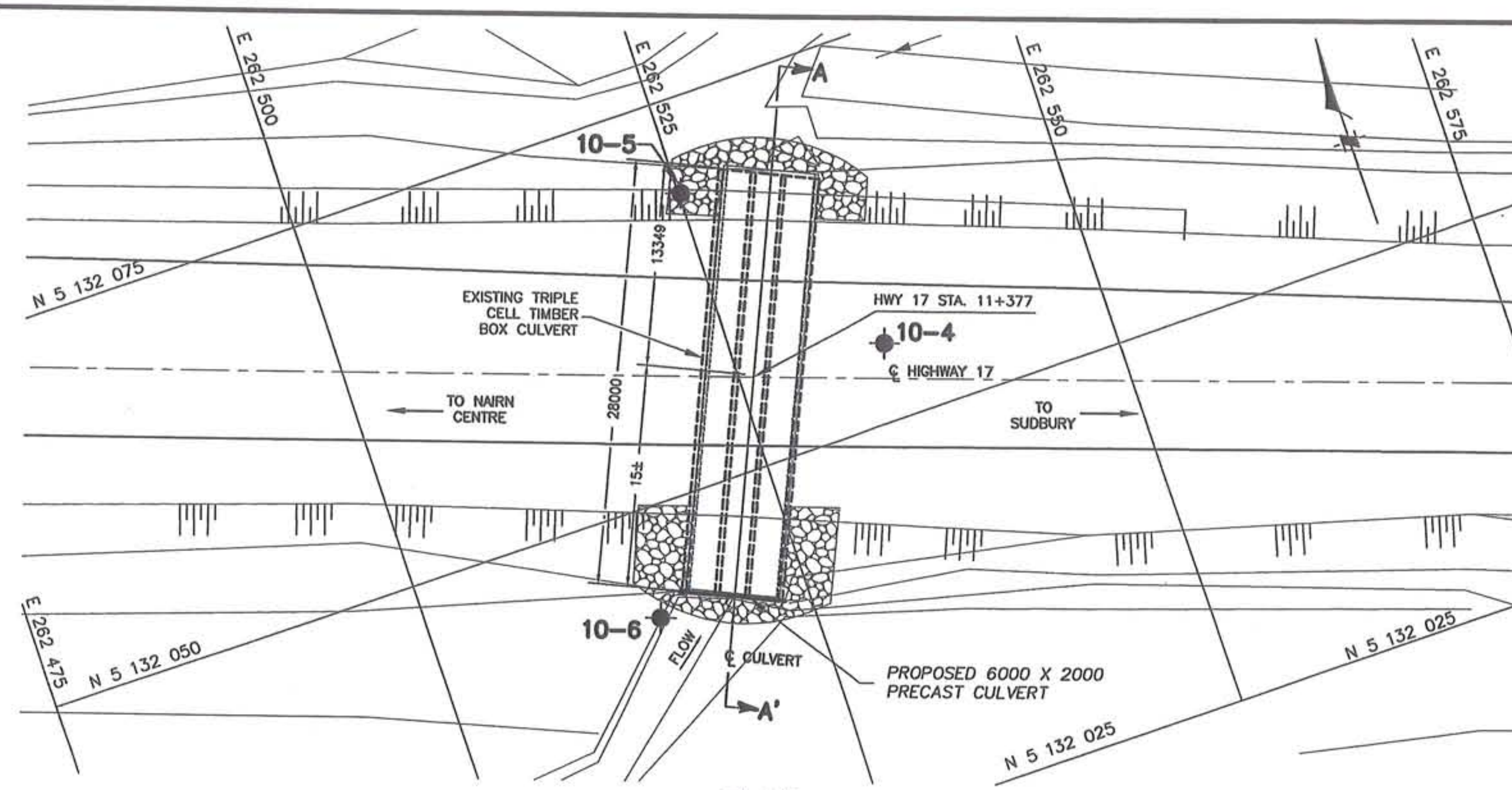
Poorly-graded sand (SP) to well-graded sand with silt and gravel (SW-SM)

Figure No. 5

Project No. 122410534

APPENDIX C

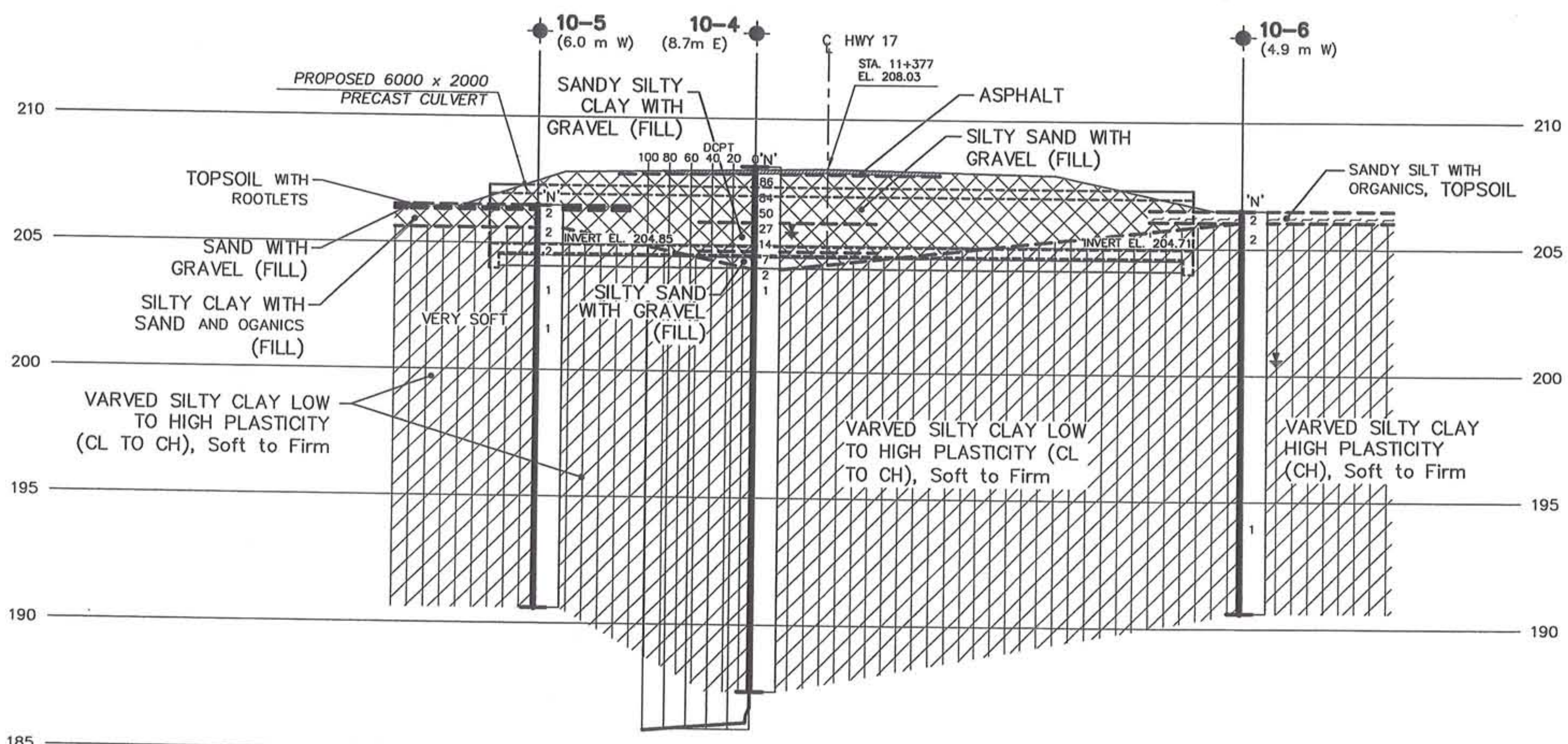
Site No. 46-397/C - Station 11+375
Site Photographs
Borehole Location Plan and Stratigraphic Section
Borehole Records
Laboratory Test Results



PLAN

SCALE

5 m 0 5 10 m



CROSS-SECTION A-A'

SCALE

2.5 m 0 2.5 5 m

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

PLATE No

CONT GWP 5182-08-00

SHEET

CULVERT AT STA 11+377

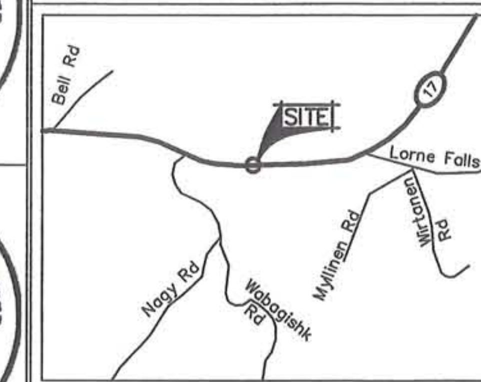
STA TO STA

BOREHOLE LOCATIONS & SOIL STRATA

LICENSED PROFESSIONAL ENGINEER F.J. GRIFFITHS Feb 17/11 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.G.A.R. HACHÉ 11/02/17 PROVINCE OF ONTARIO

Stantec



KEY PLAN

1 km 0 1 2 km

LEGEND

- Bore Hole
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at time of investigation July 2010

No	ELEVATION	MTM ZONE 12 COORDINATES NORTH	EAST
10-4	208.1	5 132 055.1	262 535.1
10-5	206.5	5 132 068.9	262 525.2
10-6	206.5	5 132 042.8	262 514.9

=NOTE=

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

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

REVISIONS DATE	BY	DESCRIPTION

GEOCRES No 41I-263

HWY No 17			DIST
SUBM'D KP	CHECKED	DATE 2010-09-03	SITE 46-397/C
DRAWN GBB	CHECKED	APPROVED	DWG 3

DRAWING NAME: 122410534_1-5 (FEB).DWG CREATED: GBB MODIFIED: 11/02/16 T:\Autocad\Drawings\Project Drawings\2011\122410534\Final\122410534_1-5 (Feb).dwg (SITE 3 (11+377) Printed: Feb 16, 2011

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-707 85-05

Photo No. C1: Site No. 46-397/c. North side of culvert of 11+373.

Photo No. C2: Site No. 46-397/c. South side of culvert at 11+373.

Photo No. C3: Site No. 46-397/c. Varved clay from BH 10-6 ST4.

Photo No. C4: Site No. 46-397/c. Varved clay from BH10-6 ST8.

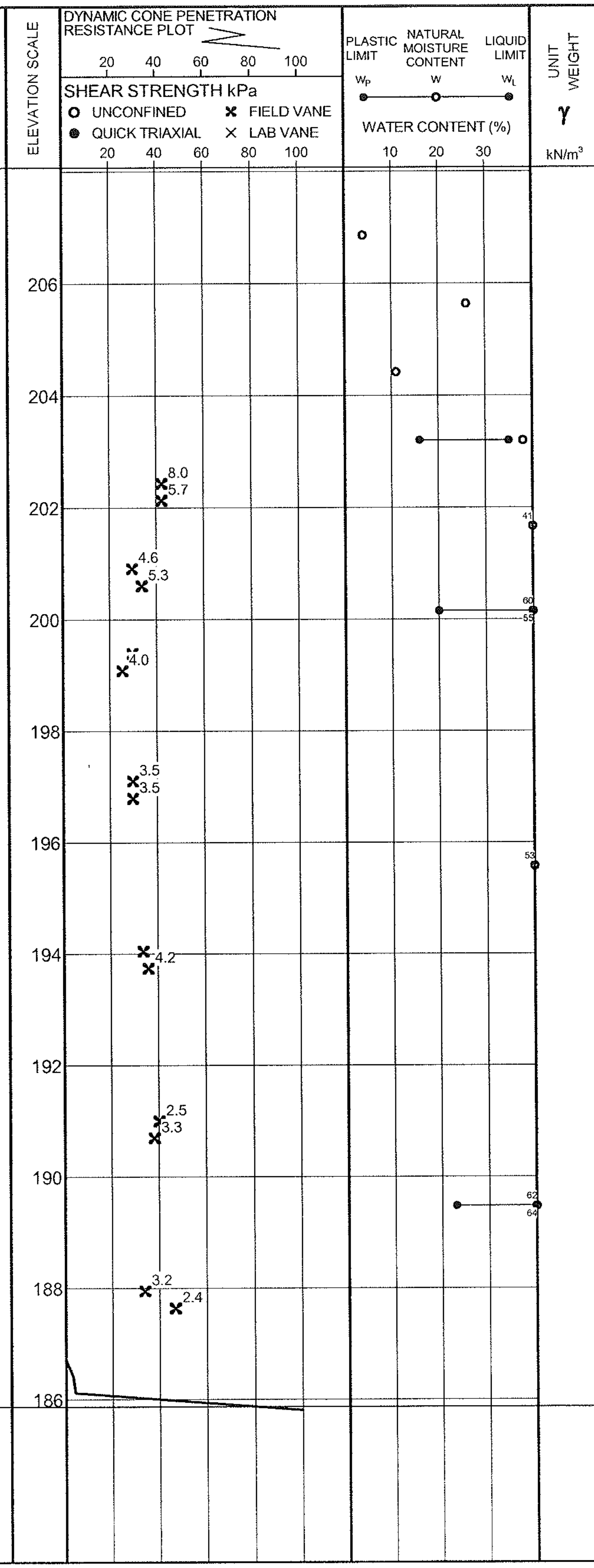
Stantec

RECORD OF BOREHOLE No 10-4

METRIC

W.P. 5182-08-00	LOCATION 11+384 2.30 m Lt CL	N: 5 132 055 E: 262 535	ORIGINATED BY AS
DIST ____ HWY 17	BOREHOLE TYPE Splitspoons, Hollow Stem Augers		COMPILED BY AS
DATUM Geodetic	DATE 2010 07 13 - 2010 07 13		CHECKED BY FG

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
208.1	Ground Surface					
208.0 / 0.0	280 mm ASPHALT					
207.8 / 0.3	Sand and gravel, brown, FILL	1	SS	86		
207.6 / 0.5	100 mm ASPHALT					
0.5	Silty sand (SM) with gravel, brown, FILL	2	SS	84		30 50 (20)
		3	SS	50		
205.9 / 2.2	Sandy silty clay with gravel, brownish grey, FILL	4	SS	27	∇	
		5	SS	14		
204.7 / 3.4	Silty sand (SM) with gravel, brownish grey, FILL	6	SS	7		25 61 (14)
204.0 / 4.1	Varved silty CLAY, low to high plasticity (CL to CH), firm to soft, grey	7	SS	2		
		8	SS	1		0 1 39 60
		9	SS	Wt of, Hmr		
		10	SS	Wt of, Hmr		0 0 21 79
		11	SS	Wt of, Hmr		
		12	SS	Wt of, Hmr		
		13	SS	Wt of, Hmr		
		14	SS	Wt of, Hmr		0 0 14 86
187.3 / 20.7	End of Borehole Start of Dynamic Cone Penetration Test at 20.7 m					
185.9 / 22.2	Dynamic Cone Penetration Test Refusal at 22.2 m Wt of Hmr = Weight of Hammer Groundwater was observed in open borehole at depth of 2.6 m					



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

Stantec

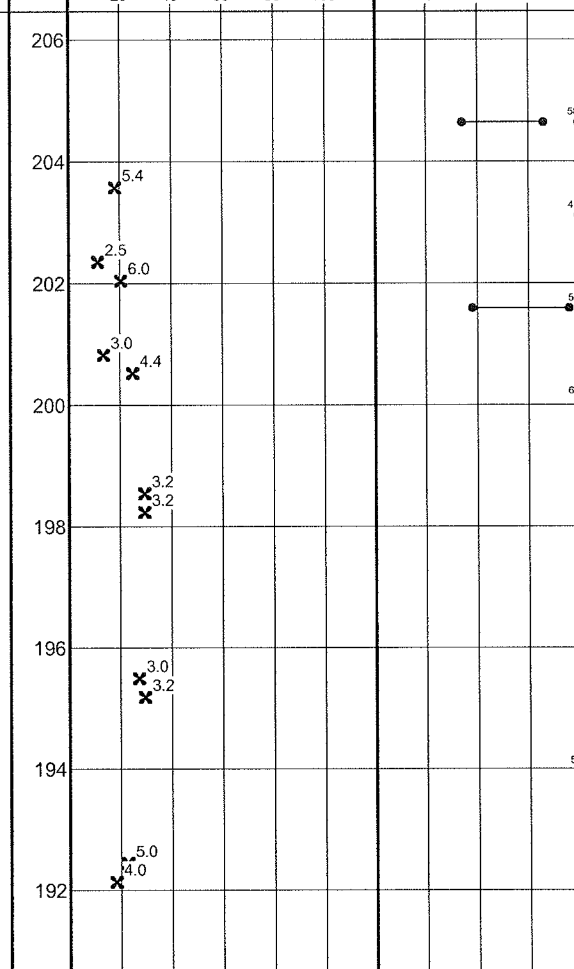
RECORD OF BOREHOLE No 10-5

 METRIC

W.P.	5182-08-00	LOCATION	11+370 12.0 m Lt CL N: 5 132 069 E: 262 525	ORIGINATED BY	AS
DIST	HWY 17	BOREHOLE TYPE	Splitspoons, B Casing	COMPILED BY	AS
DATUM	Geodetic	DATE	2010 07 21 - 2010 07 21	CHECKED BY	FG

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.5	Ground Surface				
206.4 / 0.1	Topsoil with rootlets, dark brown	1	SS	2	
206.3 / 0.2	Sand with gravel, brown, FILL				
205.6 / 0.9	Silty clay with sand and organic matter, dark grey, FILL	2	SS	2	
	Varved silty CLAY, low to high plasticity (CL to CH), soft to firm, grey	3	SS	2	
		4	SS	1	
	- very soft	5	SS	1	0 0 23 77
		6	SS	Wt of Hmr	
		7	SS	Wt of Hmr	
		8	SS	Wt of Hmr	
		9	SS	Wt of Hmr	
190.6 / 15.9	End of Borehole				
	Wt of Hmr = Weight of Hammer				

Other columns: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.



