Stantec

Preliminary Foundation Investigation and Design Report

Highway 9
Holland Drainage Canal Bridge
Replacement

Township of King
Site No. 37-030

G.W.P. 2188-08-00

Geocres No. 31D-553

Project No. 165000801

December 2012

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PRELIMINARY FOUNDATION INVESTIGATION REPORT
For
G.W.P 2188-08-00

Highway 9 – Holland Drainage Canal Bridge Replacement

Site No. 37-030
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1.0 Introduction

Stantec Consulting Ltd. (Stantec) was retained by the Ministry of Transportation, Ontario (MTO) to undertake the preliminary design for the replacement of the Holland Drainage Canal Bridge. The project site is located approximately 4.6 km west of the Highway 400/Highway 9 Interchange, in the Township of King, Ontario. The Site Location Plan is indicated on Drawing No.1 in Appendix A.

The proposed bridge replacement is anticipated to include a slight realignment of Highway 9 to the north. It is understood that the bridge replacement option being considered will take into consideration the future expansion requirements of Highway 9. It is also understood that the proposed bridge replacement will include roadway protection requirements for excavations in the vicinity of the existing approach fills and abutments.

This Preliminary Foundation Investigation Report has been prepared specifically and solely for the proposed bridge replacement and anticipated roadway protection.

Project Number:	G.W.P.: 2188-08-00
Project Location:	Highway 9, Approximately 4.6 km west of Highway 400, Township of King

The work was carried out under Agreement Number 2010-E-066 with Stantec Consulting Ltd., the Preliminary Design Consultant for this project.

2.0 Site Description and Geology

Site Location

The site location is shown on the Key Plan inset to Drawing No. 1, provided in Appendix A. The existing Holland Drainage Canal Bridge carries Highway 9 traffic across the Canal at Structure Site No. 37-030.

General Site Description

At the project site, Highway 9 is oriented in the northeast-southwest direction. For the purpose of this report, Highway 9 is assumed to be oriented in the east-west direction with the chainage increasing west to east. Highway 9 has a single lane of traffic in each direction with approximately 3 m wide shoulders (see Photographs 1 through 4 in Appendix A).

Flow in the canal is from north to south beneath the bridge to approximately 40 m south of the bridge centerline. The canal (and the flow) runs approximately easterly thereafter.

In the vicinity of the existing bridge the surrounding area is generally flat.

Existing Bridge

The existing Holland Drainage Canal Bridge has two spans, each approximately 7 m long and a width of approximately 12.7 m. The bridge structure is supported on approximately 13.7 m long timber piles. Review of available Geocres reports indicates that the original bridge had been widened to the north along with the construction of new retaining walls (wingwalls) on the north side of the bridge. The wingwalls included deadman anchors to resist lateral loads.

Physiographic Description

The site is located within a physiographic region known as the Schomberg Clay Plains at the northern foothills of the Oak Ridges Moraine (Chapman and Putnam, 1984). This region contains deep deposits of stratified clay and silt. The schomberg sediments are typically varved clays with annual layers of silt (summer) and clay (winter) having variable thicknesses. This physiographic region is also known to contain high organic content soils commonly described as muck.

It is noted that the site is also very close to the southwestern boundary of the physiographic region known as Simcoe Lowlands.

Drainage is generally toward the east and northeast toward Cook’s Bay (Lake Simcoe). In the vicinity of the site, flow is towards the Holland Drainage Canal.

3.0 Method of Investigation

3.1 DRILLING INVESTIGATION

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

A geotechnical field investigation consisting of 14 boreholes, two cone penetration test (CPT) holes and one hand auger hole was carried out for this assignment. The boreholes were designated BH12-1 through BH12-3 and BH12-5 through BH12-15. The CPT locations were designated CPT12-4 and CPT12-17 and the hand auger hole AH12-16. The investigation locations are shown on the Borehole Location Plan, Drawing No.1 in Appendix A.

A Dynamic Cone Penetration Test (DCPT) was carried out in BH12-2 from 25.0 m to 31.9 m below ground surface.

The field drilling program was carried out from March 27 to April 25, 2012. All boreholes were advanced with a track mounted Dietrich D-50 drill with a combination of hollow-stem augers and steel casings.

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 (typically every 760 mm) during the course of Standard Penetration Testing (ASTM D1586). In-situ shear vane measurements were also carried at selected locations in the cohesive deposit using MTO field vane and a pocket penetrometer. All samples recovered were returned to Stantec's Ottawa laboratory for detailed classification and testing.

The cone penetration tests (CPT) were carried out in accordance with ASTM D5778 (ASTM, 2007). Dissipation test was performed in one of the two CPT holes.

Artesian flow within borehole BH12-5 was controlled by pushing a nylon wrapped ball of bentonite chips to the bottom of the H-size casing followed by backfilling with bentonite chips and bentonite quick-gel grout. Artesian flow within borehole BH12-1 was controlled by pushing a P-size casing, concentric to the H-size casing, into the cohesive soils to capture all annular flow, followed by extending the P-size casing to 3.1 m above ground and pumping of a heavy grout mix of cement, barite, and bentonite.

A standpipe piezometer was installed within BH12-3. The section of the borehole deeper than 6 m was backfilled with a mix of bentonite chips and bentonite quick-gel grout, from 3 to 6 m a well screen with sand backfill was installed, and from 0 to 3 m a solid well pipe with bentonite backfill was installed.

Boreholes were backfilled with auger cuttings mixed with bentonite and road holes were topped with cold patch asphalt whenever applicable.

3.2 LOCATION AND ELEVATION SURVEY

The borehole location (northing and easting) and elevation (Geodetic) survey was carried out by Stantec using a Global Positioning System (GPS) apparatus Trimble Geo XH. The GPS apparatus had horizontal and vertical accuracies of 0.01 and 0.1 m, respectively.

Table 3.1 summarizes the borehole information.

Table 3.1: Borehole Summary

Borehole	MTM Zone 10 Coordinates		Ground Surface Elevation (m)	Total Depth Drilled (m)	End of Borehole Elevation (m)	Depth[1] Augered (m)	Number of Soil Samples
	Northing	Easting					
BH12-1	4875305	292810	219.7	26.2	193.5	26.2	28
BH12-2	4875312	292843	219.9	31.9	188.0	24.8	27
BH12-3	4875315	292852	220.1	25.5	194.6	25.5	27
CPT12-4	4875310	292807	220.3	10.9	209.4	-	-
BH12-5	4875323	292845	221.8	30.9	190.9	30.9	29
BH12-6	4875299	292800	219.8	9.8	210.0	9.8	13
BH12-7	4875321	292873	220.4	9.8	210.6	9.8	13
BH12-8	4875327	292863	221.2	9.8	211.4	9.8	13
BH12-9	4875305	292789	219.9	9.8	210.1	9.8	13
BH12-10	4875327	292891	220.4	8.2	212.2	8.2	11
BH12-11	4875348	292964	221.4	8.2	213.2	8.2	11
BH12-12	4875374	293033	221.6	8.2	213.4	8.2	10
BH12-13	4875400	293101	221.6	8.2	213.4	8.2	10
BH12-14	4875425	293173	221.7	8.2	213.5	8.2	10
BH12-15	4875446	293245	221.4	8.2	213.2	8.2	10
AH12-16[2]	4875308	292840	219.0	1.5	217.5	1.5	5
CPT12-17	4875316	292852	220.2	18.0	202.2	-	-

Notes: (1) No bedrock coring was carried out in any of the boreholes advanced at this site.
(2) AH refers to hand-auger hole beneath 150 mm of standing water.

3.3 LABORATORY TESTING

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

The geotechnical laboratory testing program is summarized in the following table.

Table 3.2: Geotechnical Laboratory Testing Program

Test Description	Number of Samples	Remarks
Moisture Content	224	2 by Golder
Atterberg Limits	39	2 by Golder
Grain Size Distribution	61	2 by Golder
Consolidation (oedometer)	2	By Golder
Unconfined Compression (Soil)	2	By Golder
Specific Gravity	2	By Golder

It is noted that where a value is provided for the percent of clay sized particles, the value represents the percent finer than a nominal size of 0.002 mm.

Nine samples were submitted to Parcel Laboratories of Ottawa for analysis of pH, soluble sulphate content, chloride content and resistivity.

Samples remaining after testing will be placed in storage for a period of one year after issuance of the final report. After the storage period, the samples will be discarded unless we are directed otherwise by MTO.

4.0 Subsurface Conditions

4.1 SUBSURFACE PROFILE

The subsurface conditions observed in the boreholes 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 also provided.

Results of two Cone Penetration Tests (CPTu) are also included in Appendix B.

The site is at the western limit of the Holland Marsh and the ground rises to the west resulting in differing subsurface conditions at the east and west sides of the Holland Canal bridge. The following is a generalized description of the subsurface conditions.

- On the east side of the canal the natural soil conditions include an organic silt that is up to 6 m deep underlain by silt and sand layers of limited thickness, underlain by a deep clayey silt deposit extending to about 25 to 30 m below ground surface, followed by a permeable silty sand with artesian groundwater conditions.
- On the west side of the canal the near surface soils consist of sands and silty sands which extend down to the same deep clayey silt deposit and underlying silty sand with artesian groundwater conditions.
- Boreholes drilled through the roadway platform show that the organic silt east of the canal was excavated beneath the existing roadway embankment and replaced with silty sand to sandy silt soils.

Borehole location plans and stratigraphic sections of the soils encountered within the boreholes in the vicinity of the bridge location are provided on Drawing No. 1 through 3 in Appendix A.

4.1.1 Topsoil

Topsoil was encountered in BH12-1, BH12-2, BH12-3, BH12-5, BH12-6, BH12-7, BH12-8 and BH12-9. The thickness of the topsoil ranged between 50 and 300 mm.

4.1.2 Fill

Away from the road embankment, fill was encountered in BH12-2, BH12-3, BH12-5, BH12-7 and BH12-10. The fill was 0.8 to 1.9 m thick with base elevations of 218.6 m to 220.4 m.

Within the road embankment, fill was encountered in BH12-11 to BH12-15. The fill was 3.0 to 3.7 m thick with base elevations of 217.7 m to 218.7 m.

The Standard Penetration Test (SPT) blow counts (N-values) of the fill layer ranged from 4 to 60 blows per 0.3 m.

Fill was also encountered in the hand auger hole advanced beneath standing water in the canal near the exiting northeast wingwall (AH12-16). This fill was mainly silty sand with gravel to sandy silt and had a moisture content ranging between 29 and 35%.

Index tests carried out on representative soil samples retrieved from this layer revealed the following results:

Gravel:	4 to 7%
Sand:	29 to 46%
Silt/Clay:	43 to 66%
Clay:	13 to 14%
Organic Matter:	2%
Moisture Content:	7 to 29%

The fill consists of variable mixtures of sand and silt and is described as a silty sand (SM) to sandy silt (ML).

Representative grain size distribution plots for the fill layer are provided on Figure 1 in Appendix C. Representative plasticity chart is provided on Figure 8 in Appendix C.

4.1.3 Organic Silt (Muck) – East of Canal

This deposit was encountered in BH12-2, BH12-3, BH12-5, BH12-7, BH12-8 and BH12-10 immediately beneath the topsoil and the fill layer. The deposit was encountered in boreholes advanced east of the Holland Drainage Canal and consisted predominantly of silt, clay and some organics. This deposit was 2.4 to 5.0 m thick with base elevations of 215.1 m to 216.6 m.

A buried organic silt was also encountered in BH12-13. This layer was 2.1 m thick with a base elevation of 215.8 m.

The SPT N-values for the organic silt layer were less than 4 blows per 0.3 m suggesting a soft state.

An in-situ field vane test carried out near the base of the organic silt layer indicated an undrained shear strength of 39 kPa. An unconfined compression test carried out on a thin-wall tube sample indicated an undrained shear strength of 14 kPa.

Index tests carried out on representative soil samples retrieved from this layer revealed the following results:

Gravel:	0 to 4%
Sand:	3 to 46%
Silt:	40 to 76%
Clay:	10 to 27%
Moisture content:	32 to 113%
Organic matter:	4 to 23%

This deposit is classified mainly as organic silt (OL) according to the Unified Soil Classification System. Typical grain size distribution and plasticity chart for representative samples from the organic silt deposit are provided on Figures 2 and 9, in Appendix C.

The results of a consolidation test carried out on an organic silt sample from BH12-5 obtained at a depth of 4.7 m is provided in Appendix C. The consolidation and index property test results for this sample suggest the following:

• Natural moisture content	35%
• Specific gravity	2.75
• % sand	5%
• % silt size fines	74%
• % clay size fines	21%
• Estimated preconsolidation pressure, P'_c	70 kPa
• Estimated effective overburden pressure, P'_o	70 kPa
• Compression Index, C_c	0.31
• Recompression Index, C_r	0.08

4.1.4 Sand – West of Canal

A sand layer was encountered immediately beneath the topsoil in BH12-1, BH12-6 and BH12-9. Its thickness ranged between 2.2 m and 2.9 m with base elevations of 216.9 to 217.5 m.

The SPT N-values for this sand layer ranged from 1 to 9 blows per 0.3 m suggesting a very loose to loose state.

The moisture content of the sand layer ranged between 20 and 22%.

4.1.5 Silt

A silt layer was encountered beneath the organic silt or the road embankment in boreholes 12-3, 12-7, 12-8, 12-11, 12-12, 12-14 and 12-15. This layer was 0.8 to 2.4 m thick with base elevations of 216.5 to 214.7 m.

The SPT N-values for this layer were between 2 and 10 blows per 0.3 m suggesting a very loose to compact state.

Index tests carried out on representative soil samples retrieved from the silt layer revealed the following results:

Gravel:	0 to 3%
Sand:	1 to 36%
Silt:	46 to 88%
Clay:	11 to 17%
Moisture content:	18 to 38%
Liquid Limit	20 to 24
Plasticity Index:	6 (for all tests)

According to the Unified Soil Classification System, the silt layer can be classified as CL-ML but has been carried forward as ML in the report (silt, silt with sand or sandy silt). Representative grain size distribution plots and plasticity charts for this layer are provided on Figure 3 and on Figure 10 in Appendix C.

4.1.6 Sand with Silt

A discontinuous sand with silt layer was observed beneath the organic silt layer in BH12-2 and beneath the silt layer in BH12-3. The layer was 1.6 and 2.4 m thick with base elevations of 213.4 m at both locations.

The SPT N-values for this layer were between 7 and 32 blows per 0.3 m suggesting a loose to dense state.

Index tests carried out on representative soil samples retrieved from this layer provided the following results:

Gravel:	1 to 10%
Sand:	83 to 88%
Fines (silt and clay)	7 to 11%
Moisture Content:	12 to 19%

According to the Unified Soil Classification System, this layer can be classified as SP-SM (poorly graded sand with silt). Representative grain size distribution plots for this layer are provided on Figure 4 in Appendix C.

4.1.7 Silty Sand

This layer was encountered in all boreholes except in BH12-5. West of the canal the silty sand deposit was observed directly beneath the loose sand layer. East of the canal it was generally observed beneath discontinuous layers of silt, sand, or clay which underly the organic silt deposit. This layer was not penetrated in boreholes BH12-12 through BH12-15 since drilling was terminated within this layer. Where penetrated, this layer was approximately 0.3 to 2.6 m thick with base elevations of 210.6 to 214.9 m.

The SPT N-values for this layer were between 4 and 37 blows per 0.3 m suggesting a loose to dense state.

Index tests carried out on representative soil samples retrieved from this layer revealed the following results:

Gravel:	2 to 29%
Sand:	47 to 85%
Fines (silt & clay):	6 to 39%
Moisture content:	9 to 20%

The USCS designation for this layer is SM (silty sand to silty sand with gravel). Representative grain size distribution plots for this layer are provided on Figure 5a and 5b in Appendix C.

4.1.8 Clayey Silt

This deposit was encountered in all boreholes except BH12-12 through BH12-15 which were terminated in the overlying silty sand. Where fully penetrated, the clayey silt layer was 17 to 20.8 m thick with base elevations of 195.3 to 192.7 m.

The in-situ undrained shear strength of the clayey silt layer ranged from 38 kPa to greater than 235 kPa suggesting a firm to hard consistency. The undrained shear strength is generally higher below elevation 204.0 m.

The SPT N-values for the clayey silt layer ranged from 1 to 49 blows per 0.3 m.

The results of static cone penetration tests CPT12-4 and CPT12-17 are provided in Appendix B. This result suggests the following:

- The undrained shear strength of the clayey silt is weakest between elevation 204.0 and 207.5 with undrained shear strength of approximately 50 kPa.
- That there are frequent permeable zones above elevation 207.5 m and below elevation 204.0 m.
- Dissipation test was carried out within CPT 12-17 at elevation 204.3 m. The results indicate a $C_h = 1.6\text{x}10^{-3}$ cm^2/min.

Index tests carried out on representative soil samples retrieved from the clayey silt deposit yielded the following results:

Gravel:	0 to 1%
Sand:	0 to 25%
Silt:	31 to 73%
Clay:	21 to 68%
Moisture content:	17 to 48%

The Unified Soil Classification System group designation for the clayey silt layer is predominantly CL with limited samples yielding CI or CL-ML. Representative grain size distribution plots for the silty clay deposit are provided on Figures 6A through 6c; plasticity charts are provided on Figures 11a through 11d in Appendix C.

The results of a consolidation test carried out on a clayey silt sample from BH12-3 obtained at a depth of 16.1 m is provided in Appendix D. The consolidation and index property test results for this sample suggest the following:

Natural moisture content	30%
Specific gravity	2.77
% sand	4%
% silt size fines	69%
% clay size fines	27%
Estimated preconsolidated pressure, P'_c	325 kPa
Estimated effective overburden pressure, P'_o	210 kPa
Compression Index, C_c	0.29
Recompression Index, C_r	0.02

4.1.9 Deep Silty Sand

A silty sand deposit was encountered immediately beneath the clayey silt layer in all the deep boreholes that penetrated the dry layer, namely, BH12-1, BH12-2 and BH12-5.

The SPT N-values for this layer ranged from 4 to 25 suggesting a loose to compact state.

Dynamic Cone Penetration Test (DCPT) carried out in BH12-2 indicated a blow count range of 21 to 561 per 0.3 m, generally increasing with depth. The blow count per 0.3 m of penetration increased significantly below elevation 189 m.

Index tests carried out on representative soil samples retrieved from this layer revealed the following results:

Gravel: 0%
Sand: 87%
Fines (silt & clay): 13%
Moisture content: 19%

The Unified Soil Classification System group symbol designation for this layer is SM (silty sand). Representative grain size distribution plot is provided in Figure 7 in Appendix C.

4.2 BEDROCK

Bedrock was not encountered within the depth of exploration of this investigation.

4.3 GROUNDWATER

A groundwater monitoring well was installed in BH12-3 after completion of drilling. The groundwater in this well was measured two weeks later on April 18, 2012. The depth to groundwater was also inferred in all other boreholes at the time of drilling, between March 27 and Aril 25, 2012. The measured and inferred (i.e., at the time of drilling) groundwater levels are summarized in Table 4.1 below.

Table 4.1: Measured Inferred Groundwater Levels (time of drilling)

Borehole No	Ground Surface Elevation (m)	Groundwater	
		Depth (m)	Elevation (m)
Measured			
BH12-3	220.1	0.9	219.2
Inferred			
BH12-1	219.7	0.9	218.8
BH12-2	219.9	2.7	217.2
BH12-5	221.8	4.6	217.2
BH12-6	219.7	0.9	218.8
BH12-7	220.4	5.3	215.1
BH12-8	221.2	6.7	214.5
BH12-9	219.9	0.9	219.0
BH12-10	220.4	4.9	215.5
BH12-11	221.4	4.0	217.4
BH12-12	221.6	4.1	217.5
BH12-13	221.6	5.6	216.0
BH12-14	221.7	3.4	218.3
BH12-15	221.4	4.6	216.8

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

The Holland Canal water level was surveyed by others to be at elevation 218.7 m in November 2011 and by Stantec to be at elevation 219.2 in April 2012.

Artesian Condition

Artesian conditions were observed during the course of the investigation. The following summarizes the information and observations regarding the artesian conditions.

- Artesian groundwater pressure was observed beneath the clayey silt in BH12-1, BH12-2 and BH12-5.
- Within BH12-5 a sealed casing was extended 3.1 m above ground surface and significant flow was still observed. The flow was significantly reduced by pumping a mix of cement, barite, and bentonite slurry into the cased borehole.
- Within BH12-2 and BH12-5 artesian water flows were also observed as the H-size casing was advanced through the clayey silt deposit. These observations were observed at depths of greater than 10 m below ground surface.
- In 1965 the artesian water level was recorded at 5.5 m above ground (elevation 224.7 m) at this site.

4.4 CHEMICAL TESTS

Nine samples of the native soil at the site from the different boreholes were tested for pH, water soluble sulphate and chloride concentrations, and resistivity. The analysis results are provided in Table 4.2.

Table 4.2: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
12-1	SS-3	1.22 to 1.83	7.9	480	30	9.51
12-2	SS-6B	3.66 to 4.27	7.4	311	297	11.6
12-3	SS-2	0.61 to 1.22	8.3	289	17	20.2
12-5	SS-6	3.66 to 4.27	7.5	78	105	31.9
12-7	SS-6	3.81 to 4.42	8.1	1190	30	5.62
12-8	SS-7	4.57 to 5.18	6.9	799	221	5.12
12-11	SS-6	3.81 to 4.42	7.9	948	10	8.54
12-13	SS-4	3.05 to 3.66	8.4	961	56	5.51
12-15	SS-3	2.29 to 2.90	8.2	683	42	4.93

5.0 Miscellaneous

The field work was carried out under the supervision of Mr. Jeff Forrester, Geotechnical Engineering Technologist, under the direction of Mr. Chris McGrath, P.Eng.

MultiVIEW Locates Inc. of Mississauga, Ontario, carried out the private and public utility locates for the boreholes.

The D-50 drilling equipment was supplied and operated by Walker Drilling of Utopia, Ontario.

Location and elevation survey of the boreholes was carried out by Stantec.

Geotechnical laboratory testing was carried out at Stantec’s Ottawa laboratory and Golder Associate’s Mississauga laboratory. Chemical testing for pH, soluble sulphate, and chloride content, and resistivity was carried out by Paracel Laboratories of Ottawa.

This report was prepared by Simon Gudina and reviewed by Chris McGrath and Raymond Haché.

6.0 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.

Respectfully Submitted;

STANTEC CONSULTING LTD.

LICENSED PROFESSIONAL ENGINEER
S. K. GUDINA
100162564
Jan 14, 2013
PROVINCE OF ONTARIO

S Gudina
Simon Gudina, Ph.D., P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
C. G. MCGRATH
100106606
Jan 14, 2013
PROVINCE OF ONTARIO

Chris McGrath, P.Eng.
Associate, Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
17713504
Jan 14, 2013
PROVINCE OF ONTARIO

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

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PRELIMINARY FOUNDATION DESIGN REPORT
For
G.W.P 2188-08-00

Highway 9 – Holland Drainage Canal Bridge Replacement

Site No. 37-030
Township of King

7.0 Discussions and Engineering Recommendations

7.1 PROJECT DESCRIPTION & BACKGROUND

Project Purpose/Justification

The Holland Drainage Canal Bridge is a two-span structure. It is approximately 12.7 m wide and has a total span of approximately 15 m. The bridge was constructed around or prior to the 1960s and has since been widened once. The existing bridge structure consists of reinforced concrete deck on steel beam, reinforced concrete abutments, timber piles and retaining walls (wingwalls). The bridge shows signs of deterioration and spalling off of the structural concrete, including the concrete parapet wall (see for e.g., Photo No. 2 in Appendix A). The bridge structure has been identified as requiring replacement.

Structural inspection and sampling of the timber piles indicate that the core of some of the piles are rotted, rendering them susceptible to compression damage. Stantec previously carried out preliminary settlement analysis which indicated that to avoid imposing new settlements on the existing bridge during the construction period it would be necessary to maintain a distance of at least 12 m between the existing east abutment and the new road embankment fills.

Proposed Structure

It is understood that two replacement alternatives were considered.

- The Alternative 1 replacement option involving a two-span bridge structure with new abutments, a center pier, and a total span of approximately 36 m. The proposed replacement bridge alignment would be approximately 12 m north of the existing alignment, the west abutment would be approximately 7.5 m west of the existing, and the east abutment would be approximately 16 m east of the existing. The proposed new bridge would include wingwalls at all four quadrants.
- The Alternative 2 replacement option involving the construction of a temporary detour bridge whose alignment would be approximately 23 m north of the existing alignment. Traffic would be detoured to the temporary bridge, the existing bridge would be removed, and the permanent bridge would be constructed. The permanent Alternative 2 bridge would have a single 21 m span and its alignment would be 7 m north of the existing bridge alignment. The new bridge would include wingwalls at all four quadrants. Drawing 1B in Appendix A shows the Alternative 2 bridge locations.

It is understood that Alternative 1 with a two-span bridge structure is being carried forward for the proposed bridge replacement. Drawing 1 in Appendix A shows the proposed bridge location.

Based on the preliminary general arrangement plan for proposed replacement bridge and the foundation investigation results, key elevations associated with the proposed replacement structure are as follows:

Proposed Underside of Pile Cap Elevation:	217.5 m
Proposed final grade at E/W Abutment:	222.2 m
Existing Grade Elevation at E/W:	221.5 m
Water Level Elevation:	219.2 m (Apr 2012)/218.72 (Nov 2011)
High Water Level Elevation (50 year)	220.78 m

Construction Staging & Detours

For the proposed bridge replacement option a local road detour is not anticipated. The existing bridge would remain operational with reduced lane widths while the new bridge is constructed. In order to accommodate pile driving activities at the pier location part of the existing bridge deck will need to be removed.

For the proposed bridge replacement, the anticipated excavation and removal of the soft organic silt (muck) immediately north of Highway 9 (east of Holland Drainage Canal) to accommodate the embankment widening is expected to impact on the performance of the existing highway embankment during construction if open excavation methods were used. Therefore, temporary roadway protection system will be required.

It is noted that unwatering of excavations below the canal level is anticipated at this site.

7.2 GEOTECHNICAL DESIGN PARAMETERS

For the proposed replacement bridge, the soil conditions on the east and west sides of the existing Holland Drainage Canal show some variation up to approximate elevation of 215 m. West of the canal, the observed subsurface condition consisted of topsoil over sand over a silty sand over a deep deposit of clayey silt underlain by a silty sand deposit. East of the canal, the encountered subsurface condition generally consisted of a layer of topsoil and/or fill over organic silt (muck) over clayey silt over silty sand. Representative strata plots are indicated on Drawing No. 2 and 3 in Appendix A.

For the purpose of the proposed replacement bridge, the soil conditions east of the canal will be considered representative.

Within the embankment widening area extending from the canal easterly approximately 450 m, the soils within the embankment fill generally consist of a gravelly sand layer over a sandy silt to silty sand.

For design purposes, the soil model provided in Table 7.1 will be used for the replacement bridge structure foundation.

Table 7.1: Geotechnical Model

Elevation (m)		Soil Type	Design Properties
From	To		
East of Holland Drainage Canal			
220.0	215.5	Organic silt (OL) (Muck), soft	γ = 18.2 kN/m^3 ϕ = 15°
215.5	213.5	Silt (ML) loose	γ = 20.2 kN/m^3 ϕ = 26º E' = 5 MPa
213.5	212.0	Clayey silt (CL), stiff	see Figure 12a
212.0	211.0	Silty sand (SM), loose to compact	γ = 20.6 kN/m^3 ϕ = 30° E' = 25 MPa
211.0	194.0	Clayey silt (CL), stiff to very stiff	see Figure 12a
<194.0		Silty sand (SM), compact	γ = 21.2 kN/m^3 ϕ = 35°
West of Holland Drainage Canal			
220.0	217.0	Sand (SP), loose	γ = 20 kN/m^3 ϕ = 30° E = 15 MPa
217.0	215.5	Silty sand (SM), compact	γ = 20.2 kN/m^3 ϕ = 30º E' = 25 MPa
215.5	213.5	Silt (ML) loose	γ = 20.2 kN/m^3 ϕ = 26º E' = 5 MPa
213.5	212.0	Clayey silt (CL), stiff	see Figure 12a
212.0	211.0	Silty sand (SM), loose to compact	γ = 20.6 kN/m^3 ϕ = 30° E' = 25 MPa
211.0	194.0	Clayey silt (CL), stiff to very stiff	see Figure 12a
<194.0		Silty sand (SM), compact	γ = 21.2 kN/m^3 ϕ = 35°

Notes:

(1) Significant difference in the subsurface conditions were observed between east and west sides of the Holland Drainage Canal only above approximate elevation of 215.5 m. Below this elevation, the subsurface conditions were fairly consistent across the site. For modeling purposes, identical design parameters were used below elevation of 215.5 m for both sides of the canal.

(2) The layering directly beneath the organic silt (east of the canal) varies from borehole to borehole. The above profile between elevation 215.5 m and 211.0 m is considered to be a conservative approximation of the site conditions;

(3) A design groundwater elevation of 219.2 m will be used; and

(4) γ = total unit weight, ϕ = Friction Angle, S_u = Undrained Shear Strength

7.3 FROST PROTECTION

The design frost penetration depth for foundations, f, at the site is 1.5 m based on OPSD 3090.101. Therefore, footings and pile caps should be provided with a minimum of 1.5 m of soil cover or equivalent insulation for protection against frost heaving.

Where construction is undertaken during winter, footing subgrades must be protected from freezing. Due diligence is required to ensure that granular fill materials do not include frozen material, snow or ice.

7.4 SEISMIC DESIGN CONSIDERATIONS

7.4.1 Soil Profile Type

It is recommended that a Soil Profile II as defined in CHBDC (CHBDC, 2006) Section 4.4.6 be used in the seismic design of this site. This soil profile reflects that bedrock is generally anticipated to be deeper than 60 m in the Holland Marsh area.

7.4.2 Zonal Acceleration Ratio

Table A3.1.1 of the CHBDC indicates that the Zonal Acceleration Ratio (ZAR) for Newmarket, Ontario, which is approximately 15 km northeast of the site, is 0.05.

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

7.4.3 Liquefaction Potential

The loose silts which frequently underly the organic silt would be considered liquefiable in areas with higher design peak ground acceleration (PGA) ratios. The PGA value for a 10% probability of exceedance in 50 years at this site is estimated to be 0.02; therefore, the risk of soil liquefaction is not considered to be significant for this project.

7.5 FOUNDATION OPTIONS

7.5.1 General

West of the Holland Canal, the prevailing subsurface soil at the site includes a shallow deposit of loose to compact silty sand overlying a deep deposit of stiff to very stiff clayey silt.

East of the Holland Canal, the prevailing subsurface soils include organic silt (muck) overlying a deep deposit of stiff to very stiff clayey silt.

Due to the prevalence of loose subsurface conditions in the top 5 m from the ground surface, shallow (spread footing) foundations are not considered suitable for the proposed replacement structure. Deep foundations terminating within the very stiff clayey silt deposit are recommended for supporting the bridge abutments and the centre pier.

Table 7.2 compares deep foundation options from a foundation design and constructability perspective.

Table 7.2: Comparison of Deep Foundation Options for Replacement Bridge

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
H-Piles **Frictional Pile**	▪ Minimize disturbance to the clayey silt (non-displacement) ▪ Commonly used for integral abutment bridges ▪ Will penetrate harder layers with relative ease	▪ Cannot be internally inspected	Medium	Non-displacement piles are at greater risk of enabling artesian flow at the pile perimeter. This would impact the pile capacities.
Closed End Pipe Piles filled with Concrete **Frictional Pile**	▪ Can be internally inspected for alignment and damage ▪ Displacement pile will create a barrier to artesian flow along the perimeter of the pile.	▪ Significantly stiffer than H-Pile and therefore may not be suitable for integral abutments ▪ Will pose more difficulty driving ▪ More risk of overdriving /damaging of piles	Medium	▪ Risk of refusal within the hard clay ▪ Risk of pile damage during installation would require additional piles
Drilled Caissons	▪ Very high axial capacity	▪ Would require a liner due to presence of less stiff clayey silt layers ▪ Very stiff cross-section may not be suitable for integral abutment	High	▪ Risk of cave-in below groundwater ▪ Dewatering may become an issue; Tremie concrete methods could resolve this issue ▪ Artesian flow observed within the clayey silt renders this a high risk option.

It is recommended that pipe piles be carried forward as the preferred deep foundation type for this project. It is recommended that the frictional piles terminate within the stiff to very stiff clayey silt to avoid puncturing the sandy artesian layer encountered at approximate elevation of 194.0 m.

The following foundation recommendations are provided for a frictional pile terminating within the stiff to very stiff (hard) clayey silt.

7.5.2 Deep Foundations

7.5.2.1 General

The design recommendations presented in this section have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2006). Pile foundations consisting of steel pipe piles are recommended to support the proposed abutments and the centre pier of the bridge. Based on the preliminary general arrangement drawing, it is anticipated that the underside of the pile caps will be at approximate elevation of 217.5 m. The piles would extend approximately up to 23 m to approximate elevation of 195.0 m. This minimum elevation was determined based upon avoiding the sandy aquifer during pile installation and hence minimizing the risk of artesian pressures.

7.5.2.2 Axial Pile Resistance in Compression

The axial pile resistance at ULS in compression for 324 mm O.D. x 63 mm wall thickness, 356 mm O.D. x 6.3 mm wall thickness, 406 mm O.D. x 6.3 mm wall thickness, and 508 mm O.D. x 11.13 mm wall thickness pipe piles was assessed using the American Petroleum Institute (API) design method using the program APile developed by Ensoft (Ensoft, 2007) and the geotechnical model presented in Table 7.1.

Ontario Ministry of Transportation Report EM-48 (Rev. 93) Pile Load and Extraction Tests 1954-1992 Pile Test Site 26 data, which is located 5 km from the project site, encountered a similar clayey silt deposit. The test data was used to confirm the suitability of the soil model and the API design method.

Figure 13a in Appendix D provides a profile of geotechnical axial resistance at ULS in compression for the above noted pile sizes and includes a resistance factor of 0.4 applied to the calculated ultimate capacity. For example, using Figure 13a, a 356 mm O.D. x 6.3 mm wall thickness pipe pile driven to 22 m would have a geotechnical resistance at ULS of 500 kN. Our analysis indicates that this factored geotechnical resistance is reached prior to undergoing 25 mm of pile top settlement. Hence, an SLS geotechnical reaction of 500 kN is appropriate for this site.

Drag loads are not considered applicable for this project due to the following:

- The total estimated settlement at the new abutment locations due to the embankment fill is less than 20 mm (see Section 7.7.2).
- The road embankment is anticipated to be constructed prior to pile driving. The embankment settlement anticipated at the abutment locations within the overconsolidated clayey silt is expected to occur prior to pile driving activities commencing.

7.5.2.3 Geotechnical Lateral Resistance of Piles

The geotechnical resistance of the pile against lateral loads is mobilized due to the passive resistance of the surrounding soil. The passive lateral resistance for vertical piles can be calculated according to Sections C6.8.7.1 and C6.8.7.2 of the CHBDC, 2006.

7.5.2.4 ULS Lateral Resistance

The following ULS lateral resistances may be used.

Pile Outside Diameter	ULS Lateral Resistance
324 mm x 6.3 mm	120 kN
356 mm x 6.3 mm	130 kN
406 mm x 6.3 mm	150 kN
508 mm x 11.13 mm	190 kN

The above ULS lateral resistances include a resistance factor of 0.5 applied against the ultimate calculated resistance.

7.5.2.5 SLS Lateral Resistance

The lateral capacity of piles was evaluated using the program called LPile Plus v5.0 developed by Ensoft, Inc. (Ensoft, 2004). The input parameters are given in Table 7.3. A 508 mm O.D. pipe pile with an 11.13 mm wall thickness was evaluated to develop the soil response spring stiffness. A modulus of elasticity of 200 GPa was used for the pile material (steel). The pile was modelled with a total length of 23 m and terminating within the very stiff clayey silt. The p-y modulus values were based on values suggested by Ensoft, Inc. (Ensoft, 2004).

The resistance can be calculated with the unfactored soil parameters presented in Table 7.3.

Table 7.3: Recommended Parameters for Lateral Pile Capacity Evaluation

Soil Layer	Depth Range (m)		Unit weight, γ	Friction angle (ϕ) or Undrained Shear strength (S_u)	p-y Modulus, k
	From	To	kN/m^3		kN/m^3
SSM	220	215.5	20.5	32°	6,000
Loose Silt (ML)	215.5	213.5	20.2	26°	7,180
Stiff Clayey Silt (CL)	213.5	212.0	20.6	60 KPa	4,285
Loose to Compact Silty Sand (SM)	212.0	211.0	20.6	30°	19,500
Stiff Clayey Silt (CL)	211.0	204.0	20.6	60 kPa	4,285
Very Stiff Clayey Silt (CL)	204.0	194.0	20.6	120-150 kPa	10,500

Notes:

(1) Base of pile cap was assumed at elevation of 217.5 m.
(2) The entire pile is assumed to be entirely beneath the groundwater level. Hence, the soil k-values provided in Table 7.3 is applicable for submerged conditions.

Two plots from LPILE analysis are presented in Figures 14a and 14b in Appendix D. Figure 14a shows the deformed shape of the pile for lateral force (shear) ranging from 150 to 250 kN. This plot indicates that for the conditions modeled the pile will undergo negligible lateral deflection below a depth of approximately 6.0 m from the underside of the pile cap.

Figure 14a illustrates the displacement of the pile with depth for different lateral loads. According to this figure, a lateral load of 80 kN would correspond to a pile top displacement of 10 mm. Therefore, the SLS geotechnical resistance of a 508 mm outside diameter pipe pile with a wall thickness of 11.13 mm would be estimated to be 200 kN.

Figure 14b presents the p-y plot that gives the non-linear response of the pile-soil interaction. It provides a series of curves obtained from LPILE generated for selected depths below the pile head. These plots can be used in the structural evaluation of the proposed bridge founded on pipe piles. The interpreted Linear Spring Constants, k from Figure 14b are summarized in Table 7.3.

7.5.2.6 Group Action

Group action of piles (pile interaction) for lateral loading should be considered if centerline spacing of piles is less than 8 pile diameters (or least lateral dimension of pile) parallel to the direction of lateral load, or less than 4 pile diameters, perpendicular to the load. The effect of interaction between piles can be considered by applying a reduction factor to the coefficient of lateral subgrade reaction (p-y modulus). The following reduction factors may be used to account for pile group action:

Table 7.4: Recommended Reduction Factors for Pile Groups

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

7.5.2.7 Axial Pile Resistance in Tension

The geotechnical axial resistances at ULS in tension for the selected pipe pile diameters are provided on Figure 13b in Appendix D; these resistances include a resistance factor of 0.3 applied against the ultimate calculated capacity.

For example, a 356 mm O.D. x 6.3 mm wall thickness steel pipe pile driven to 22 m would have a geotechnical axial resistance at ULS in tension of 330 kN.

7.6 LATERAL EARTH PRESSURES

7.6.1 Backfill

Earth pressures will need to be considered in the design of abutments, retaining walls and roadway protection systems. The bridge abutments should be backfilled with granular material in accordance with OPSD 3101.150. The Granular backfill should consist of OPSS Granular A.

Computation of earth pressures should be in accordance with Section 6.9 of the Canadian CHBDC. For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used.

7.6.2 Lateral Earth Pressures under Static Conditions

For static conditions, the unfactored soil parameters provided in Table 7.5 may be used for design of walls with a horizontal backfill and those provided in Table 7.6 for walls with a 2H:1V 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), at rest (P_o) and passive (P_P) thrusts under static loading conditions can be calculated using the following equations:

$$P_A = ½ K_a \gamma H^2$$

$$P_O = ½ K_o \gamma H^2$$

$$P_P = ½ K_p \gamma H^2$$

Where H is the height of the wall. Values for K_a, K_o, K_p and γ are provided in Tables 7.5 and 7.6 below. The thrust acts at a point one third up the height of the wall.

Table 7.5: Recommended Non-Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill	Sandy Silt to Silty Sand and SSM
Bulk Unit Weight, γ (kN/m^3)	21.2	22.0	19.0	21
Effective Friction Angle	32º	35º	30º	32°
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43	0.50	0.47
Coefficient of Active Earth Pressure (K_a)	0.31	0.27	0.33	0.31
Coefficient of Passive Earth Pressure (K_p)	3.25	3.69	3.00	3.25

Table 7.6: Recommended Non-Seismic Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill	Sandy Silt to Silty Sand and SSM
Bulk Unit Weight, γ (kN/m^3)	21.2	22.0	19.0	21
Effective Friction Angle	32º	35º	30º	32°
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43	0.50	0.47
Coefficient of Active Earth Pressure (K_a)	0.47	0.39	0.54	0.47
Coefficient of Passive Earth Pressure (K_p)	8.62	10.84	7.84	8.62

7.6.3 Lateral Earth Pressures under Seismic Conditions

The abutments and the retaining walls should be designed to resist the earth pressures induced under seismic loading conditions. The seismic earth pressures may be calculated using the parameters detailed in Tables 7.7 and 7.8 below.

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

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

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

where:

- K_{AE} = active earth pressure coefficient (combined static and seismic)
- K_{PE} = passive earth pressure coefficient (combined static and seismic)
- H = height of wall
- k_h = horizontal acceleration coefficient
- k_v = vertical acceleration coefficient
- γ = total unit weight

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

Zonal Acceleration Ratio, A or PGA	0.05	
Horizontal Acceleration Coefficient, k_h	0.025 yielding	0.075 non-yielding
Vertical Acceleration Coefficient, kv	0.017 yielding	0.05 non-yielding
Horizontal Backslope to wall		
Vertical back of wall		

The above k_h value corresponds to ½ of the A value for yielding walls and 1.5 times for non-yielding walls. The k_v value corresponds to 0.67 of the k_h value. The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 7.7: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran B Type I		OPSS Gran A and Gran B Type II		Existing Embankment Fill		Sandy Silt to Silty Sand and SSM	
Bulk Unit Weight, γ (kN/m^3)	21.2		22.0		19.0		21	
Effective Friction Angle	32º		35º		30º		32°	
	Yielding wall	Non-yielding	Yielding wall	Non-yielding	Yielding wall	Non-yielding	Yielding wall	Non-yielding
Active Earth Pressure (K_{AE})	0.32	0.35	0.28	0.31	0.35	0.38	0.32	0.31
Height of Application of P_{AE} from base as a ratio of wall height, (H)	0.341	0.356	0.342	0.358	0.340	0.355	0.341	0.356
Passive Earth Pressure, (K_{PE})	3.24	3.77	3.64	3.54	2.96	2.84	3.21	3.11
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.341	0.306	0.342	0.307	0.340	0.305	0.341	0.306

Table 7.8: Recommended Seismic Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Gran B Type I		OPSS Gran A and Gran B Type II		Existing Embankment Fill		Sandy Silt to Silty Sand and SSM	
Bulk Unit Weight, γ (kN/m^3)	21.2		22.0		19.0		21	
Effective Friction Angle	32º		35º		30º		32°	
	Yielding wall	Non-yielding	Yielding wall	Non-yielding	Yielding wall	Non-yielding	Yielding wall	Non-yielding
Active Earth Pressure (K_{AE})	0.51	0.66	0.43	0.51	0.60	Note 1	0.51	0.66
Height of Application of P_{AE} from base as a ratio of wall height, (H)	0.352	0.400	0.349	0.384	0.356	Note 1	0.352	0.400
Passive Earth Pressure, (K_{PE})	8.57	8.45	10.77	10.64	7.42	7.32	8.57	8.45
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.327	0.313	0.327	0.314	0.327	0.313	0.327	0.313

Note 1 Under seismic conditions these materials are not suitable for retaining wall backslopes constructed at 2H:1V. Either flatter backslopes or the use of OPSS Granular B Type II or Granular A would be required.

7.7 EMBANKMENT DESIGN

7.7.1 Stability of Slopes

A slope stability evaluation was carried out using commercially available limit equilibrium based software called SLOPE/W (GEO-SLOPE, 2007). The analysis included dynamic loading due to traffic by considering an equivalent static load equivalent to 0.8 m of additional fill, as per Section 6.9.5 of the CHBDC. The analysis also considered seismic loading using one-half of the ZAR.

It is assumed that the muck in the vicinity of the east bridge abutment will be removed to a depth of approximately 4.2 m (elevation of 215.8 m) and replaced with Select Subgrade Material (SSM). The geometric configuration of the organic silt removal should be similar to that shown in OPSD 203.030 Embankments Over Swamps; copy included in Appendix E.

Static (long- and short-term) and seismic slope stability analysis results for Select Subgrade Material (SSM) are presented in Figures 15a through 15c.

A 2H:1V slope is required for embankments constructed of SSM under the conditions modeled herein.

7.7.2 Evaluation of Potential Ground Settlement due to Embankment

Settlement of the underlying soil due to the embankment was evaluated. The following assumptions were made in evaluating the settlement of the site soil under the proposed embankments:

- Typical soil profile given Table 7.1 was considered representative for the respective east and west sides of the canal;

- The load from the bridge abutment (and the center pier) will be transferred to deeper and more competent strata by the piles and hence does not contribute to the settlement of the site soil;
- The design consolidation parameters for the clayey silt summarized on Figures 12a and 12b in Appendix D were used for calculation of the settlement;
- The embankment fill height corresponds to elevation 222.2 m representing embankment height of approximately 2 m; and
- Settlement estimates at existing structure (i.e., at east and west abutment locations) were carried out for two different offsets of the proposed east and west abutments with respect to the existing abutments:
  - Proposed east abutment – 16 and 12 m offsets east of existing;
  - Proposed west abutment – 7.5 and 6 m west of existing.

Evaluation of soil settlement due to the effects discussed above was performed using a computer program called Settle3D (Rocscience, 2009). It is a three-dimensional computer program for the analysis of the immediate vertical settlement and consolidation of soil under surface loads such as embankments. Settlement evaluation was carried out for embankments constructed using Select Subgrade Material (SSM) with 2H:1V slopes constructed as per OPSD 203.030.

Figures 16a and 16b represent the settlements anticipated east of the canal as a result of the roadway embankment construction for the east abutment offsets of 16 m and 12 m, respectively, east of the existing east abutment.

Figures 17a and 17b represent the settlements anticipated west of the canal as a result of the roadway embankment construction for the west abutment offsets of 7.5 m and 6 m, respectively, west of the existing west abutment.

The following table summarizes the anticipated critical settlements.

Table 7.9: Summary of Estimated Settlements at Existing East and West Abutments

Location	New East Embankment		New West Embankment	
	Offset=16 m East	Offset=12 m East	Offset=7.5 m West	Offset=6 m West
Existing East Abutment	<1 mm	< 1 mm	Negligible	Negligible
Existing West Abutment	Negligible	Negligible	<2 mm	<2 mm

The respective maximum settlements caused by the east and west embankments are 34 mm and 40 mm. The corresponding maximum settlements at the proposed abutments are 17 mm and 20 mm, respectively, at the new east and west abutments. The critical location of predicted settlements is at the existing bridge abutments. As shown in Table 7.9 above, the proposed bridge replacement configuration and the abutment offsets will satisfy the requirements of minimizing settlement at the existing bridge east and west abutments; a requirement driven by the poor condition of the existing timber piles.

It is noted that the settlement within the clayey silt strata is within the recompression portion of the consolidation model; i.e., the new overburden stresses will not exceed historic geological pressures. Therefore, settlements are anticipated to occur within a few weeks of embankment construction.

It is noted that there will also be a minor amount of self-weight settlement of the embankment fill. This self-weight settlement was estimated using charts provided by Poulos and Davis (1974) for embankments having similar geometries to the SSM embankments presented herein. The estimated self-weight settlement was approximately 10 to 15 mm, for the SSM embankment fill. This settlement is also expected to be completed by the end of construction.

No settlement monitoring is recommended for this project.

7.8 CEMENT TYPE AND CORROSION POTENTIAL

Nine samples of the native soil at different locations across the site were tested for pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in Table 4.2.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate concentrations for the samples tested ranged between 10 and 297 µg/g. 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. 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 values were between 6.9 and 8.4 which are 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. The test results provided in Table 4.2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

8.0 Construction Considerations

8.1 CONSTRUCTION STAGING

Construction staging is not anticipated for the proposed bridge replacement; ie., the traffic will be maintained on the appropriate detour routes throughout the construction period.

Construction of the approach embankments and the roadway widening, which will extend to approximately 450 m east of the east abutment and will require organic silt removal similar to OPSD 203.030. Open cut excavation will not be possible and temporary roadway protection will be required.

8.2 EXCAVATION AND BACKFILLING

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

Excavation and removal of muck should be carried out in accordance with OPSD 203.030.

All vegetation, fill, organic soils and other deleterious materials must be removed from beneath the proposed bridge foundation pile caps. Where deleterious materials are encountered, the material should be excavated, wasted and replaced.

Side slopes for open cut excavations should conform to the Occupational Health and Safety Act (OHSA) regulations for Construction Projects. The organic silt and the very loose to loose underlying silt should be considered a Type 4 soil. The remaining soils encountered in the boreholes may be considered a Type 3 Soil in accordance with the OHSA.

Grading work should be carried out in accordance with OPSS 206 Construction Specification for Grading and SP 206S03.

It is noted that excavated material containing organics such as organic silt (muck) encountered at the site will not be suitable as backfill behind walls or for embankment construction.

8.3 ROADWAY PROTECTION SYSTEM

Construction of the approach embankment fills along the proposed new alignment will require the removal of soft organic silt (muck) in the vicinity of the bridge. The extent of the muck removal is anticipated to be approximately 450 m to accommodate the anticipated length of embankment widening. This will involve excavations in the vicinity of the existing approach fills. It is understood that two traffic lanes are required to be open during construction. Hence, a temporary roadway protection is required to reduce any adverse impacts on the existing traffic during construction.

A conceptual drawing showing the anticipated location of the roadway protection is provided in Appendix E. It is anticipated that cantilevered sheet piles will be required to support the roadway while organic silt removal is being carried out. Although organic silt was not encountered within the boreholes drilled through the existing road embankment, it is anticipated that organic silt removal will extend to between 2.0 and 4.0 m below the road profile; this height can generally be supported by cantilevered sheet piles. The contractor will be responsible for designing his proposed support system.

Computation of earth pressures should be in accordance with Section 6.9 of the CHBDC. For roadway protection with a horizontal backfill, the unfactored soil parameters provided in Table 7.5 may be used for design.

The total active (P_A) and passive (P_P) thrusts can be calculated using the following equations.

$P_A = ½ K_a \gamma H^2$

$P_P = ½ K_p \gamma H^2$

Where H is the height of the wall and the values for K_a, K_p, and γ are provided in Table 7.5. The thrust typically acts at a point one third up the height of the wall, however, roadway protection types and materials will dictate the actual pressure distribution.

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.

Roadway protection 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 construction process as described in OPSS 539. The monitoring requirements outlined in OPSS 539 are considered to be appropriate for this project.

8.4 TEMPORARY CONSTRUCTION UNWATERING

It is understood that the abutment excavations are going to extend below the water level in the Holland Drainage Canal. Construction will require removal of the organic silt to elevation 215.5 m or about 3 m below the canal water level.

Control of the water flow in the canal will require a sheet pile cofferdam driven into the clayey silt to prevent flow into the excavations. A Dewatering NSSP should be included in the contract to cover the possibility that the groundwater level increases during construction to the point where some dewatering effort is required to provide a stable earth platform for construction purposes. A copy of the NSSP is provided in Appendix E.

The clayey silt into which the sheet piles will be driven to form a cofferdam has a low coefficient of permeability estimated to be less than 10^{-8} m/sec. Provided the sheet piles are embedded within the clayey silt layer unwatering within cofferdams should be achieved using conventional sumps and pumps from within.

8.5 EROSION AND SCOUR PROTECTION

Scour protection will be required to ensure the long-term surficial stability of the embankment slopes and adjacent canal banks. All slopes within 3 m of upstream and downstream of the bridge should be surfaced with rip-rap at least 300 mm thick placed on a Class II non-woven filter fabric. The rip-rap should extend up the slopes to 0.3 m above the design high water level.

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 PILE INSTALLATION

It is essential that the compatibility of the pile driving equipment, the soil conditions, and the pile type being driven is properly accounted for in order to achieve the required pile penetration and a satisfactory pile foundation.

Piles shall have reinforced tips according to Ontario Provincial Detail OPSD 3001.100 Type I.

The pile driving equipment shall be appropriate to the driving conditions and capable of delivering a minimum specified energy of 50kJ.

It is recommended that pile driving note 2 be used in the contract drawings. “Piles to be driven to el. 195 m”.

9.0 Specifications

The following specifications are referenced in this report:

Table 9.1: Specifications Referenced in Report

Document	Title
OPSD 203.010	Embankments over Swamp – New Construction
OPSD 203.030	Embankments over Swamps
OPSD 3001.100	Foundation Piles Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirements
OPSS 206	Construction Specification for Grading
OPSS 517	Construction Specification for Dewatering of Pipeline, Utility and Associated Structure
OPSS 518	Construction Specification for Control of Water from Dewatering Operations
OPSS 539	Construction Specification for Temporary Protection System
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
SP 206S03	Earth Excavation, Grading
SS 103-11	MTO Structural Manual

10.0 References

ASTM. 1999. Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). ASTM International, West Conshohocken, PA.

ASTM, 2007. Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils (ASTM D5778). ASTM International, West Conshohocken, PA.

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

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

Ensoft, 2004. User’s Manual for Computer Program LPILE Plus Version 5.0. Ensoft, Inc., Austin, Texas.

Ensoft, 2007. Computer Program APILE Plus Version 5.0. Ensoft, Inc., Austin, Texas.

Geo-Slope International, Ltd. 2010. GeoStudio 2007 (Slope/W 2007), Calgary, Alberta.

Report EM-48 (Rev. 93). Pile Load and Extraction Tests 1954-1992 (MTO).

11.0 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 was prepared by Simon Gudina and reviewed by Chris McGrath and Raymond Haché.

Respectfully submitted,

STANTEC CONSULTING LTD.

LICENSED PROFESSIONAL ENGINEER
SG
S. K. GUDINA
100162564
Jan 14, 2013
PROVINCE OF ONTARIO

S Gudina

Simon Gudina, Ph.D., P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
CM
C. G. MCGRATH
100108606
Jan 14, 2013
PROVINCE OF ONTARIO

Chris McGrath, P.Eng.
Associate, Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
17713504
Jan. 14, 2013
PROVINCE OF ONTARIO

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

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APPENDIX A

Drawing No. 1 through 3 – Borehole Location Plan and Soil Strata Plot

Site Photos

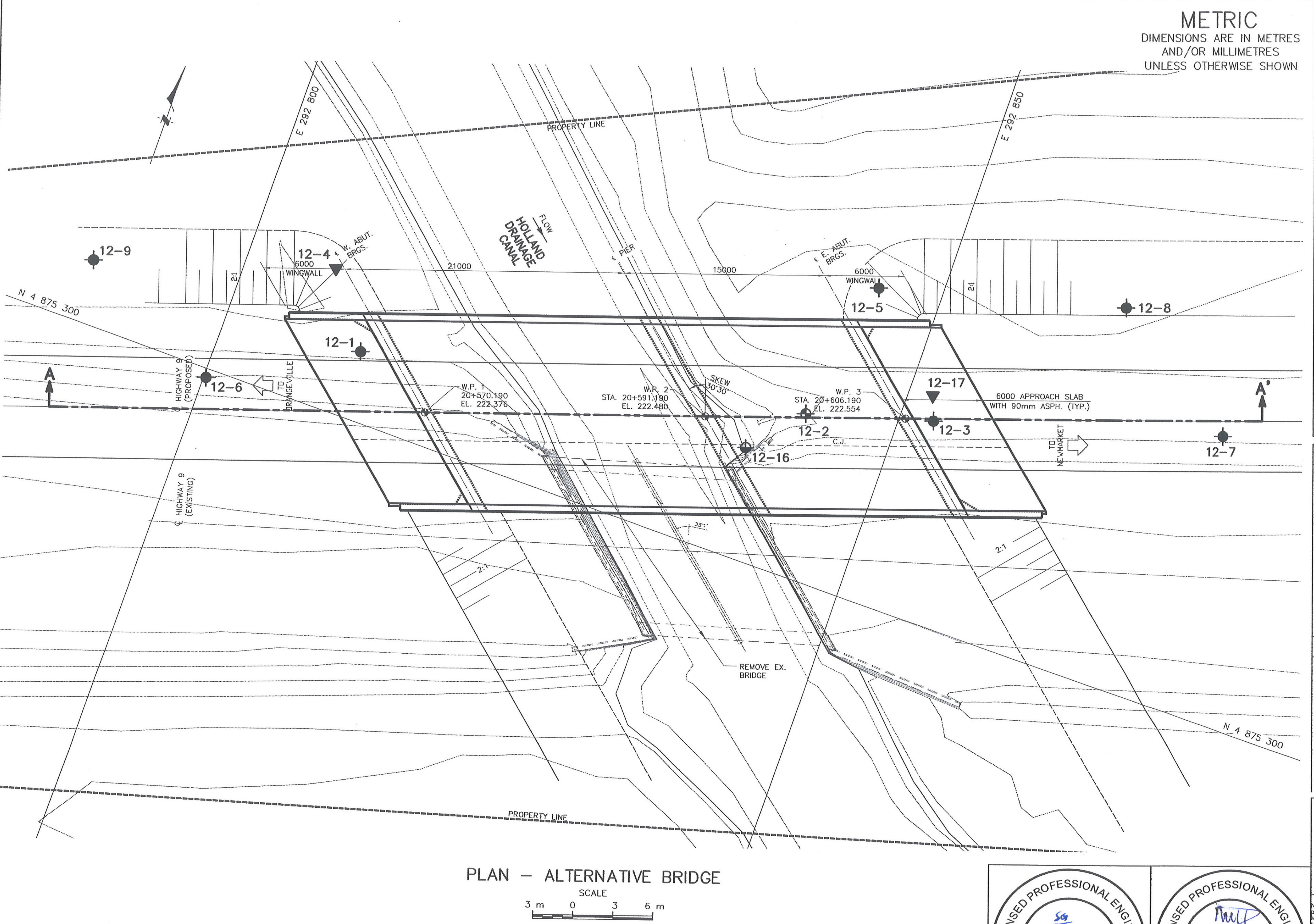
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No
CONT
WP 2188-08-00

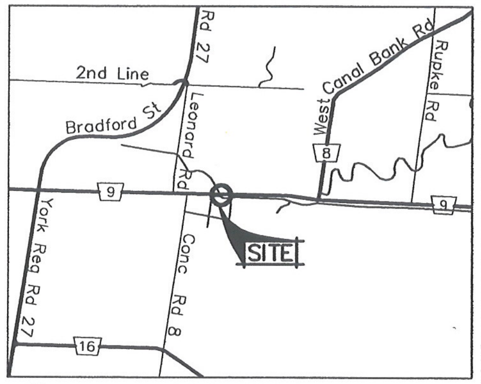
HOLLAND DRAINAGE CANAL BRIDGE BOREHOLE LOCATIONS

SHEET

Stantec



PLAN – ALTERNATIVE BRIDGE



KEY PLAN
1 km 0 1 2 km

LEGEND

⊕	Borehole
⊕	Borehole and Cone
⊕	Auger Hole
▼	Cone Penetration Test
N	Blows/0.3m (Std Pen Test, 475 J/blow)
▽	Inferred WL at time of investigation April 2012
▼	WL at Time of Investigation, April 18, 2012
(m NORTH)	Offset from Cross Section Line (m)

No	ELEVATION	MTM ZONE 10 COORDINATES NORTH	EAST
12-1	219.7	4 875 305.0	292 810.3
12-2	219.9	4 875 312.0	292 843.3
12-3	220.1	4 875 314.6	292 852.5
12-4	220.3	4 875 310.2	292 806.5
12-5	221.8	4 875 322.5	292 845.4
12-6	219.8	4 875 299.4	292 800.2
12-7	220.4	4 875 320.8	292 873.3
12-8	221.2	4 875 327.3	292 863.2
12-9	219.9	4 875 304.7	292 789.3
12-16	219.0	4 875 308.1	292 839.9
12-17	220.2	4 875 316.4	292 851.8

NOTES

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

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

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

LICENSED PROFESSIONAL ENGINEER
SG
S. K. GUDINA
Dec. 14, 2012
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
2013/01/14
PROVINCE OF ONTARIO

REVISIONS DATE	BY	DESCRIPTION

GEOCRES No 31D-553

HWY No 9			DIST
SUBM'D SKG	CHECKED	DATE 2012-12-12	SITE 37-30
DRAWN GBB	CHECKED	APPROVED SG	DWG 1

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

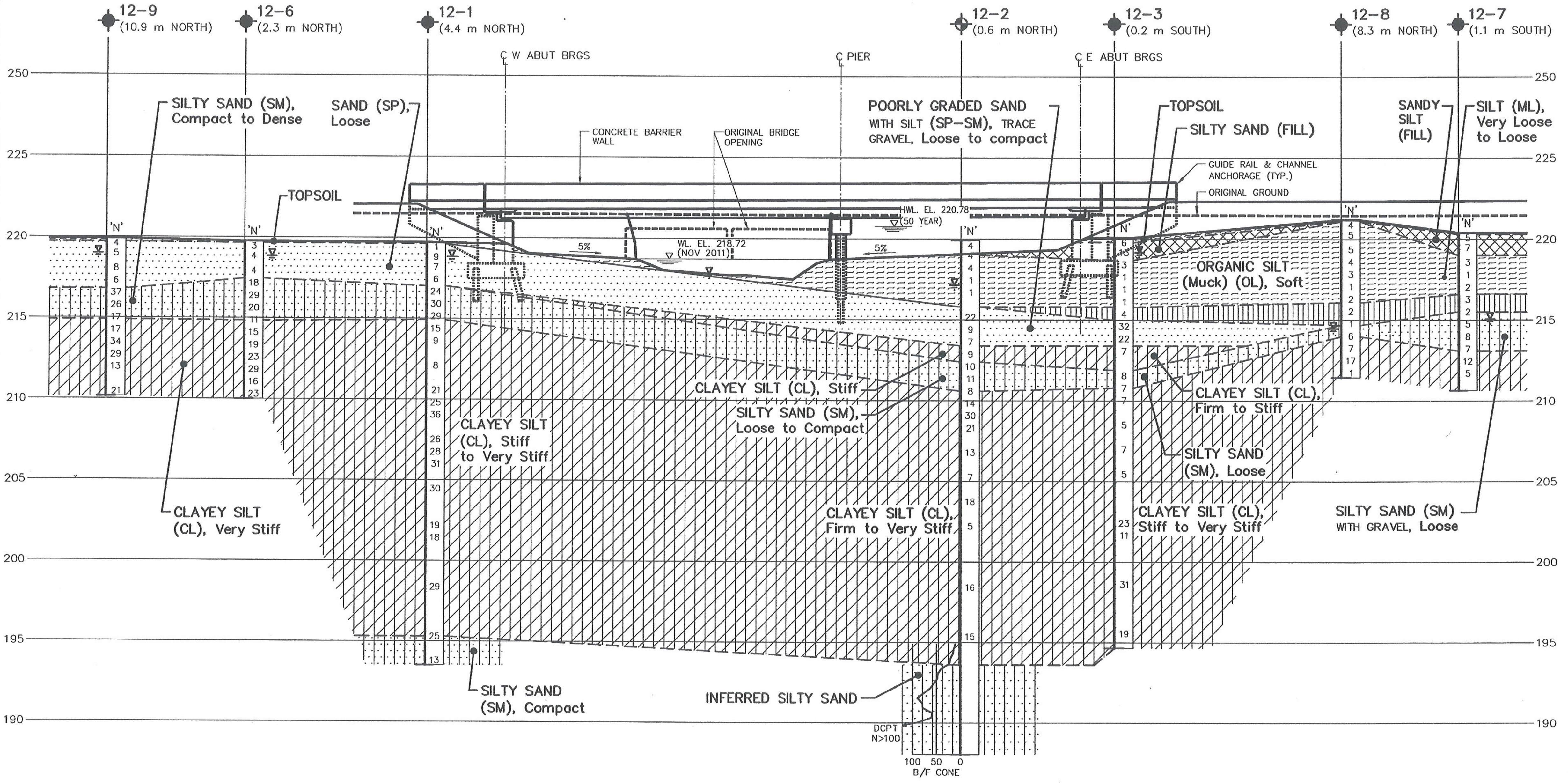
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No
CONT
WP 2188-08-00

SHEET

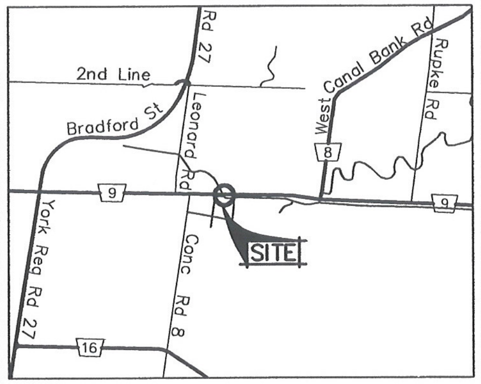
HOLLAND DRAINAGE CANAL BRIDGE CROSS SECTION

Stantec



CROSS SECTION A-A'

SCALE
3 m 0 3 6 m



KEY PLAN
1 km 0 1 2 km

LEGEND

●	Borehole
⊕	Borehole and Cone
⊕	Auger Hole
▼	Cone Penetration Test
N	Blows/0.3m (Std Pen Test, 475 J/blow)
▽	Inferred WL at time of investigation April 2012
▼	WL at Time of Investigation, April 18, 2012
(m NORTH)	Offset from Cross Section Line (m)

No	ELEVATION	MTM ZONE 10 COORDINATES NORTH	EAST
12-1	219.7	4 875 305.0	292 810.3
12-2	219.9	4 875 312.0	292 843.3
12-3	220.1	4 875 314.6	292 852.5
12-4	220.3	4 875 310.2	292 806.5
12-6	219.8	4 875 299.4	292 800.2
12-7	220.4	4 875 320.8	292 873.3
12-8	221.2	4 875 327.3	292 863.2
12-9	219.9	4 875 304.7	292 789.3
12-16	219.0	4 875 308.1	292 839.9
12-17	220.2	4 875 316.4	292 851.8

NOTES

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

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S. K. GUDINA
Dec. 14, 2012

J.G.A.R. HACHÉ
2013/01/14

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No 31D-553			
HWY No 9			DIST
SUBM'D SKG	CHECKED	DATE 2012-12-12	SITE 37-30
DRAWN GBB	CHECKED	APPROVED SG	DWG 2