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

Stantec

RECORD OF BOREHOLE No 10-6

METRIC

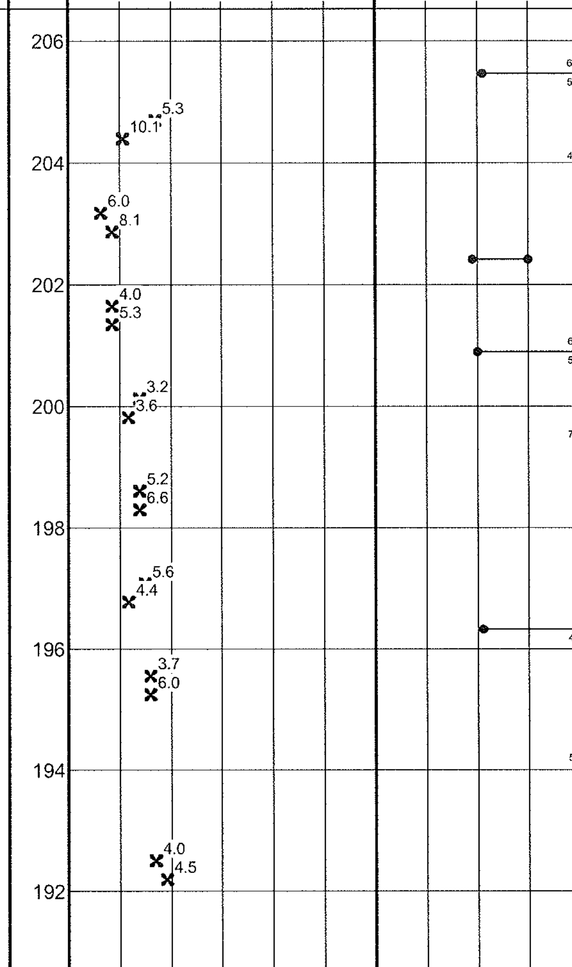
W.P. 5182-08-00 LOCATION 11+369 16.0 m Rt CL N: 5 132 043 E: 262 515 ORIGINATED BY AS

DIST ___ HWY 17 BOREHOLE TYPE Splitspoons, Hollow Stem Augers COMPILED BY AS

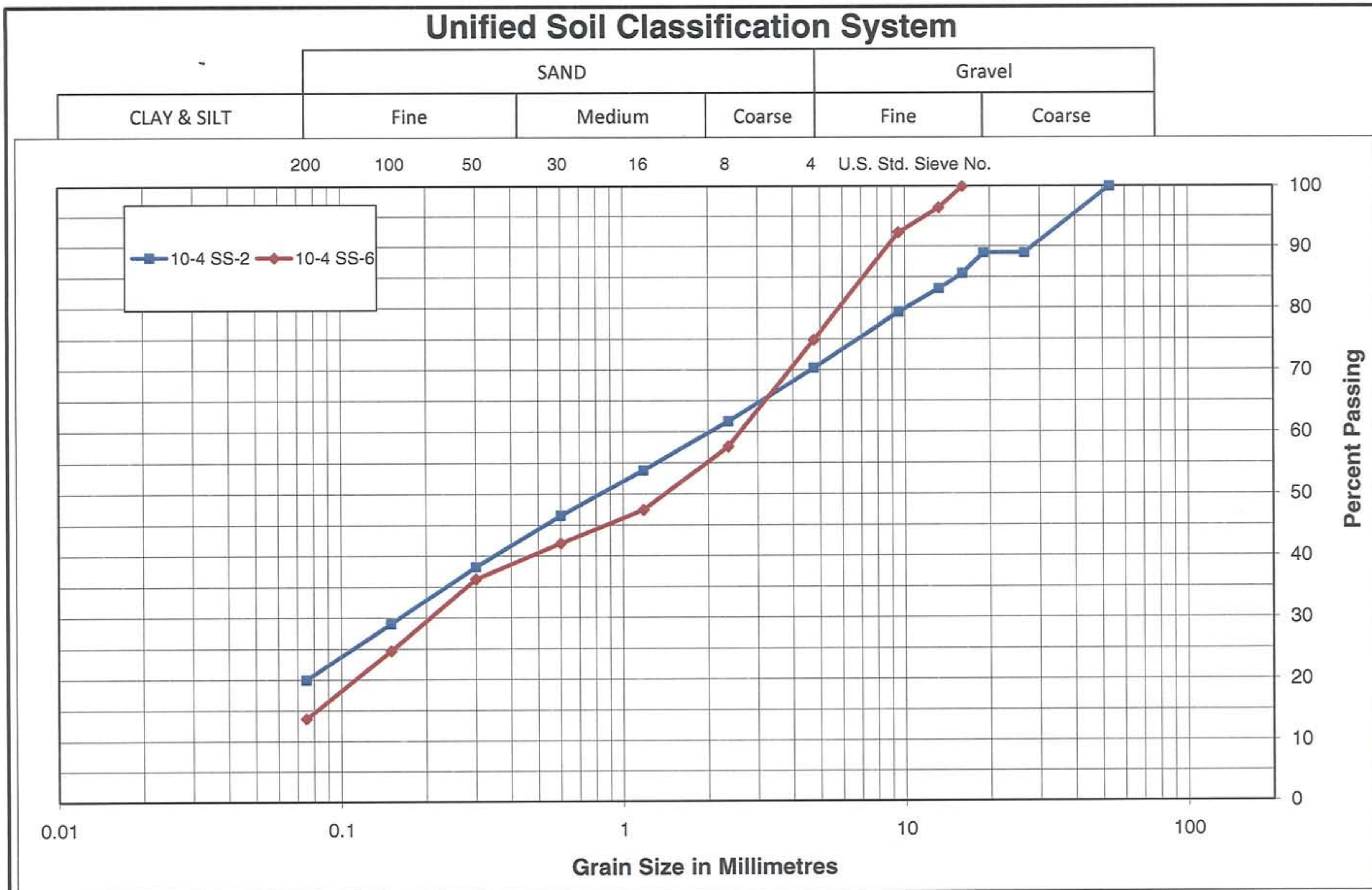
DATUM Geodetic DATE 2010 07 16 - 2010 07 19 CHECKED BY FG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
206.5	Ground Surface					
206.1 0.0	Sandy silt with organic matter, dark brown,	1	SS	2		
0.5	Varved silty CLAY, high plasticity clay (CH), soft to firm, grey	2	SS	2		
		3	SS	Wt of Hmr		
		4	ST	-		0 0 35 65
		5	SS	Wt of Hmr	∇	0 0 23 77
		6	SS	Wt of Hmr		
		7	SS	Wt of Hmr		
		8	ST	-		0 0 20 80
		9	SS	1		
190.7		10	ST	-		
15.9	End of Borehole Wt of Hmr = Weight of Hammer					

SOIL PROFILE · SAMPLES · STRAT PLOT · ELEVATION SCALE · DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 · 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



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

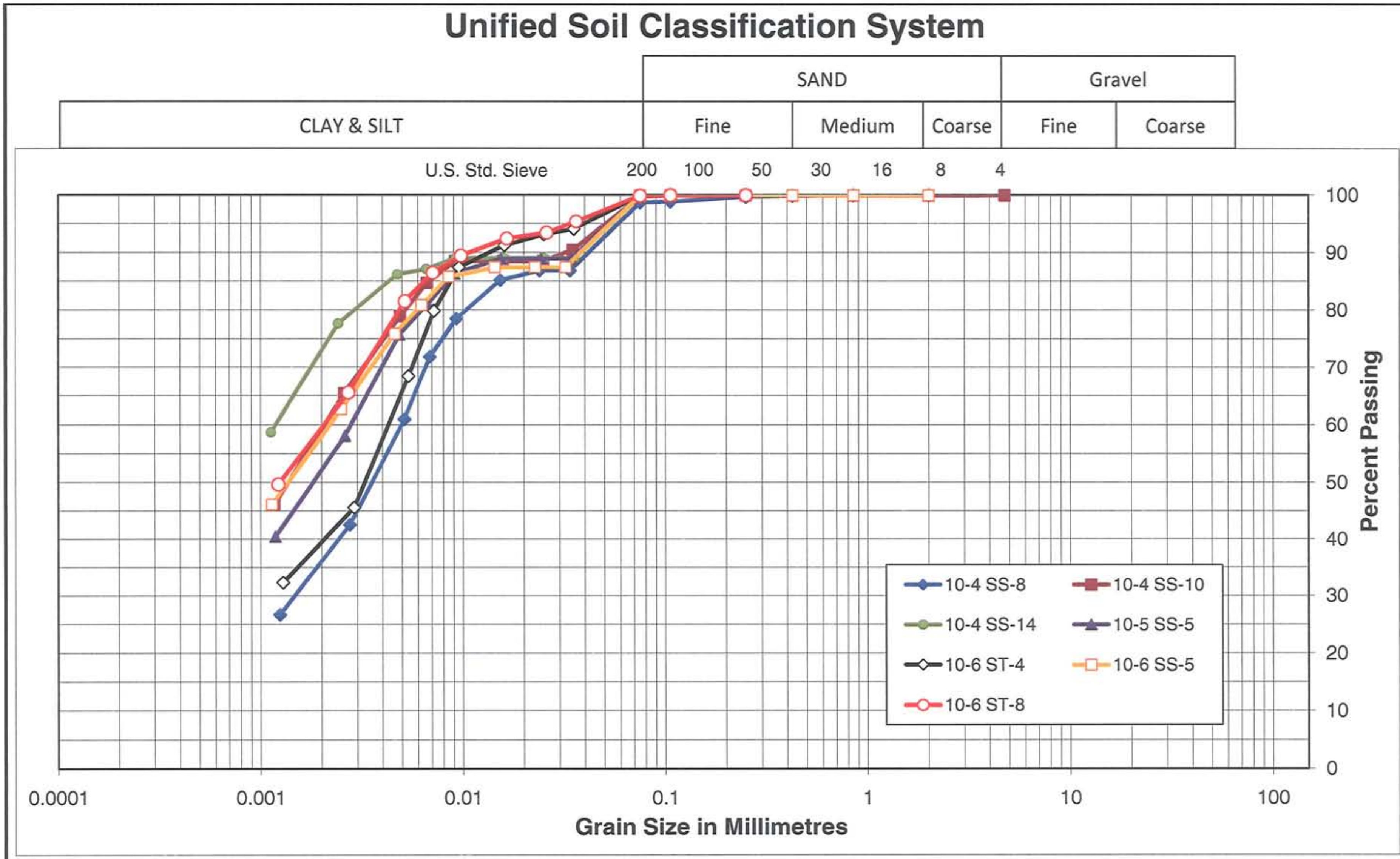


GRAIN SIZE DISTRIBUTION

FILL: Silty sand (SM) with gravel

Figure No. 6

Project No. 122410534



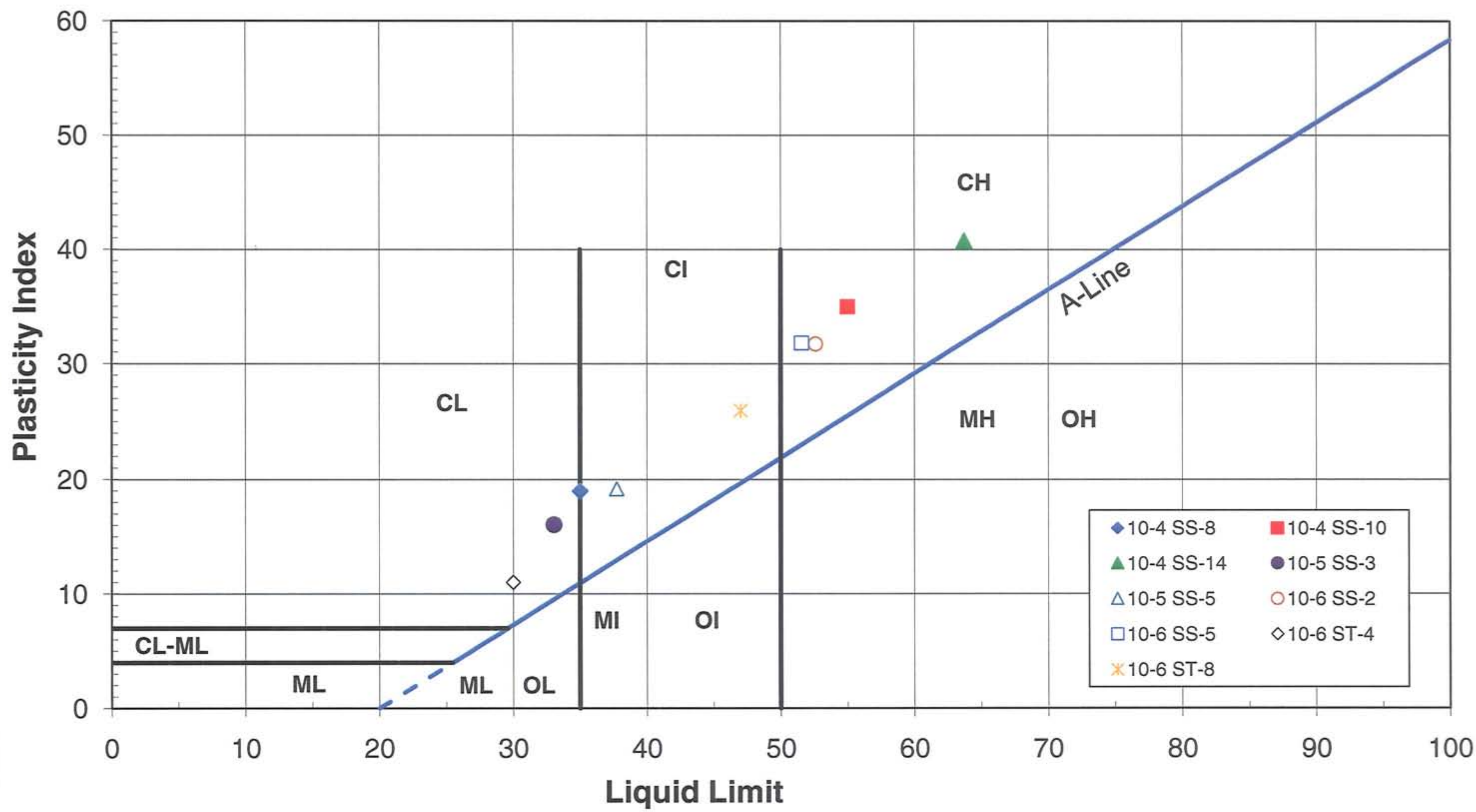
Stantec

GRAIN SIZE DISTRIBUTION

Silty Clay: low to high plasticty (CL to CH)

Figure No. 7

Project No. 122410534



Stantec	PLASTICITY CHART	Figure No. 8
		Project No. 122410534

APPENDIX D

Site No. 46-395/C - Station 13+506

Site Photographs

Borehole Location Plan and Stratigraphic Section

Borehole Records

Laboratory Test Results

Field Core Logs

Photos of Rock Cores

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

PLATE No

CONT GWP 5182–08–00

CULVERT AT STA 13+506

STA TO STA

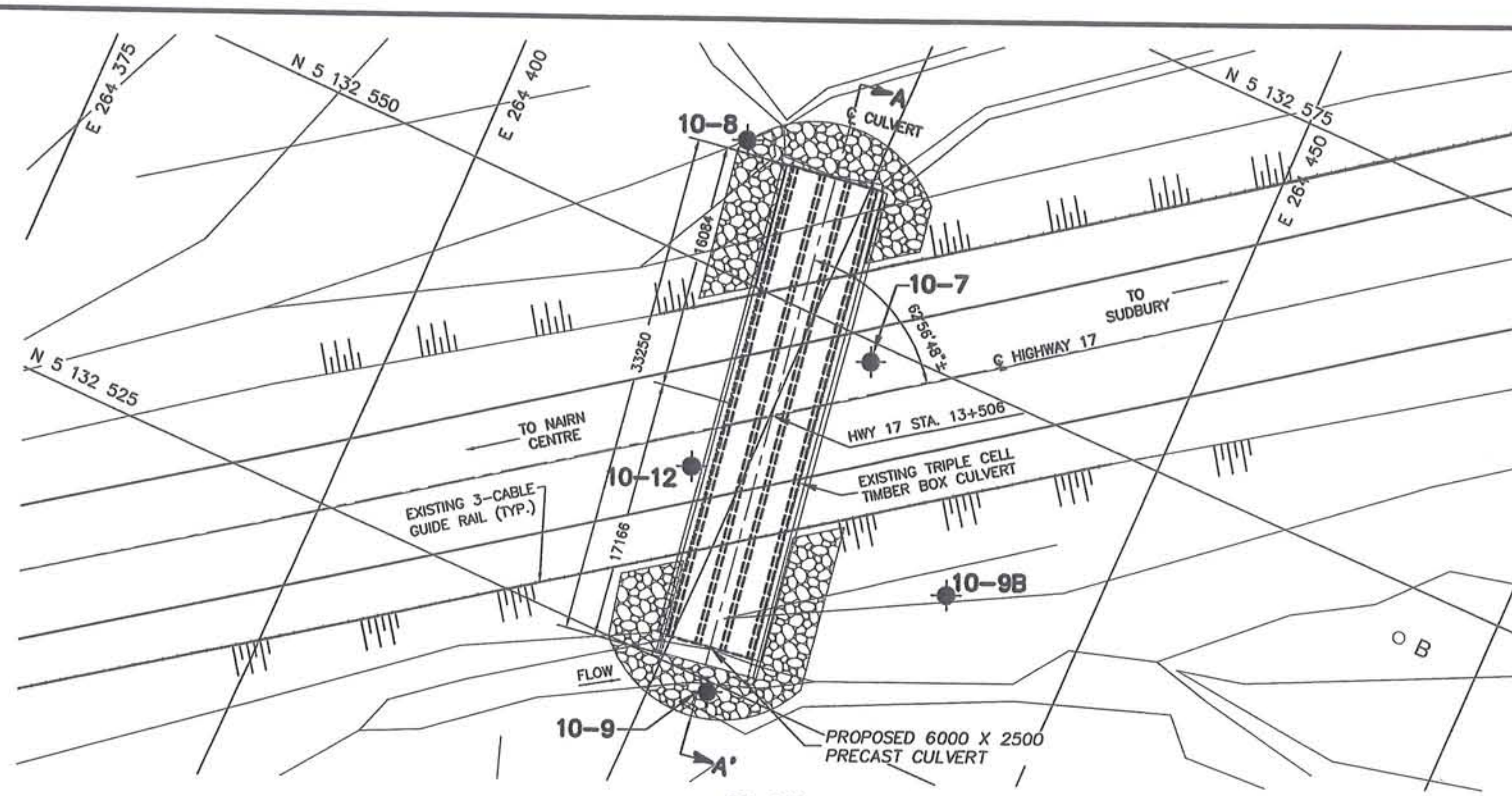
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Stantec

LICENSED PROFESSIONAL ENGINEER
F.J. GRIFFITHS
Feb 17/11
PROVINCE OF ONTARIO

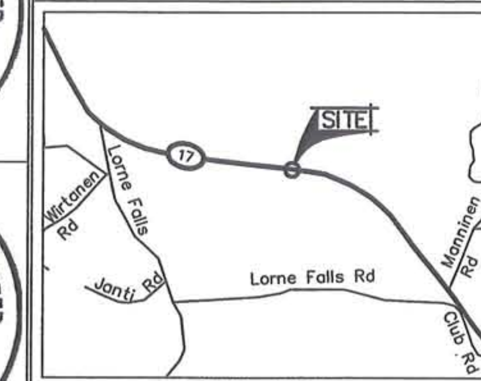
LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
11/02/17
PROVINCE OF ONTARIO



PLAN

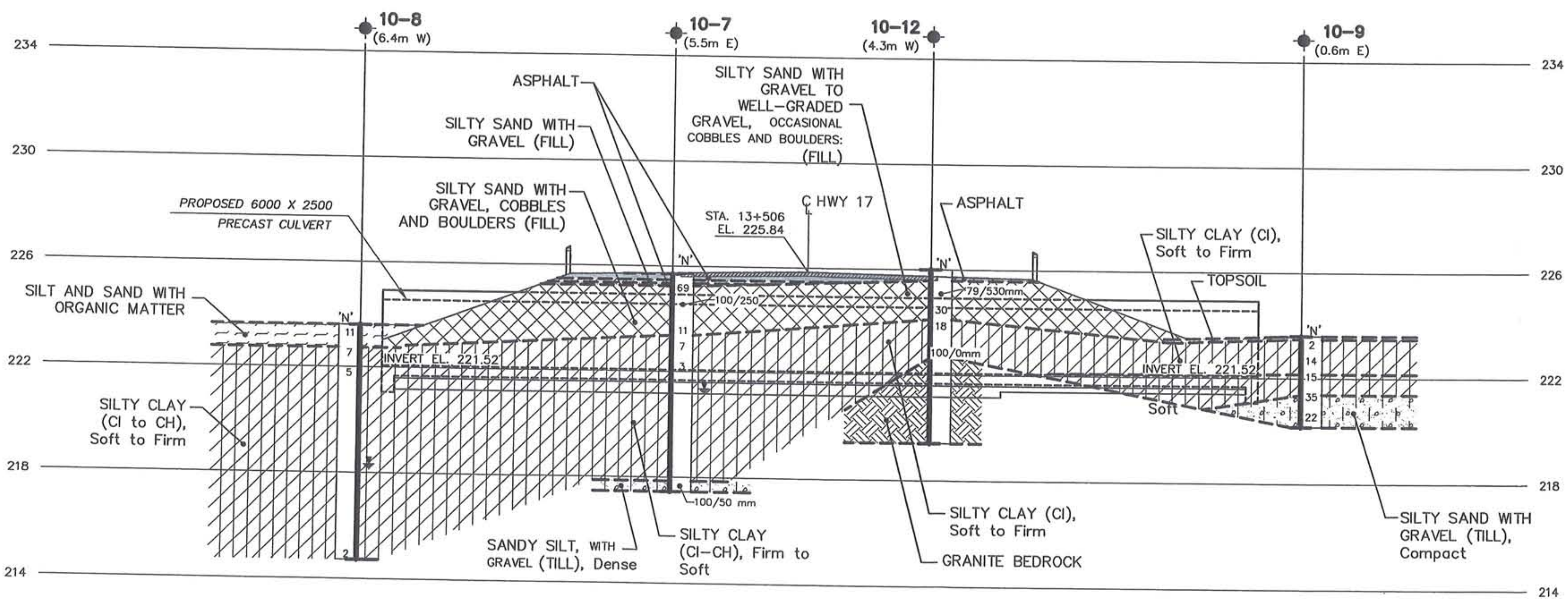
SCALE

5 m 0 5 10 m



KEY PLAN

1 km 0 1 2 km



CROSS–SECTION A–A'

SCALE

2 m 0 2 4 m

LEGEND

- Bore Hole
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at time of investigation July 2010

No	ELEVATION	MTM ZONE 12 COORDINATES NORTH	EAST
10–7	225.7	5 132 548.7	264 429.7
10–8	223.6	5 132 558.5	264 416.1
10–9	223.6	5 132 524.3	264 428.6
10–9B	223.3	5 132 536.8	264 440.6
10–12	226.0	5 132 537.5	264 421.6

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

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

REVISIONS DATE	BY	DESCRIPTION

GEOCRES No 41I–263

HWY No 17			DIST
SUBM'D KP	CHECKED	DATE 2010–08–24	SITE 46–395/C
DRAWN GBB	CHECKED	APPROVED	DWG 4

DRAWING NAME: 122410534_1–5 (FEB).DWG
CREATED: GBB MODIFIED: 11/02/16
T:\Autocad\Drawings\Project Drawings\2011\122410534\Final\122410534_1–5 (Feb).dwg (SITE 4 (13+506)) Printed: Feb 16, 2011

MINISTRY OF TRANSPORTATION, ONTARIO PH–D–707 86–05

Photo No. D1: Site No. 46-395/c. North side of culvert at 13+506.

Photo No. D2: Site No. 46-395/c. South side of culvert at 13+506.

Stantec

RECORD OF BOREHOLE No 10-7

METRIC

W.P. 5182-08-00 | LOCATION 13+513 2.10 m Lt CL | N: 5 132 549 E: 264 430 | ORIGINATED BY AS

DIST ____ HWY 17 | BOREHOLE TYPE Splitspoons, Hollow Stem Augers | COMPILED BY AS

DATUM Geodetic | DATE 2010 07 06 - 2010 07 06 | CHECKED BY FG

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L	UNIT WEIGHT γ kN/m³	GR SA SI CL
225.7	Ground Surface									
225.5 / 0.0	220 mm ASPHALT									
225.3 / 0.2	Silty sand with gravel, brown, FILL	1	SS	69						
225.2 / 0.4 / 0.5	100 mm ASPHALT									
	Silty sand with gravel, brown to grey, FILL	2	SS	100/ 250 mm		224				
224.0 / 1.7	- frequent cobbles - boulder at 1.4 m									
223.3	Silty gravel with sand, trace rootlets, dark brown to grey, FILL	3	SS	11				o		53 28 (19)
2.3	Silty CLAY, intermediate to high plasticity (CI to CH), firm to soft, brownish grey to grey	4	SS	7				●—o—● 43		0 14 49 37
		5	SS	3		222				
					∇		8.1, 8.9			
		6	SS	Wt of Hmr				●—● 59 53		0 0 17 83
						220	4.0, 4.0			
		7	SS	Wt of Hmr						
						218	7.1, 3.1			
217.8 / 7.9	Sandy silt (ML) with gravel, dense, grey, TILL	8	SS	100/ 80mm				o		22 24 (54)
217.5 / 8.2	End of Borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

WATER CONTENT (%): 10 20 30

REMARKS & GRAIN SIZE DISTRIBUTION (%)

End of Borehole

Auger Refusal at 8.2 m

Wt of Hmr = Weight of Hammer

Groundwater was observed in open borehole at depth of 4.3 m

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

Stantec

RECORD OF BOREHOLE No 10-8

METRIC

W.P. 5182-08-00 LOCATION 13+508 18.0 m Lt CL N: 5 132 559 E: 264 416 ORIGINATED BY AS

DIST ___ HWY 17 BOREHOLE TYPE Splitspoons, Hollow Stem Augers COMPILED BY AS

DATUM Geodetic DATE 2010 07 16 - 2010 07 16 CHECKED BY FG

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH (FIELD VANE, sensitivity)	WATER CONTENT	GR	SA	SI	CL
223.6	Ground Surface											
0.0	Silt and sand with organic matter, dark brown	1	SS	11				57, 95				
222.8 / 0.9	Silty CLAY, intermediate to high plasticity (CI to CH), firm to soft, brownish grey to grey	2	SS	7								
		3	SS	5		222		43, 46				
		4	ST	-			5.5, 6.0					
		5	SS	Wt of Hmr		220		61, 46	0	2	24	74
					∇		3.0, 4.0					
		6	SS	Wt of Hmr		218	4.0, 5.5					
		7	ST	-		216	3.3, 4.3					
214.7 / 8.9	- some sand and gravel	8	SS	2					0	12	(88)	

End of Borehole

Auger Refusal at 8.9 m

Wt of Hmr = Weight of Hammer

Groundwater was observed in open borehole at depth of 5.3 m

$\times^3$, $\times^3$: Numbers refer to Sensitivity   O$^{3\%}$ STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No 10-9

METRIC

W.P. 5182-08-00 LOCATION 13+498 17.0 m Rt CL N: 5 132 524 E: 264 429 ORIGINATED BY AS

DIST ____ HWY 17 BOREHOLE TYPE Splitspoons, Hollow Stem Augers COMPILED BY AS

DATUM Geodetic DATE 2010 07 15 - 2010 07 15 CHECKED BY FG

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
223.6	Ground Surface											
223.5 / 0.1	120 mm TOPSOIL											
	Silty CLAY, intermediate plastcity (CI), stiff to firm, brownish grey to grey	1	SS	2								
		2	SS	14			wP–w–wL (wL 46)		0	1	26	73
		3	SS	15		222						
221.3 / 2.2	Silty sand (SM) with gravel, compact, grey, TILL	4	SS	35								
		5	SS	22			w ≈ 10		20	46	(34)	
220.1 / 3.5	End of Borehole Auger Refusal at 3.5 m											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Stantec

RECORD OF BOREHOLE No 10-9B

METRIC

W.P. 5182-08-00	LOCATION 13+515 14.0 m Rt CL	N: 5 132 537 E: 264 441	ORIGINATED BY AS
DIST ___ HWY 17	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY AS
DATUM Geodetic	DATE 2010 07 15 - 2010 07 15		CHECKED BY FG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, ✕ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
223.3	Ground Surface										
0.0	Probe Hole Auger to Refusal						222 220				
219.3 4.0	End of Borehole Auger Refusal at 4.0 m										

✕ 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No 10-12

METRIC

W.P. 5182-08-00 LOCATION 13+500 2.2 m Rt CL N: 5 132 538 E: 264 422 ORIGINATED BY AS

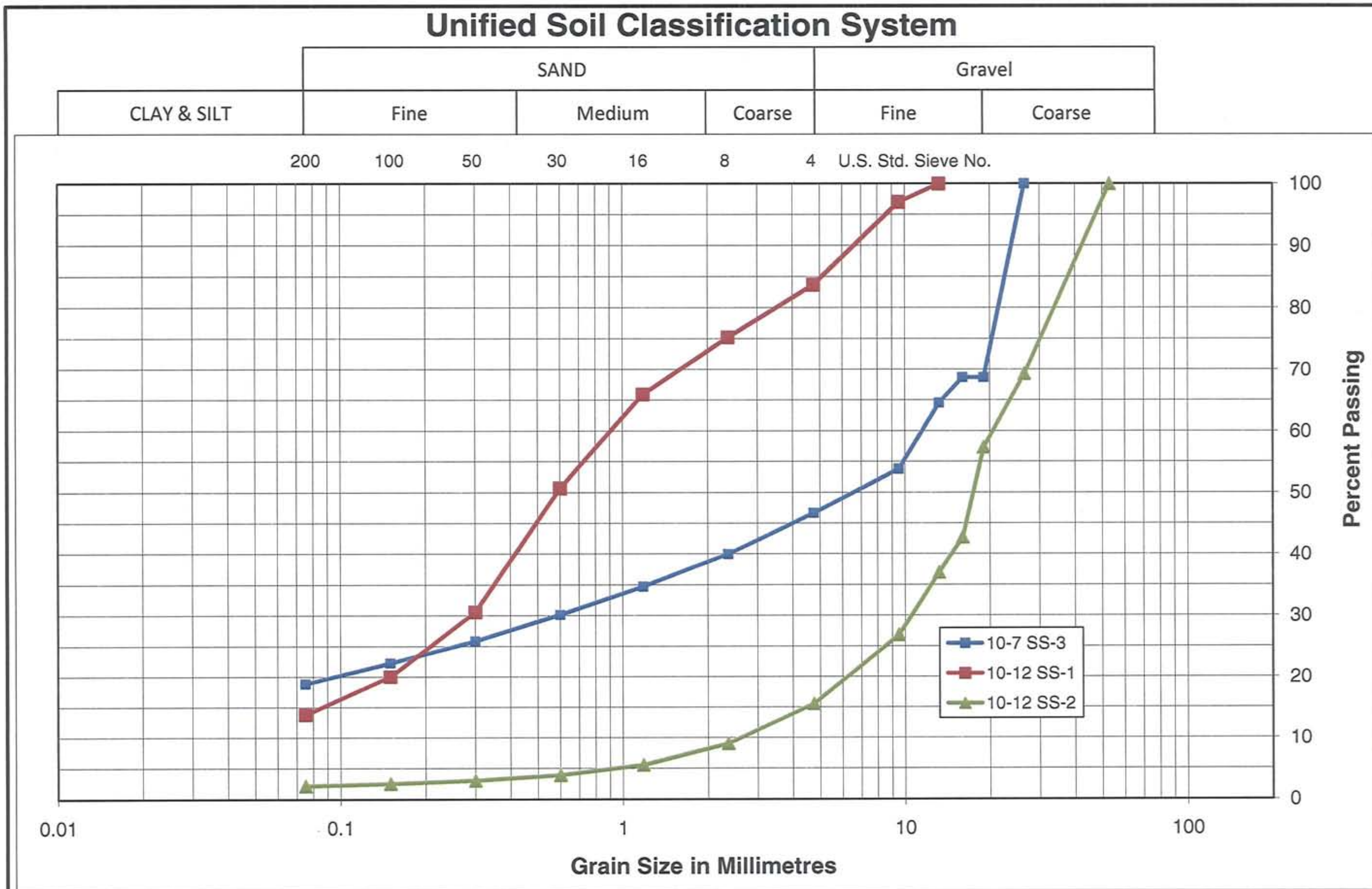
DIST ___ HWY 17 BOREHOLE TYPE Splitspoons, Hollow Stem Augers COMPILED BY AS

DATUM Geodetic DATE 2010 08 05 - 2010 08 05 CHECKED BY FG

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
226.0	Ground Surface						
225.8 / 0.0	420mm ASPHALT						
0.4	Silty sand (SM) with gravel, brown, FILL	1	SS	79/ 530mm			16 70 (14)
225.0 / 1.0	Well-graded gravel (GW), grey, FILL - occasional cobbles and boulders	2	SS	30			84 13 (2)
224.1 / 1.9	Silty CLAY, intermediate plasticity (CI), stiff to firm, brown to grey	3	SS	18	224		
		4	SS	5		52	0 2 31 67
222.7 / 3.4	Grey GRANITE bedrock - very poor to excellent quality - fresh weathering - flat orientation (0 - 20°) - vertical cracking from 3.3 m to 3.9 m - close to wide joint spacing - rough planar	5	SS	100/ 0mm			
		6	NQ	7%	222		
		7	NQ	66%	220		
219.4 / 6.6	End of Borehole	8	NQ	100%			

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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



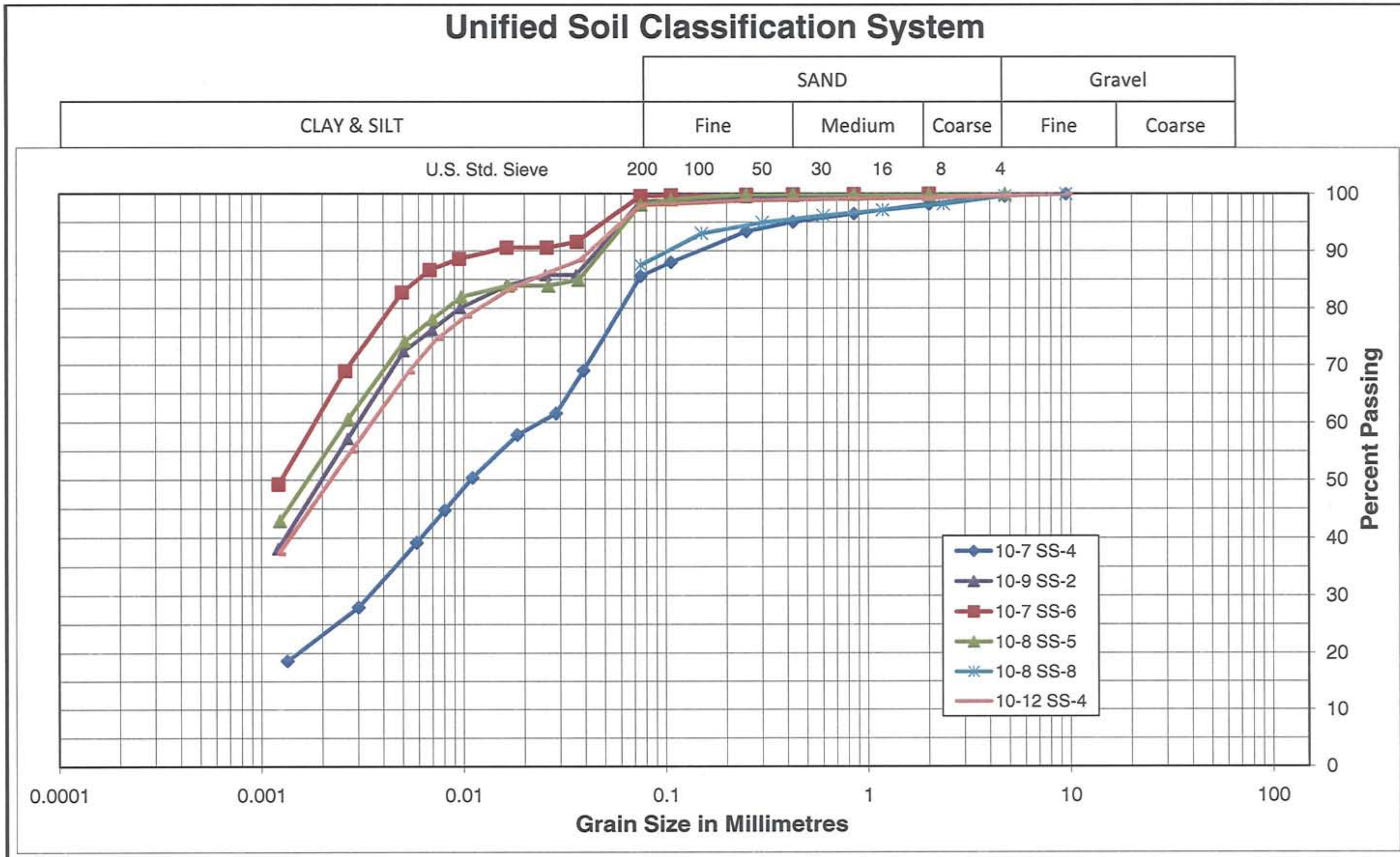
Stantec

GRAIN SIZE DISTRIBUTION

FILL: Silty sand (SM), silty gravel (GM) with sand and well-graded gravel (GW)