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

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PLATE No

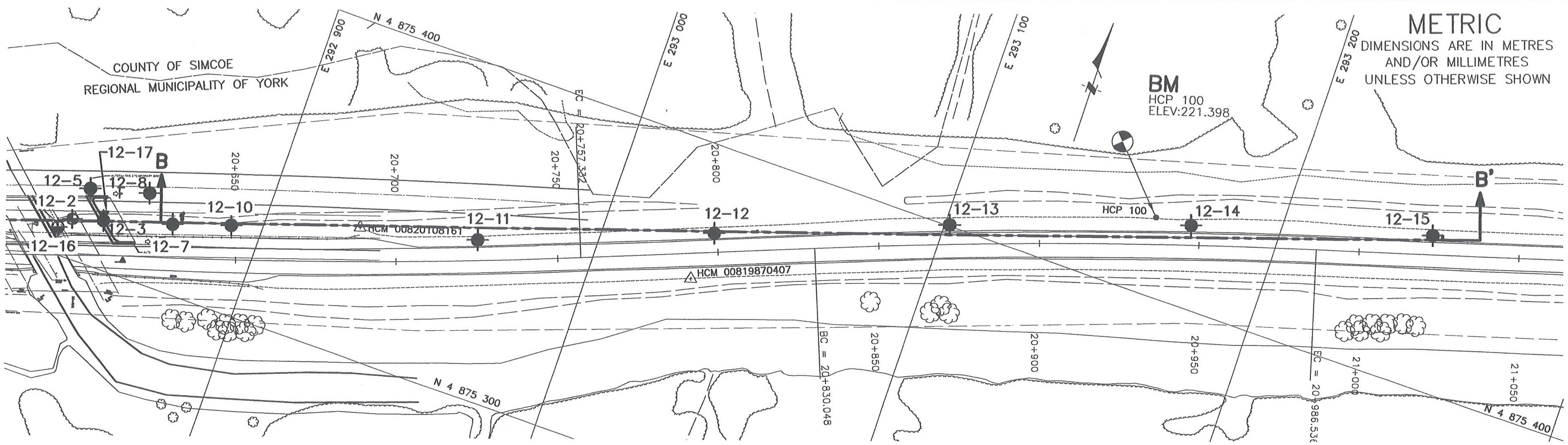
CONT
WP 2188-08-00

SHEET

HOLLAND DRAINAGE CANAL BRIDGE
BOREHOLE LOCATIONS & SOIL STRATA

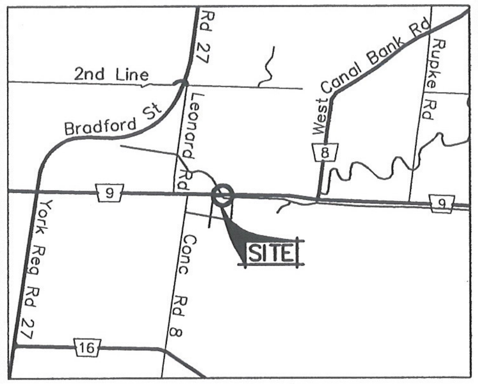
Stantec

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



PLAN

SCALE
15 m 0 15 30 m

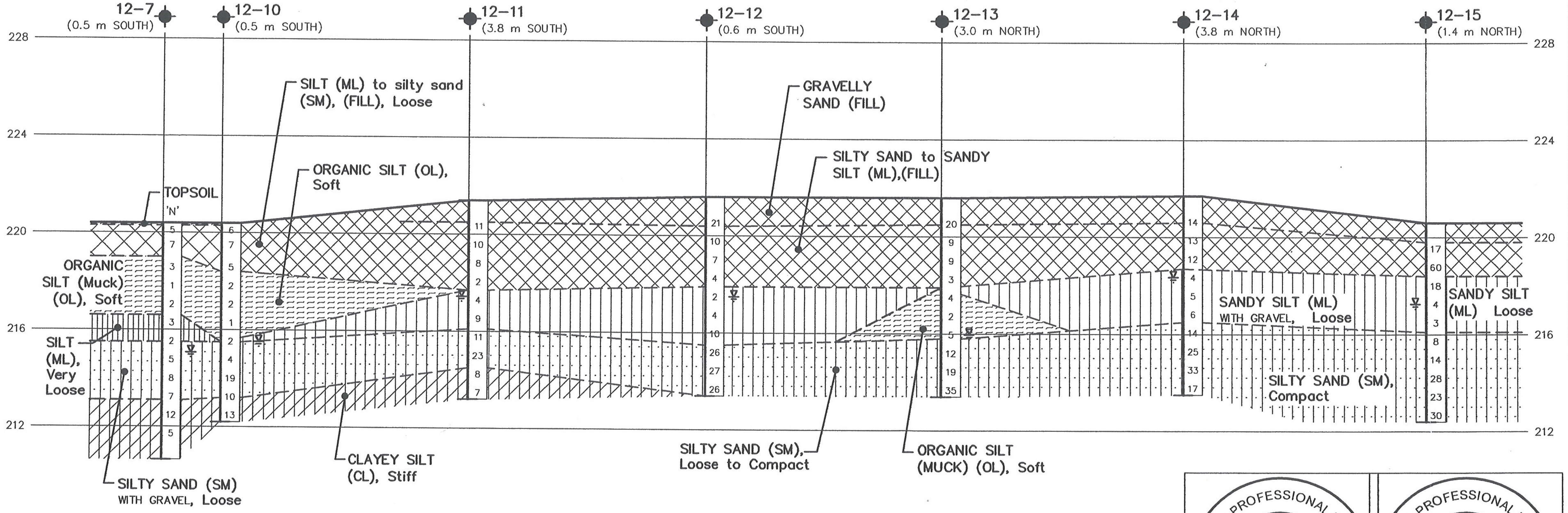


KEY PLAN
1 km 0 1 2 km

LEGEND

- ● Borehole
- ⊕ Borehole and Cone
- ⊕ Auger Hole
- ▼ Cone Penetration Test
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- ⊻ Inferred WL at time of investigation April 2012
- (m NORTH) Offset from Cross Section Line (m)

No	ELEVATION	MTM ZONE 10 COORDINATES NORTH	MTM ZONE 10 COORDINATES EAST
12-2	219.9	4 875 312.0	292 843.3
12-3	220.1	4 875 314.6	292 852.5
12-7	220.4	4 875 320.8	292 873.3
12-8	221.2	4 875 327.3	292 863.2
12-10	220.4	4 875 326.6	292 890.7
12-11	221.4	4 875 347.6	292 964.2
12-12	221.6	4 875 373.9	293 033.2
12-13	221.6	4 875 400.4	293 101.5
12-14	221.7	4 875 424.9	293 172.9
12-15	221.4	4 875 446.4	293 245.3



CROSS SECTION B–B'

SCALE
15m 0 15 30m HORIZONTAL
2m 0 2 4m VERTICAL

NOTES

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

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LICENSED PROFESSIONAL ENGINEER
SG
S. K. GUDINA
Dec. 14, 2012
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
2013/01/14
PROVINCE OF ONTARIO

REVISIONS DATE	BY	DESCRIPTION

GEOCRES No 31D-553

HWY No 9			DIST
SUBM'D SKG	CHECKED	DATE 2012-12-12	SITE 37-30
DRAWN GBB	CHECKED	APPROVED SG	DWG 3

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Photo No. 1: Looking east on Hwy 9 at Holland Drainage Canal Bridge – near the northwest quadrant of the bridge

Photo No. 2: Looking east on Hwy 9 at Holland Drainage Canal Bridge – near existing wingwall at the northwest quadrant of the bridge

Photo No. 3: Looking west on Hwy 9 at Holland Drainage Canal Bridge – near the northeast quadrant of the bridge

Photo No. 4: Looking west along Hwy 9 at Holland Drainage Canal Bridge with the northeast quadrant of bridge near the top left corner

APPENDIX B

Symbols and Terms Used on Borehole Records
Borehole Records
Terminology Used on SCPTu and CPTu Records
CPTu Results
MTO 1965 Borehole Records

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

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

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

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

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

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

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

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

ROCK DESCRIPTION

Terminology describing rock quality:

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

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

Terminology describing rock mass:

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

Terminology describing rock strength:

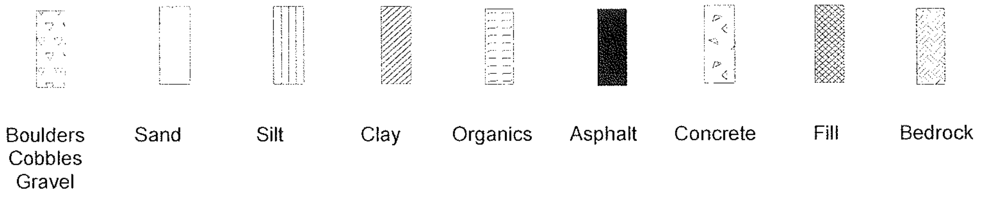
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

Terminology describing rock weathering:

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

STRATA PLOT

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



SAMPLE TYPE

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

WATER LEVEL MEASUREMENT

▼	measured in standpipe, piezometer, or well
▽	inferred

RECOVERY

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

N-VALUE

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

DYNAMIC CONE PENETRATION TEST (DCPT)

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

OTHER TESTS

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

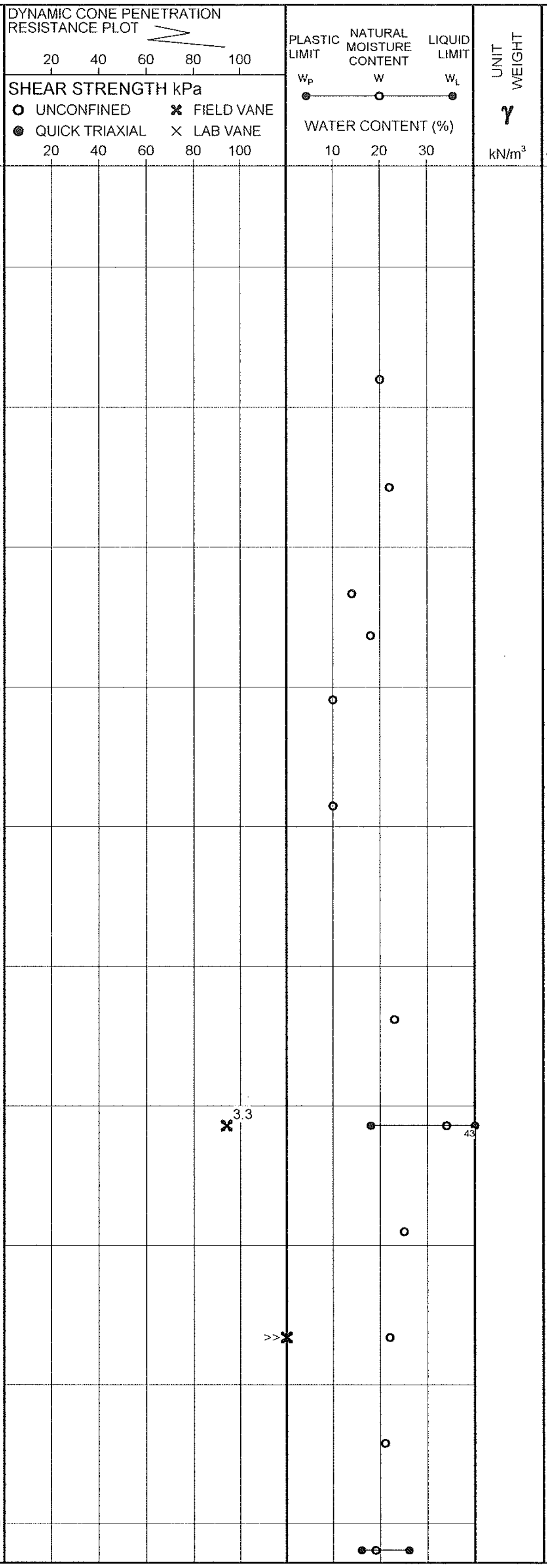
	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

RECORD OF BOREHOLE No BH 12-1

1 OF 3 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 305 E: 292 810
ORIGINATED BY JF
DIST HWY 9
BOREHOLE TYPE H Casing, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 12 - 2012 04 13
CHECKED BY CM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
219.7	Tall Grass									
219.6 0.0	TOPSOIL									
0.1	SAND (SP) Loose Grey to brown, wet	1	SS	1						
		2	SS	9	∇	219				
		3	SS	7		218				
		4	SS	6						
217.0 2.7	SILTY SAND (SM) with gravel Compact Grey, moist	5	SS	24		217				
	- with gravel below 3.7 m	6	SS	30		216				
		7	SS	29		215	29	55	(16)	
214.9 4.8	CLAYEY SILT (CL) Stiff to very stiff Grey, moist	8	SS	15		214				
		9	SS	9		213				
		10	SS	-			0	1	31	68
		11	SS	8		212				
		12	SS	-		211	- S_u > 235 kPa			
		13	SS	21		210				
		14	SS	-			0	1	71	28



Continued Next Page

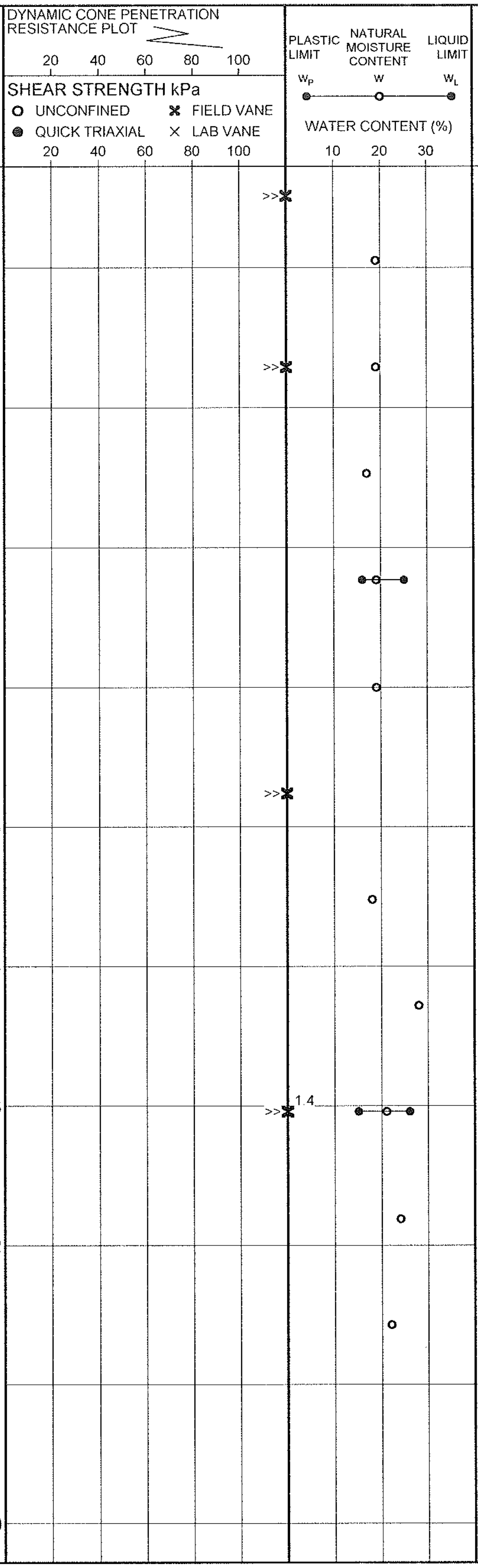
✖³, ×³ Numbers refer to Sensitivity
O³% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH 12-1 2 OF 3 METRIC

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 305 E: 292 810
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE H Casing, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 12 - 2012 04 13
CHECKED BY CM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR SA SI CL
	CLAYEY SILT (CL) Stiff to very stiff Grey, moist *(continued)*						- S_u > 235 kPa
		15	SS	36		209	
		16	SS	-		208	- S_u > 235 kPa
		17	SS	26		207	
		18	SS	28			0 1 74 25
		19	SS	31		206	
		20	SS	-		205	- S_u > 235 kPa
		21	SS	30		204	
		22	SS	-			
		23	SS	-		203	0 1 68 31 - S_u = 141 kPa
		24	SS	19		202	
		25	SS	18		201	
						200	



UNIT WEIGHT γ kN/m³

Continued Next Page

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

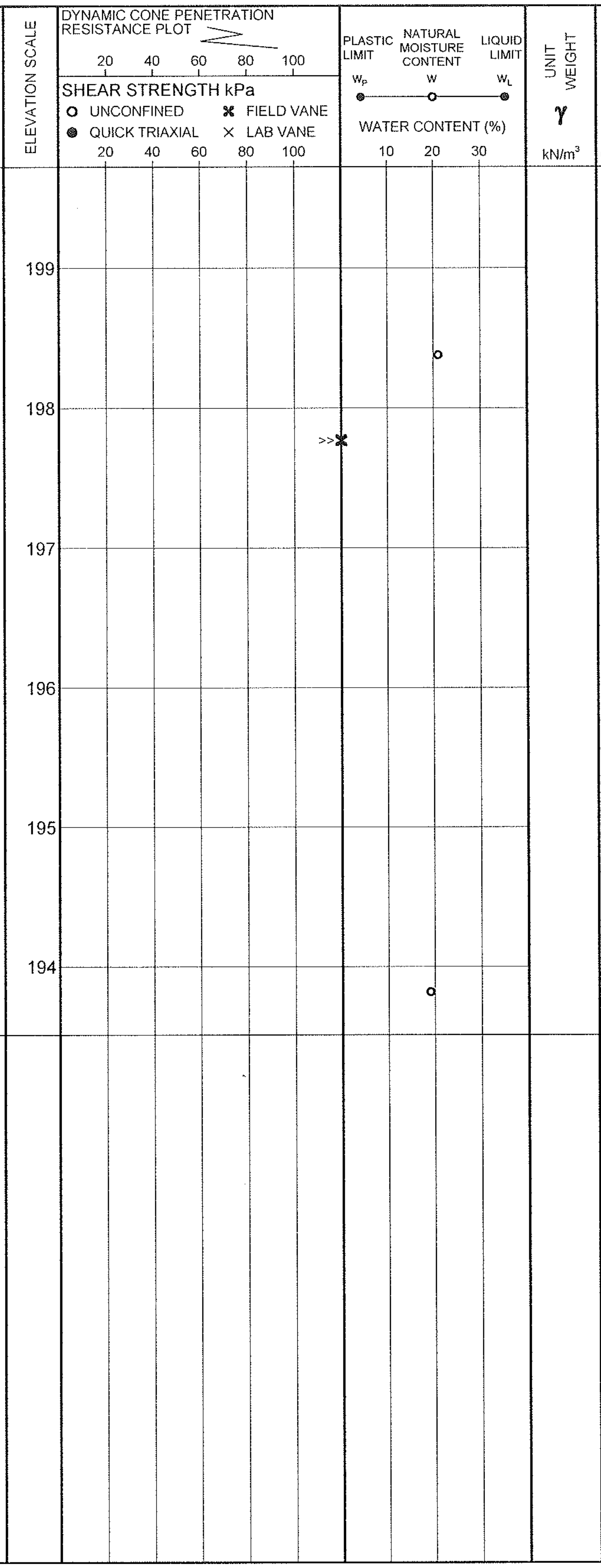
Stantec

RECORD OF BOREHOLE No BH 12-1

3 OF 3 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 305 E: 292 810
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE H Casing, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 12 - 2012 04 13
CHECKED BY CM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT (CL) Stiff to very stiff Grey, moist *(continued)*					
		26	SS	29		
						- S_u > 235 kPa
195.3 24.4	SILTY SAND (SM) Compact Grey, wet	27	SS	25		
		28	SS	13		0 87 (13)
193.5 26.2	End of Borehole - artesian condition encountered below clayey silt					



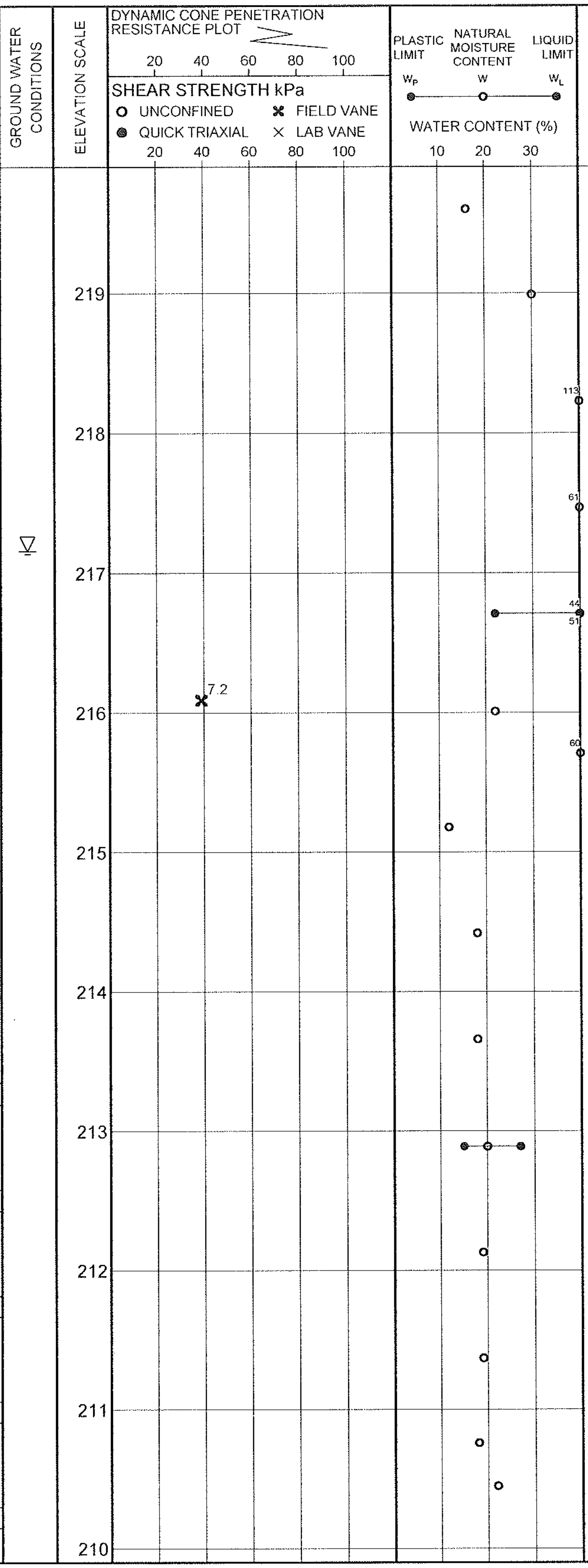
✖^{3}, ×^{3}: Numbers refer to Sensitivity
O$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 12-2

1 OF 4 **METRIC**

W.P. 2188-08-00 LOCATION Hwy 9 Holland Canal N: 4 875 312 E: 292 843 ORIGINATED BY JF
DIST ______ HWY 9 BOREHOLE TYPE H Casing, Splitspoon Sampler COMPILED BY JF
DATUM Geodetic DATE 2012 04 02 - 2012 04 03 CHECKED BY CM

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
219.9	Tall Grass				
219.8 0.1	TOPSOIL				
	FILL: Silty SAND (SM) Loose Brown, moist	1	SS	4	
		2	SS	4	
219.0 0.9	ORGANIC SILT (muck) (OL) Soft Black, moist to wet - frequent coiled shells throughout	3	SS	4	Org M = 23%
		4	SS	1	
		5	SS	1	0 3 70 27
		6	SS	-	
215.8 4.1	Poorly graded SAND WITH SILT (SP-SM) Loose to compact Grey, trace gravel, wet	7	SS	22	
		8	SS	9	10 83 (7)
		9	SS	7	
213.4 6.5	CLAYEY SILT (CL) Stiff Grey, wet	10	SS	9	0 0 62 38
212.4 7.5	SILTY SAND (SM) Loose to compact Grey, wet	11	SS	10	
		12	SS	11	6 81 (13)
		13	SS	8	
210.6 9.3	CLAYEY SILT (CL) Firm to very stiff Grey, moist				



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

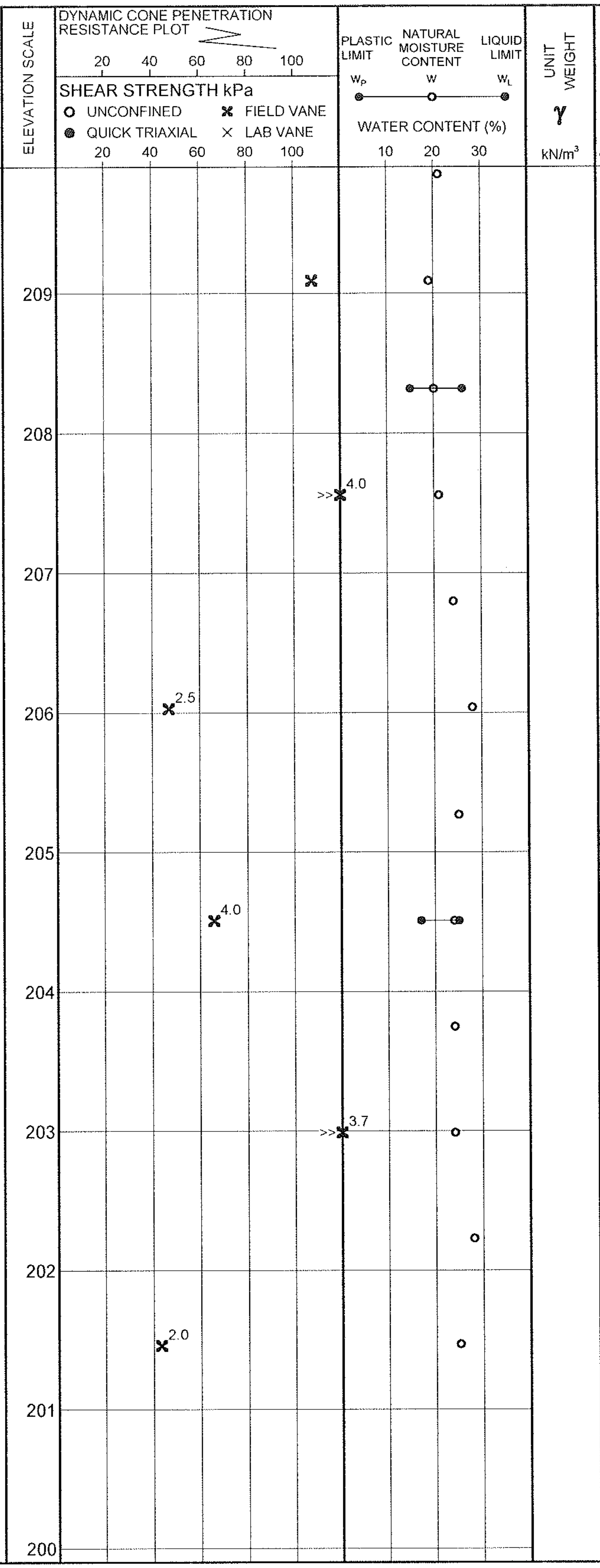
Stantec

RECORD OF BOREHOLE No BH 12-2

2 OF 4 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 312 E: 292 843
ORIGINATED BY JF
DIST ___ HWY 9
BOREHOLE TYPE H Casing, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 02 - 2012 04 03
CHECKED BY CM

SOIL PROFILE			SAMPLES				REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS	GR	SA	SI	CL
	CLAYEY SILT (CL) Firm to very stiff Grey, moist *(continued)*		14	SS	14						
			15	SS	-		- S_u > 108 kPa				
			16	SS	21			0	2	72	26
			17	SS	-		- S_u = 132 kPa				
			18	SS	13						
			19	SS	-						
			20	SS	7						
			21	SS	-			0	0	68	32
			22	SS	18						
			23	SS	-		- S_u = 122 kPa				
			24	SS	5						
			25	SS	-						



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

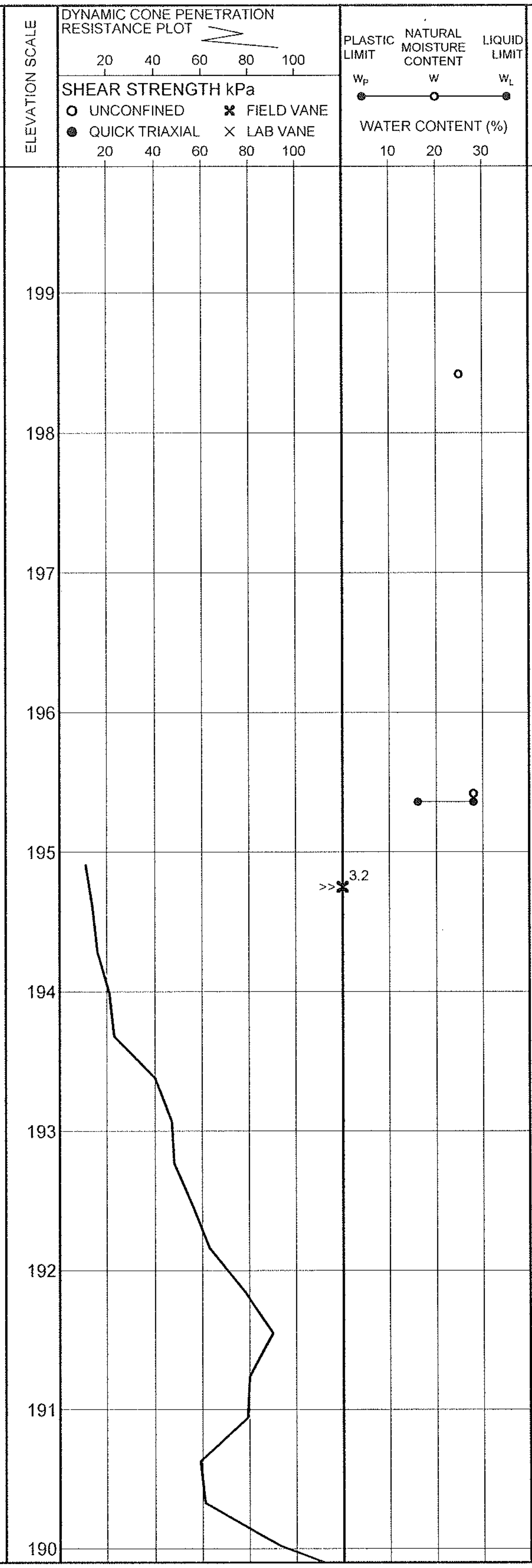
ONTARIO MTO STANTEC 165000801 - HIGHWAY 9 HOLLAND CANAL.GPJ ONTARIO MOT.GDT 12/10/3

RECORD OF BOREHOLE No BH 12-2

3 OF 4 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 312 E: 292 843
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE H Casing, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 02 - 2012 04 03
CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			
	CLAYEY SILT (CL) Firm to very stiff Grey, moist *(continued)*							
			26	SS	16			
			27	SS	15			0 1 50 49
								- S_u = 178 kPa
193.6 26.3	Start of Dynamic Cone Penetration Test Inferred silty sand							



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

ONTARIO MTO STANTEC 165000801 - HIGHWAY 9 HOLLAND CANAL.GPJ ONTARIO MOT.GDT 12/10/3

Stantec

RECORD OF BOREHOLE No BH 12-2

4 OF 4 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 312 E: 292 843
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE H Casing, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 02 - 2012 04 03
CHECKED BY CM

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ 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
	Inferred silty sand *(continued)*										DCPT blows/0.3m 141
											174
							189				196
	- inferred to be dense below 31 m										326
											468
188.0											561
31.9	End of Borehole - artesian condition encountered below clayey silt						188				

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

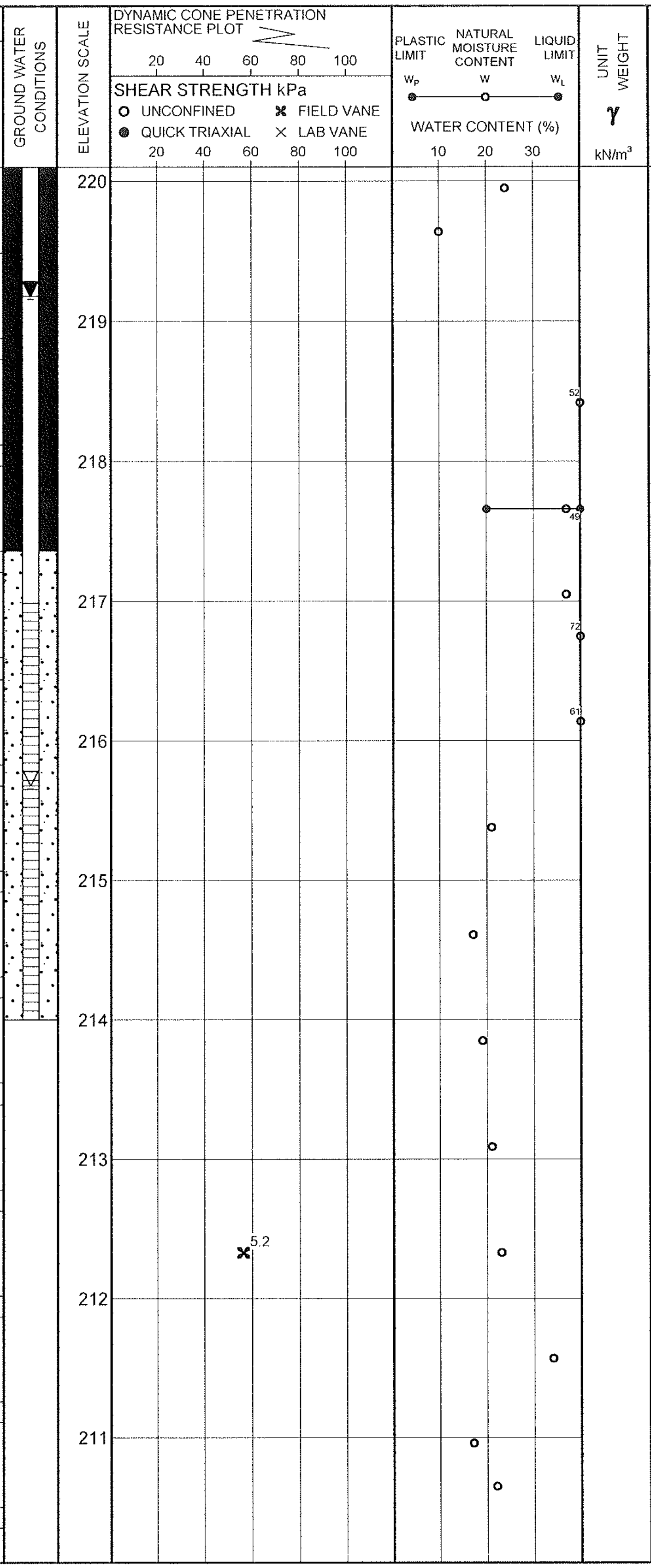
Stantec

RECORD OF BOREHOLE No BH 12-3

1 OF 3 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 315 E: 292 852
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE H Casing, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 04 - 2012 04 05
CHECKED BY CM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
220.1	Tall Grass				
0.0	TOPSOIL				
219.8 0.3	FILL: Silty sand (SM) Compact Brown, moist	1	SS	6	
		2	SS	13	
218.7 1.4	ORGANIC SILT (muck) (OL) Soft Black	3	SS	3	
	- frequent coiled shells throughout	4	SS	1	0 9 66 25
		5	SS	1	Org M = 10%
		6	SS	1	Org M = 7%
215.8 4.3	SILT (ML) Loose Grey, moist	7	SS	4	0 13 72 15 Non-Plastic
215.0 5.1	Poorly graded SAND with silt (SP-SM) Compact to dense Grey, wet	8	SS	32	
		9	SS	22	1 88 (11)
213.4 6.7	CLAYEY SILT (CL) Firm to stiff Grey, moist	10	SS	7	
		11	SS	-	
211.9 8.2	SILTY SAND (SM) Loose Grey, wet	12	SS	8	
210.8 9.3	CLAYEY SILT (CL) Stiff to very stiff Grey, moist	13	SS	7	



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

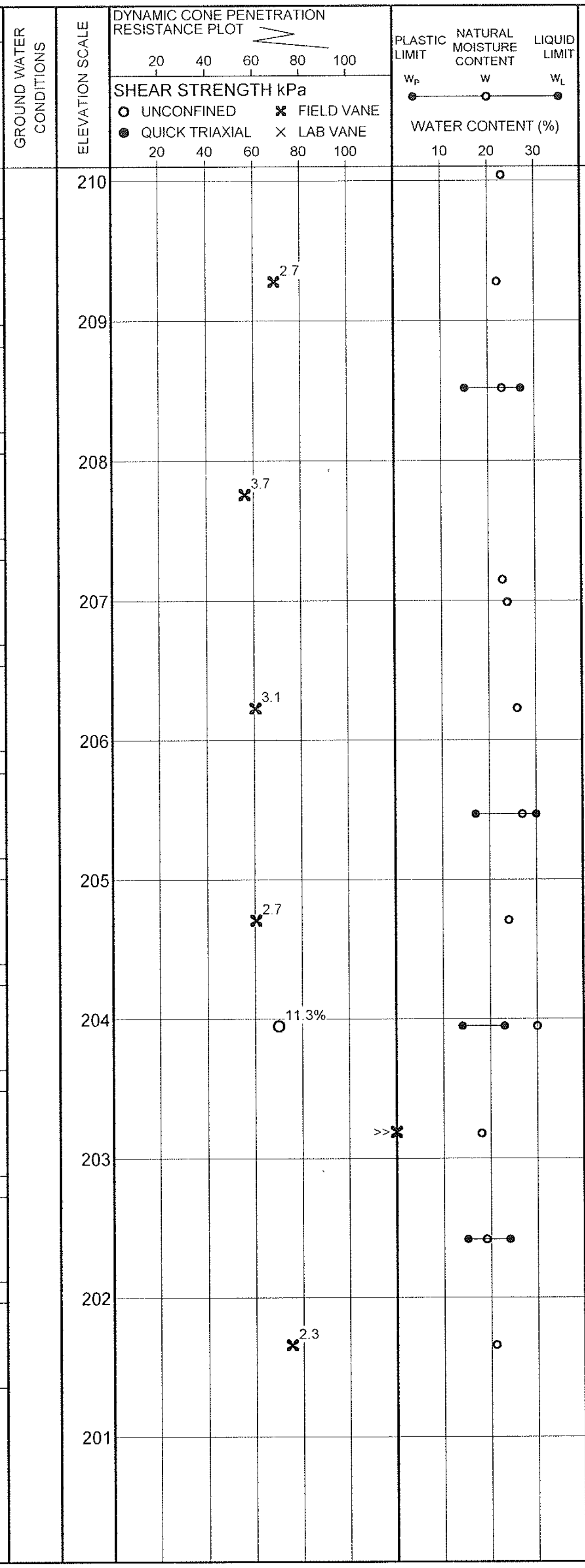
Stantec

RECORD OF BOREHOLE No BH 12-3

2 OF 3 **METRIC**

W.P. 2188-08-00	LOCATION Hwy 9 Holland Canal	N: 4 875 315 E: 292 852	ORIGINATED BY JF
DIST HWY 9	BOREHOLE TYPE H Casing, Splitspoon Sampler		COMPILED BY JF
DATUM Geodetic	DATE 2012 04 04 - 2012 04 05		CHECKED BY CM

SOIL PROFILE		SAMPLES			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
	CLAYEY SILT (CL) Stiff to very stiff Grey, moist *(continued)*	14	SS	7					
		15	SS	-					
		16	SS	5		0	2	63	35
		17	SS	-					
		18	SS	7					
		19	SS	-					
		20	SS	5		0	0	56	44
		21	SS	-					
		22	ST		19.3	0	4	69	27
		23	SS	-		- S_u > 108 kPa			
		24	SS	23		0	3	65	32
		25	SS	11					



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✖³, ×³ Numbers refer to Sensitivity

O³% STRAIN AT FAILURE

ONTARIO MTO STANTEC 165000801 - HIGHWAY 9 HOLLAND CANAL.GPJ ONTARIO MOT.GDT 12/10/3

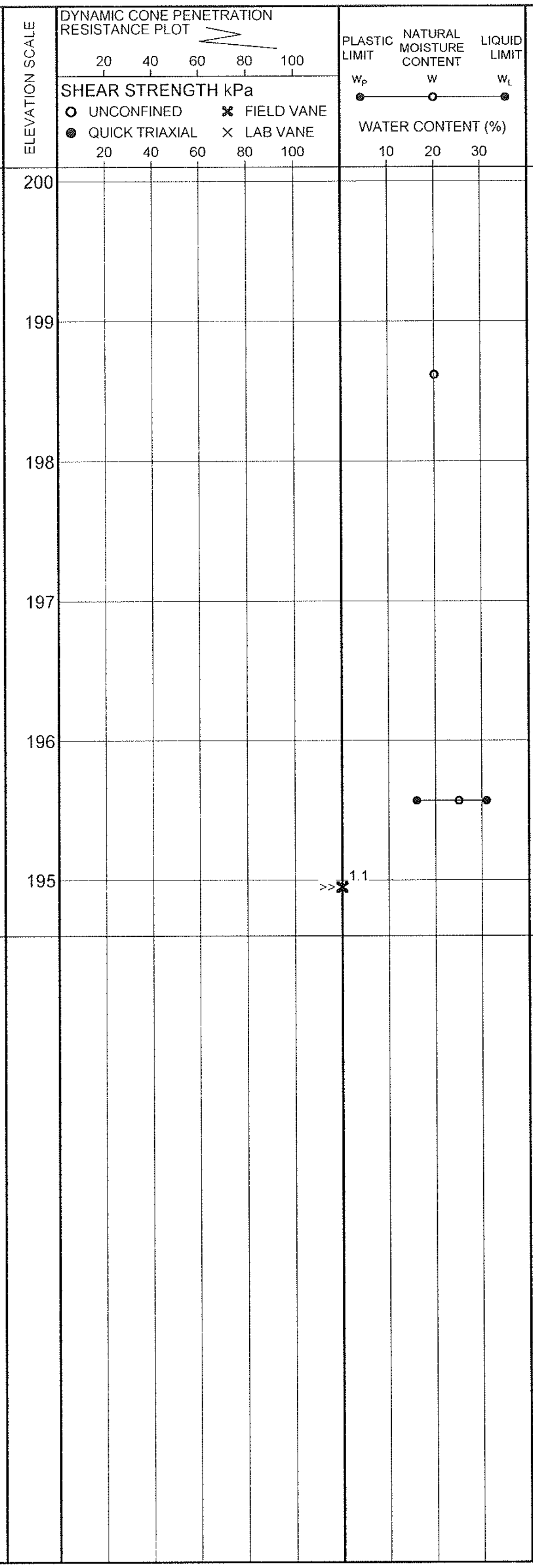
Stantec

RECORD OF BOREHOLE No BH 12-3

3 OF 3 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 315 E: 292 852
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE H Casing, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 04 - 2012 04 05
CHECKED BY CM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	CLAYEY SILT (CL) Stiff to very stiff Grey, moist *(continued)*									
		26	SS	31						
		27	SS	19			0	1	48	51
							- S_u = 132 kPa			
194.6 25.5	End of Borehole Monitor Well Installed to 6.10 m									



✖³, ×³ Numbers refer to Sensitivity

O³% STRAIN AT FAILURE

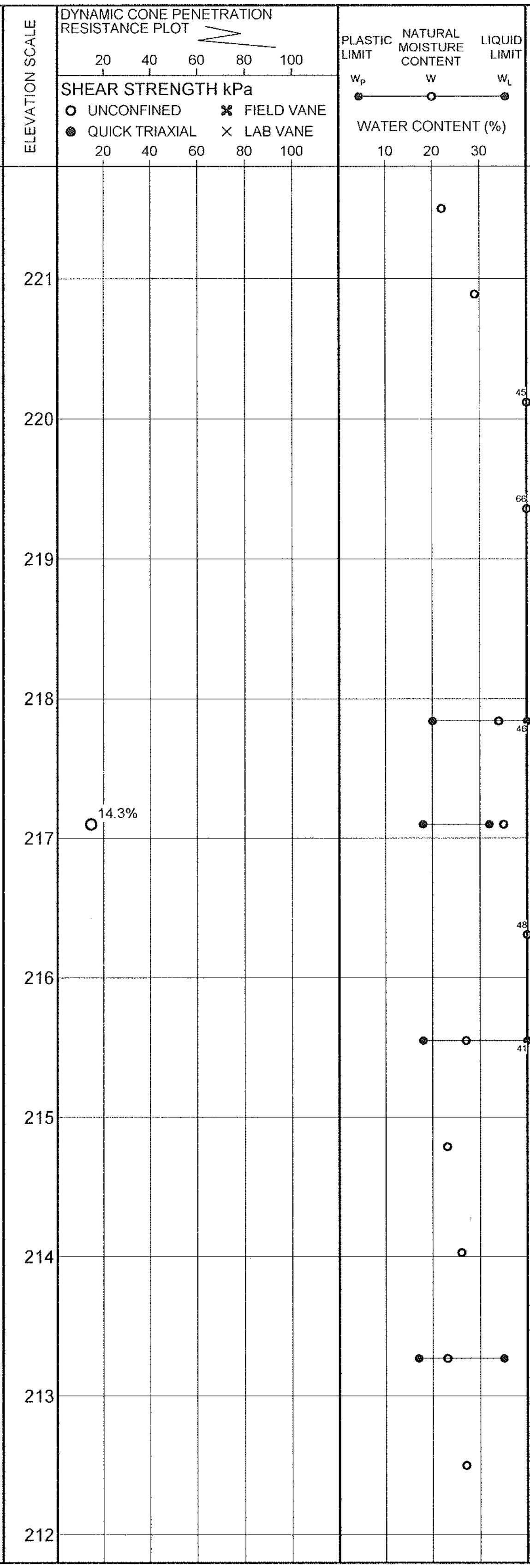
Stantec

RECORD OF BOREHOLE No BH 12-5

1 OF 4 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 323 E: 292 845
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE H Casing, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 03 28 - 2012 03 29
CHECKED BY CM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	GR	SA	SI	CL
221.8	Tall Grass										
[illegible] 0.1	TOPSOIL Silty sand (SM), FILL Loose Brown, moist	1	SS	5							
		2	SS	4							
220.4 1.4	ORGANIC SILT (muck) (OL) Soft Black, moist	3	SS	2			Org M = 6%				
	- frequent coiled shells throughout	4	SS	1							
		5	ST								
		6	SS	1				0	7	66	27
		7	ST		∇	18.4		0	5	73	22
		8	SS	3							
215.9 5.9	CLAYEY SILT (CI) Firm to very stiff Grey, moist	9	SS	9				0	2	31	67
		10	SS	16			PP = 200 kPa				
		11	SS	10			PP = 92 kPa				
		12	SS	7			PP = 67 kPa	1	2	50	47
		13	SS	5							



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

ONTARIO MTO STANTEC 165000801 - HIGHWAY 9 HOLLAND CANAL.GPJ ONTARIO MOT.GDT 12/10/3

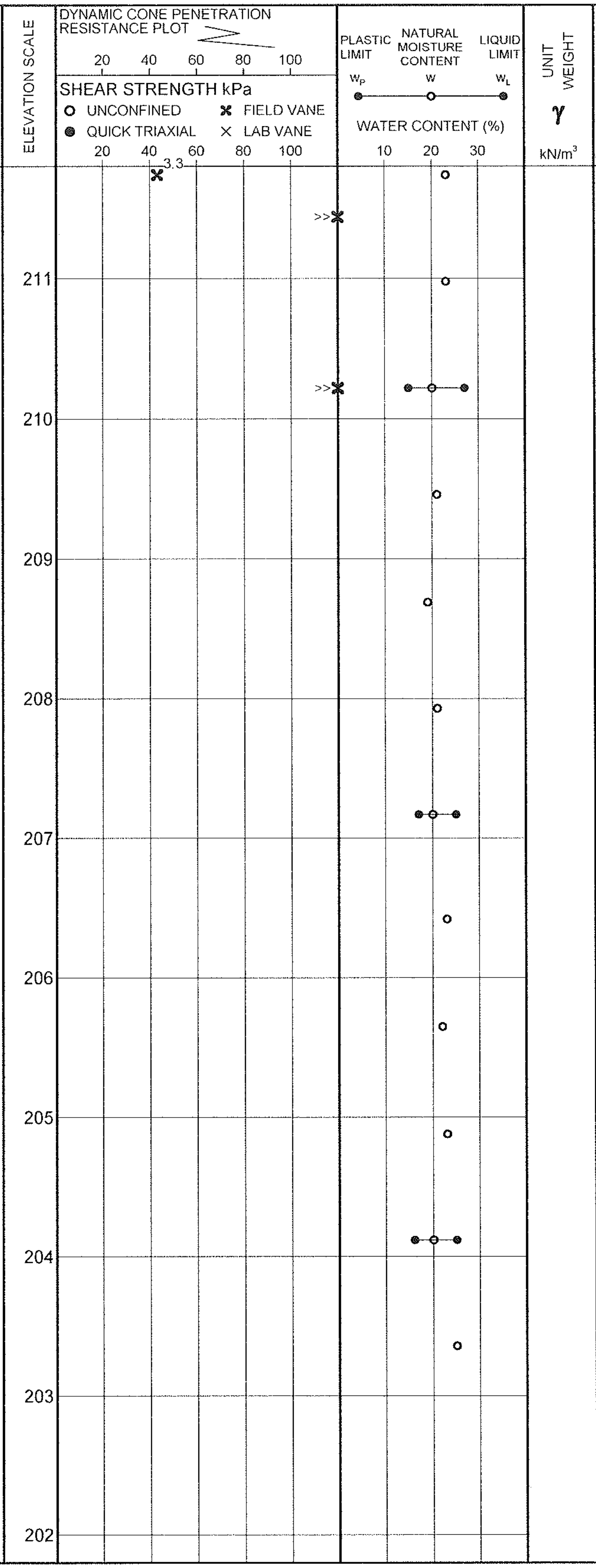
Stantec

RECORD OF BOREHOLE No BH 12-5

2 OF 4 **METRIC**

W.P. 2188-08-00	LOCATION Hwy 9 Holland Canal	N: 4 875 323 E: 292 845	ORIGINATED BY JF
DIST ____ HWY 9	BOREHOLE TYPE H Casing, Splitspoon Sampler		COMPILED BY JF
DATUM Geodetic	DATE 2012 03 28 - 2012 03 29		CHECKED BY CM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT (CI) Firm to very stiff Grey, moist *(continued)*		14	SS	-		- S_u > 108 kPa
			15	SS	16		
			16	SS	-		0 0 66 34 - S_u > 108 kPa
			17	SS	17		PP = 187 kPa
			18	SS	19		
			19	SS	28		
			20	SS	16		0 0 67 33
			21	SS	21		
			22	SS	18		
			23	SS	21		
			24	SS	22		0 0 73 27 PP = 208 kPa PP = 183 kPa
			25	SS	20		



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✖³, ×³ : Numbers refer to Sensitivity

O³% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH 12-5

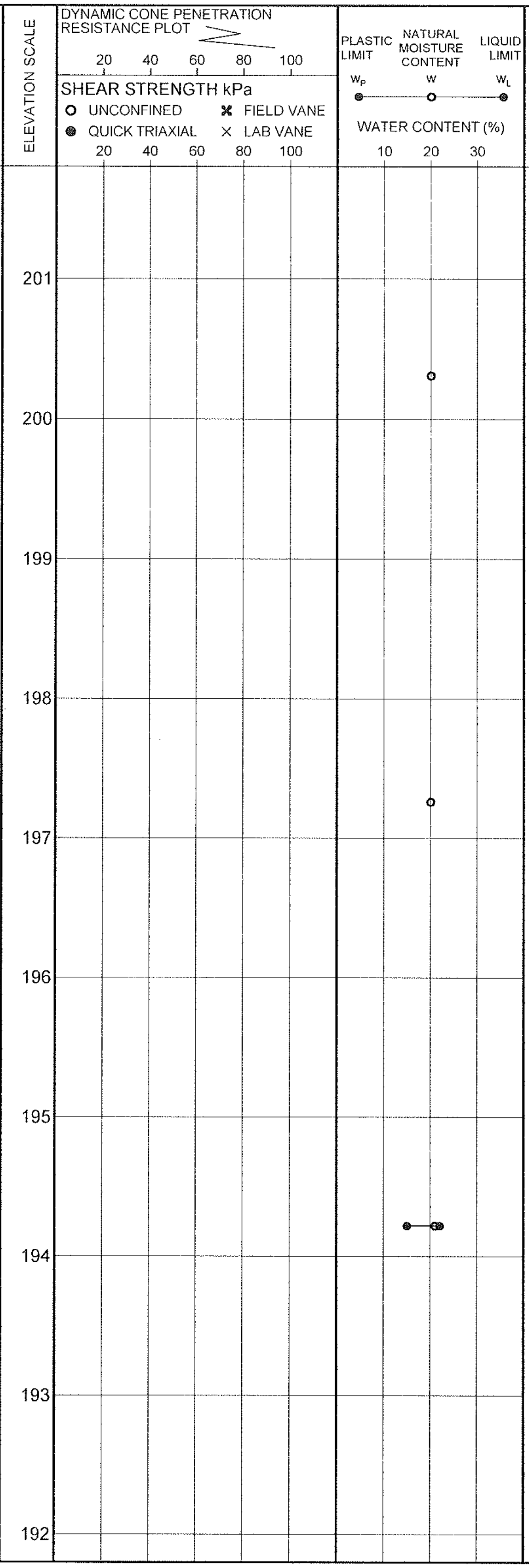
3 OF 4 **METRIC**

W.P. 2188-08-00 LOCATION Hwy 9 Holland Canal N: 4 875 323 E: 292 845 ORIGINATED BY JF

DIST ______ HWY 9 BOREHOLE TYPE H Casing, Splitspoon Sampler COMPILED BY JF

DATUM Geodetic DATE 2012 03 28 - 2012 03 29 CHECKED BY CM

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT (CI) Firm to very stiff Grey, moist *(continued)*							
			26	SS	21			PP = 192 kPa
			27	SS	49			PP = 175 kPa
			28	SS	29			0 3 66 31 PP = 175 kPa
192.7 29.1	SILTY SAND (SM) Loose Grey, trace gravel, wet							



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

ONTARIO MTO STANTEC 165000801 - HIGHWAY 9 HOLLAND CANAL.GPJ ONTARIO MOT.GDT 12/10/3

Stantec

RECORD OF BOREHOLE No BH 12-5

4 OF 4 **METRIC**

W.P.	2188-08-00	LOCATION	Hwy 9 Holland Canal N: 4 875 323 E: 292 845	ORIGINATED BY	JF
DIST	HWY 9	BOREHOLE TYPE	H Casing, Splitspoon Sampler	COMPILED BY	JF
DATUM	Geodetic	DATE	2012 03 28 - 2012 03 29	CHECKED BY	CM

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						
190.9	SILTY SAND (SM) Loose Grey, trace gravel, wet *(continued)*		29	SS	4		191		o		
30.9	End of Borehole - artesian condition encountered below clayey silt										

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

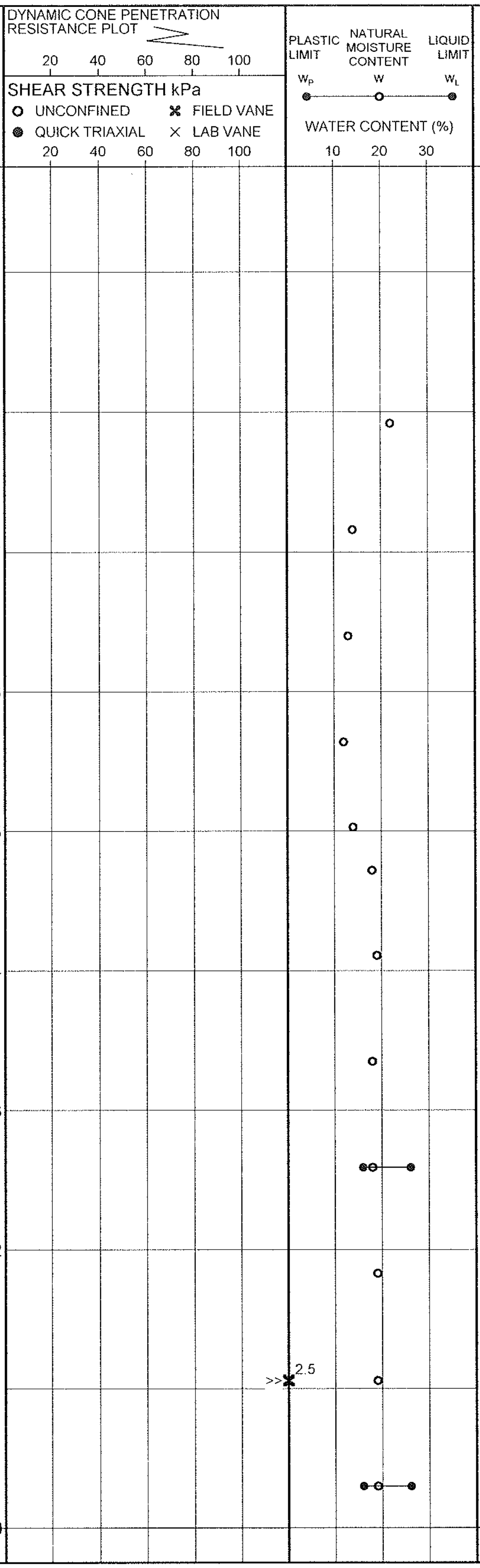
Stantec

RECORD OF BOREHOLE No BH 12-6

1 OF 1 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 299 E: 292 800
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 11 - 2012 04 11
CHECKED BY CM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
219.8	Tall Grass						
0.0	TOPSOIL						
219.7 0.1	SAND (SP) Loose Grey to brown, moist to wet	1	SS	3			
		2	SS	4	∇	219	
		3	SS	4		218	
217.5 2.3	SILTY SAND (SM) Compact Grey, moist	4	SS	18		217	
		5	SS	29		216	
		6	SS	20			11 77 (12)
214.9 4.9	CLAYEY SILT (CL) Very stiff Grey, moist	7	SS	11		215	
		8	SS	15		214	
		9	SS	19		213	
		10	SS	23			0 4 64 32
		11	SS	29		212	
		12	SS	-		211	- S_u = 226 kPa
		13	SS	23			0 1 67 32
210.0 9.8	End of Borehole					210	



UNIT WEIGHT γ kN/m³

×3, ×3 Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

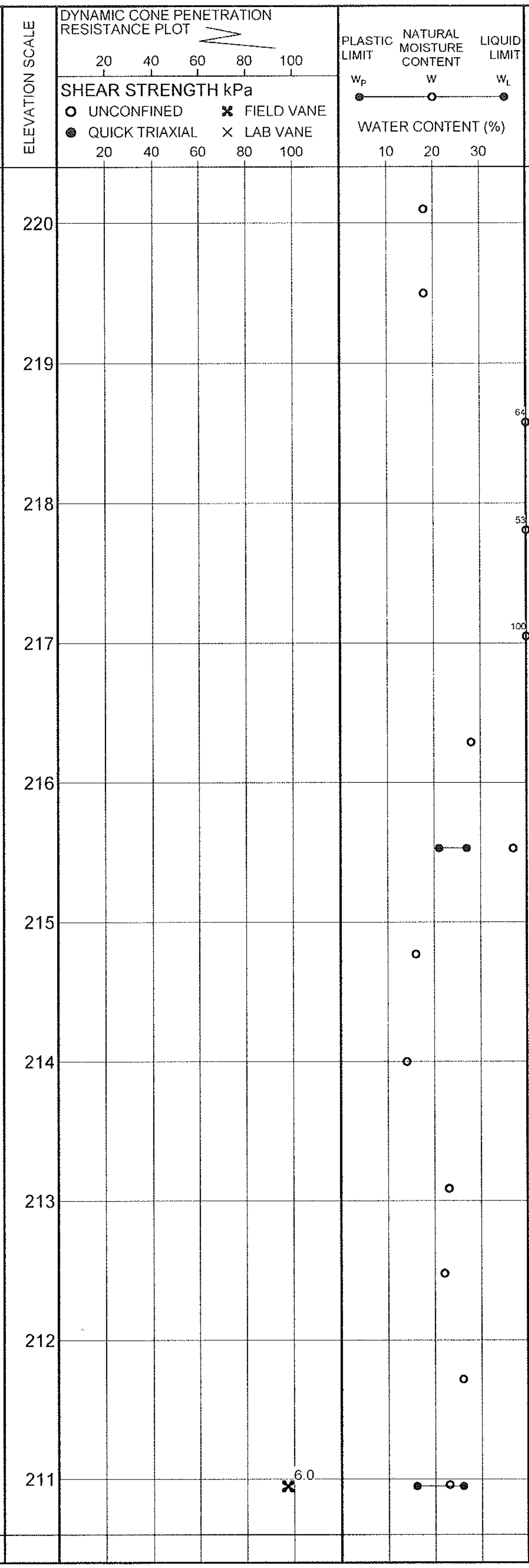
Stantec

RECORD OF BOREHOLE No BH 12-7

1 OF 1 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 321 E: 292 873
ORIGINATED BY JF
DIST ____ HWY 9
BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 03 27 - 2012 03 27
CHECKED BY CM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
220.4	Tal Grass						
220.3 0.1	TOPSOIL						
	FILL: Sandy silt (ML) Loose Brown to grey, trace gravel, moist	1	SS	5			
		2	SS	7			5 39 (56)
219.0 1.4	ORGANIC SILT (muck) (OL) Soft Black	3	SS	3			Org M = 14%
	- frequent coiled shells throughout	4	SS	1			Org M = 6%
		5	SS	2			Org M = 18%
216.6 3.8	SANDY SILT (ML) Very loose Grey, moist	6	SS	3			
215.5 4.9	SILTY SAND with gravel (SM) Loose Grey, wet	7	SS	2	∇		3 36 46 15
		8	SS	5			21 63 (16)
		9	SS	8			
213.3 7.1	CLAYEY SILT (CL) Stiff Grey, wet	10	SS	7			9 25 (66)
		11	SS	12			
		12	SS	5			PP = 58 kPa
		13	SS	-			1 6 57 36
210.6 9.8	End of Borehole						



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

ONTARIO MTO STANTEC 165000801 - HIGHWAY 9 HOLLAND CANAL.GPJ ONTARIO MOT.GDT 12/10/3

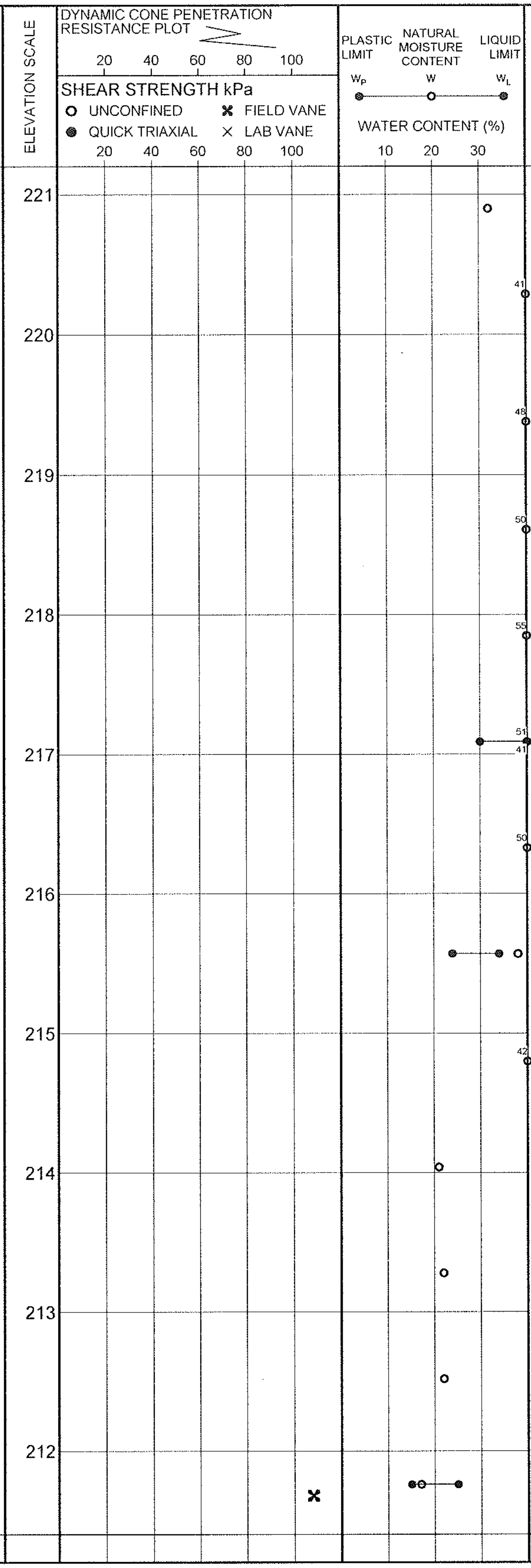
Stantec

RECORD OF BOREHOLE No BH 12-8

1 OF 1 **METRIC**

W.P. 2188-08-00	LOCATION Hwy 9 Holland Canal	N: 4 875 327 E: 292 863	ORIGINATED BY JF
DIST	HWY 9	BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler	COMPILED BY JF
DATUM Geodetic	DATE 2012 03 27 - 2012 03 27		CHECKED BY CM

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
221.2	Tall Grass						
[illegible] 0.1	TOPSOIL						
	ORGANIC SILT (muck) (OL) Soft Black	1	SS	4			Org M = 9%
	- frequent coiled shells throughout	2	SS	5			
		3	SS	5			Org M = 11%
		4	SS	4			Org M = 11%
		5	SS	3			
		6	SS	1			0 27 55 18
		7	SS	2			Org M = 6%
216.1 5.1	SILT with SAND (ML) Very loose Grey, wet	8	SS	2			0 20 63 17
	- trace organics at 6.2 m	9	SS	1			
214.7 6.6	SILTY SAND (SM) Loose Grey, very wet				∇		
214.3 6.9	CLAYEY SILT (CL) Very stiff Grey, wet	10	SS	6			2 19 (79)
		11	SS	7			PP = 116 kPa
		12	SS	17			PP = 156 kPa
		13	SS	1			0 1 68 31 PP = 133 kPa - S_u > 108 kPa
211.4 9.8	End of borehole						