Figure No. 9

Project No. 122410534



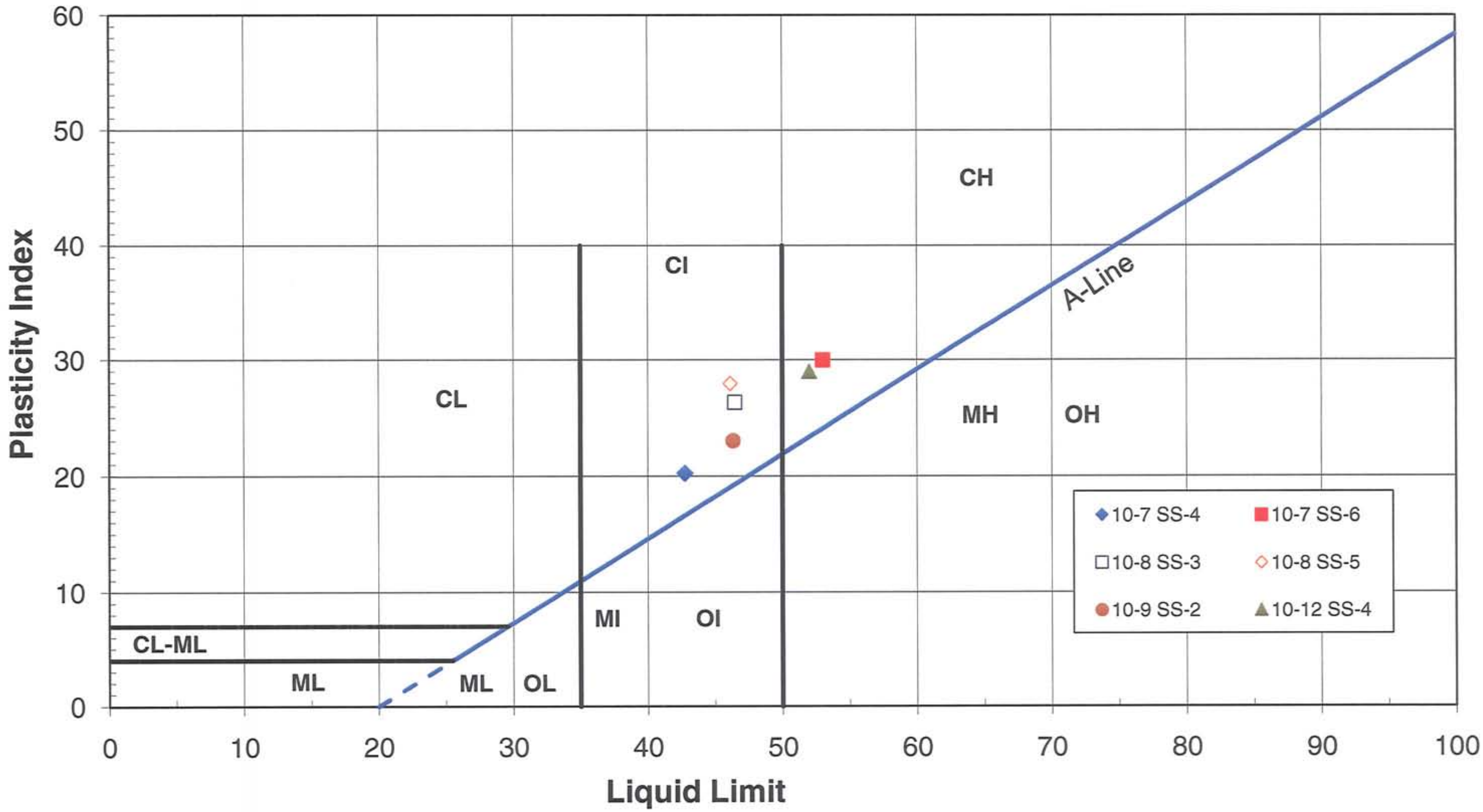
GRAIN SIZE DISTRIBUTION

Silty Clay: Intermediate to high plasticity (CI to CH)

Figure No. 10

Project No. 122410534

Stantec

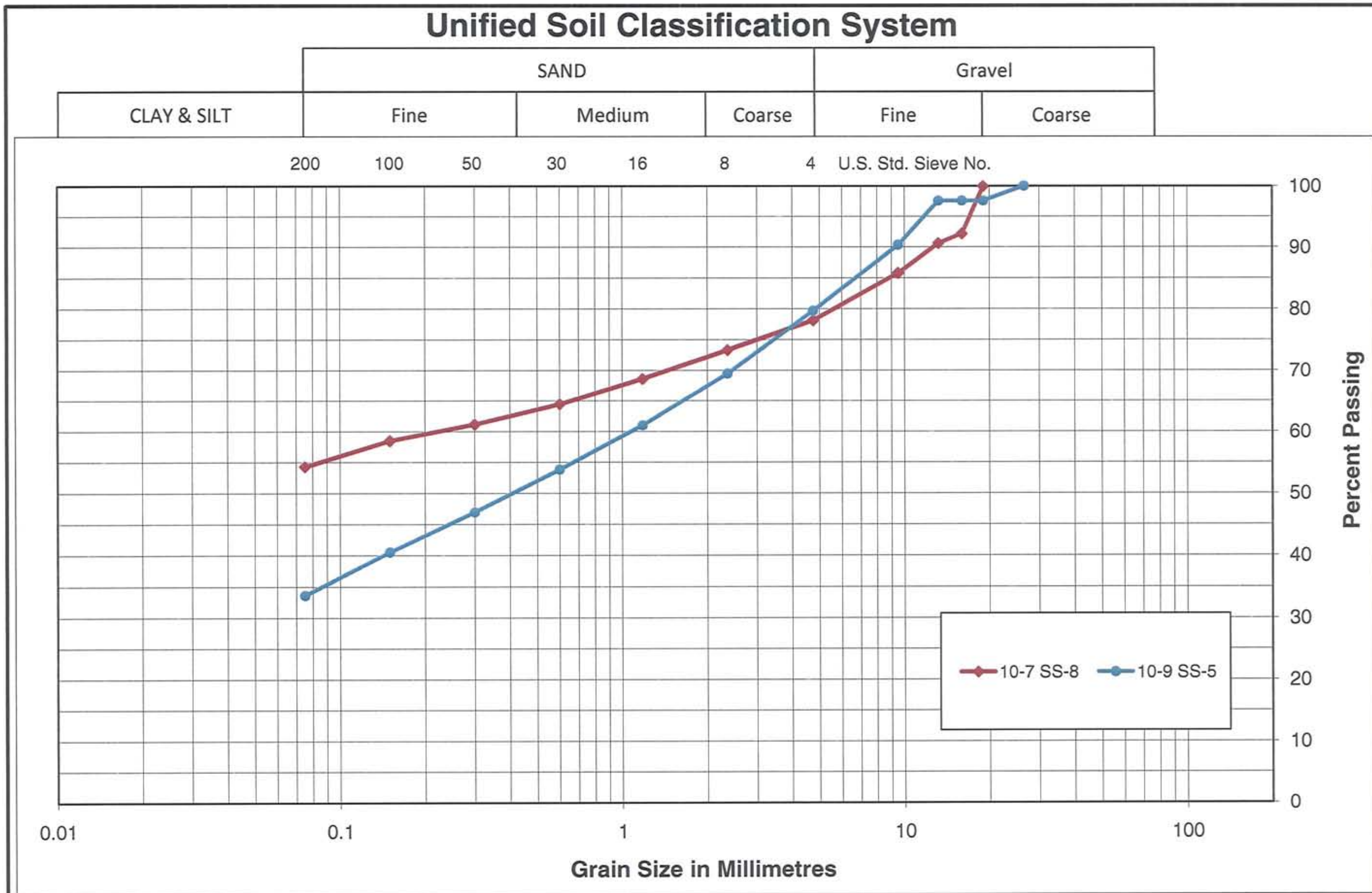


Stantec

PLASTICITY CHART

Figure No. 11

Project No. 122410534



GRAIN SIZE DISTRIBUTION

TILL: silty sand (SM) with gravel to silt (ML) with sand

Figure No. 12

Project No. 122410534

Stantec

Photo No. 12: Bedrock core Hwy 17 BH10-12

Field Core Log

Stantec

Client: McIntosh Perry Consulting Engineers
Project: Highway 17 - Blake Creek Culverts W.P. 5182-08-00
Contractor: Abraflex Drilling CME55 Track Mount

Project No.: 122410534
Date: August 18, 2010
Borehole No.: 10-12
Logger: Kenton C. Power

DEPTH FROM	RUN NO.	% CORE RECOVERY	% RQD	DEPTH TO	GENERAL DESCRIPTION (Rock Type/s, %, Colour, Texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES: NO. OF SETS	TYPE/S	ORIENTATION	SPACING	ROUGHNESS	APERTURE	FILLING	OCCASIONAL FEATURES	DRILLING OBSERVATIONS
3.4	6	86	7	4.9	Granitic Igneous Bedrock, grey to dark grey, medium texture	VS	U	1	S	V		RP		O		
4.9	7	85	66	6.2	Granitic Igneous Bedrock, grey to dark grey, medium texture	VS	U	1	S	D	C/M	RP		O		
6.2	8	100	100	6.6	Granitic Igneous Bedrock, grey to dark grey, medium texture	VS	U	1	S	D	M/W	RP		O		

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

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

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

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

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

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

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

APPENDIX E

Site No. 46-396/C - Station 13+631
Site Photographs
Borehole Location Plan and Stratigraphic Section
Borehole Records
Laboratory Test Results

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

PLATE No

CONT GWP 5182-08-00

CULVERT AT STA 13+631

STA TO STA

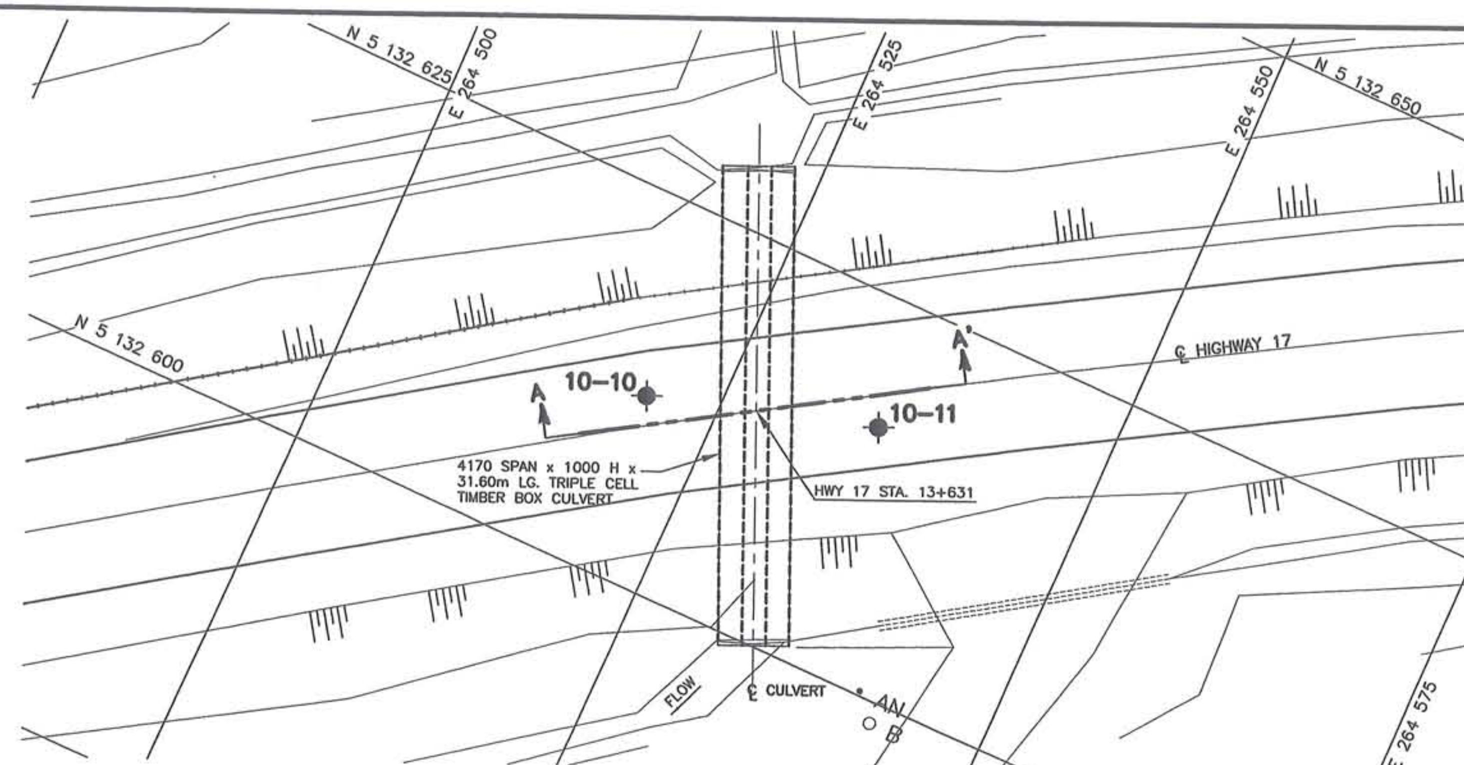
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Stantec

LICENSED PROFESSIONAL ENGINEER
F.J. GRIFFITHS
PROVINCE OF ONTARIO

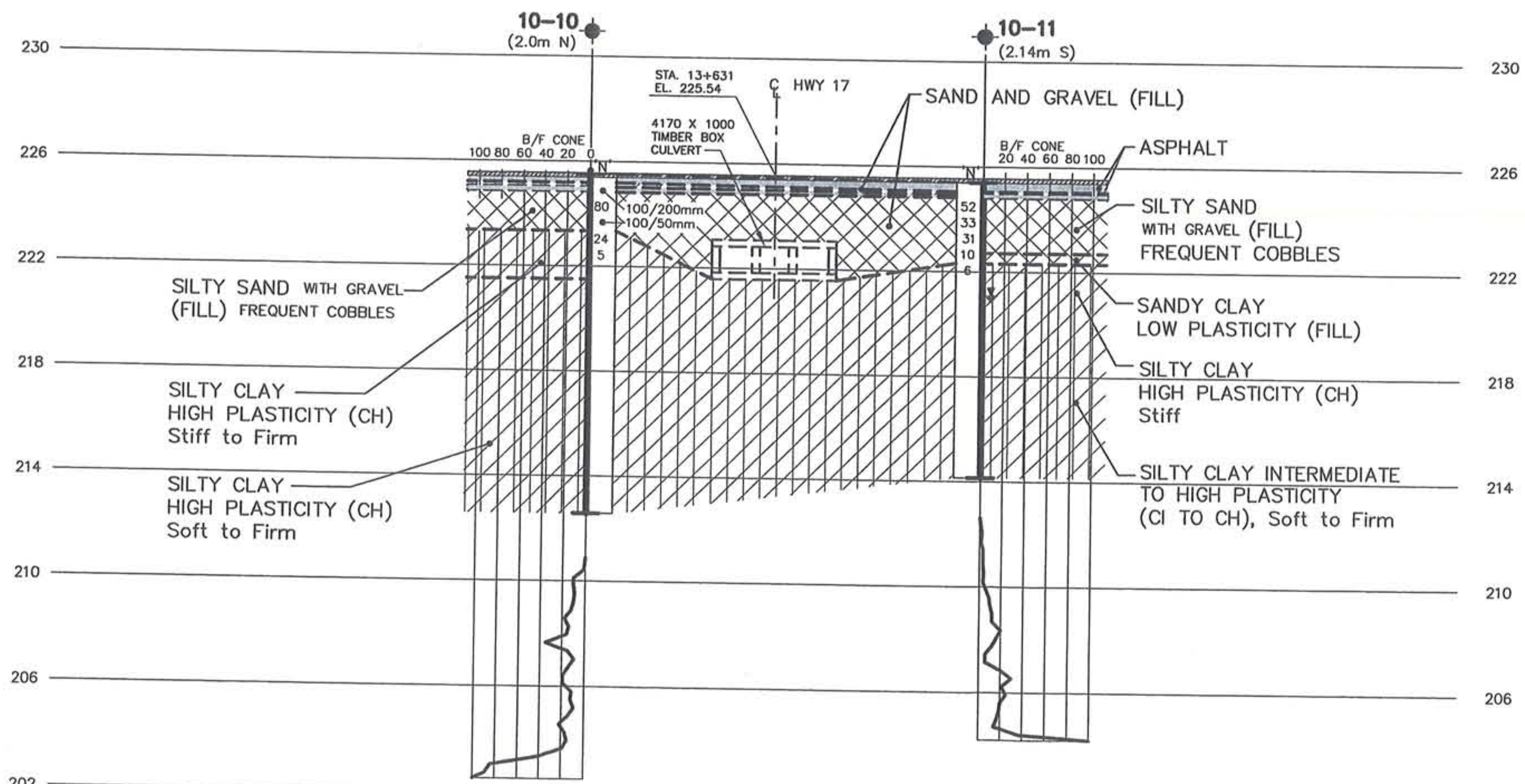
LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
PROVINCE OF ONTARIO



PLAN

SCALE

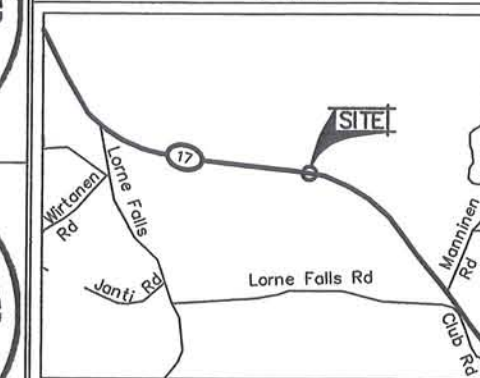
5 m 0 5 10 m



CROSS-SECTION A-A'

SCALE

2.5 m 0 2.5 5 m



KEY PLAN

0.5 km 0 0.5 1 km

LEGEND

- Bore Hole
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at time of investigation August 2010

No	ELEVATION	MTM ZONE 12 COORDINATES NORTH	EAST
10-10	225.7	5 132 612.1	264 520.3
10-11	225.4	5 132 616.7	264 535.2

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

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

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No 41I-263

HWY No 17			DIST
SUBM'D KP	CHECKED	DATE 2010-08-23	SITE 46-396/C
DRAWN GBB	CHECKED	APPROVED	DWG 5

DRAWING NAME: 122410534_1-5 (FEB).DWG
CREATED: GBB 11/02/08 MODIFIED:
T:\Autocad\Drawings\Project Drawings\2011\122410534\Final\122410534_1-5 (Feb).dwg (SITE 5 (13+631)) Printed: Feb 08, 2011

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

Photo No. E1: Site No. 46-396/c. North side of culvert at 13+631.

Photo No. E2: Site No. 46-396/c. South side of culvert at 13+631 looking west.

Photo No. E3: Site No. 46-395/c. Split spoon sampler that was bent trying to penetrate fill in BH 10-11 that contains cobbles and boulders.

Stantec

RECORD OF BOREHOLE No 10-10

 METRIC

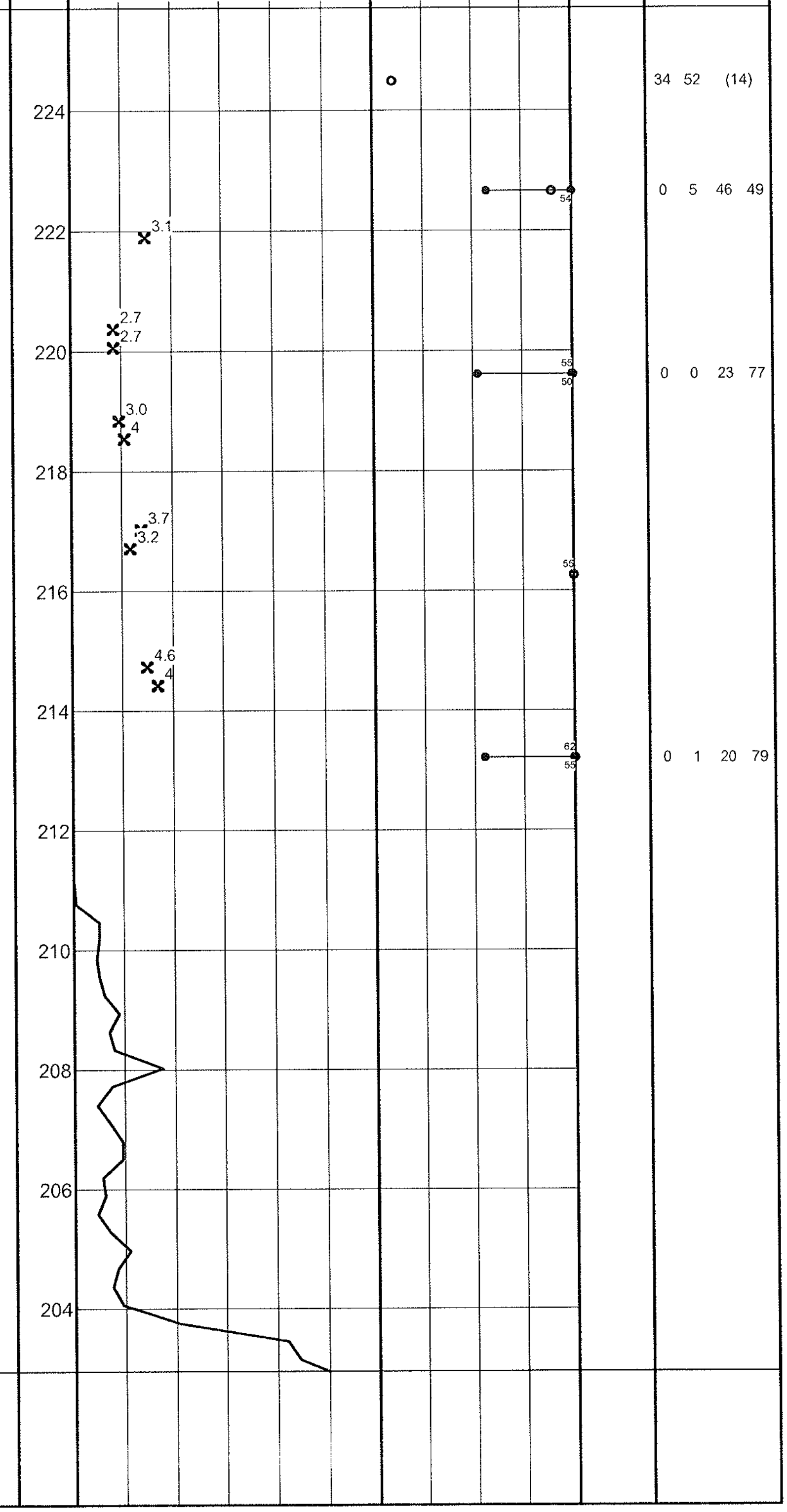
W.P. 5182-08-00 | LOCATION 13+624 2.0 m Lt CL | N: 5 132 612 E: 264 520 | ORIGINATED BY AS

DIST ____ HWY 17 | BOREHOLE TYPE Splitspoons, Hollow Stem Augers, NQ Casing, Dynamic Cone Test | COMPILED BY AS

DATUM Geodetic | DATE 2010 07 07 - 2010 07 07 | CHECKED BY FG

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
225.7	Ground Surface							
[illegible]	320 mm ASPHALT							
[illegible]	Sand and gravel, brown, FILL	1	SS	100/ 200mm				
[illegible]	150 mm ASPHALT							
[illegible]	Sand and gravel, brown, FILL	2	SS	80	34	52	(14)	
1.0	Silty sand (SM) with gravel, frequent cobbles, brown, FILL	3	SS	100/ 50mm				
223.6 / 2.1	Silty CLAY, high plasticity (CH), stiff to firm, dark brown to grey	4	SS	24				
		5	SS	5	0	5	46	49
221.7 / 4.0	Silty CLAY, high plasticity (CH), soft to firm, grey	6	SS	Wt of Hmr				
		7	SS	Wt of Hmr	0	0	23	77
		8	SS	Wt of Hmr				
		9	SS	Wt of Hmr				
	- occasional silt seams	10	SS	Wt of Hmr	0	1	20	79
212.9 / 12.8	End of Borehole Start of Dynamic Cone Penetration Test Cone sank under weight of hammer to 14.6 m							
202.9 / 22.8	Dynamic Cone Refusal at 22.8 m Wt of Hmr = Weight of Hammer							

GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | 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



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

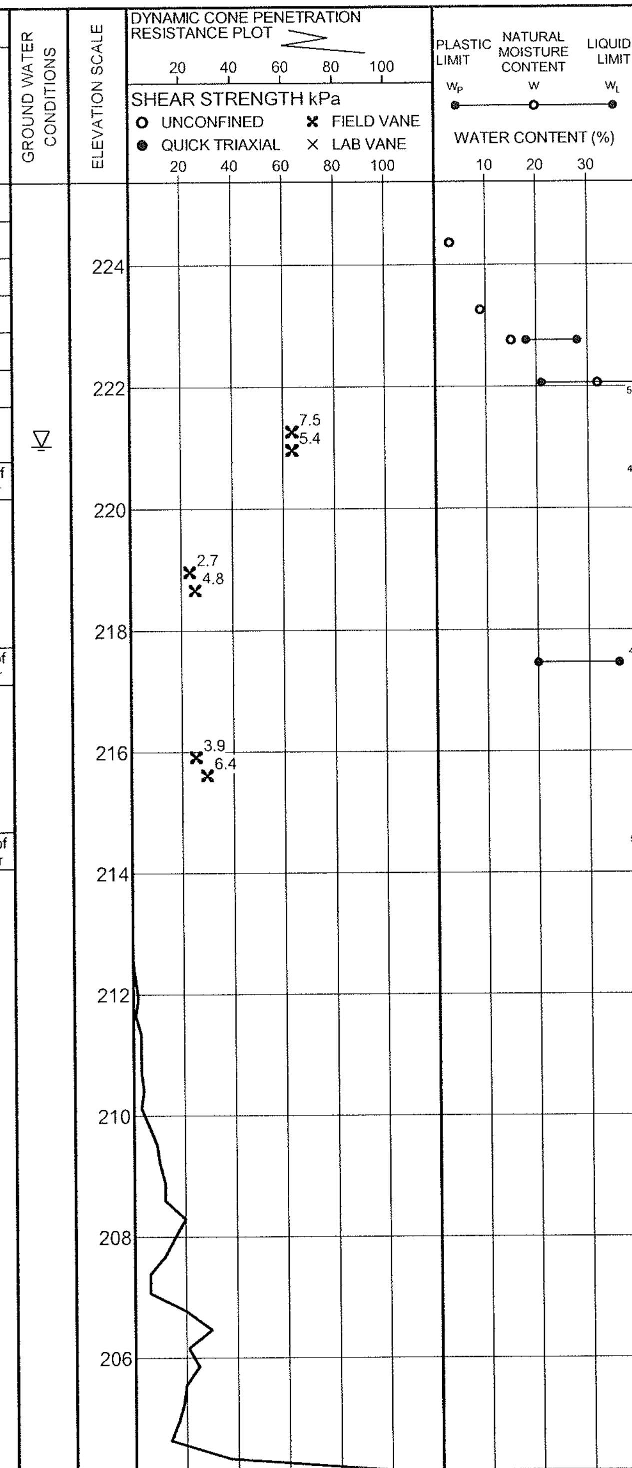
Stantec

RECORD OF BOREHOLE No 10-11

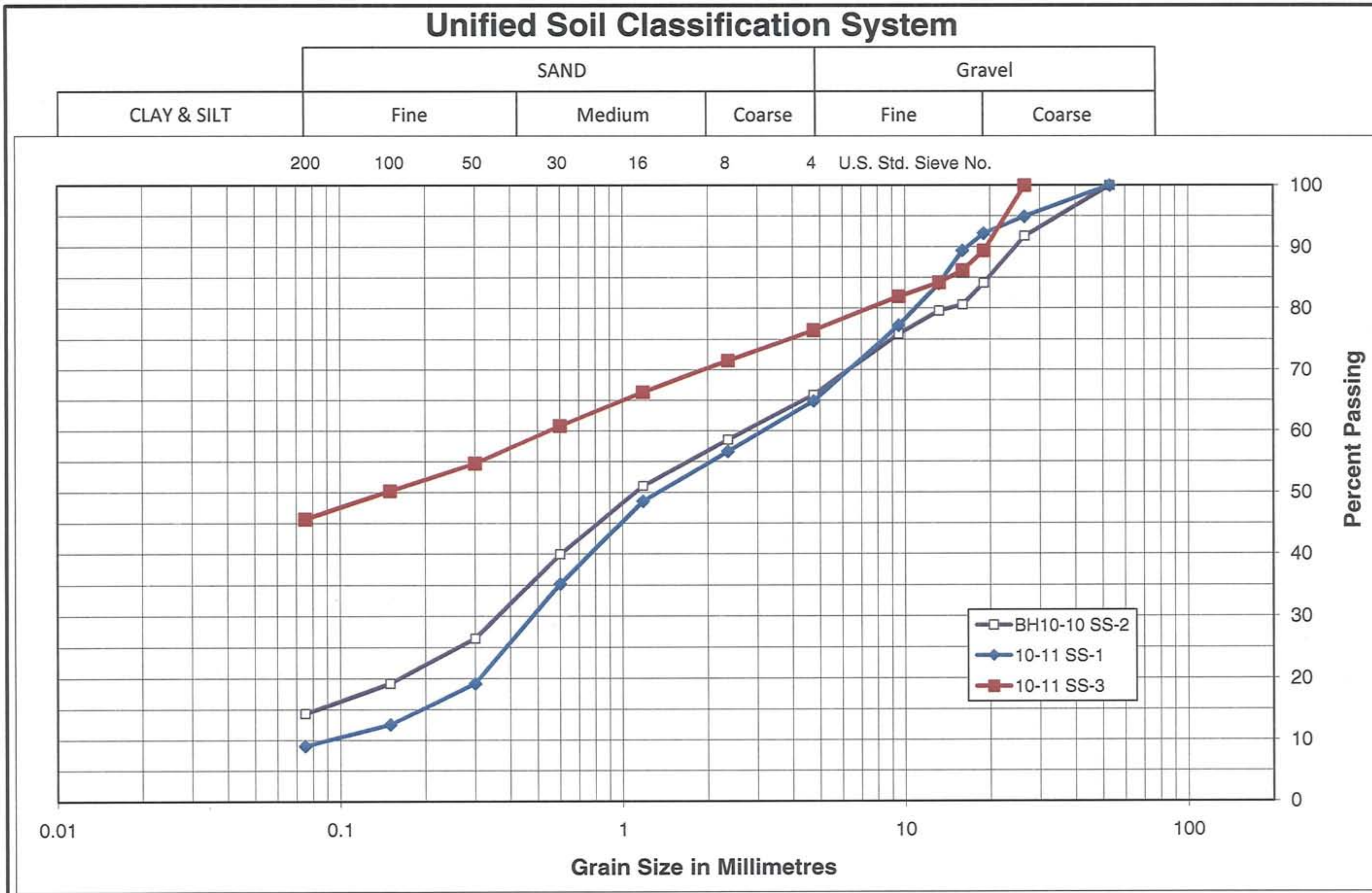
 METRIC

W.P. 5182-08-00	LOCATION 13+639 2.1 m Rt CL	N: 5 132 617 E: 264 535	ORIGINATED BY AS
DIST	HWY 17	BOREHOLE TYPE Splitspoons, Hollow Stem Augers	COMPILED BY AS
DATUM Geodetic	DATE 2010 08 04 - 2010 08 04		CHECKED BY FG

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
225.4	Ground Surface					
[illegible]	350 mm - ASPHALT					
[illegible]	Sand and gravel, brown, FILL					
[illegible]	110 mm ASPHALT					
[illegible]	Sand and gravel, brown, FILL					
0.8	Silty sand (SM) with gravel, brown, FILL - frequent cobbles	1	SS	52	224	35 56 (9)
		2	SS	33		
		3	SS	31		23 31 (46)
222.7 / [illegible]	Sandy clay, low plasticity, dark grey, FILL	4	SS	10		9 22 41 28
[illegible] / 3.1	Silty CLAY, high plasticity (CH), stiff, brownish grey	5	SS	6	222	0 1 28 71
220.8 / 4.6	Silty CLAY, intermediate to high plasticity (CI to CH), soft to firm, grey	6	SS	Wt of Hmr	220	
		7	SS	Wt of Hmr	218	0 3 27 70
		8	SS	Wt of Hmr	216	
214.1 / 11.3	End of Borehole Start of Dynamic Cone Penetration Test at 11.3 m Cone sank under weight of hammer to 12.8 m				214	
204.1 / 21.2	Dynamic Cone Penetration Refusal at 21.2 m Wt of Hmr = Weight of Hammer Groundwater was observed in open borehole at depth of 4.3 m					