✖ 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

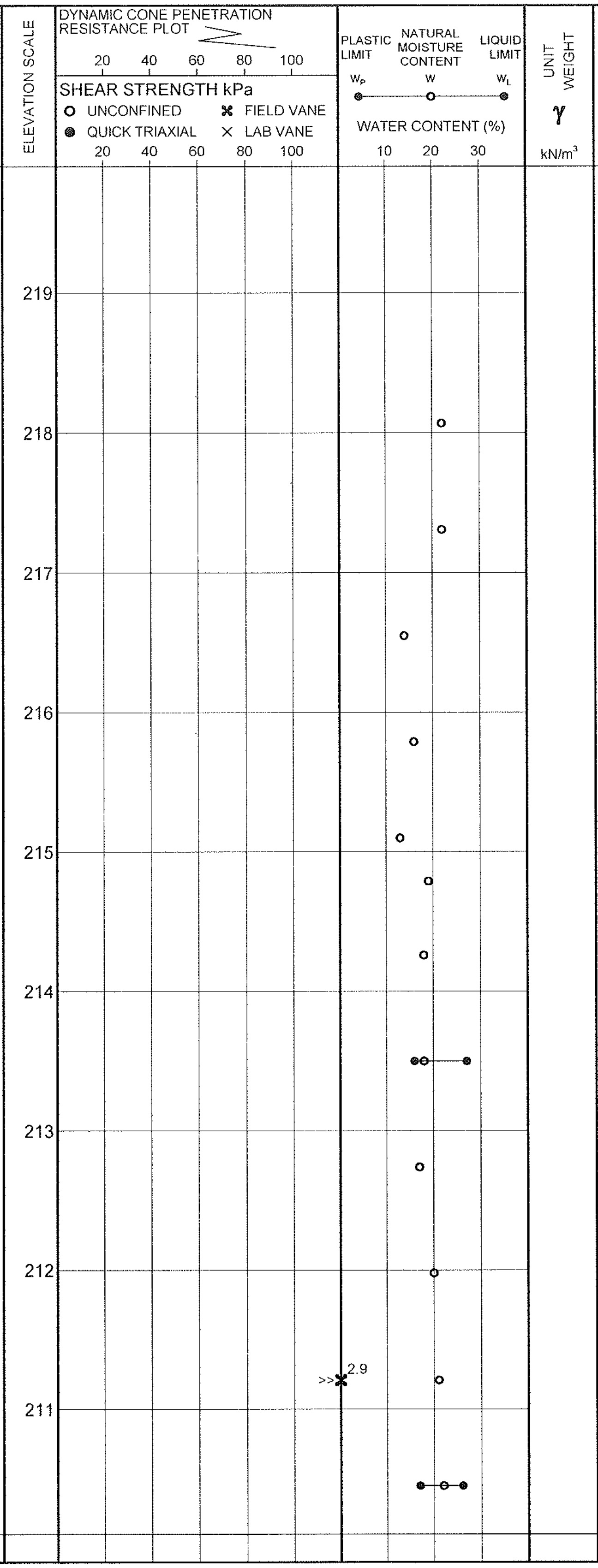
Stantec

RECORD OF BOREHOLE No BH 12-9

1 OF 1 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 305 E: 292 789
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 11 - 2012 04 11
CHECKED BY CM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
219.9	Tall Grass								
219.8 0.0	TOPSOIL								
0.2	SAND (SP) Loose Grey to brown, moist to wet	1	SS	4					
		2	SS	5	∇				
		3	SS	8					
		4	SS	6					
216.9 3.1	SILTY SAND (SM) Compact to dense Grey, moist to wet	5	SS	37					
		6	SS	26					
		7	SS	17		9	85	(6)	
214.9 5.0	CLAYEY SILT (CL) Very stiff Grey, moist	8	SS	17					
		9	SS	34		0	2	63	35
		10	SS	29					
		11	SS	13					
		12	SS	-					
		13	SS	21		0	0	63	37
210.1 9.8	End of Borehole								



REMARKS & GRAIN SIZE DISTRIBUTION (%)

✖3, ×3: Numbers refer to Sensitivity

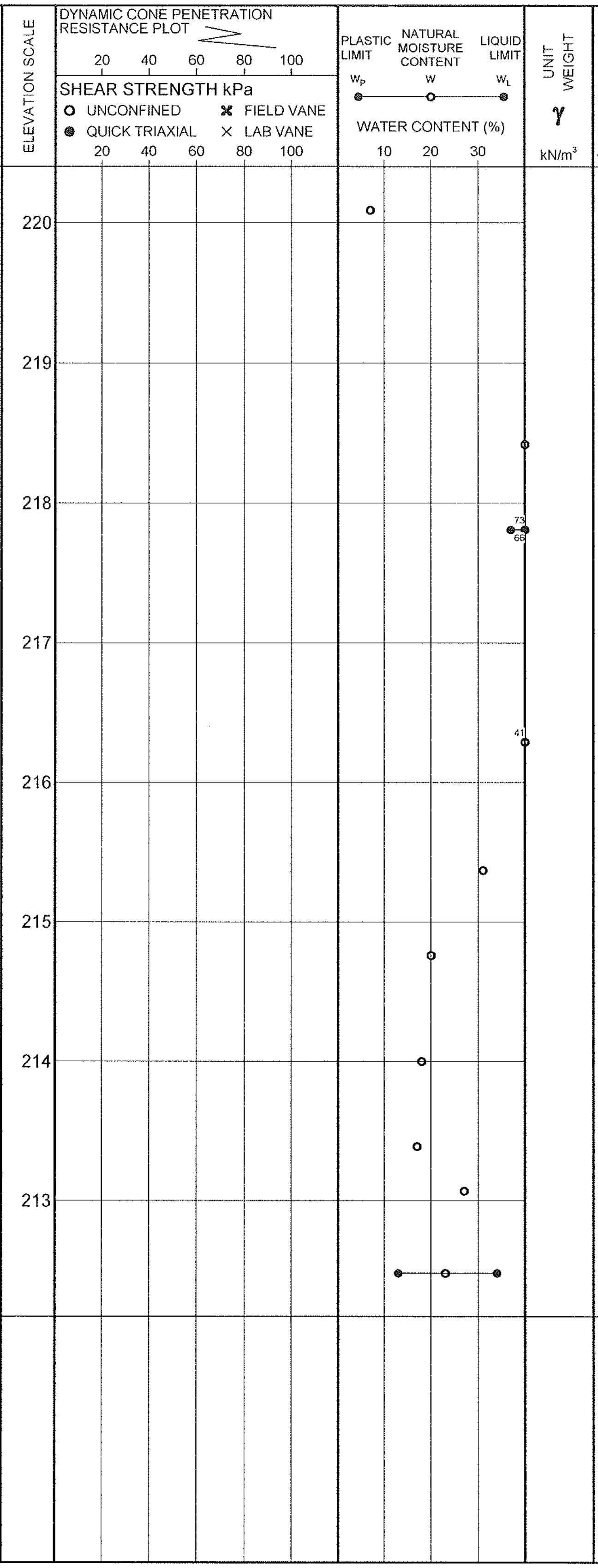
O3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 12-10

1 OF 1 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 327 E: 292 891
ORIGINATED BY JF
DIST ____ HWY 9
BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 10 - 2012 04 10
CHECKED BY CM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
220.4	Tall Grass						
0.0	FILL: Silt (ML) to silty sand (SM) Loose Brown, moist to wet	▨	1	SS	6		
		▨	2	SS	7		
218.6 1.9	ORGANIC SILT (OL) Soft Black, moist, with organics and shells		3	SS	5		Org M = 20%
	- frequent coiled shells throughout		4	SS	2		0 8 76 16
			5	SS	2		Org M = 11%
			6	SS	1		
			7	SS	2	▽	Org M = 10%
215.1 5.3	SILTY SAND (SM) with gravel Loose to compact Grey, wet		8	SS	4		20 52 (28)
			9	SS	19		
213.2 7.2	CLAYEY SILT (CL) Stiff Grey, moist		10	SS	10		
			11	SS	13		3 5 42 50
212.2 8.2	End of Borehole						



✖3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

Stantec

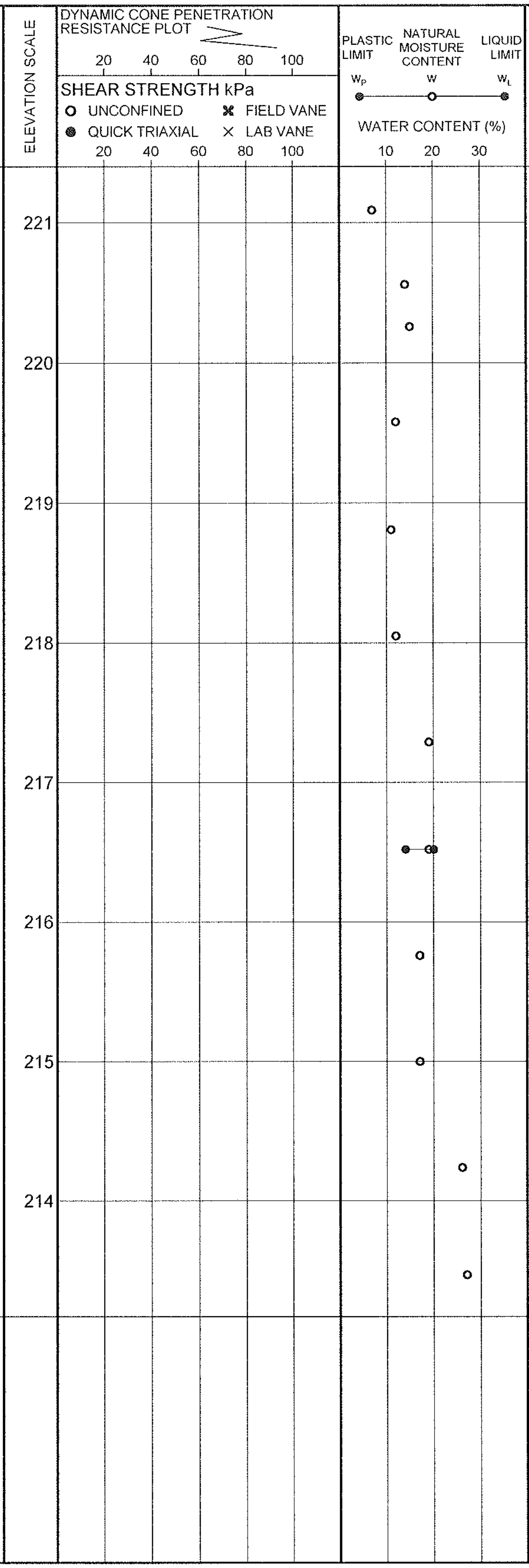
RECORD OF BOREHOLE No BH 12-11

1 OF 1 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 348 E: 292 964
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 24 - 2012 04 24
CHECKED BY CM

SOIL PROFILE			SAMPLES				REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR SA SI CL
221.4	Roadbase Granulars						
0.0	FILL: gravelly sand, brown		1	BS			
220.5 0.9	FILL: Silty sand (SM) to sandy silt (ML) Brown to grey, moist to wet		2	SS	11		
			3	SS	10		Org M = 2%
			4	SS	8		7 36 (57)
			5	SS	2		
217.7 3.7	SILT with SAND (ML) Very loose to loose Grey, moist, trace organics		6	SS	4	∇	
			7	SS	9		0 18 68 14
216.1 5.3	SILTY SAND (SM) Compact Grey, moist		8	SS	11		
			9	SS	23		3 79 (18)
214.5 6.9	CLAYEY SILT (CL) Stiff Grey, moist		10	SS	8		
			11	SS	7		
213.2 8.2	End of Borehole						

UNIT WEIGHT γ kN/m³



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

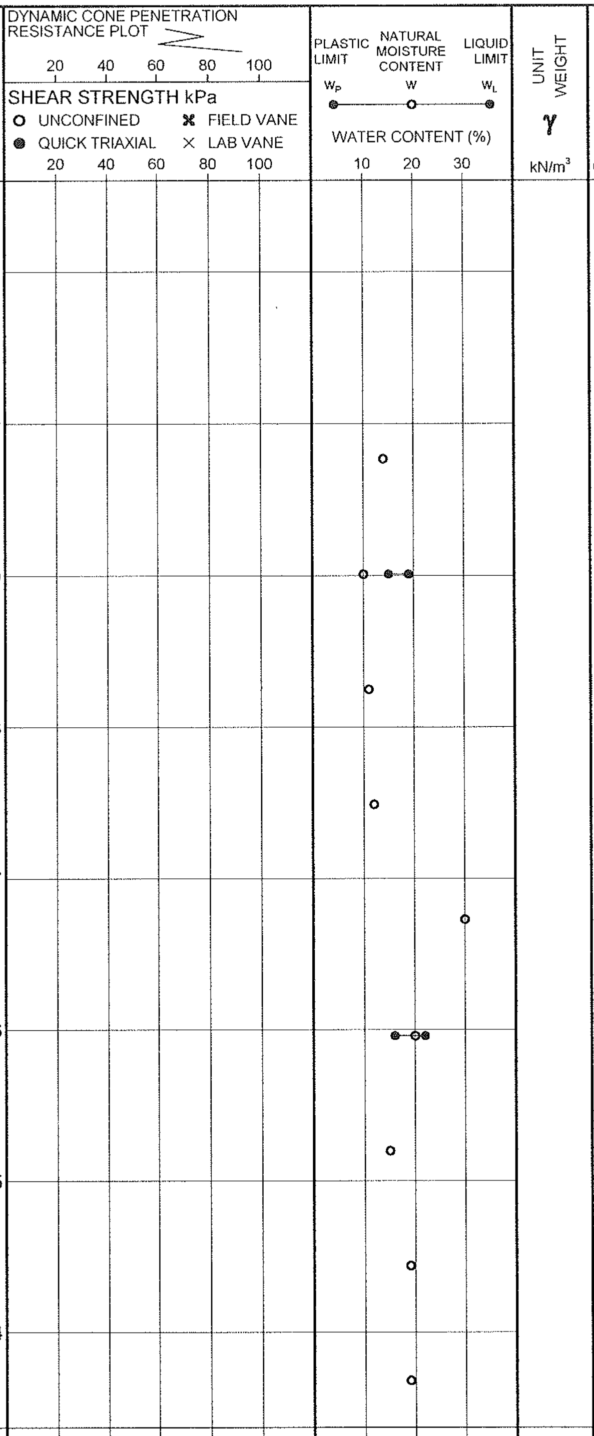
Stantec

RECORD OF BOREHOLE No BH 12-12

1 OF 1 **METRIC**

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 374 E: 293 033
ORIGINATED BY JF
DIST
HWY 9
BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 25 - 2012 04 25
CHECKED BY CM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
221.6	Roadbase Granulars					
0.0	FILL: Gravelly sand, brown	1	SS	21		
220.4 1.2	FILL: Silty sand (SM) to sandy silt (ML) Grey to brown	2	SS	10		
		3	SS	7		4 33 50 13
		4	SS	4		
217.9 3.7	Sandy SILT (ML) Very loose to compact Grey, trace gravel and organics	5	SS	2	▽	
		6	SS	4		Org M = 3%
		7	SS	10		0 34 37 29
215.5 6.1	SILTY SAND (SM) Compact Grey, wet	8	SS	26		
		9	SS	27		
		10	SS	26		2 59 (39)
213.4 8.2	End of Borehole					



✖³, ×³: Numbers refer to Sensitivity

O³% STRAIN AT FAILURE

ONTARIO MTO STANTEC 165000801 - HIGHWAY 9 HOLLAND CANAL.GPJ ONTARIO MOT.GDT 12/10/3

RECORD OF BOREHOLE No BH 12-13

1 OF 1 **METRIC**

W.P. 2188-08-00	LOCATION Hwy 9 Holland Canal	N: 4 875 400 E: 293 101	ORIGINATED BY JF
DIST HWY 9	BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler		COMPILED BY JF
DATUM Geodetic	DATE 2012 04 25 - 2012 04 25		CHECKED BY CM

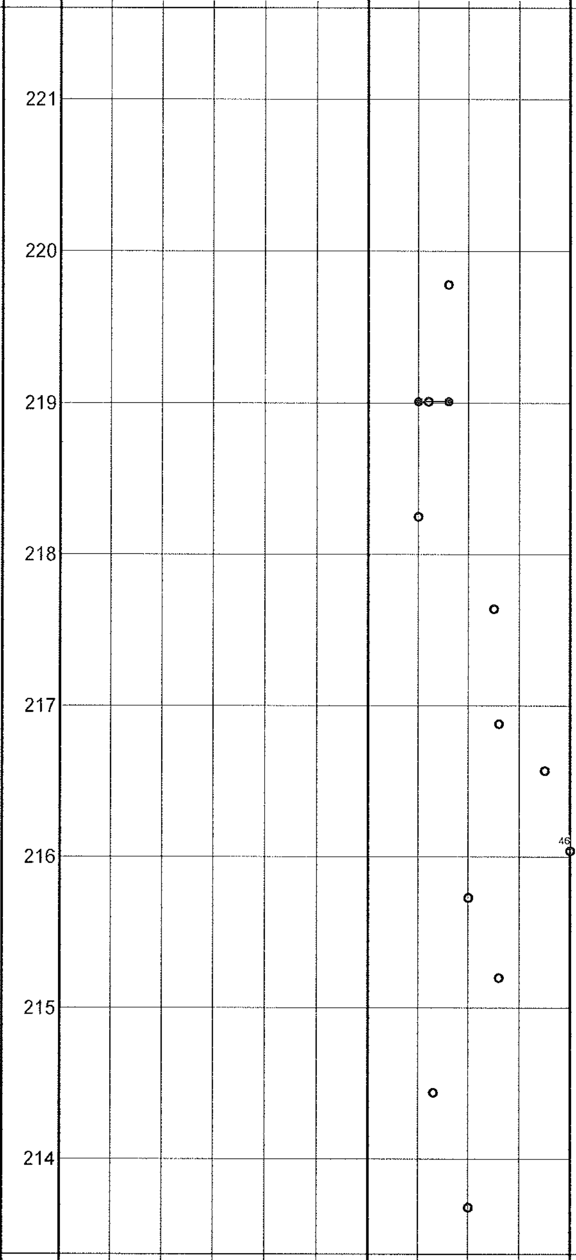
ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
221.6	Roadbase Granulars					
0.0	FILL: Gravelly sand, brown					
220.5 1.1	FILLL: Silty sand (SM) to sandy silt (ML) Brown to grey, moist to wet	1	SS	20		
		2	SS	9		
		3	SS	9		5 29 52 14
		4	SS	3		
217.9 3.7	ORGANIC SILT (OL) Soft Black - frequent coiled shells throughout	5	SS	4		
		6	SS	2		Org M = 3%
		7	SS	5	▽	4 46 (50) Org M = 6%
215.8 5.8	SILTY SAND (SM) Compact to dense Grey, wet	8	SS	12		
		9	SS	19		
		10	SS	35		5 71 (24)
213.4 8.2	End of Borehole					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ 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



ONTARIO MTO STANTEC 165000801 - HIGHWAY 9 HOLLAND CANAL.GPJ ONTARIO MOT.GDT 12/10/3

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

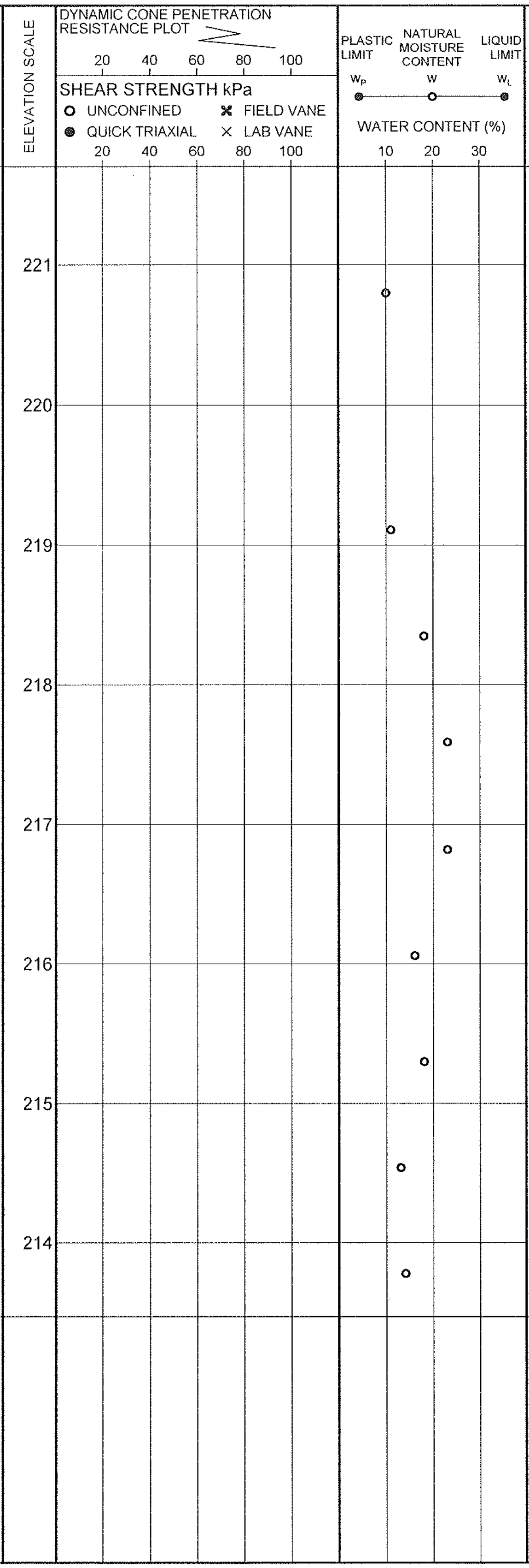
Stantec

RECORD OF BOREHOLE No BH 12-14

1 OF 1 **METRIC**

W.P. 2188-08-00	LOCATION Hwy 9 Holland Canal	N: 4 875 425 E: 293 173	ORIGINATED BY JF
DIST ______ HWY 9	BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler		COMPILED BY JF
DATUM Geodetic	DATE 2012 04 25 - 2012 04 25		CHECKED BY CM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
221.7	Roadbase Granulars								
0.0	FILL: Gravelly sand, brown								
220.6 1.1	FILL: Silty sand (SM) to sandy silt (ML) Brown, moist	1	SS	14					
		2	SS	13					
		3	SS	12		8	49	(43)	
218.7 3.0	SILT (ML) Loose Brown to grey, wet - trace organics	4	SS	4	▽				
		5	SS	5					
		6	SS	6					
216.5 5.2	SILTY SAND (SM) Compact to dense Grey, moist to wet	7	SS	14					
		8	SS	25					
	- with gravel below 6.7 m	9	SS	33		15	76	(9)	
		10	SS	17					
213.5 8.2	End of Borehole								



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

✖3, ×3: Numbers refer to Sensitivity

O3% STRAIN AT FAILURE

Stantec

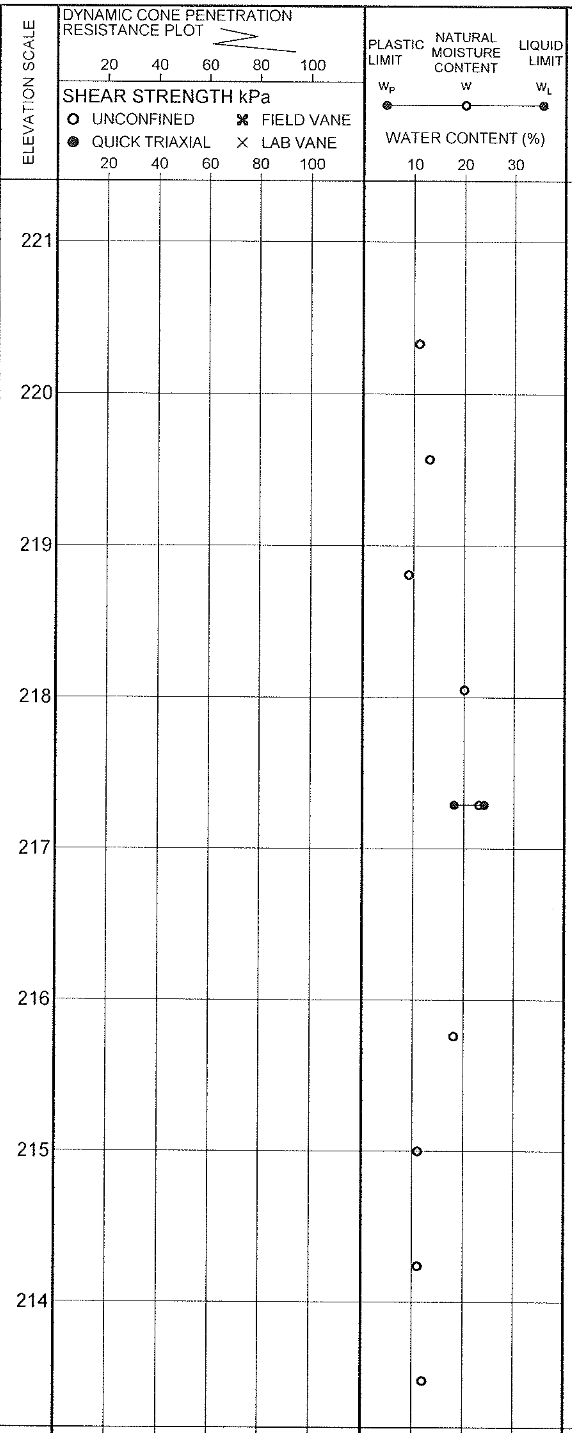
RECORD OF BOREHOLE No BH 12-15

1 OF 1

METRIC

W.P. 2188-08-00
LOCATION Hwy 9 Holland Canal N: 4 875 446 E: 293 245
ORIGINATED BY JF
DIST HWY 9
BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler
COMPILED BY JF
DATUM Geodetic
DATE 2012 04 25 - 2012 04 25
CHECKED BY CM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
221.4	Roadbase Granulars								
0.0	FILL: Gravelly sand, brown								
220.6 0.8	FILL: Silty sand (SM) to sandy silt (ML) Brown, moist	1	SS	17					
	- cobbles at 1.8 m	2	SS	60					
		3	SS	18		4	47	(49)	
218.4 3.0	SILT (ML) Loose Brown to grey, moist to wet	4	SS	4	∇				
		5	SS	3		0	1	87	12
		6	SS	8					
216.1 5.3	SILTY SAND (SM) Compact Grey, moist to wet	7	SS	14					
	- with gravel below 6.1 m	8	SS	28					
		9	SS	23		14	79	(7)	
		10	SS	30					
213.2 8.2	End of Borehole								



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

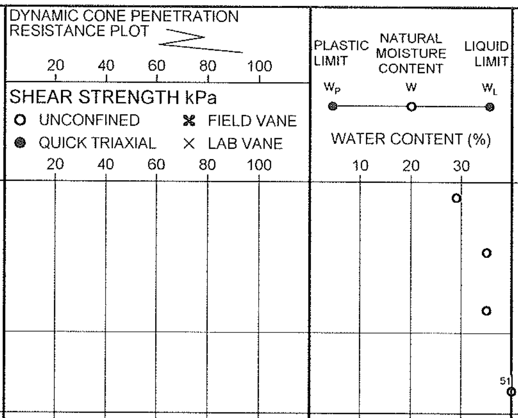
✖ 3, × 3 : Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No AH 12-16 1 OF 1 METRIC

W.P. 2188-08-00 LOCATION Hwy 9 Holland Canal N: 4 875 308 E: 292 840 ORIGINATED BY JF
DIST HWY 9 BOREHOLE TYPE Hand Augers COMPILED BY JF
DATUM Geodetic DATE 2012 04 24 - 2012 04 24 CHECKED BY CM



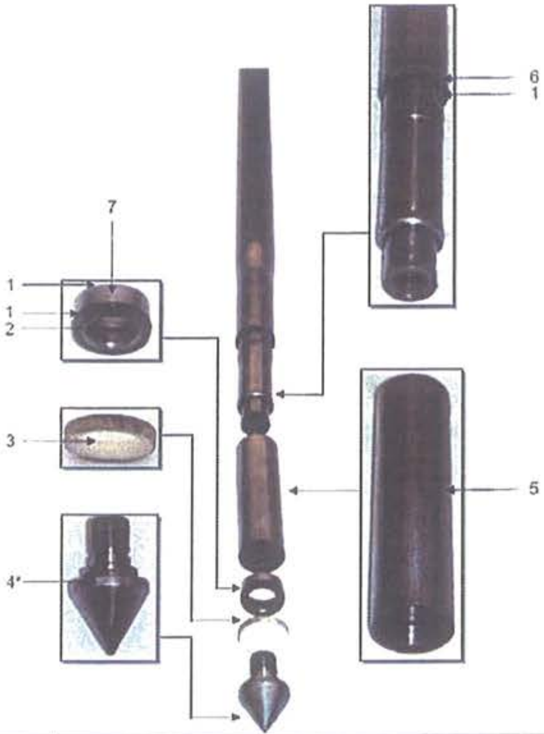
SOIL PROFILE			SAMPLES						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
219.0	Creek bed								
0.0 218.8	FILL: Dark grey silty sand with gravel	▨	1	BS					
0.2	FILL: Grey to dark grey sandy silt	▨	2	BS					
218.2 0.8	ORGANIC SILT (muck) (OL)		3	BS					
	Soft		4	BS			218		
	Black, wet		5	BS					Org M = 4%
217.5 1.5	End of Augerhole								

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

CPT Equipment and Details

Stantec Limited's standard method for cone penetration testing (CPT) uses a Vertek 4579 Digital Piezocone (serial number 2659.109) with a cone area of 10 cm^2 (1.5 in^2) and a mass of 14500 kg (32,000 lb). It is capable of recording the following parameters:

- Tip Resistance, q_c
- Sleeve Friction, f_s (Side Friction)
- Pore Water Pressure, u_2
- Shear Wave Arrival Time
- Compression Wave Arrival Time
- Inclination
- Temperature



Item	Description
1	O-Ring
2	O-Ring
3	Piezo filter, saturated in de-aired silicone oil
4	Piezo tip
*	Standard tip (For use without piezo filter)
5	Friction Sleeve
6	X-Ring
7	Retainer Ring

The CPTu unit can be used to interpret subsurface stratigraphy. The piezocone is pushed at a rate of 2 cm/s, with a drill rig providing the thrust and reaction force. The piezocone measures force in two locations – at the tip of the penetrometer, as well as along the sleeve. The tip load cell, which measures tip resistance (q_c) has a range of 100 kN (22,000 lb) and an accuracy of 0.2%. Other penetrometer specifications include:

- Cone Area: 10 cm^2
- Net Area Ratio: 0.83
- Zero Drift: 0.006 %FS/degF
- Linearity: 0.10%FS (max)
- Overload Cap (%): 150

A load cell along the sleeve measures sleeve friction (f_s) with a range of 20 kN (4,400 lb) and an accuracy of 0.2%. Further specifications include:

- Sleeve Area: 150 cm^2
- Net Area Ratio: 1.00
- Zero Drift: 0.003 %FS/degF
- Linearity: 0.25 %FS (max)
- Overload Cap (%) 150

A piezofilter, saturated in de-aired silicone oil, acts as a pore pressure transducer. This instrument, which is situated behind the cone of the penetrometer (commonly referred to as position u_2), has a standard range of 3.5 MPA (500 psi) and an accuracy of 0.5%. Further specifications are as follows:

- Burst Pressure: 150 %
- Rise Time (10-90%) <1 ms
- Zero Drift: 0.03 %FS/degF
- Static Error Band: 0.03 %FS (max)

The built-in inclinometer has a range of ± 15°, and an accuracy of 1°.

Terminology Used on SCPTu and CPTu Records

Key Terminology and Principles

***SCPTu*:**

- Seismic Piezocone (SCPTu);
- A piezocone (CPTu) is an enhanced cone penetration test (CPT) probe that is able to measure porewater pressure (u);
- A seismic piezocone (SCPTu) is further enhanced to measure surface generated compression and shear waves at depth; used to define the shear wave velocity of soils.

***Equipment Type and Governing Standard*:**

- 10 cm^2 seismic piezocone;
- 150 cm^2 friction sleeve;
- manufactured by Applied Research Associates, Inc.;
- ASTM Specification D3441.

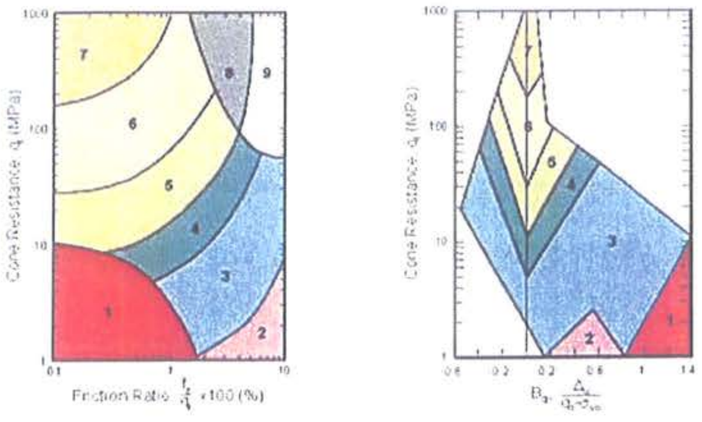
***SCPTu Investigation Objectives*:**

- evaluate soil type and soil stratigraphy;
- estimate the relative density of granular soils and in situ undrained shear strength of cohesive soils.

***Soil Behavior Type (SBT)*:**

- The SBT is selected based on a soil's response to cone penetration, which is different from an explicit soil type defined by specified laboratory testing procedures, but is normally what the geotechnical engineer requires for design purposes.
- The SBT can be classified on the basis of the soil friction ratio, F_r; ratio between the side shear on the friction sleeve and cone tip resistance.
- The SBT can also be classified on the basis of the normalized pore pressure, B_q; a function of the pore water response to penetration and the cone tip resistance.
- The "CPTu Soil Behavior Type Legend" used for this project is presented below.