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



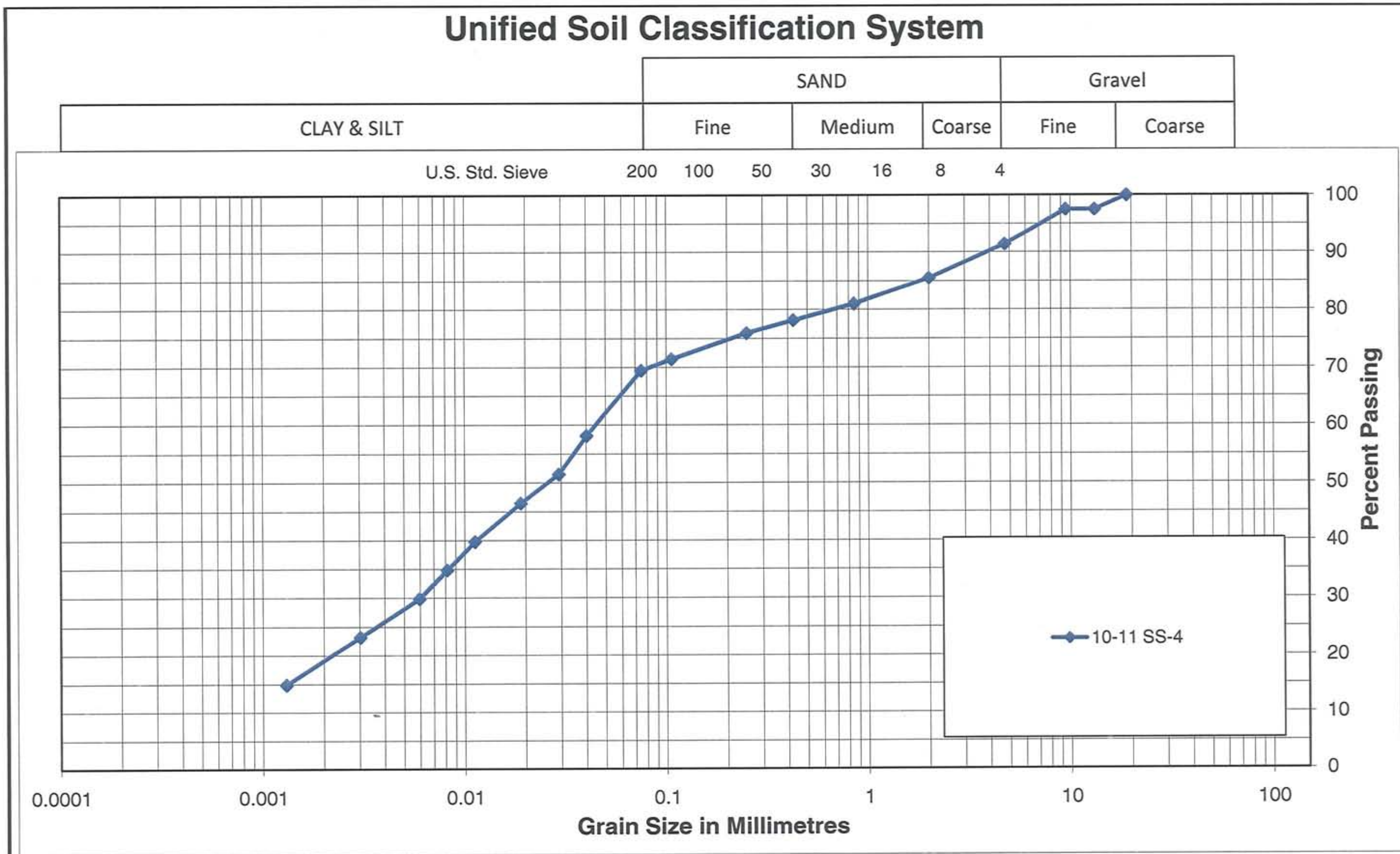
GRAIN SIZE DISTRIBUTION

FILL: silty sand (SM) with gravel

Figure No. 13

Project No. 122410534

Stantec

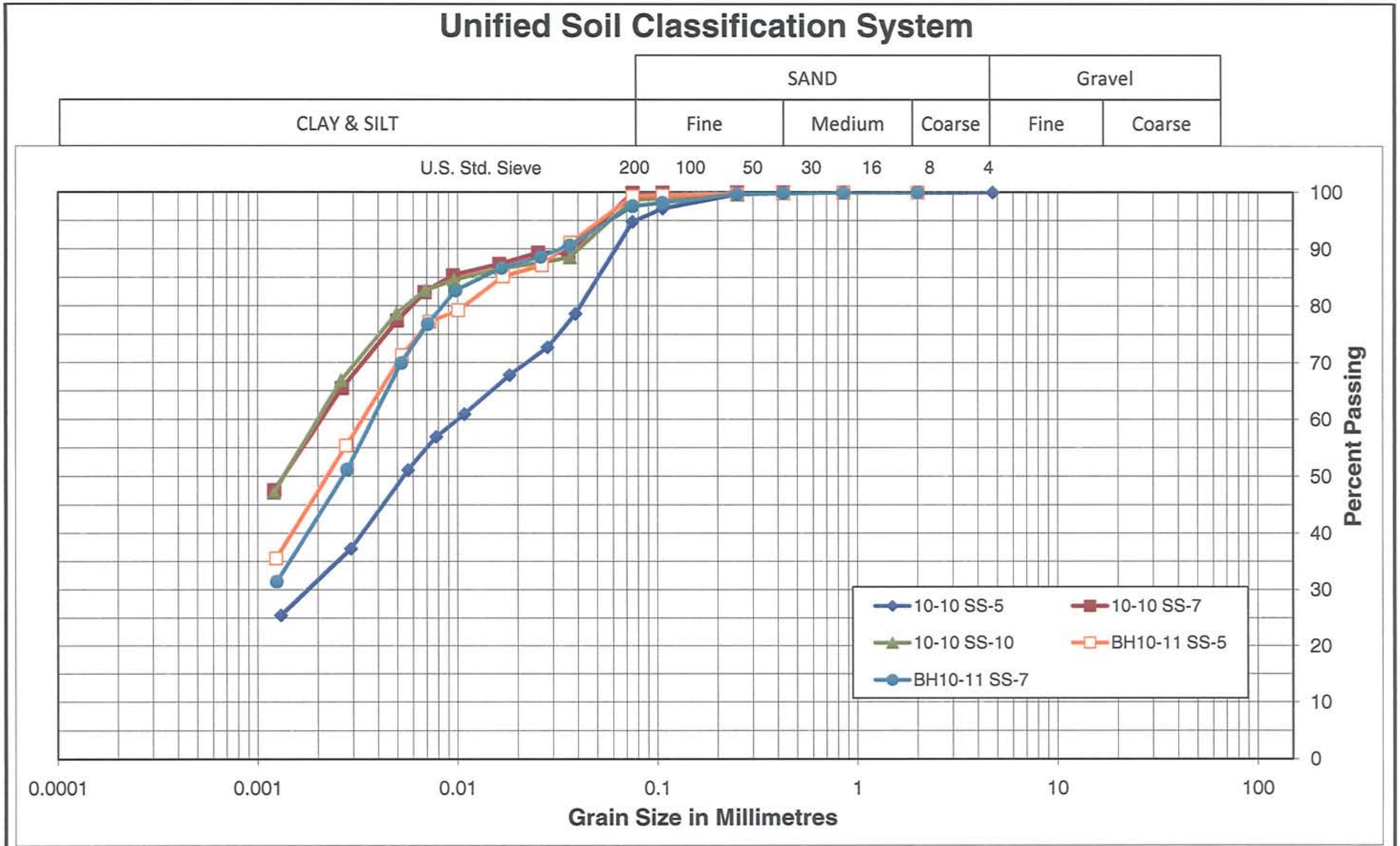


GRAIN SIZE DISTRIBUTION

FILL: Sandy Clay (CL)

Figure No. 14

Project No. 122410534



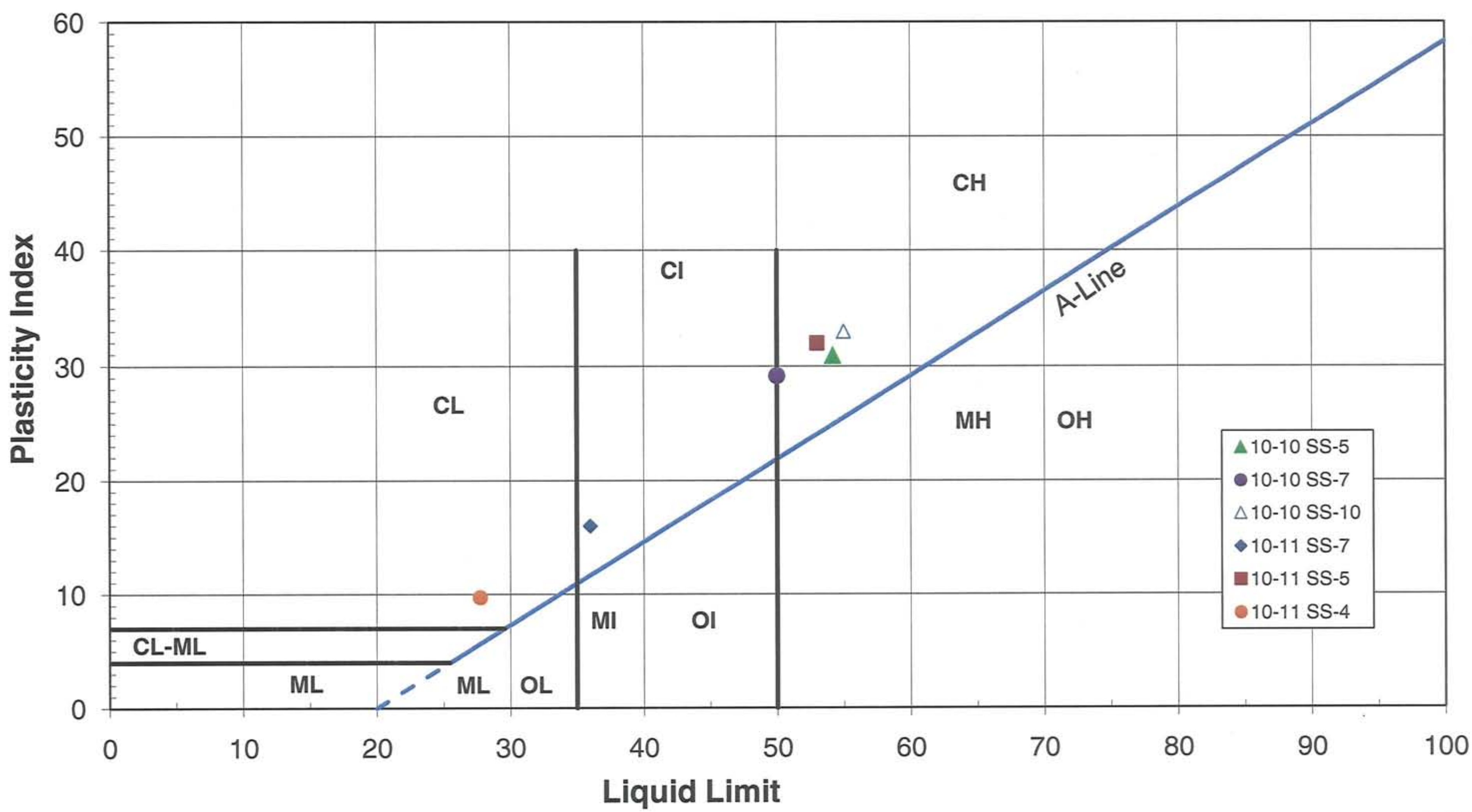
Stantec

GRAIN SIZE DISTRIBUTION

Silty Clay: Intermediate to high plasticity
(CI to CH)

Figure No. 15

Project No. 122410534



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PLASTICITY CHART

Figure No. 16

Project No. 122410534

APPENDIX F

Temporary Detour Staging Plans
GSC Seismic Hazard Calculation Sheet
Soil Parameter Design Models
Slope Stability Output
Temporary Protection System Locations
Settlement Estimates

DISTRICT OF SUDBURY
TOWNSHIP OF NAIRN

METRIC

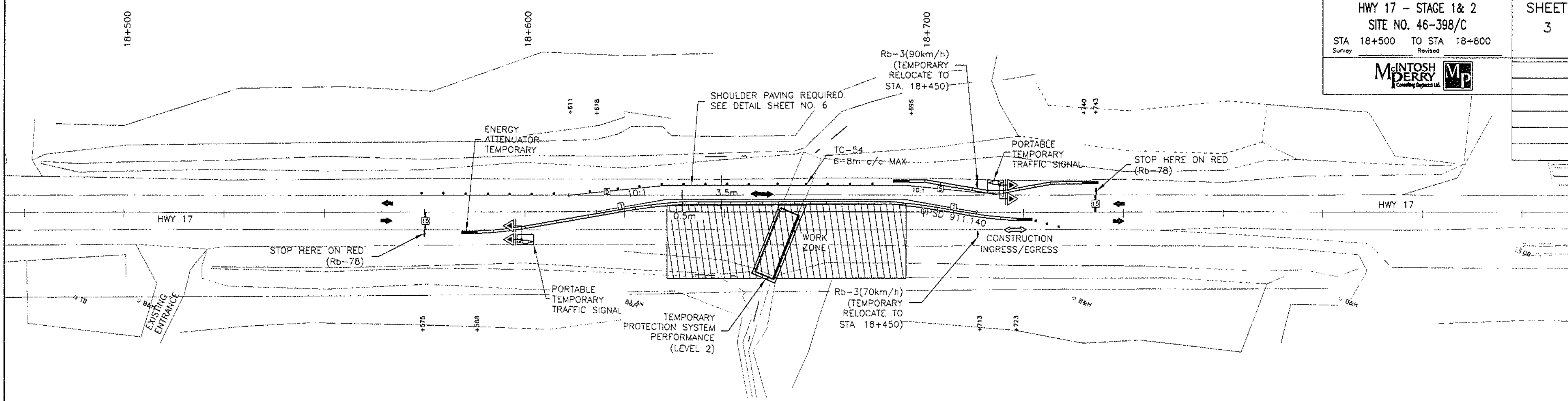
PLATE No
CONT 2011-XXXX
WP 5128-08-00

HWY 17 - STAGE 1& 2
SITE NO. 46-398/C
STA 18+500 TO STA 18+800
Survey Revised

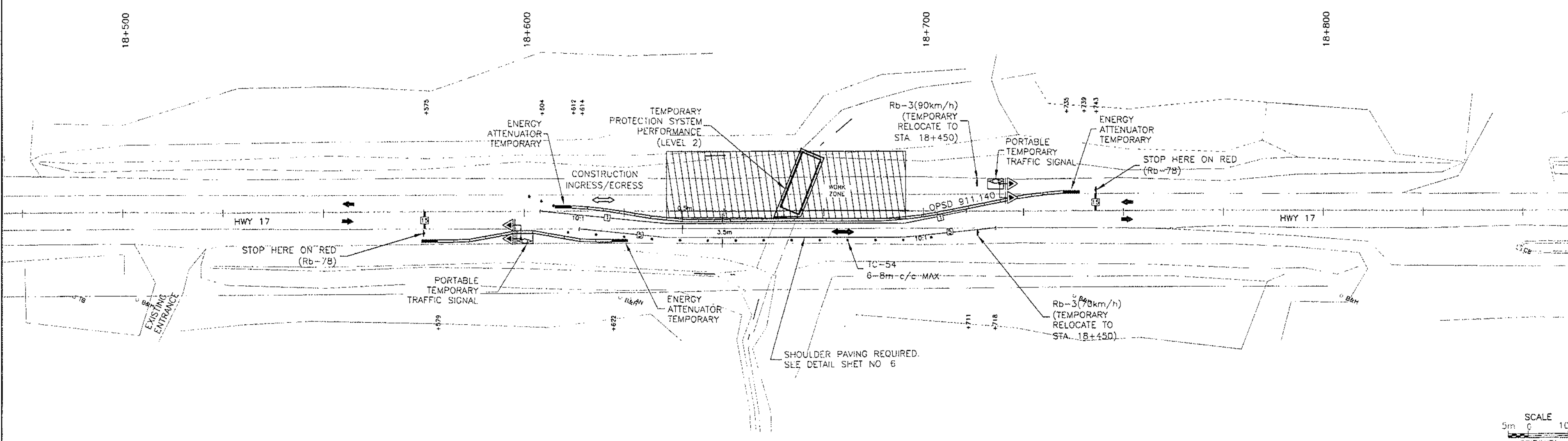
McINTOSH PERRY Consulting Engineers Ltd.

SHEET 3

STAGE 1



STAGE 2



SCALE
5m 0 10m
HORIZONTAL

PLATE No
CONT 2011-XXXX
WP 5128-08-00

HWY 17 - STAGE 1 & 2
SITE NO. 46-397/C
STA 11+200 TO STA 11+500

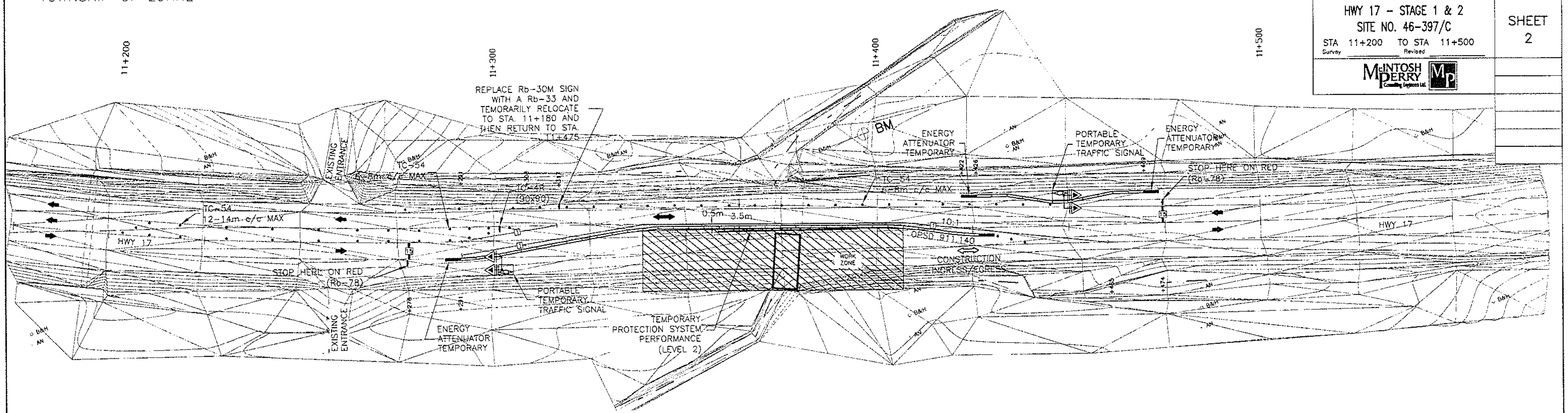
SHEET 2

McINTOSH PERRY

METRIC

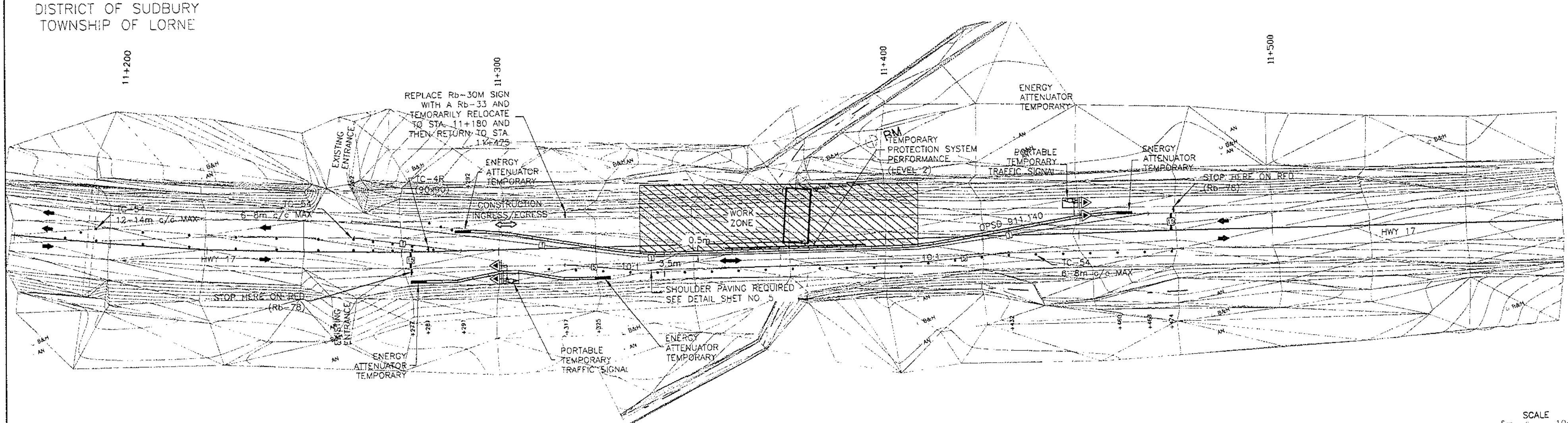
STAGE 1

DISTRICT OF SUDBURY
TOWNSHIP OF LORNE



STAGE 2

DISTRICT OF SUDBURY
TOWNSHIP OF LORNE



SCALE
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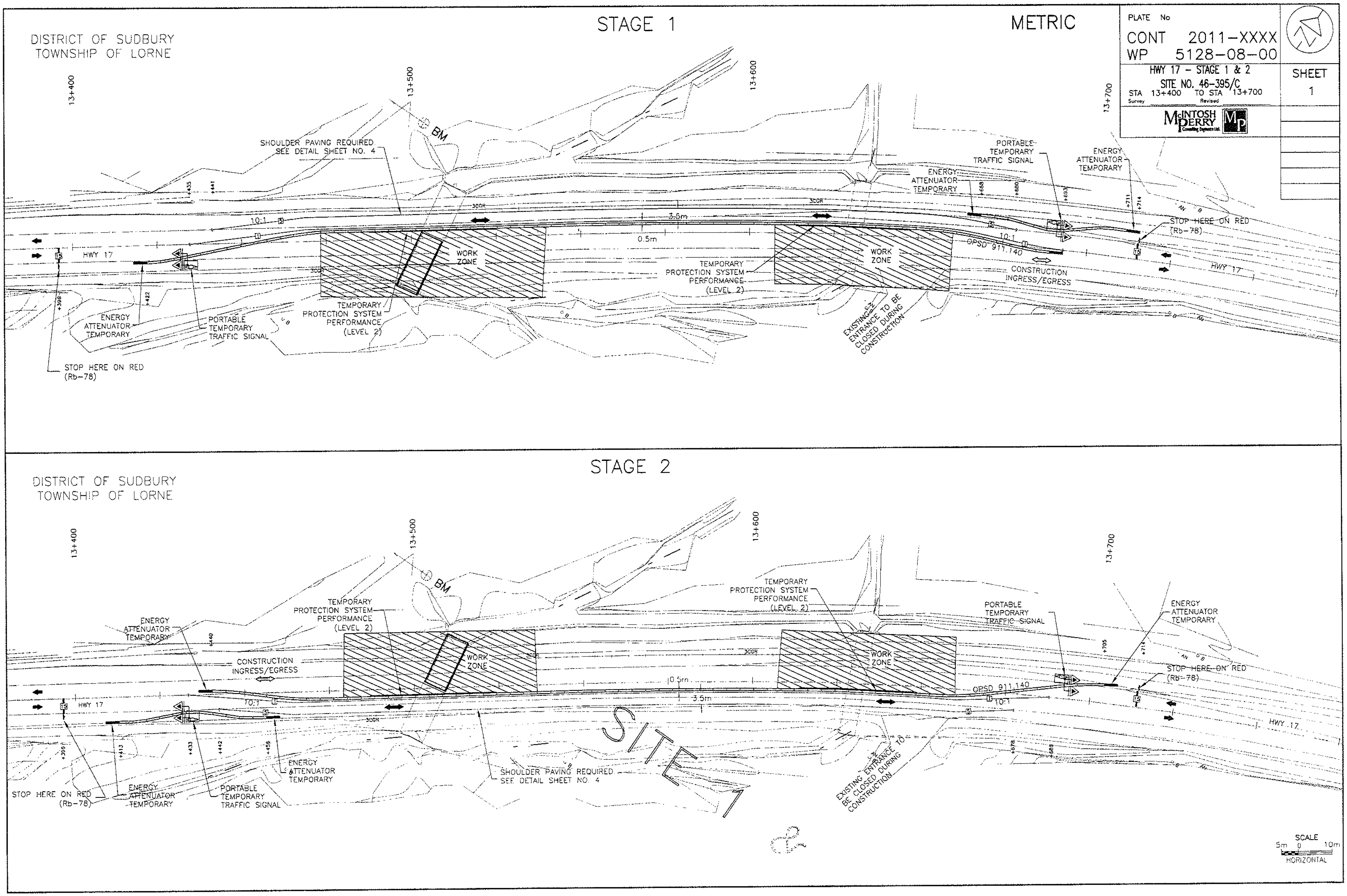
HORIZONTAL


STAGE 1
METRIC
DISTRICT OF SUDBURY
TOWNSHIP OF LORNE
PLATE No
CONT 2011-XXXX
WP 5128-08-00
HWY 17 - STAGE 1 & 2
SITE NO. 46-395/C
STA 13+400 TO STA 13+700
Survey
Revised
McINTOSH PERRY
SHEET
1
13+400
13+500
13+600
13+700
BM
SHOULDER PAVING REQUIRED SEE DETAIL SHEET NO. 4
PORTABLE TEMPORARY TRAFFIC SIGNAL
ENERGY ATTENUATOR TEMPORARY
STOP HERE ON RED (Rb-78)
HWY 17
WORK ZONE
TEMPORARY PROTECTION SYSTEM PERFORMANCE (LEVEL 2)
3.5m
0.5m
10:1
OPSD 911.140
CONSTRUCTION INGRESS/EGRESS
EXISTING ENTRANCE TO BE CLOSED DURING CONSTRUCTION
STAGE 2
DISTRICT OF SUDBURY
TOWNSHIP OF LORNE
13+400
13+500
13+600
13+700
BM
TEMPORARY PROTECTION SYSTEM PERFORMANCE (LEVEL 2)
ENERGY ATTENUATOR TEMPORARY
CONSTRUCTION INGRESS/EGRESS
WORK ZONE
PORTABLE TEMPORARY TRAFFIC SIGNAL
STOP HERE ON RED (Rb-78)
HWY 17
10.5m
3.5m
10:1
OPSD 911.140
SHOULDER PAVING REQUIRED SEE DETAIL SHEET NO. 4
EXISTING ENTRANCE TO BE CLOSED DURING CONSTRUCTION
SITE 1
SCALE
5m 0 10m
HORIZONTAL

2005 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: Paul Carnaffan, Stantec

September 22, 2010

Site Coordinates: 46.33 North 81.55 West

User File Reference: Hwy 17 Site 46-397/c

National Building Code ground motions:
2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.120	**0.078**	**0.039**	**0.013**	**0.059**

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2005 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.025	0.057	0.081
Sa(0.5)	0.014	0.036	0.052
Sa(1.0)	0.006	0.016	0.026
Sa(2.0)	0.002	0.005	0.008
PGA	0.010	0.024	0.035

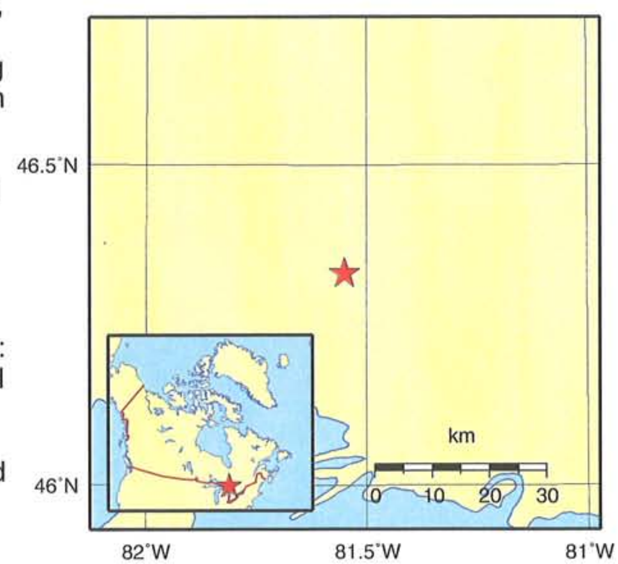
References

National Building Code of Canada 2005 NRCC no. 47666; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

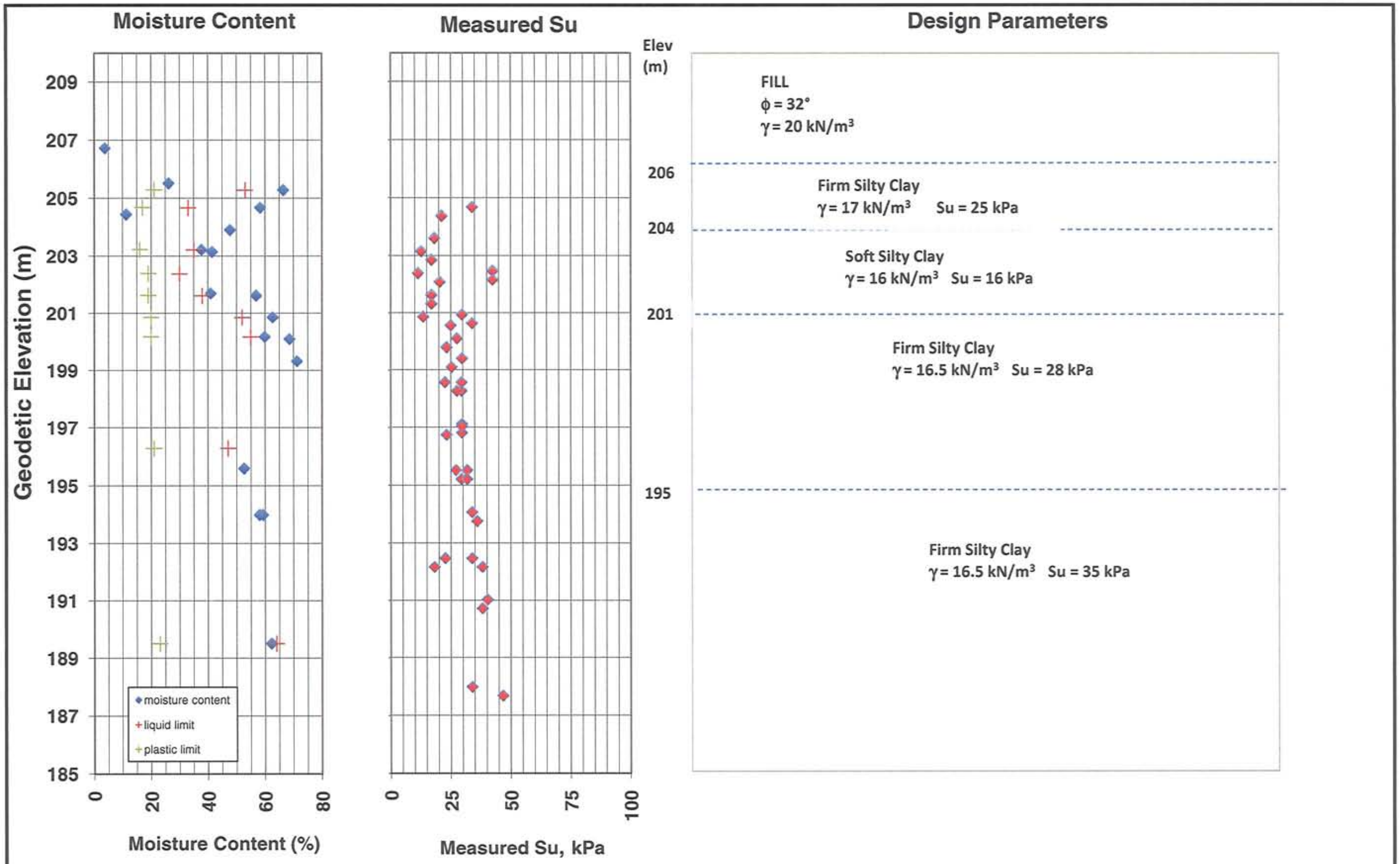
User's Guide - NBC 2005, Structural Commentaries NRCC no. 48192
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Grid values to be used with the 2005 National Building Code of Canada (in preparation)

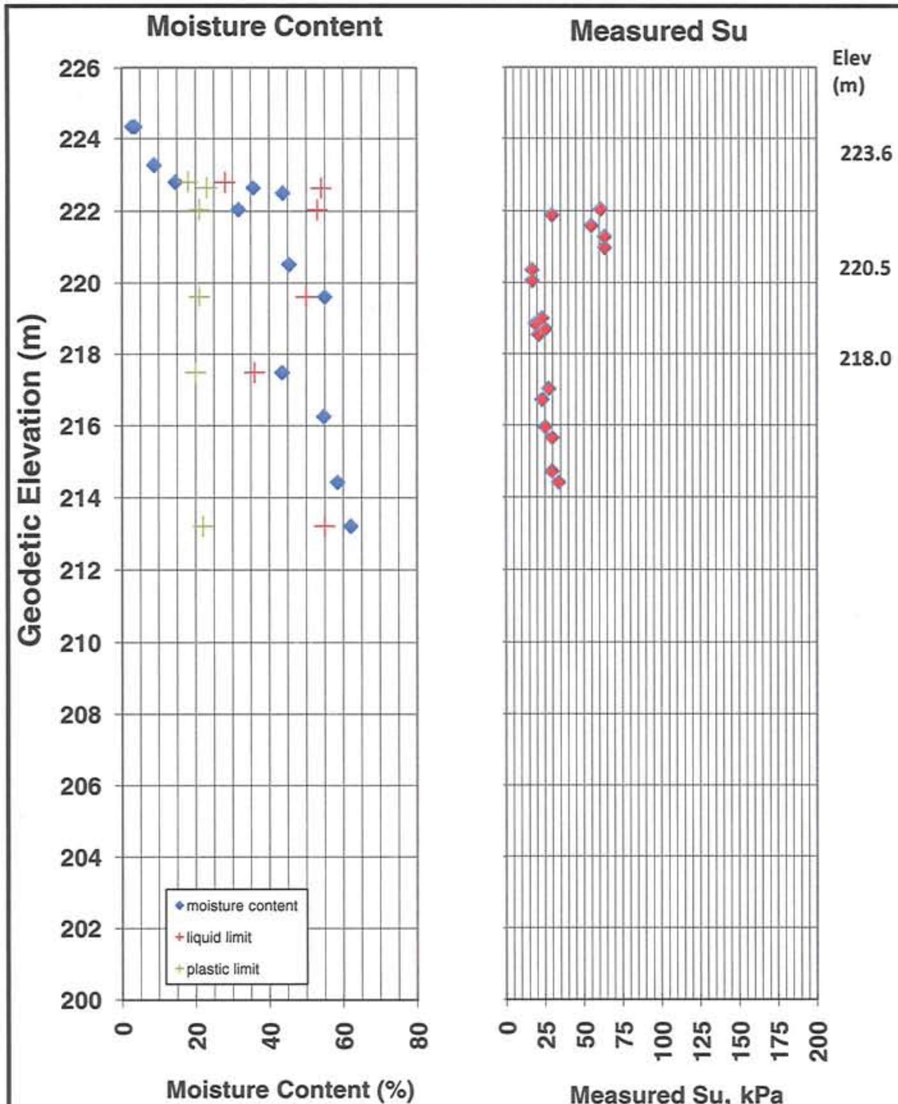
See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information



Aussi disponible en français



Stantec Consulting Ltd.	Project No. 122410534 Site 46-397/C - Station 11+373 Lorne Township	Figure F-1



Design Parameters

FILL
$\phi = 32°$ $\gamma = 20\ kN/m^3$ E = 100 MPa

Firm Silty Clay
$\gamma = 17\ kN/m^3$ Su = 50 kPa Cc=0.356 Cr =0.1 $C\alpha = 0.014$ $e_o = 1.38$

Soft Silty Clay
$\gamma = 16\ kN/m^3$ Su = 21 kPa Cc = 0.544 Cr = 0.1 $C\alpha = 0.022$ $e_o = 1.788$

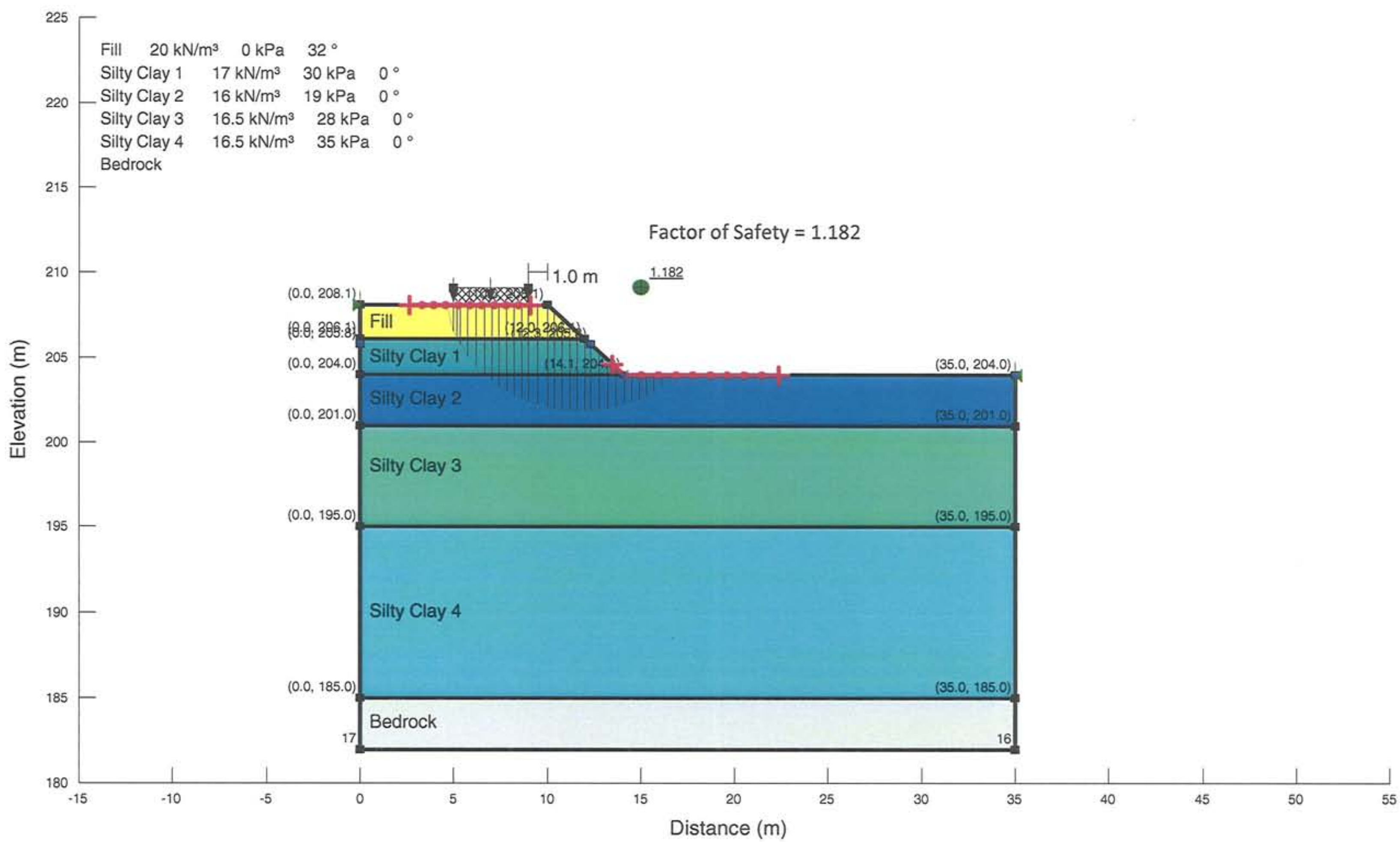
Firm Silty Clay
$\gamma = 16.5\ kN/m^3$
Su = 35 kPa
Cc = 0.427
Cr = 0.1
$C\alpha = 0.017$
$e_o = 1.54$

Stantec Consulting Ltd.