CPTu Soil Behavior Type Legend
(Robertson et al. 1990)



Zone		Soil Behavior Type
1	🟥	Sensitive, Fine Grained
2	▨	Organic Soils-Peats
3	🟦	Clays; Clay to Silty Clay
4	🟦	Silt Mixtures; Clayey Silt to Silty Clay
5	🟨	Sand Mixtures; Silty Sand to Sandy Silt
6	🟨	Sands; Clean Sands to Silty Sands
7	🟨	Gravelly Sand to Sand
8	▩	Very Stiff Sand to Clayey Sand*
9	☐	Very Stiff Fine Grained*

*Overconsolidated or Cemented

Terminology and Key Engineering Relationships

Parameter	Description	Symbol/Equation
Depth/Elevation	Measured at the centroid of the sensor	
Sleeve Stress	Measured friction stress on the friction sleeve located above the cone tip	f_s
Tip Stress, Uncorrected	Measured compression stress on the cone tip surface	q_c
Corrected Tip Stress	Tip stress, corrected for probe geometry	$q_t = q_c + u_2 \cdot (1-a)$ *where a is a geometry based ratio relating the diameters of the inner load cell and the cone*
Ratio (%)	Friction ratio	$R_f = \frac{f_s}{q_t} \cdot 100\%$
In situ Pore Pressure	In situ equilibrium or static value	u_0
Measured Pore Pressure	Penetration pore pressure value	u_2
Overburden Stress		σ_{vo}
Effective Overburden Stress		$\sigma'_{vo} = \sigma_{vo} - u_o$
Normalized Tip Stress		$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}}$
Normalized Friction Ratio		$F_r = \frac{f_s}{q_t - \sigma_{vo}}$
Normalized Pore Pressure		$B_q = \frac{\Delta u}{q_t - \sigma_{vo}}$ *where* $\Delta u = u_2 - u_0$

Key References:

T. Lunne, P.K. Robertson, and J.J.M. Powell (1997). "Cone Penetration Testing in Geotechnical Practice"; Spon Press.

P.W. Mayne (1986). "CPT indexing of in situ OCR in Clays"; Proceedings of the ASCE Specialty Conference In Situ '86: Use of In Situ Tests in Geotechnical Engineering, Blacksburg, 780-93, ASCE.

P.K. Robertson and R.G. Campanella (1988). "Guidelines for geotechnical design using CPT and CPTU"; University of British Columbia, Vancouver, Department of Civil Engineering, Soil Mechanics Series 120.

P.K. Robertson (1990) "Soil classification using the cone penetration test", Canadian Geotechnical Journal, Vol. 27, No. 1, pp. 151-158.

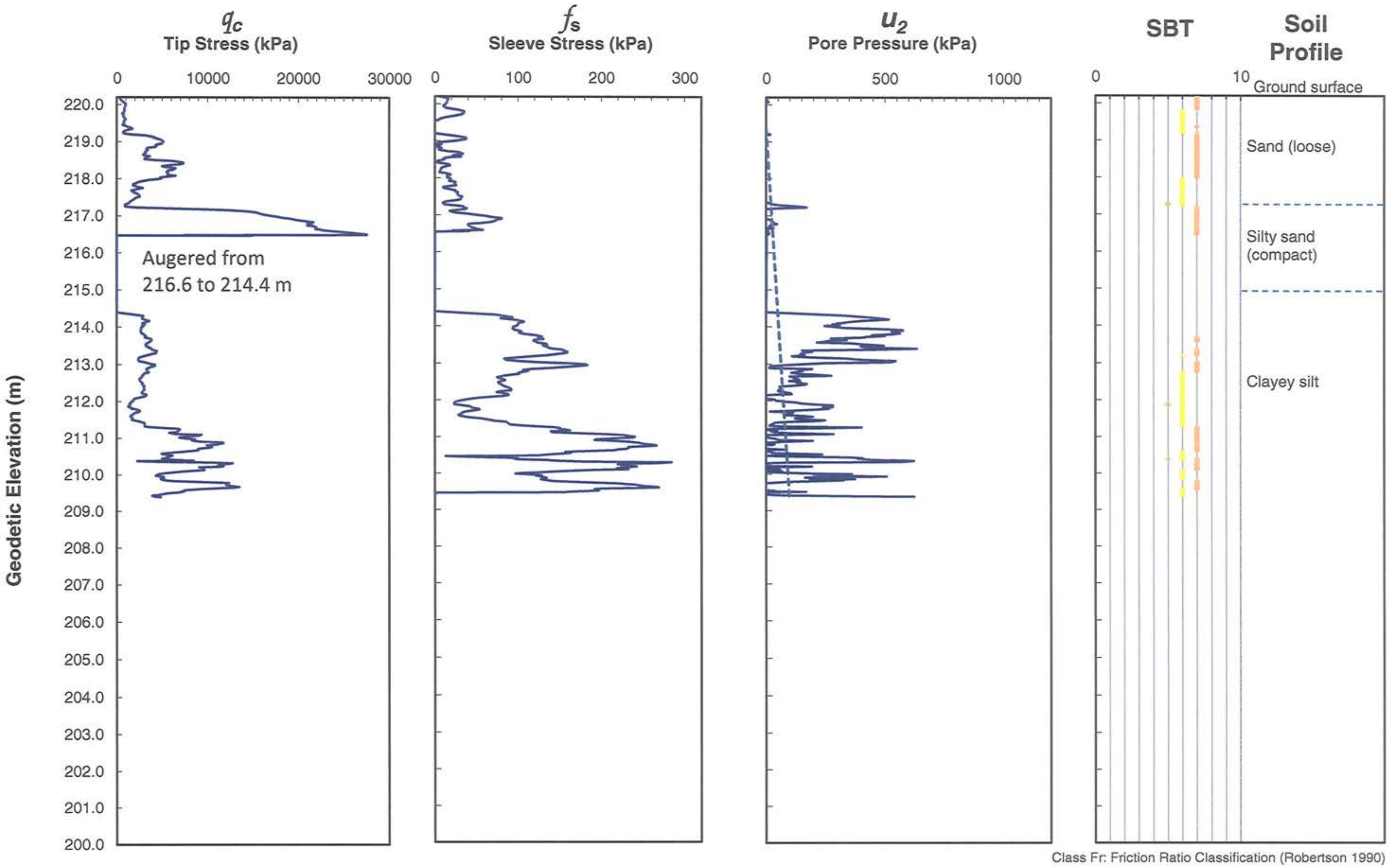
Stantec Consulting Ltd.
2781 Lancaster Road, Suite 200
Ottawa ON, K1B 1A7

Elevation: 220.26 m
SCPTu Start Elevation: 220.26 m
Groundwater Elevation: 219.35 m

Test Date: April 17, 2012
Project No. 165000801

CPT12-4

Client: MTO
Project: Hwy 9 Holland Canal Drainage Bridge



Class Fr: Friction Ratio Classification (Robertson 1990)

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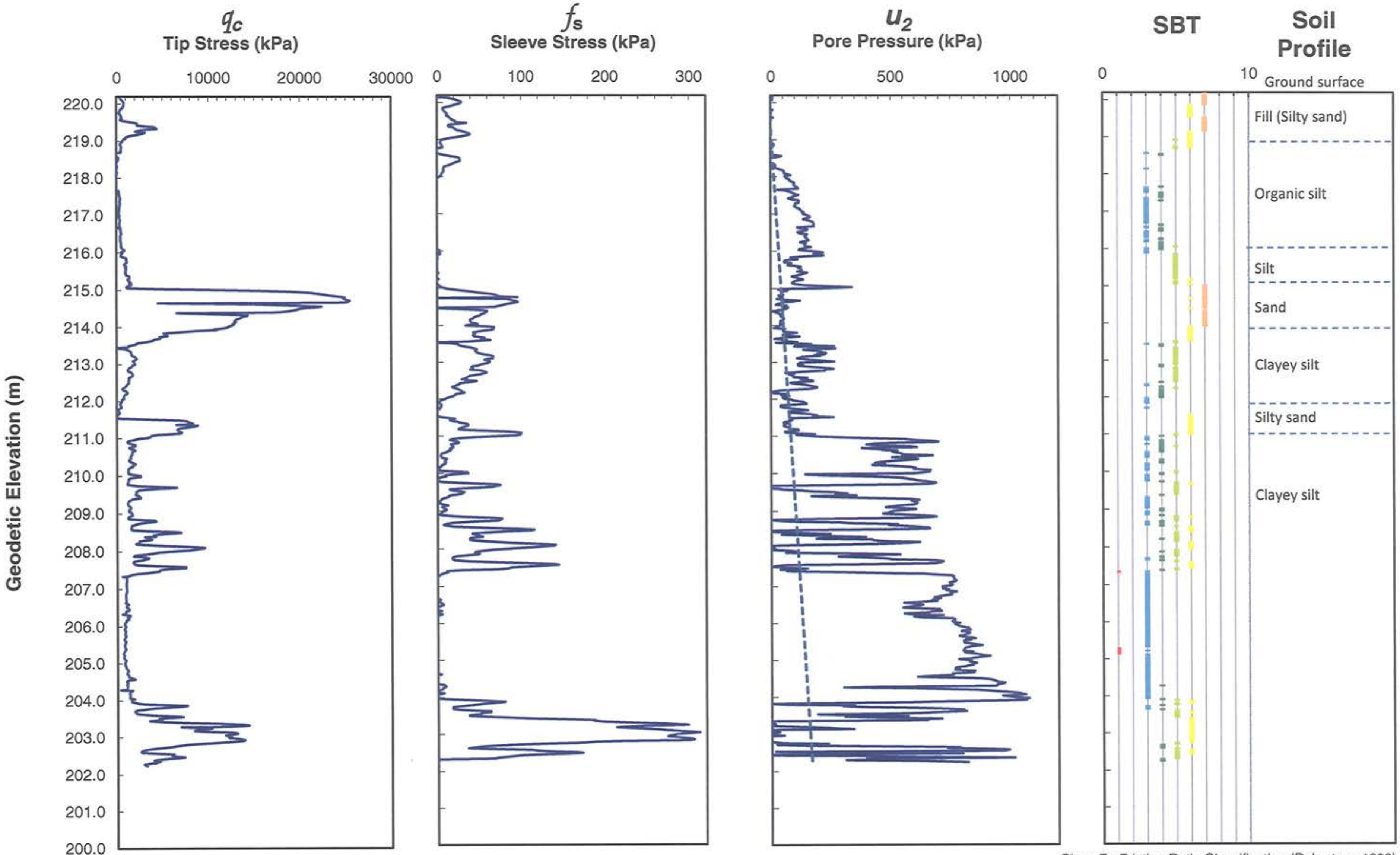
Elevation: 220.20 m
SCPTu Start Elevation: 220.20 m
Groundwater Elevation: 219.28 m

Test Date: March 28, 2012
Project No. 165000801

CPT12-17

Client: MTO
Project: Hwy 9 Holland Canal Drainage Bridge



Class Fr: Friction Ratio Classification (Robertson 1990)

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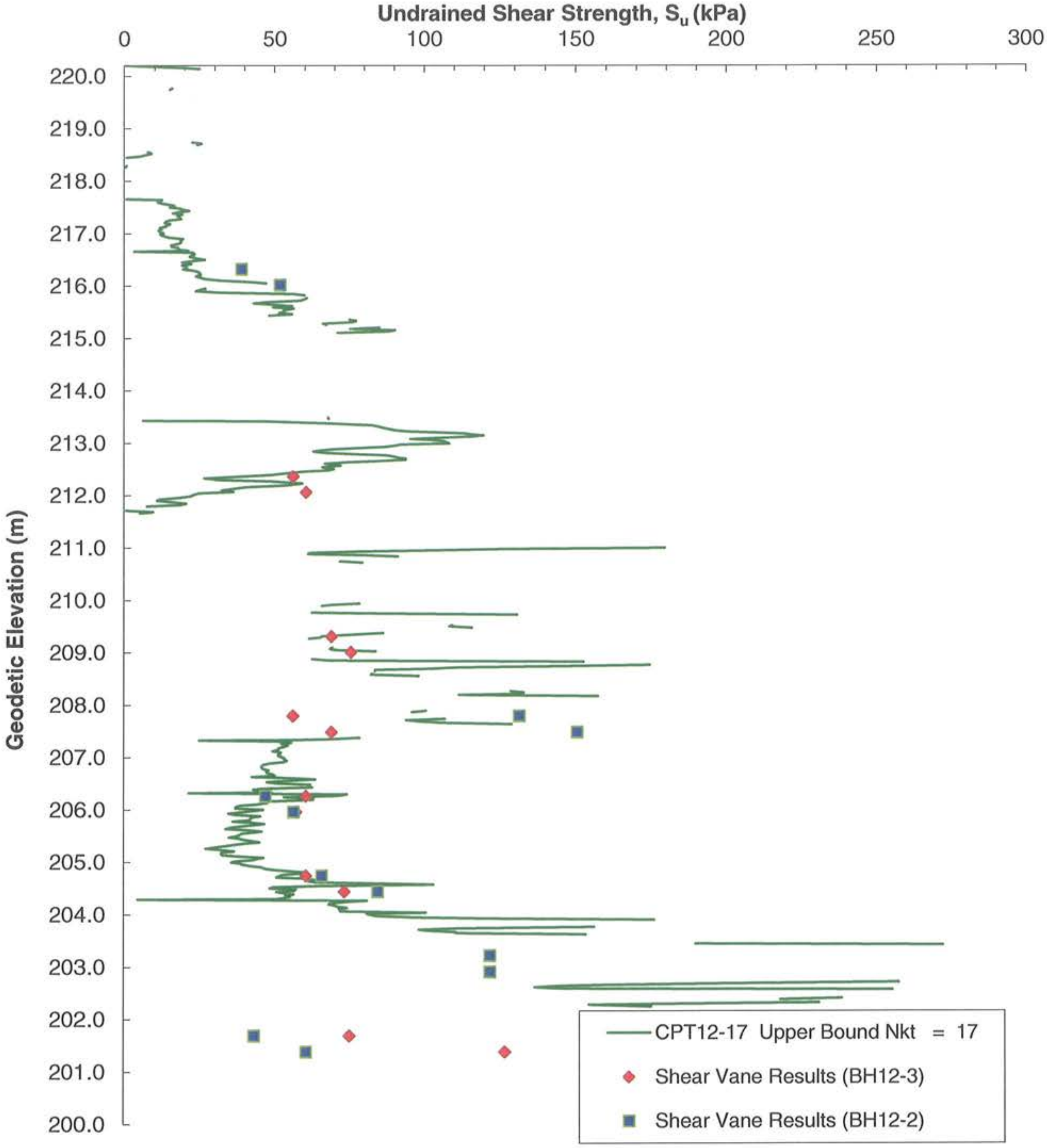
SCPTu RESULTS
Undrained Shear Strength, S_u



Project No. 165000801
CPT12-17

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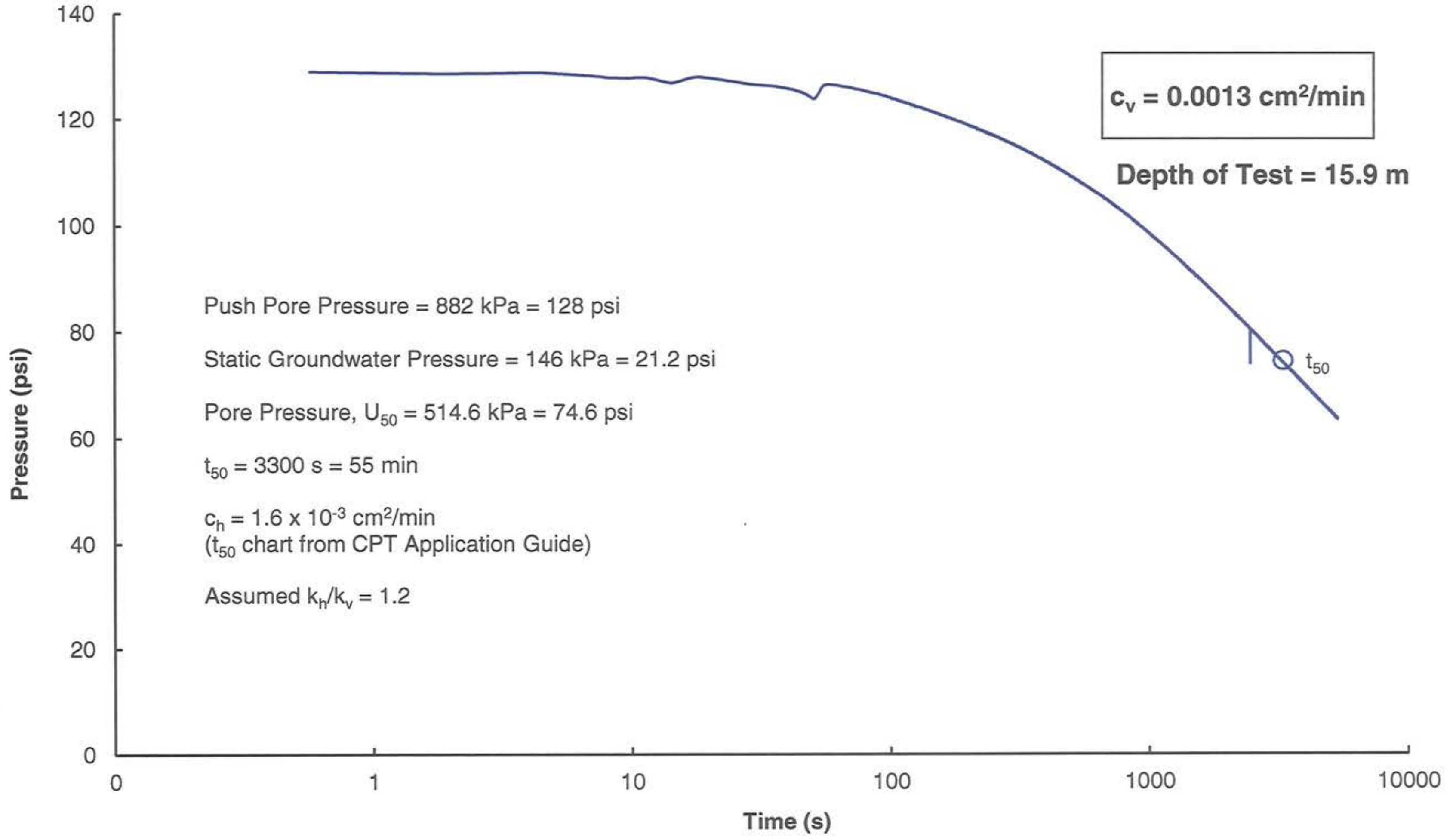
SCPTu DISSIPATION RESULTS
Coefficient of Consolidation



c_v = 0.0013 cm²/min

Depth of Test = 15.9 m

Push Pore Pressure = 882 kPa = 128 psi

Static Groundwater Pressure = 146 kPa = 21.2 psi

Pore Pressure, U_{50} = 514.6 kPa = 74.6 psi

t_{50} = 3300 s = 55 min

$c_h = 1.6 \times 10^{-3}$ cm²/min
(t_{50} chart from CPT Application Guide)

Assumed k_h/k_v = 1.2

Project No. 165000801
CPT12-17

V:\01224\active\other_pc_projects\165000801\CPT\STATIC SCPTu Interpreter V3-11-2 - CPT17.xlsm

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	'N' BLOWS / FT.	c LB. / SQ. FT.	DENSENESS	'N' BLOWS / FT.
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		

P.H.	SAMPLE ADVANCED HYDRAULICALLY
P.M.	SAMPLE ADVANCED MANUALLY

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)	
γ_s	UNIT WEIGHT OF SOLID PARTICLES	
γ_w	UNIT WEIGHT OF WATER	
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)	
γ'	UNIT WEIGHT OF SUBMERGED SOIL	
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$	
e	VOID RATIO	
n	POROSITY	
w	WATER CONTENT	
S_r	DEGREE OF SATURATION	
w_L	LIQUID LIMIT	
w_P	PLASTIC LIMIT	
I_P	PLASTICITY INDEX	
$_s$	SHRINKAGE LIMIT	
I_L	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$	
I_C	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$	
e_{max}	VOID RATIO IN LOOSEST STATE	
e_{min}	VOID RATIO IN DENSEST STATE	
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$	
	RELATIVE DENSITY D_r IS ALSO USED	
h	HYDRAULIC HEAD OR POTENTIAL	
q	RATE OF DISCHARGE	
v	VELOCITY OF FLOW	
i	HYDRAULIC GRADIENT	
k	COEFFICIENT OF PERMEABILITY	
j	SEEPAGE FORCE PER UNIT VOLUME	
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma'}$	
c_v	COEFFICIENT OF CONSOLIDATION	
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma'}$	
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)	
U	DEGREE OF CONSOLIDATION	
τ_f	SHEAR STRENGTH	
c'	EFFECTIVE COHESION INTERCEPT	IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION	
c_u	APPARENT COHESION	IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION	
μ	COEFFICIENT OF FRICTION	
S_t	SENSITIVITY	

GENERAL

π	$= 3{\cdot}1416$
e	BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ε	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS, REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

66-1831

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

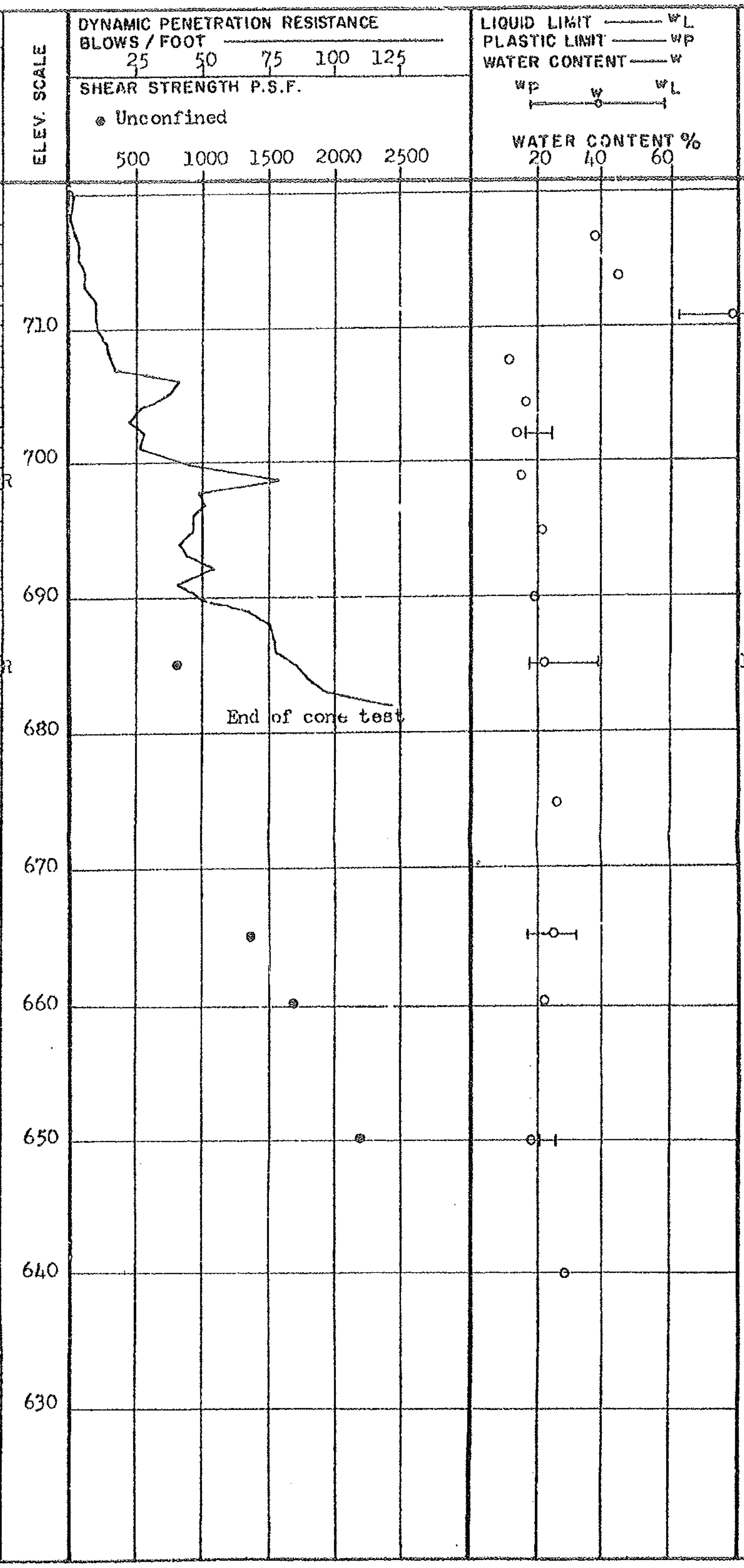
JOB 65-F-114
W.P. 172-65
DATUM Geodetic
LOCATION East Abutment - North Corner.
BORING DATE Oct. 20, 22, 25, 1965.
BOREHOLE TYPE Washbore - NX & BX Casings.
ORIGINATED BY P.P.
COMPILED BY P.P.
CHECKED BY

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT
720.9	Groundlevel	
0.0	Organic Material (Sand, silt and clay) Muck Black	~
707.9 13.0	Sand with layers of clayey silt and some gravel. Brown Loose to compact.	⁚
691.4 29.5	Clayey silt with occasional layers of sand and silt. Grey Firm to v. stiff.	▨
633.9 87.0	Sandy silt Grey V. loose	▥
629.4 91.5	End of borehole.	

SAMPLES

NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
1	SS	2		
2	TW	PM		
3	TW	PM		
4	TW	PM		Gr 22% Sa 72% Si) Cl) 6%
5	SS	10		
6	SS	19		
7	SS	PM.DR		Gr 1% Sa 88% Si) Cl) 11%
8	SS	7		
9	SS	22		
10	TW	PM.DR	127.5	
11	SS	13		
12	TW	PM	123	
13	TW	PM	132	Gr 0% Sa 2% Si 66% Cl 32%
14	TW	32	130	
15	SS	16		
16	SS	6		



68-1031

DEPARTMENT OF HIGHWAYS - ONTARIO

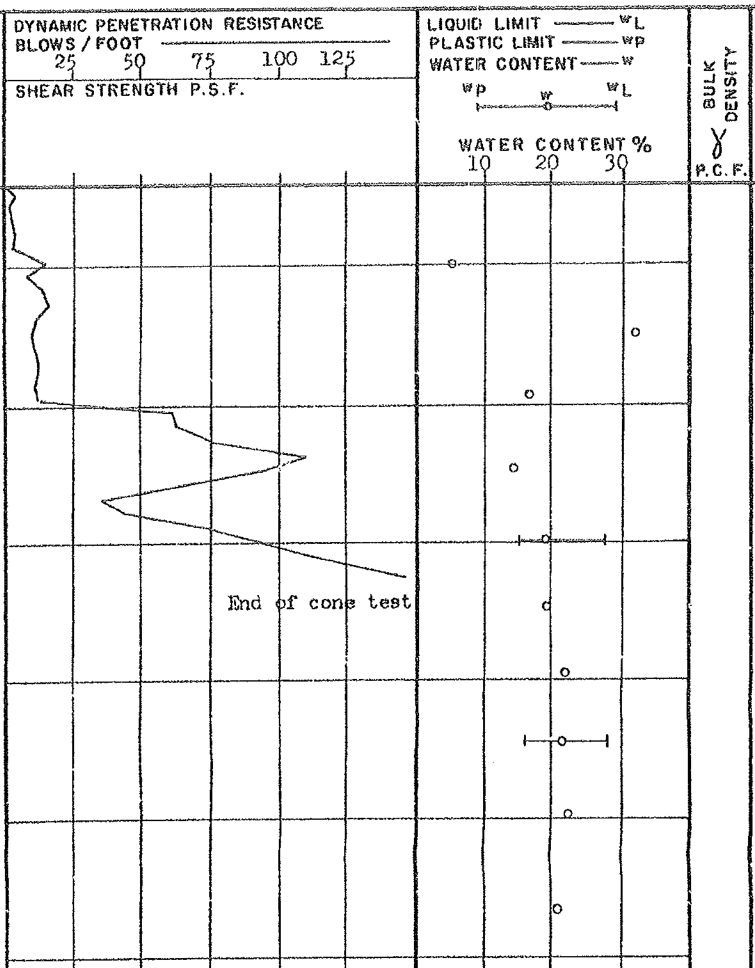
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 65-F-114
W.P. 172-65
DATUM Geodetic
LOCATION West Abutment - North Corner
BORING DATE Oct. 25 & 26, 1965.
BOREHOLE TYPE Washbore - NX Casing.
ORIGINATED BY P.P.
COMPILED BY P.P.
CHECKED BY

SOIL PROFILE			SAMPLES				
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
726.0	Groundlevel						
0.0	Sand Brown V. loose Fill		1	SS	3	720	
717.0 9.0	Sandy silt to silty sand. Greyish brown Loose to compact.		2	SS	9		Gr 0% Sa 12% Si) Cl)88%
			3	SS	29	710	
			4	SS	39		Gr 6% Sa 84% Si) Cl)10%
702.8 23.2	Clayey silt with traces of sand. Grey. Firm to hard.		5	SS	37	700	
			6	SS	29		
			7	SS	22	690	
			8	SS	32		
			9	SS	19	680	Gr 0% Sa 1% Si 58% Cl 41%
			10	SS	30		
			11	SS	30		
673.0 53.0	End of borehole.					670	

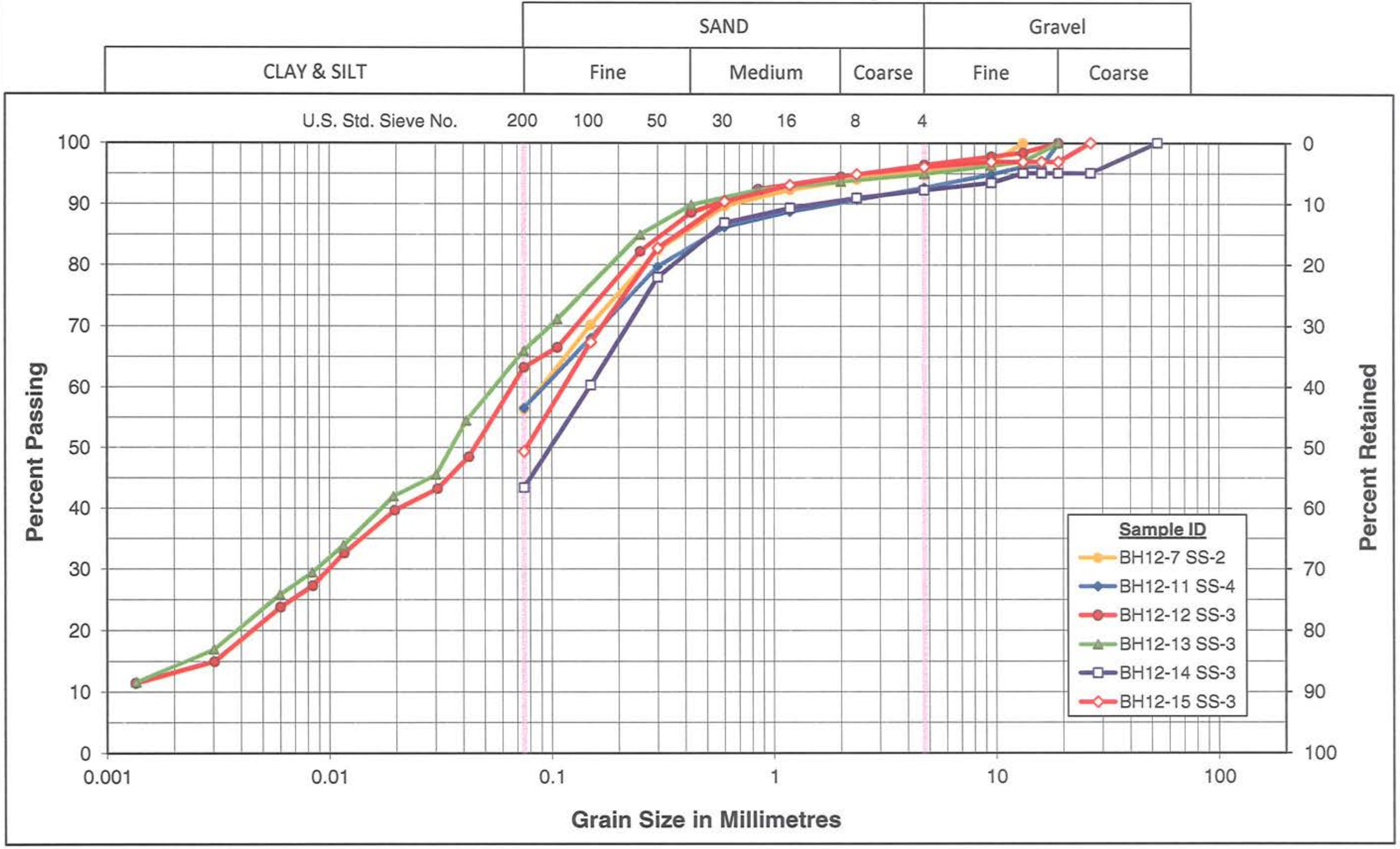


APPENDIX C

Laboratory Test Results

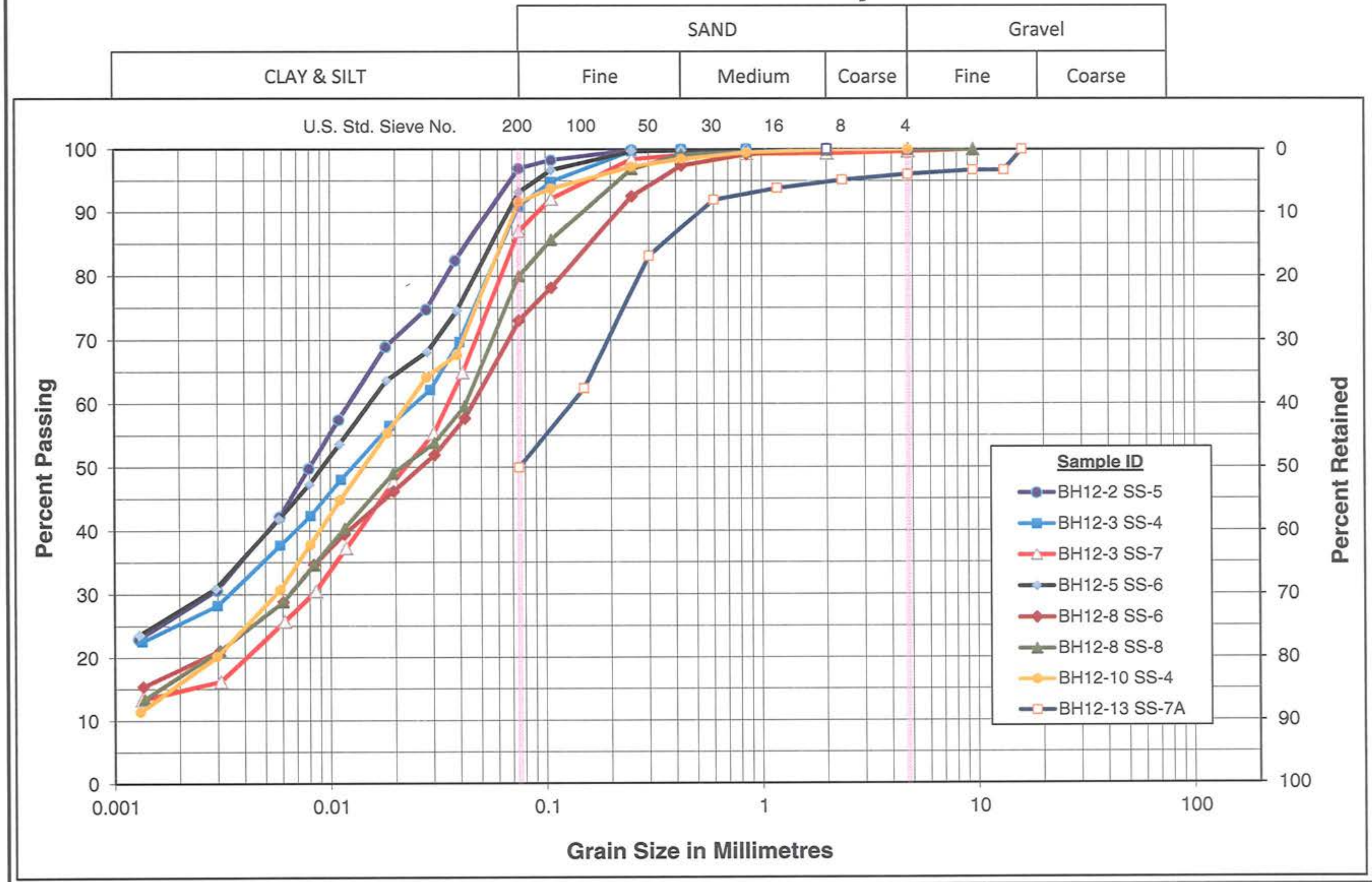
Consolidation Test Results (from Golder)

Unified Soil Classification System



Stantec	GRAIN SIZE DISTRIBUTION	Figure No. 1
	FILL: Silty sand (SM) to sandy silt (ML)	Project No. 165000801 GWP No. 2188-08-00

Unified Soil Classification System



Stantec

GRAIN SIZE DISTRIBUTION

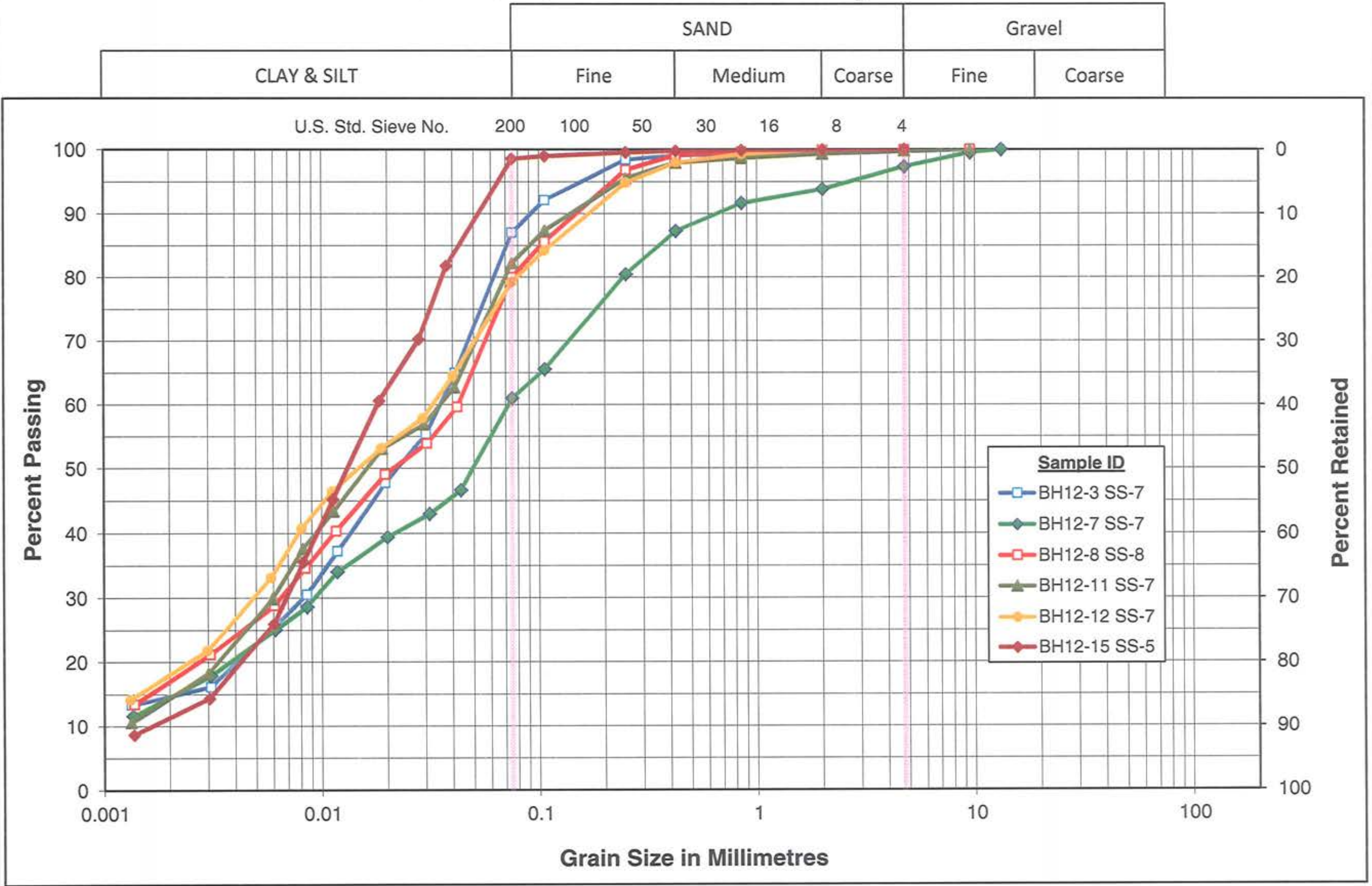
Organic silt (OL) (Muck)

Figure No. 2

Project No. 165000801

GWP No. 2188-08-00

Unified Soil Classification System



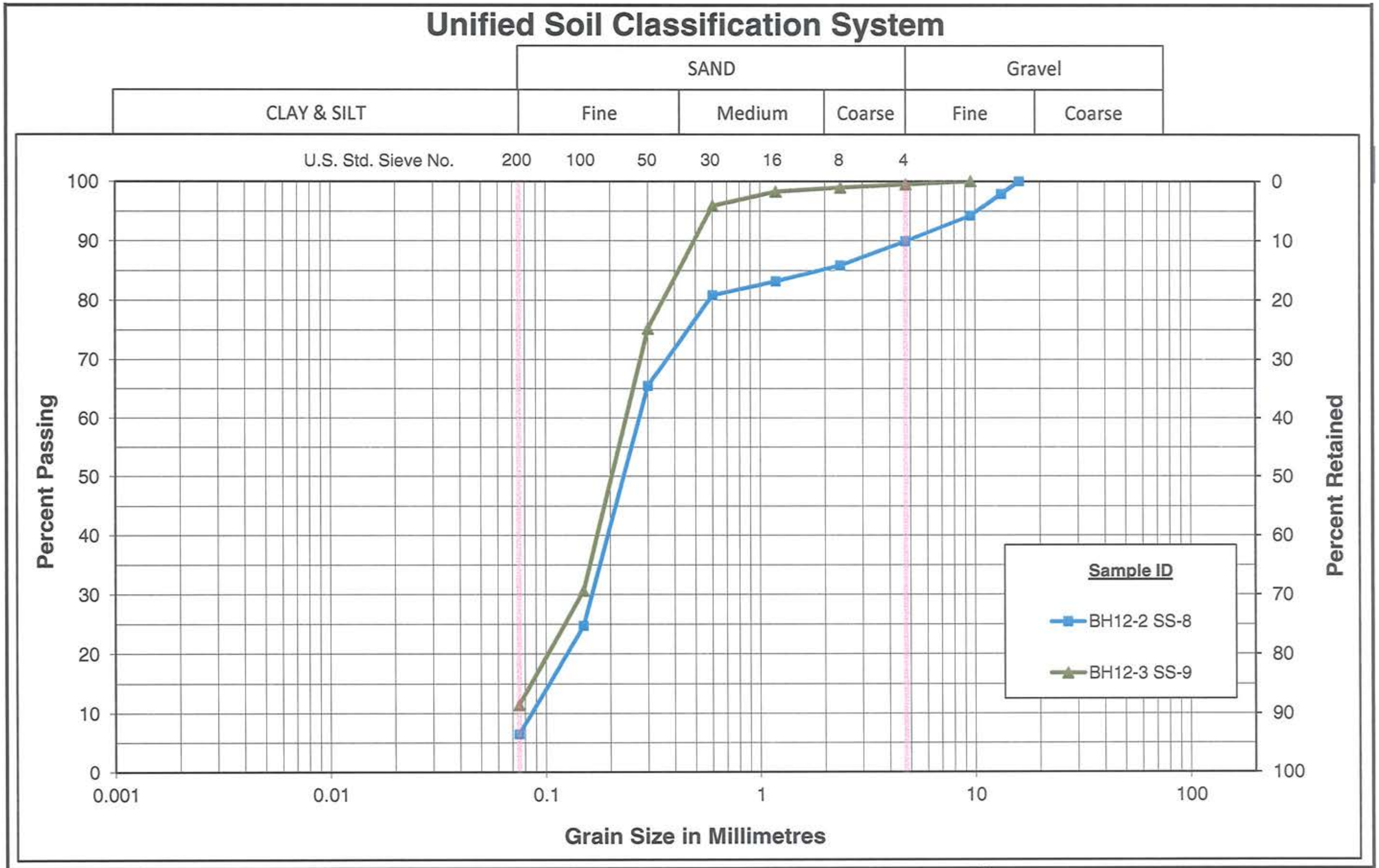
Stantec

GRAIN SIZE DISTRIBUTION

Silt to sandy silt (ML)

Figure No. 3

Project No. 165000801
GWP No. 2188-08-00



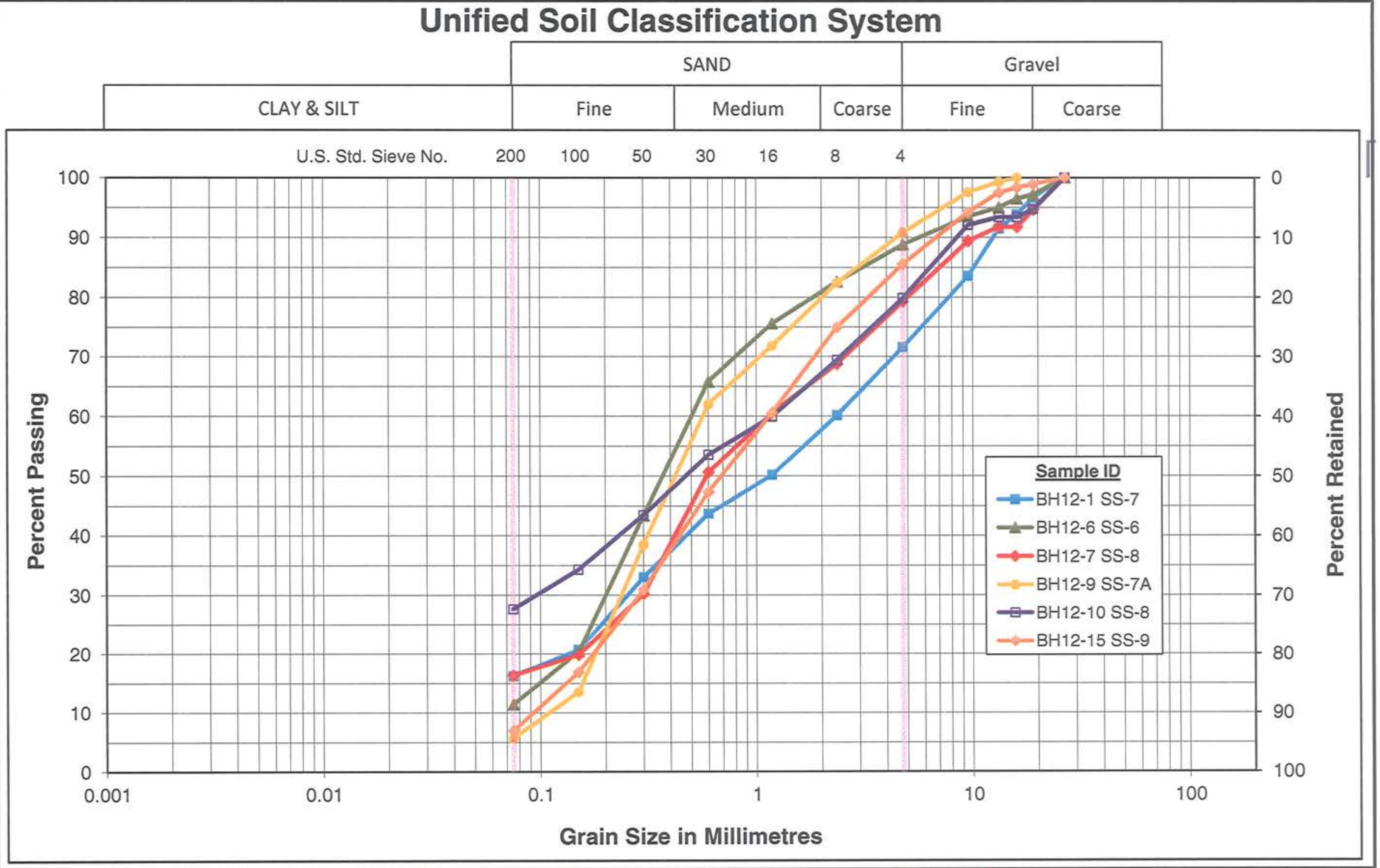
Stantec

GRAIN SIZE DISTRIBUTION

Poorly graded sand with silt (SP-SM)

Figure No. 4

Project No. 165000801
GWP No. 2188-08-00



Stantec

GRAIN SIZE DISTRIBUTION

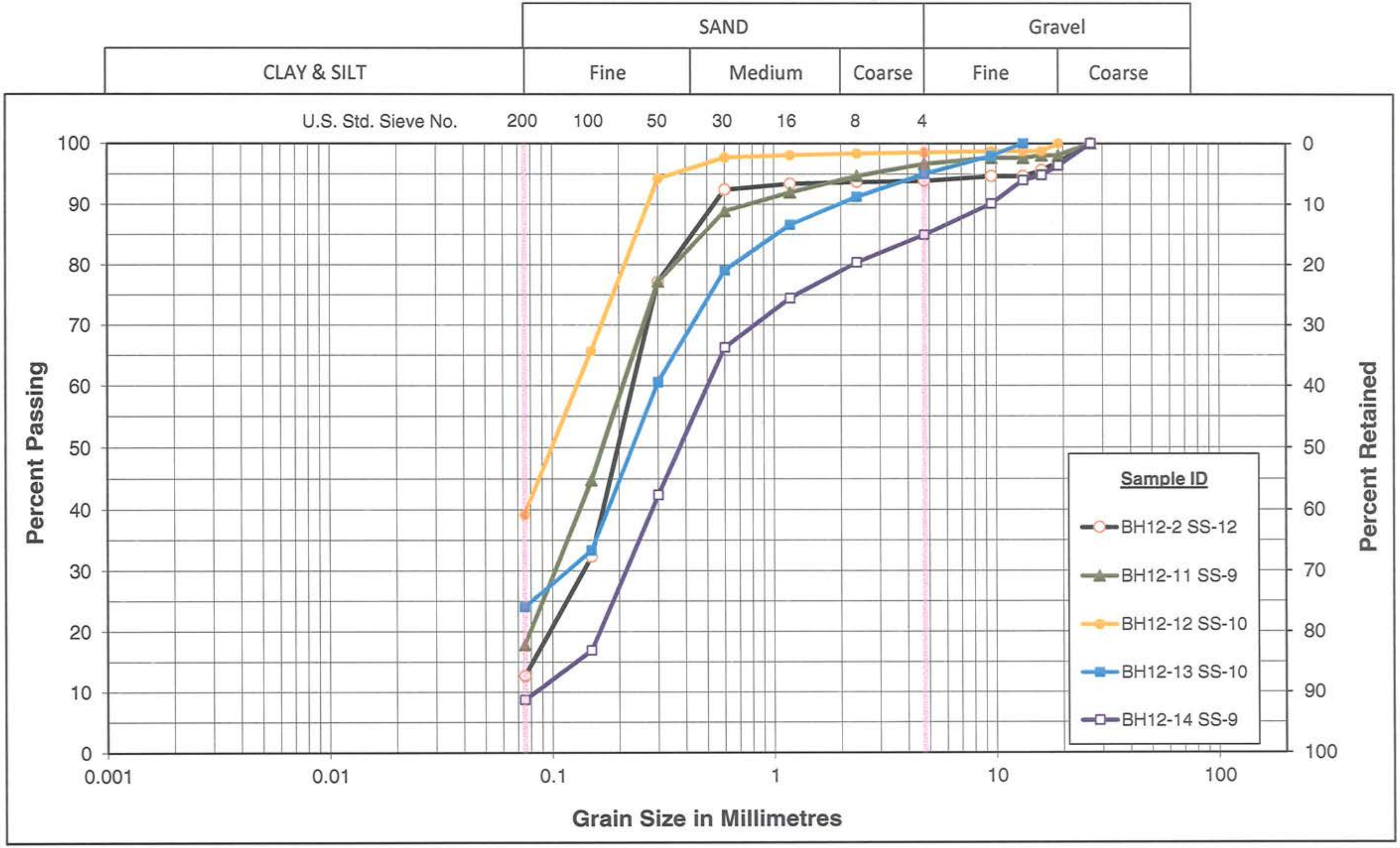
Silty sand (SM) to silty sand with gravel (SM)

Figure No. 5a

Project No. 165000801

GWP No. 2188-08-00

Unified Soil Classification System



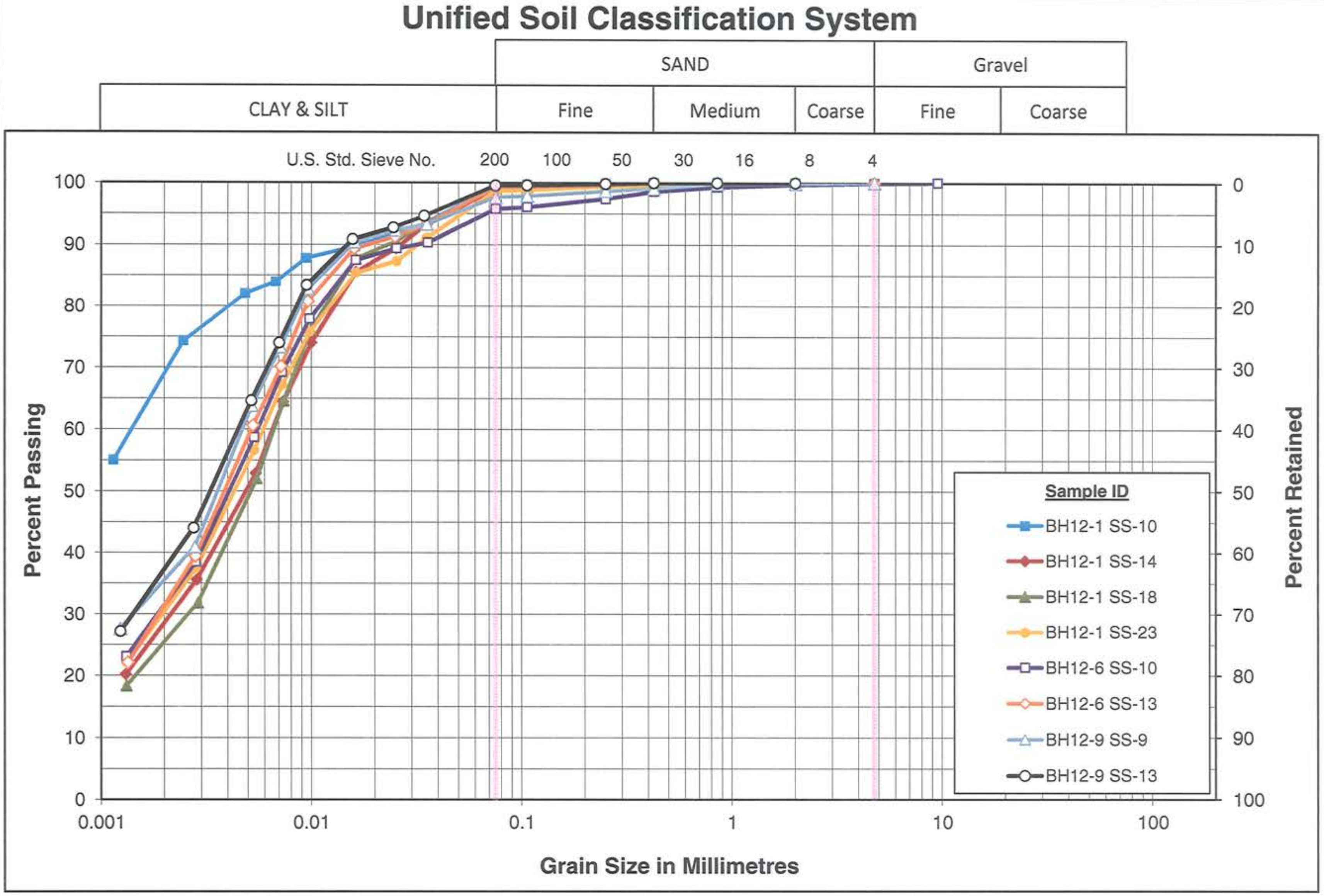
Stantec

GRAIN SIZE DISTRIBUTION

Silty sand (SM)

Figure No. 5b

Project No. 165000801
GWP No. 2188-08-00



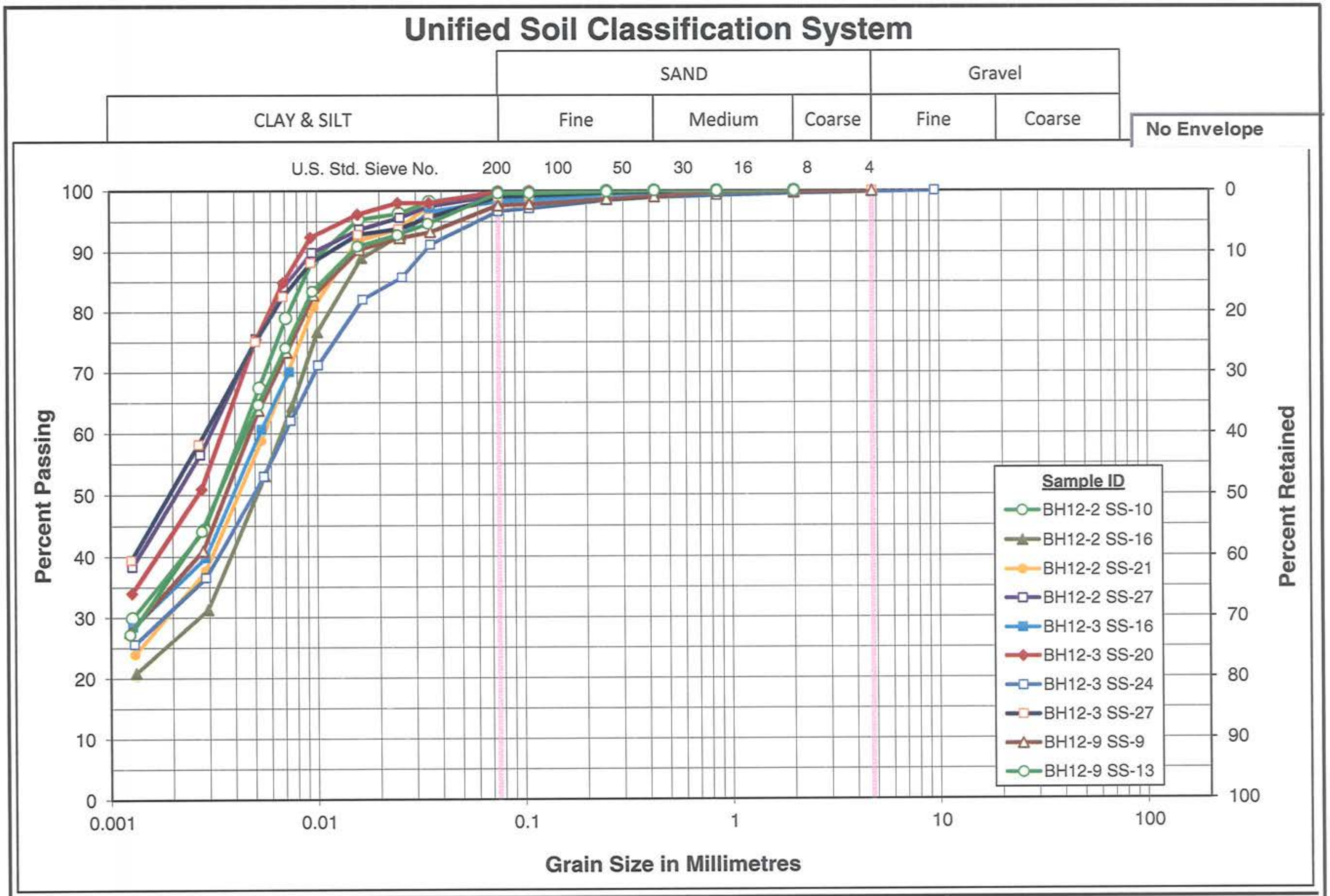
Stantec

GRAIN SIZE DISTRIBUTION

Clayey silt (CL)

Figure No. 6a

Project No. 165000801
GWP No. 2188-08-00



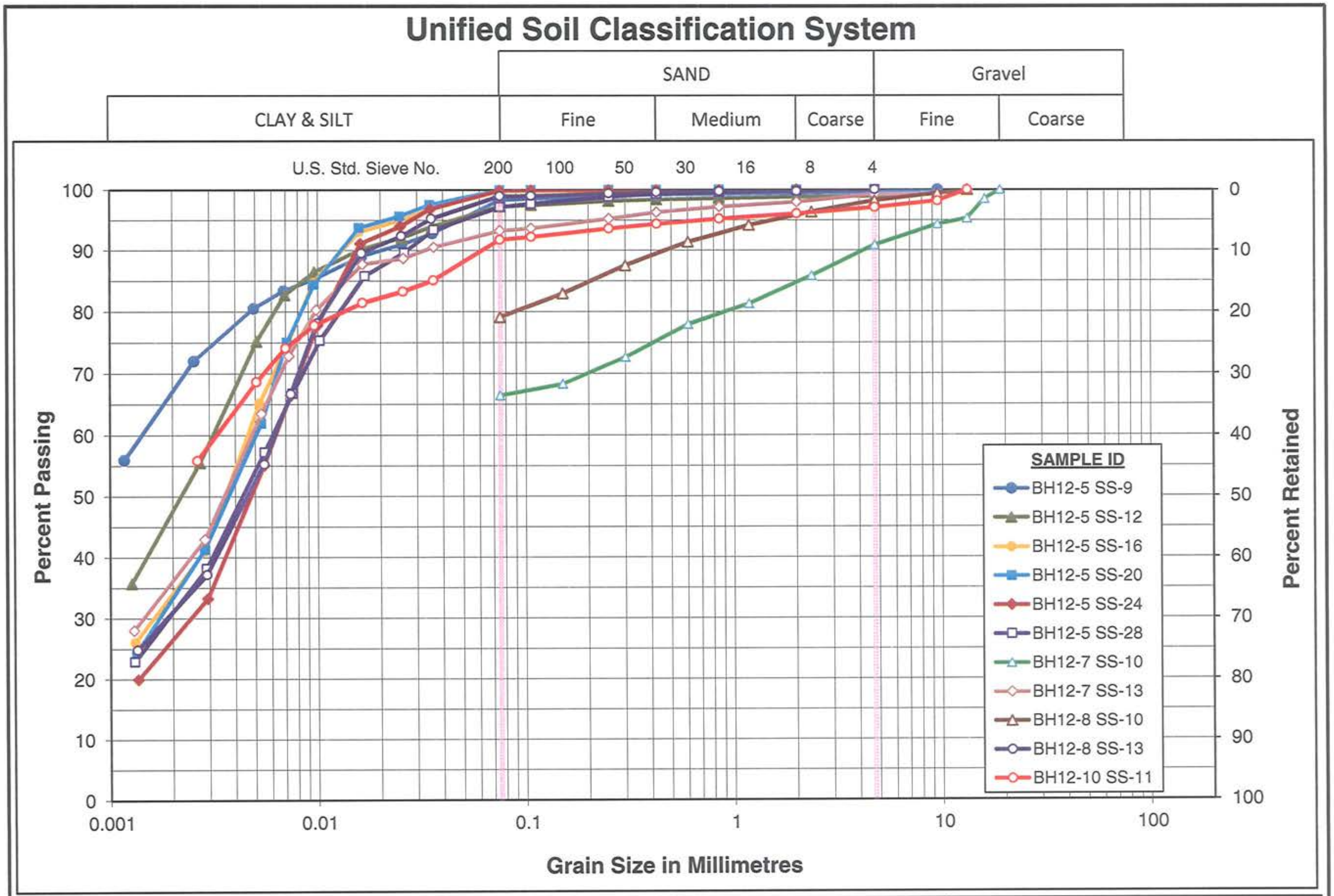
Stantec

GRAIN SIZE DISTRIBUTION

Clayey silt (CL)

Figure No. 6b

Project No. 165000801
GWP No. 2188-08-00



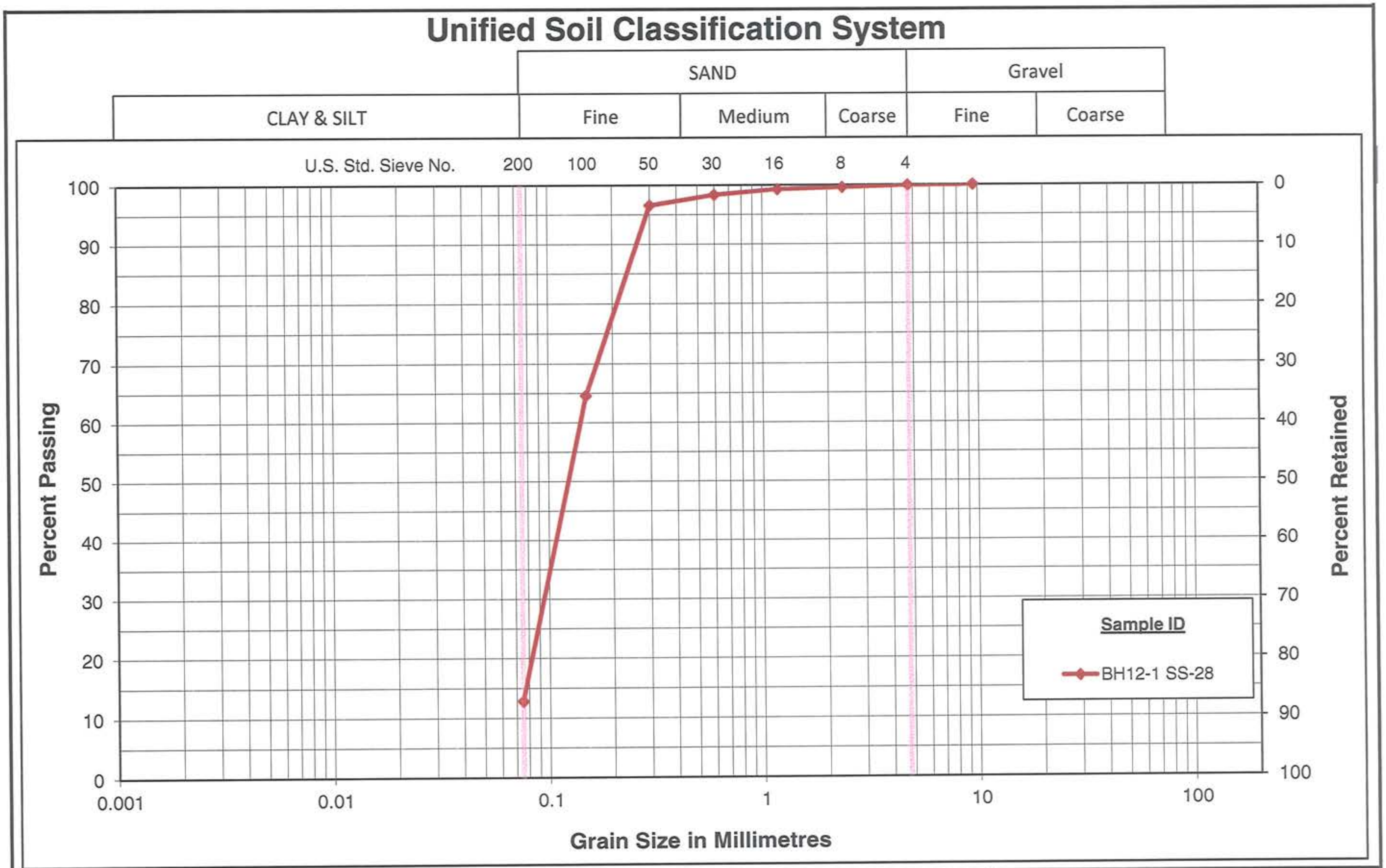
Stantec

GRAIN SIZE DISTRIBUTION

Clayey silt (CL)