Project No. 122410534 Site 46-396/C Station 13+631 Lorne Township

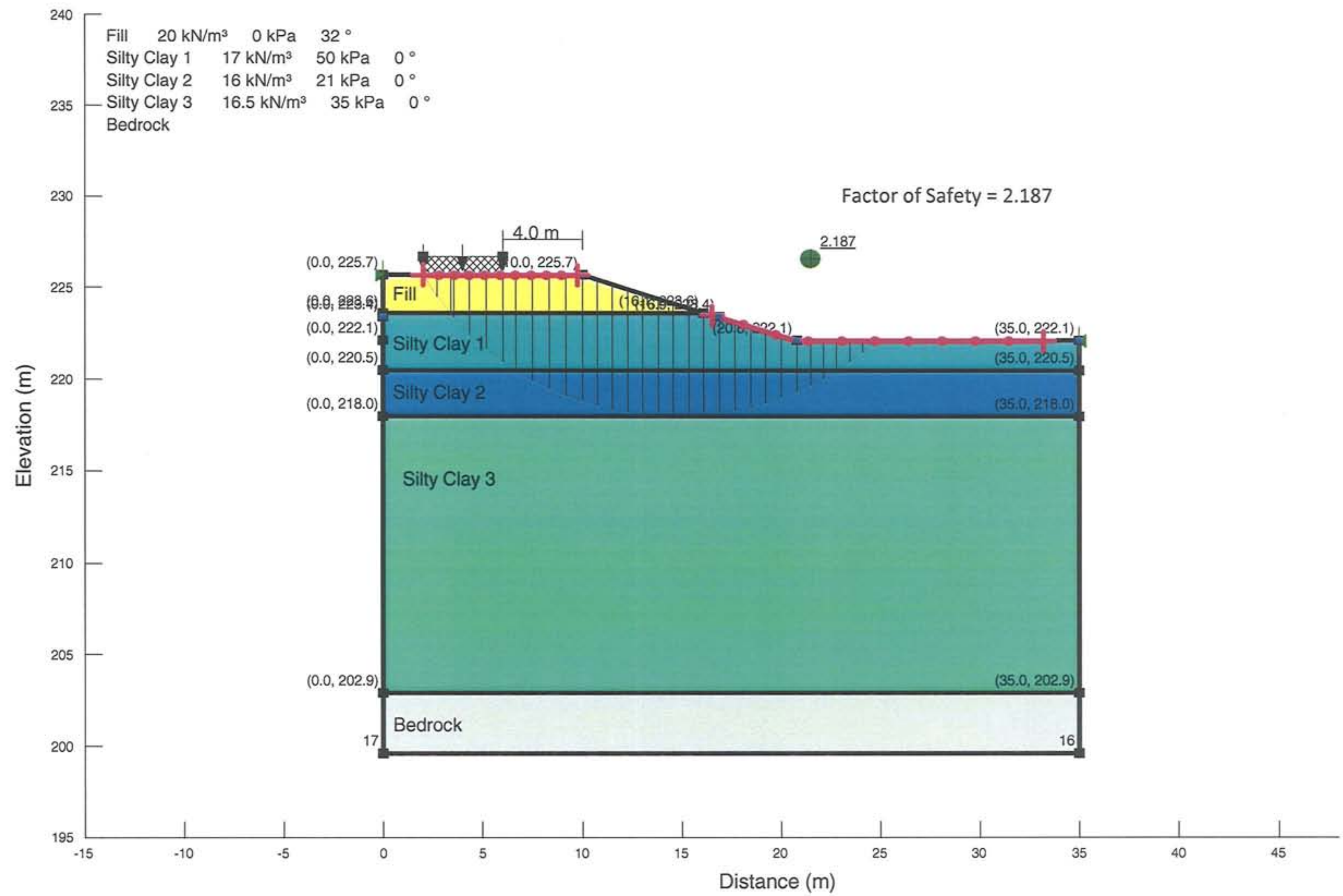
Figure F-2

V:\01224\active\1224105XX\122410534\Foundations\Report\Dwgs\Site 46-396C 13+631 - Soil Profile



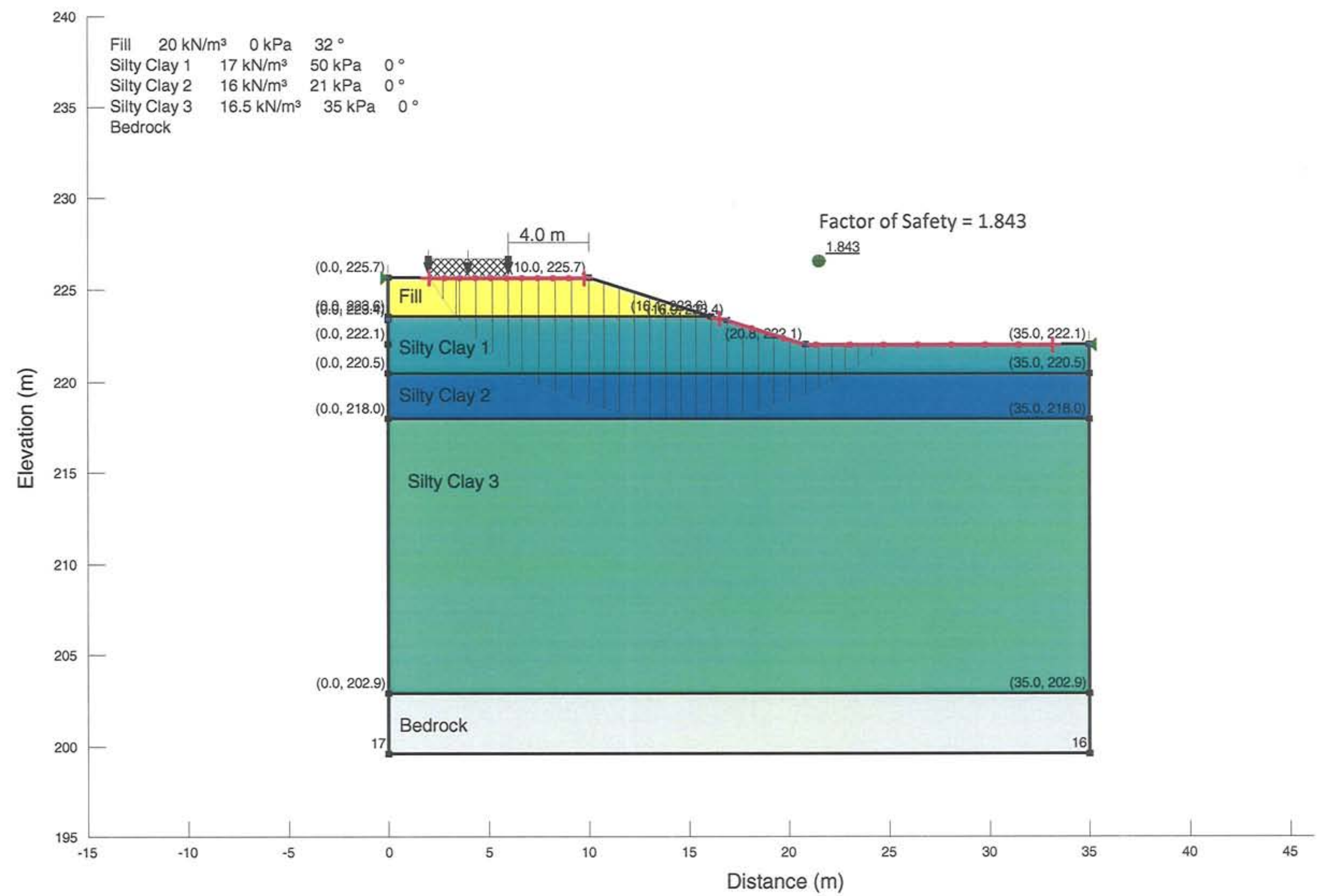
Stantec	**Slope Stability Analysis** Culvert at Station 11+373 - 1H:1V Slope Configuration Static Conditions Temporary Excavation
Highway 17 G.W.P. 5182-08-00	Figure No. F-3

V:\01224\active\1224105XX\122410534\Foundations\SlopeW\Embankment SlopeW Figures



Stantec	**Slope Stability Analysis** Culvert at Station 13+631 - 3H:1V Slope Configuration Static Conditions
Highway 17 G.W.P. 5182-08-00	Figure No. F-4

V:\01224\active\1224105XX\122410534\Foundations\SlopeW\Embankment SlopeW Figures



Slope Stability Analysis Culvert at Station 13+631 - 3H:1V Slope Configuration Seismic Conditions	Highway 17 G.W.P. 5182-08-00 Figure No. F-5

V:\01224\active\1224105XX\122410534\Foundations\SlopeW\Embankment SlopeW Figures

Table F-1

Site No. 46-398/C – Station 18+667

Comparison of Roadway Protection Systems

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
Steel sheet pile (SSP); soil anchors	• easier to install below waterline (no dewatering required during roadway protection installation) • penetration of sheet piles below base of excavation can limit water inflow and stabilize base	• will require removal and re-installation of soil anchors during staging	Moderate	
Steel sheet pile (SSP); cantilevered	• easier to install below waterline (no dewatering required during roadway protection installation) • penetration of sheet piles below base of excavation can limit water inflow and stabilize base • can be used for both stages		Moderate	
H-Piles with timber lagging; soil anchors	• simple installation	• will require removal and re-installation of soil anchors during staging • dewatering required prior to excavation • portion of system often left in place beneath pavement	Low	• seepage or flow of retained material/ possibility of settlement
H-Piles with timber lagging; rakers	• simple installation	• rakers can act in both tension and compression, avoiding need to re-install support system • dewatering required prior to excavation • portion of system often left in place beneath pavement	Low	• seepage or flow of retained material/ possibility of settlement

Table F-2

Site No. 46-397/C – Station 11+373

Comparison of Roadway Protection Systems

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
Steel sheet pile (SSP); soil anchors	• easier to install below waterline (no dewatering required during roadway protection installation)	• will require removal and re-installation of soil anchors during staging	High	
Steel sheet pile (SSP); cantilevered	• easier to install below waterline (no dewatering required during roadway protection installation) • can be used for both stages		High	• disturbance during extraction after second stage – could loosen soils and cause settlement
H-Piles with timber lagging; soil anchors	• simple installation	• will require removal and re-installation of soil anchors during staging • dewatering required prior to excavation • portion of system often left in place beneath pavement • limited capacity for soil anchors in soft clay	Low	• seepage or flow of retained material/ possibility of settlement
H-Piles with timber lagging; rakers	• simple installation	• rakers can act in both tension and compression, avoiding need to re-install support system • dewatering required prior to excavation • limited bearing resistance for rakers • portion of system often left in place beneath pavement	Low	• seepage or flow of retained material/ possibility of settlement

Table F-3

Site No. 46-395/C – Station 13+506

Comparison of Roadway Protection Systems

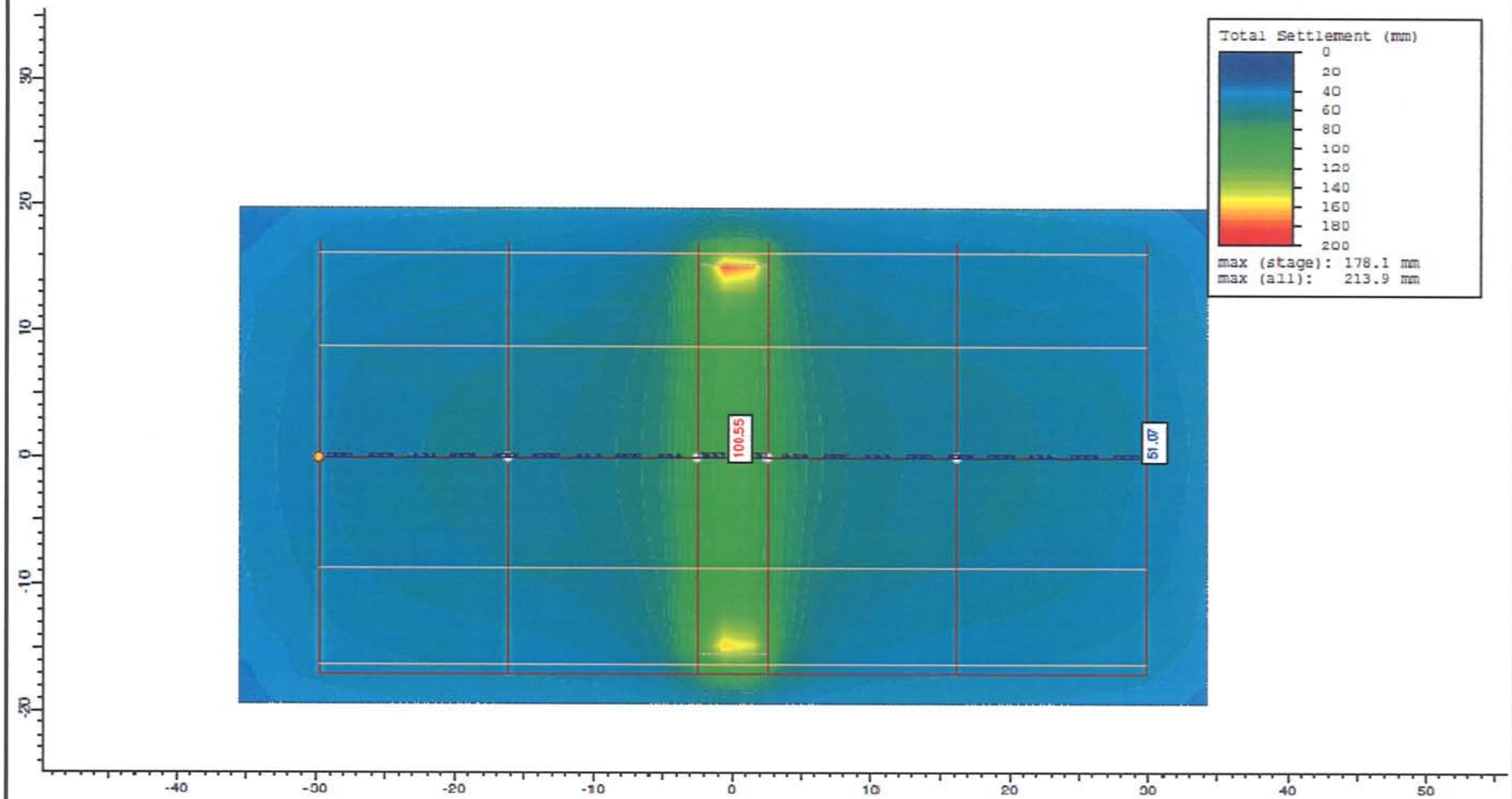
Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
Steel sheet pile (SSP); soil/rock anchors	• easier to install below waterline (no dewatering required during roadway protection installation)	• will require removal and re-installation of soil/rock anchors during staging	High	
H-Piles with timber lagging; soil anchors	• simple installation	• will require removal and re-installation of soil anchors during staging • dewatering required prior to excavation • portion of system often left in place beneath pavement	Low	• seepage or flow of retained material/ possibility of settlement
H-Piles with timber lagging; rakers	• simple installation	• rakers can act in both tension and compression, avoiding need to re-install support system • dewatering required prior to excavation • portion of system often left in place beneath pavement	Low	• seepage or flow of retained material/ possibility of settlement

Table F-4

Site No. 46-396/C – Station 13+631

Comparison of Roadway Protection Systems

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
Steel sheet pile (SSP); soil anchors	• easier to install below waterline (no dewatering required during roadway protection installation)	• will require removal and re-installation of soil anchors during staging • easily damaged where obstructions are encountered	High	
Steel sheet pile (SSP); cantilevered	• easier to install below waterline (no dewatering required during roadway protection installation) • can be used for both stages	• easily damaged where obstructions are encountered	High	• disturbance during extraction after second stage – could loosen soils and cause settlement
H-Piles with timber lagging; soil anchors	• simple installation	• will require removal and re-installation of soil anchors during staging • dewatering required prior to excavation • portion of system often left in place beneath pavement • limited capacity from soil anchors in soft clay	Low	• seepage or flow of retained material/ possibility of settlement
H-Piles with timber lagging; rakers	• simple installation	• rakers can act in both tension and compression, avoiding need to re-install support system • dewatering required prior to excavation • portion of system often left in place beneath pavement	Low	• seepage or flow of retained material/ possibility of settlement



Settlement Analysis

Culvert at Station 13+361

Total Settlement 20 yrs After Culvert Removal

Highway 17

G.W.P. 5182-08-00

Figure No. F-6

Stantec

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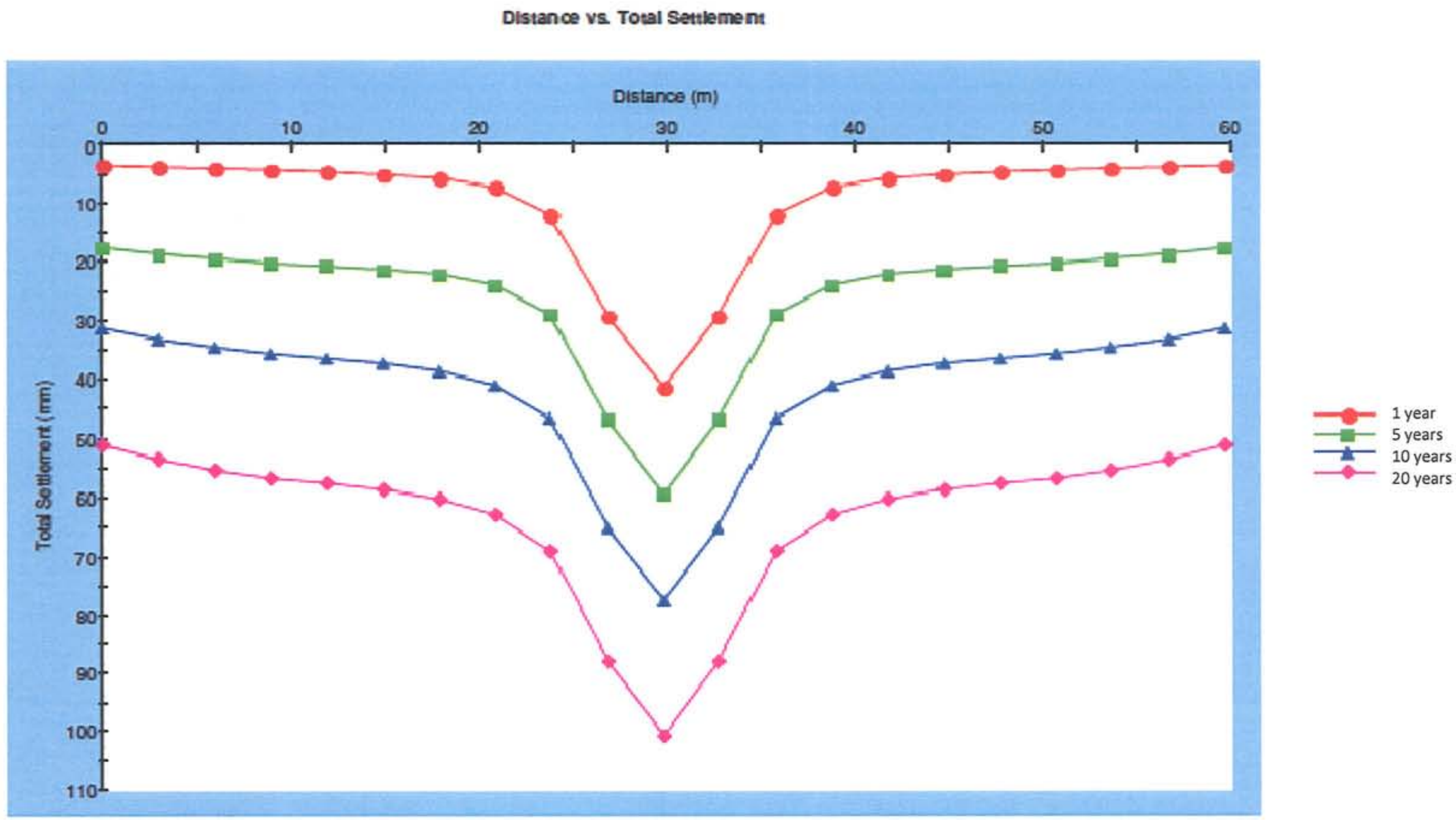

Stantec	Settlement Analysis Culvert at Station 13+361 Settlement along Centreline After Culvert Removal	Highway 17 G.W.P. 5182-08-00 Figure No. F-7

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