Figure No. 6c

Project No. 165000801
GWP No. 2188-08-00

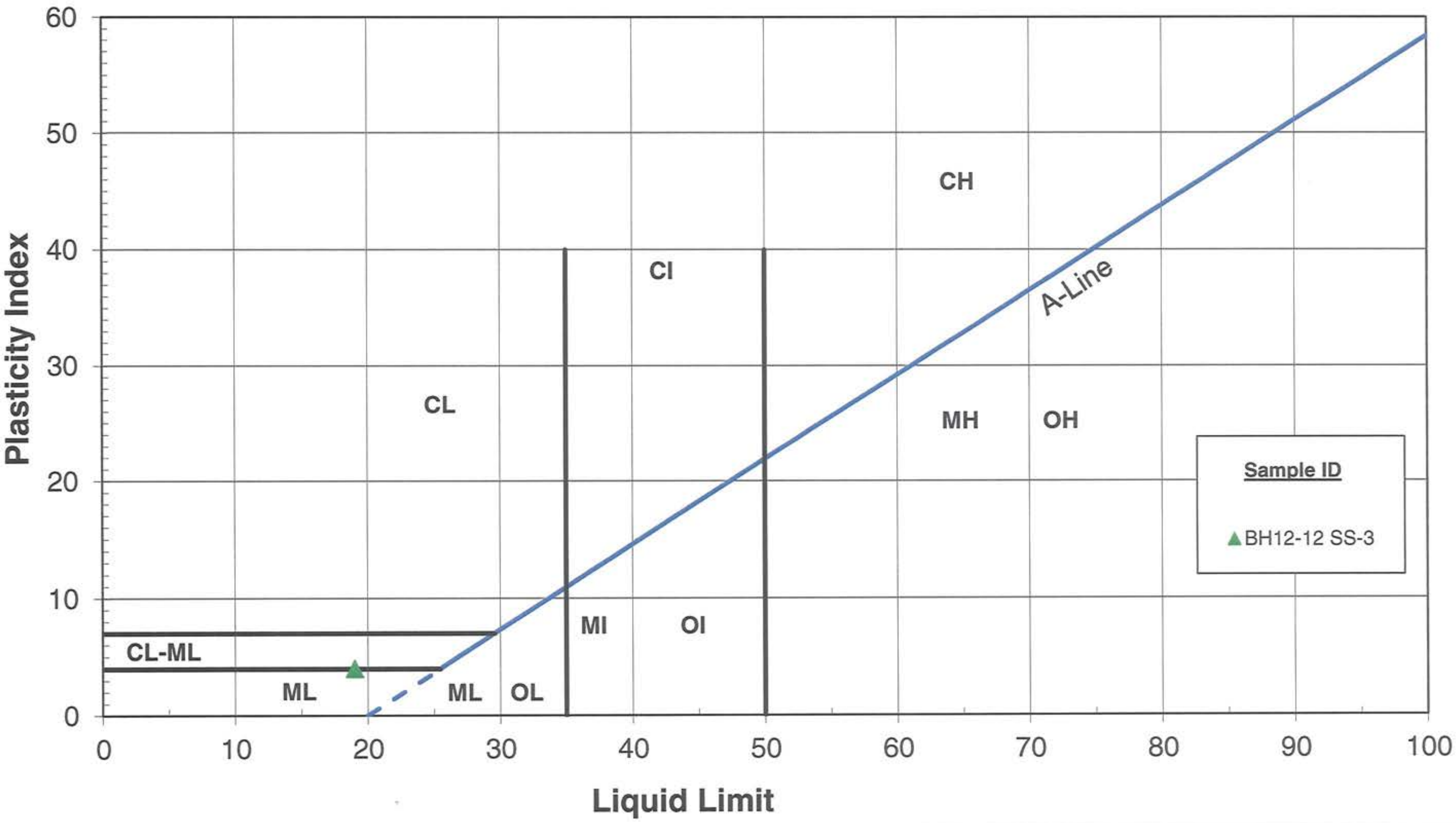


GRAIN SIZE DISTRIBUTION

Silty sand (SM)

Figure No. 7

Project No. 165000801
GWP No. 2188-08-00

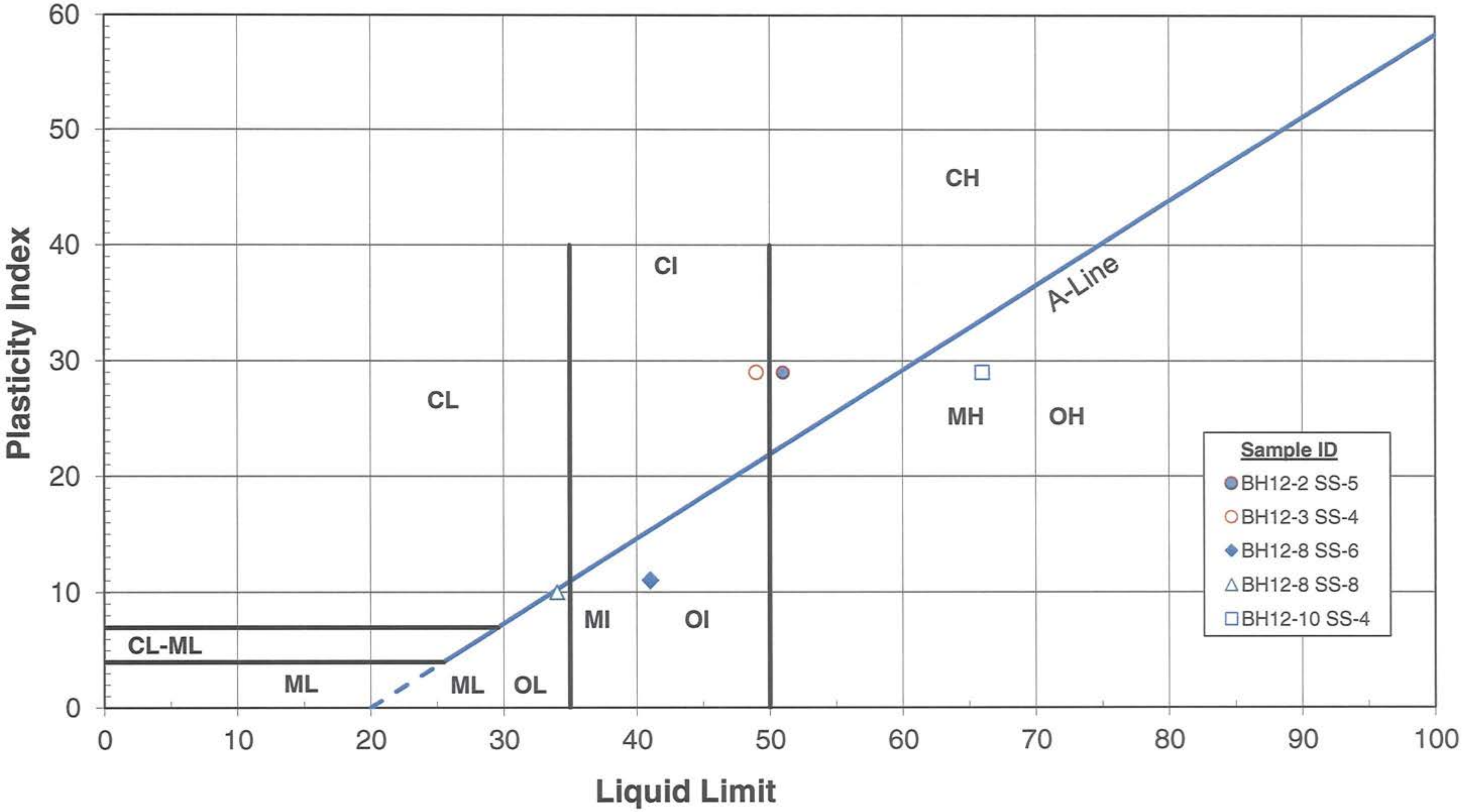


PLASTICITY CHART

Fill: Silt

Figure No. 8

Project No. 165000801
GWP No. 2188-08-00



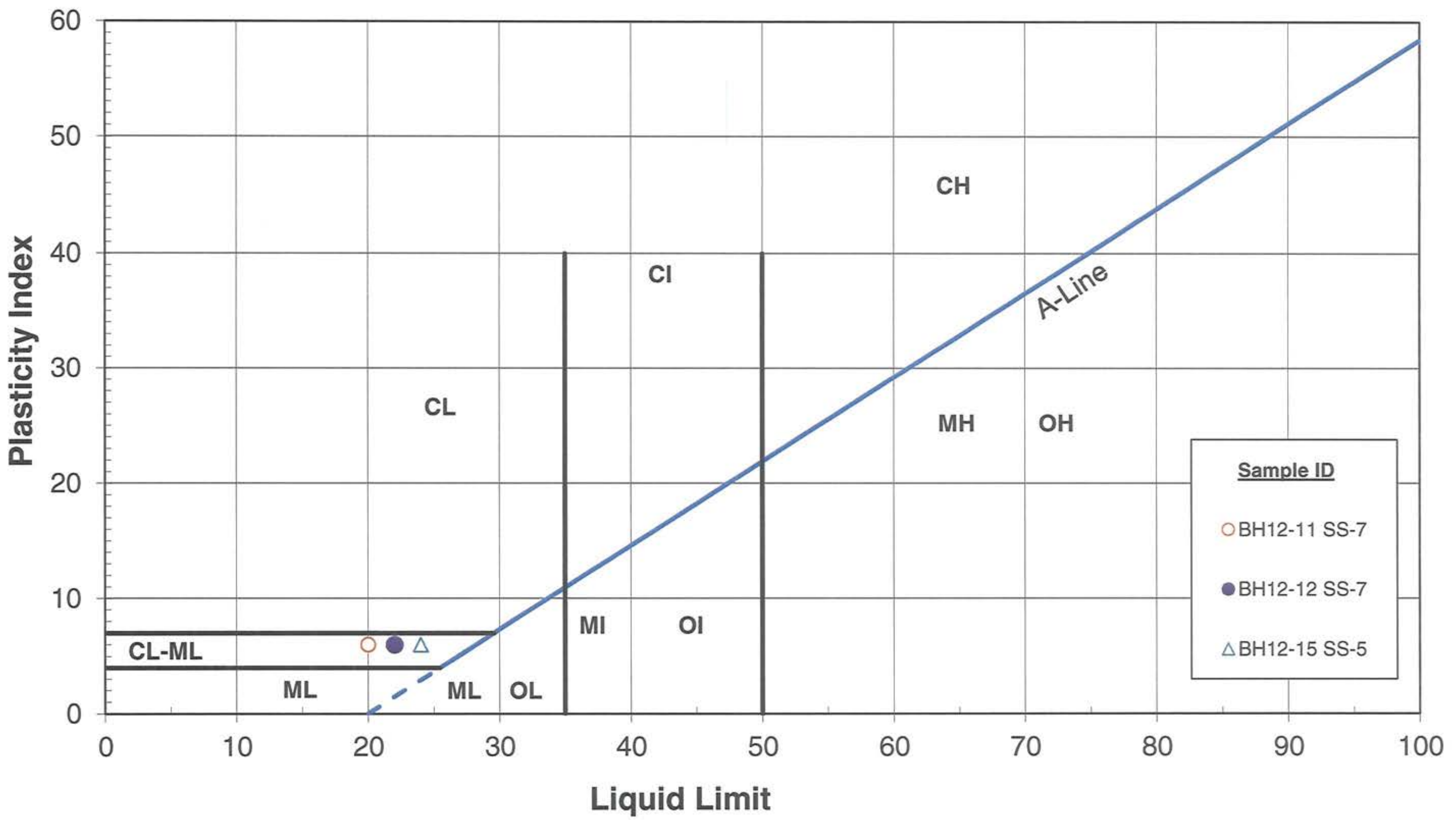
Stantec

PLASTICITY CHART

Organic silt (Muck)

Figure No. 9

Project No. 165000801
GWP No. 2188-08-00



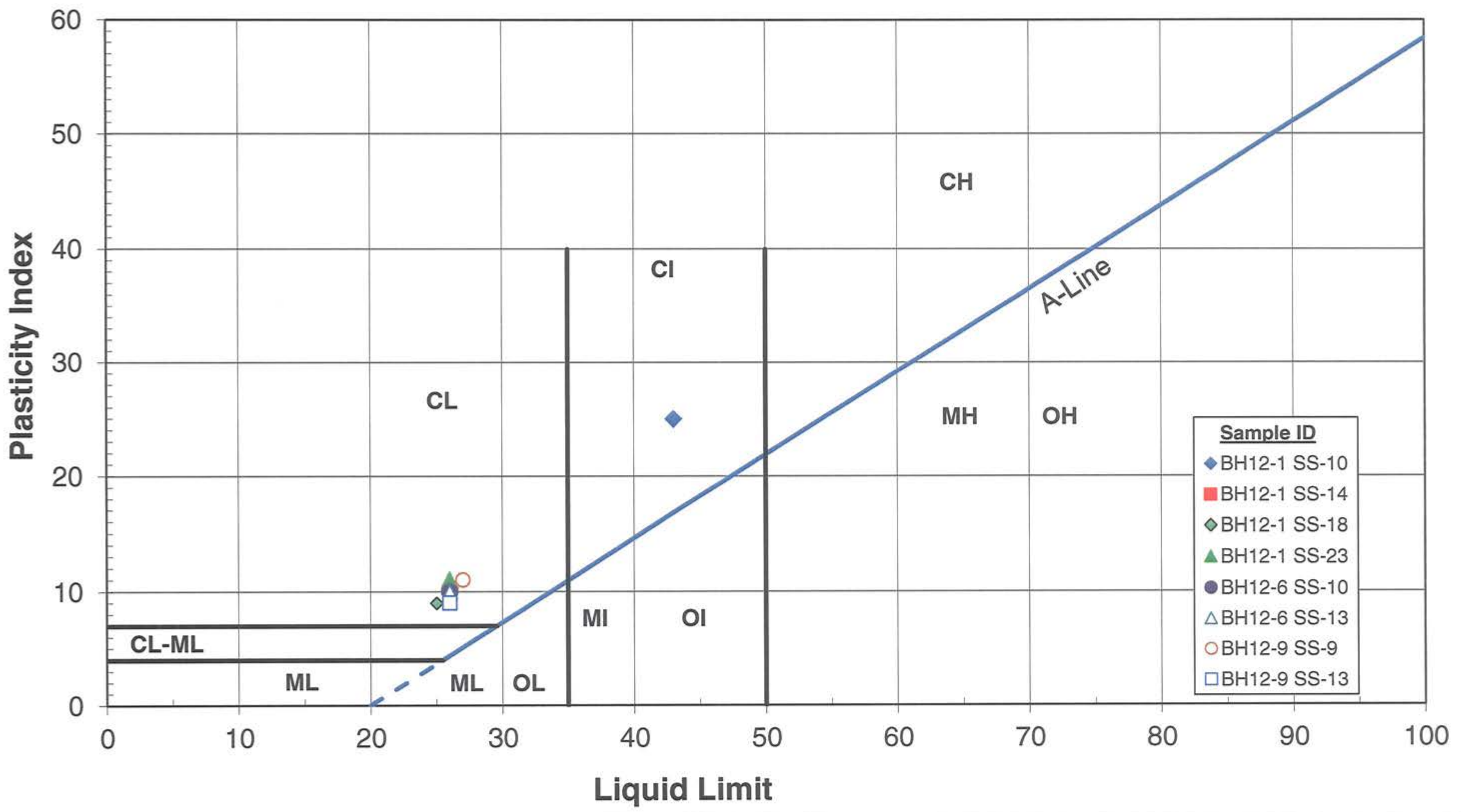
Stantec

PLASTICITY CHART

Silt

Figure No. 10

Project No. 165000801
GWP No. 2188-08-00



Stantec

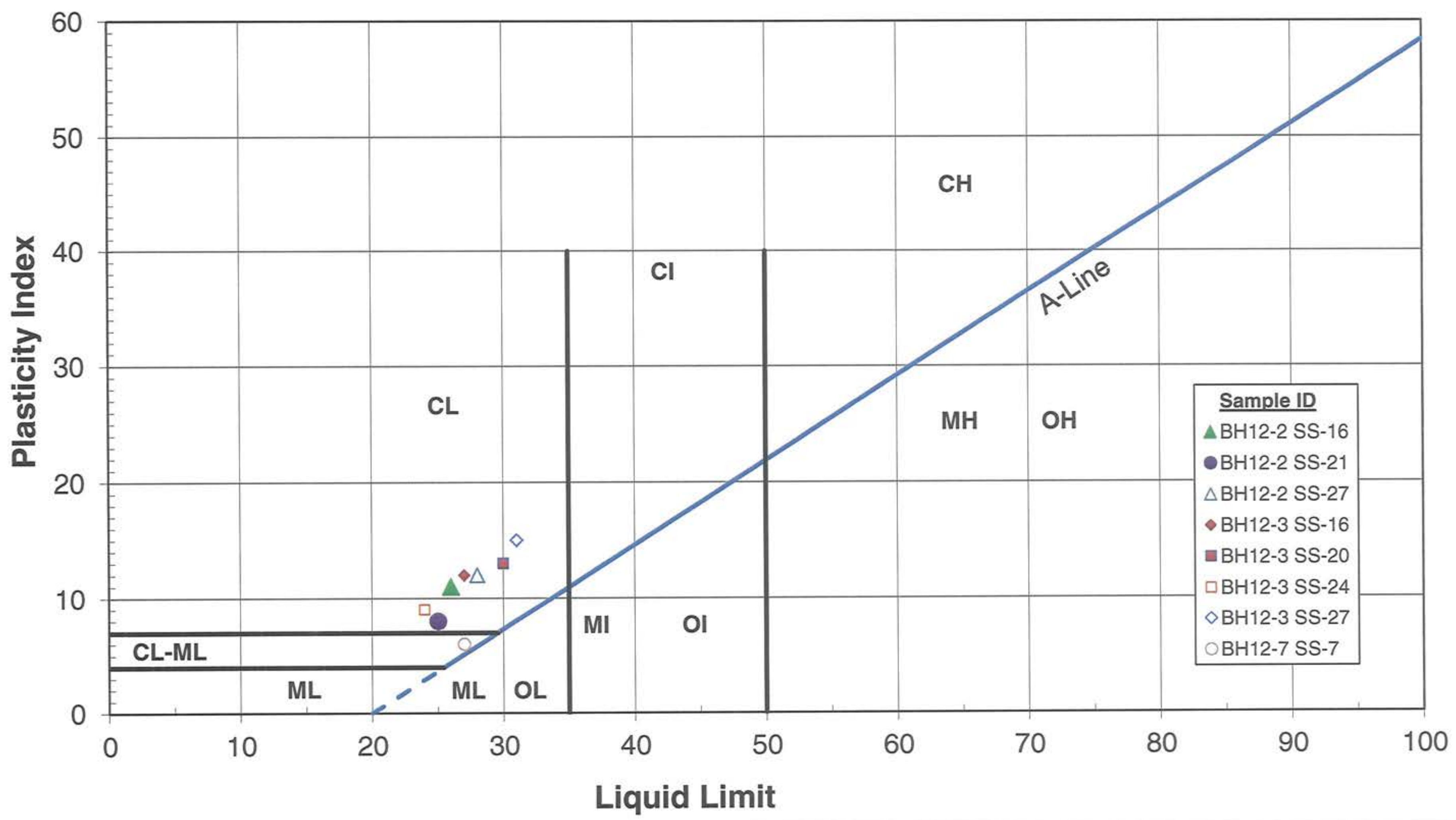
PLASTICITY CHART

Clayey silt

Figure No. 11a

Project No. 165000801

GWP No. 2188-08-00



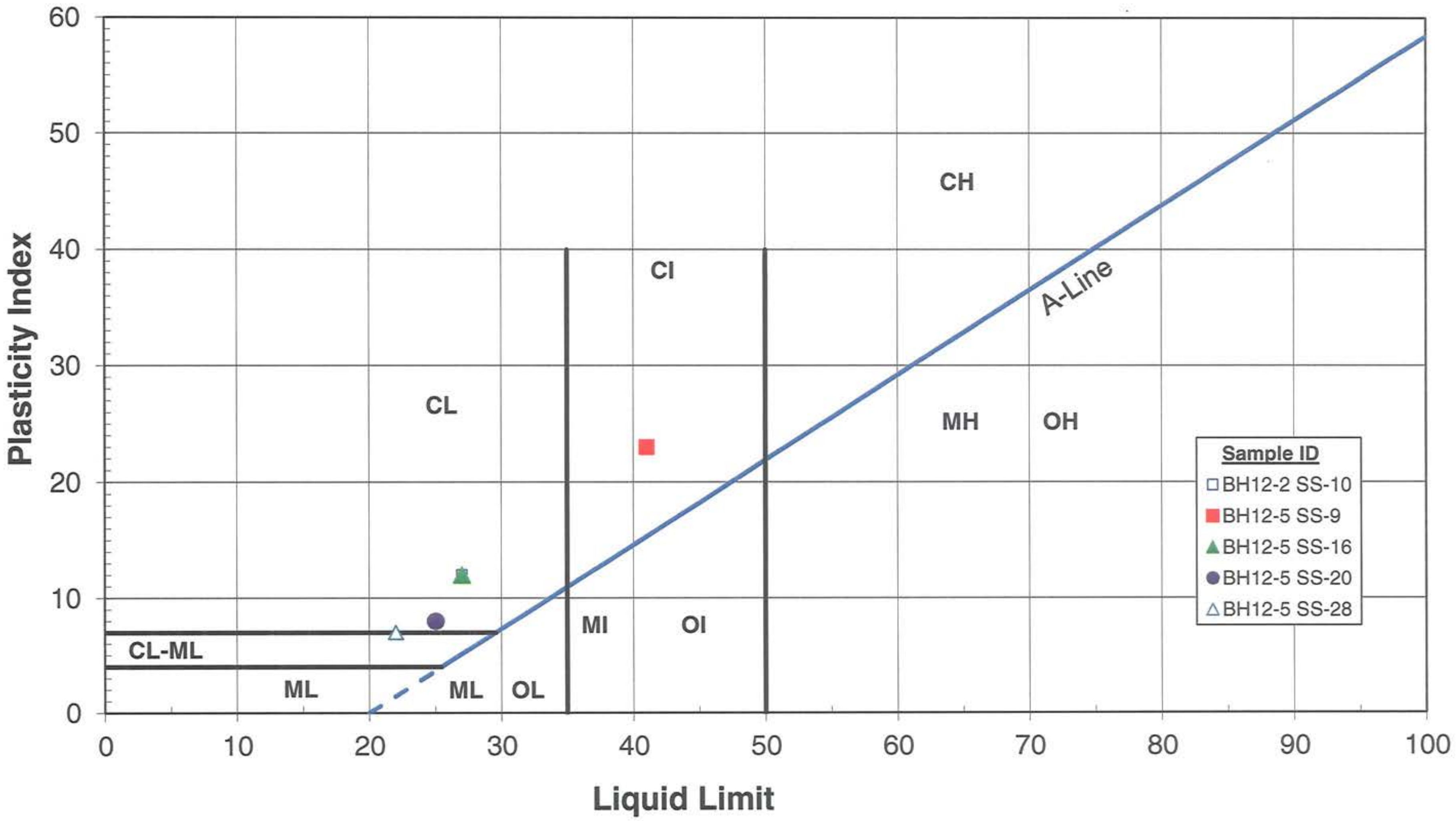
Stantec

PLASTICITY CHART

Clayey silt

Figure No. 11b

Project No. 165000801
GWP No. 2188-08-00



Stantec

PLASTICITY CHART

Clayey silt

Figure No. 11c

Project No. 165000801
GWP No. 2188-08-00

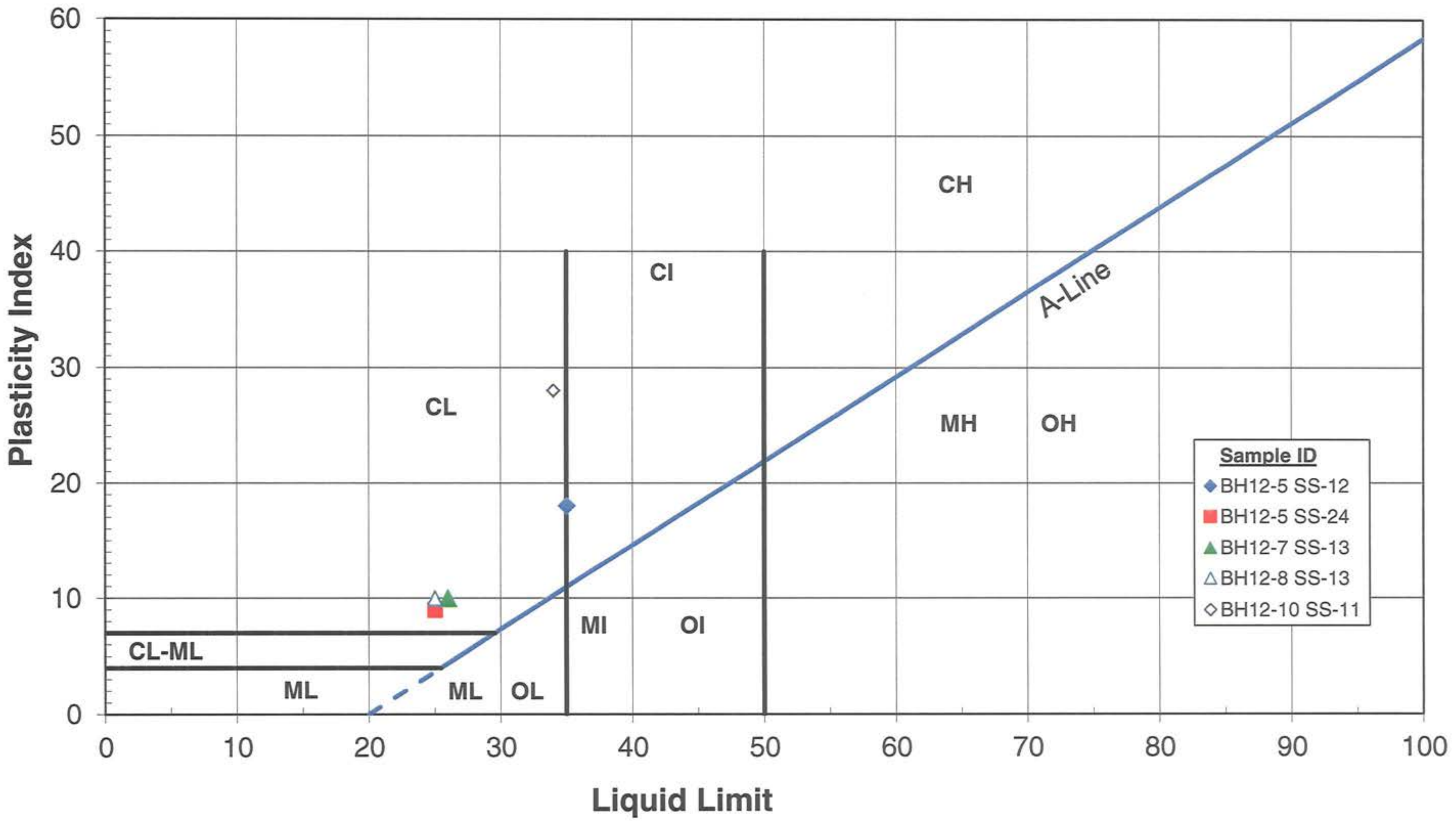


Figure No. 11d

PLASTICITY CHART

Clayey silt

Project No. 165000801
GWP No. 2188-08-00

Stantec

Golder Associates

May 31, 2012

Project No. 12-1183-0048

165000801

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

GEOTECHNICAL LABORATORY TESTING

Dear Sir

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

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

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

GOLDER ASSOCIATES LTD.

Marijana Manojlovic

Marijana Manojlovic
Laboratory Manager

MM/lg

Golder Associates Ltd.
2390 Argentia Road, Mississauga, Ontario, Canada L5N 5Z7
Tel: +1 (905) 567 4444 Fax: +1 (905) 567 6561 www.golder.com

Golder Associates: Operations in Africa, Asia, Australasia, Europe, North America and South America

CONSOLIDATION TEST SUMMARY

FIGURE

SAMPLE IDENTIFICATION

Project Number	12-1183-0048	Sample Number	ST-22
Borehole Number	12-3	Sample Depth, m	15.8-16.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	4/23/2012		
Date Completed	5/4/2012		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	19.27
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	14.81
Area, cm^2	31.55	Specific Gravity, measured	2.77
Volume, cm^3	59.88	Solids Height, cm	1.035
Water Content, %	30.16	Volume of Solids, cm^3	32.64
Wet Mass, g	117.69	Volume of Voids, cm^3	27.24
Dry Mass, g	90.42	Degree of Saturation, %	100.1

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	c_v cm^2/s	m_v m^2/kN	k cm/s
0.00	1.898	0.834	1.898				
5.86	1.895	0.832	1.897	22	3.47E-02	2.61E-04	8.86E-07
10.66	1.888	0.825	1.892	126	6.02E-03	7.35E-04	4.34E-07
20.44	1.883	0.820	1.886	689	1.09E-03	2.86E-04	3.06E-08
39.92	1.874	0.812	1.879	265	2.82E-03	2.38E-04	6.59E-08
78.77	1.861	0.798	1.867	240	3.08E-03	1.86E-04	5.61E-08
156.26	1.839	0.778	1.850	290	2.50E-03	1.44E-04	3.53E-08
311.42	1.805	0.744	1.822	265	2.66E-03	1.18E-04	3.07E-08
621.60	1.734	0.675	1.769	342	1.94E-03	1.21E-04	2.30E-08
1242.77	1.644	0.589	1.689	392	1.54E-03	7.60E-05	1.15E-08
2483.77	1.567	0.514	1.605	289	1.89E-03	3.27E-05	6.06E-09
1242.77	1.582	0.529	1.574				
311.42	1.607	0.553	1.594				
78.77	1.643	0.588	1.625				
20.44	1.689	0.633	1.666				
5.86	1.725	0.667	1.707				

Note:
k calculated using cv based on t_{90} values.
Specimen taken 20cm from bottom of the tube.
Loading stages assigned by the client.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.72	Unit Weight, kN/m^3	20.45
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	16.30
Area, cm^2	31.55	Specific Gravity, measured	2.77
Volume, cm^3	54.41	Solids Height, cm	1.035
Water Content, %	25.49	Volume of Solids, cm ^{3}	32.64
Wet Mass, g	113.47	Volume of Voids, cm ^{3}	21.77
Dry Mass, g	90.42		

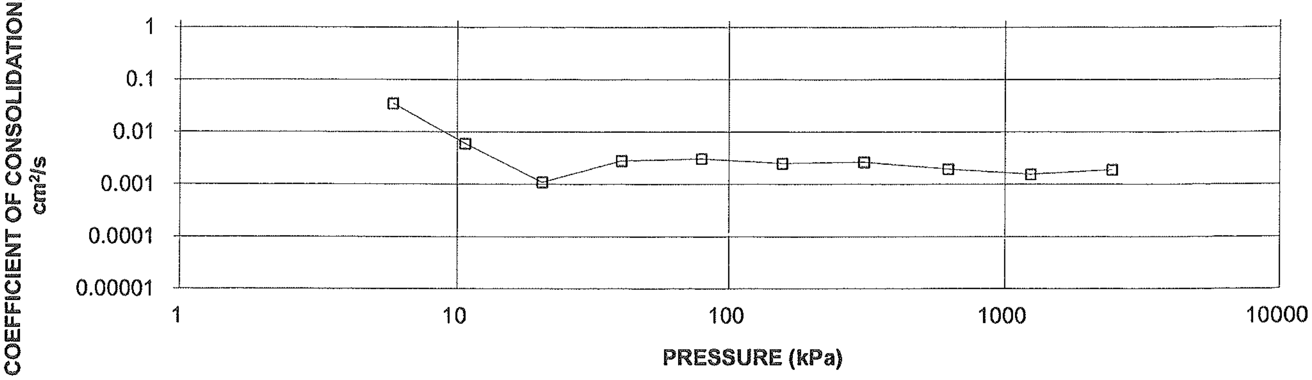
Prepared By: LFG

Golder Associates

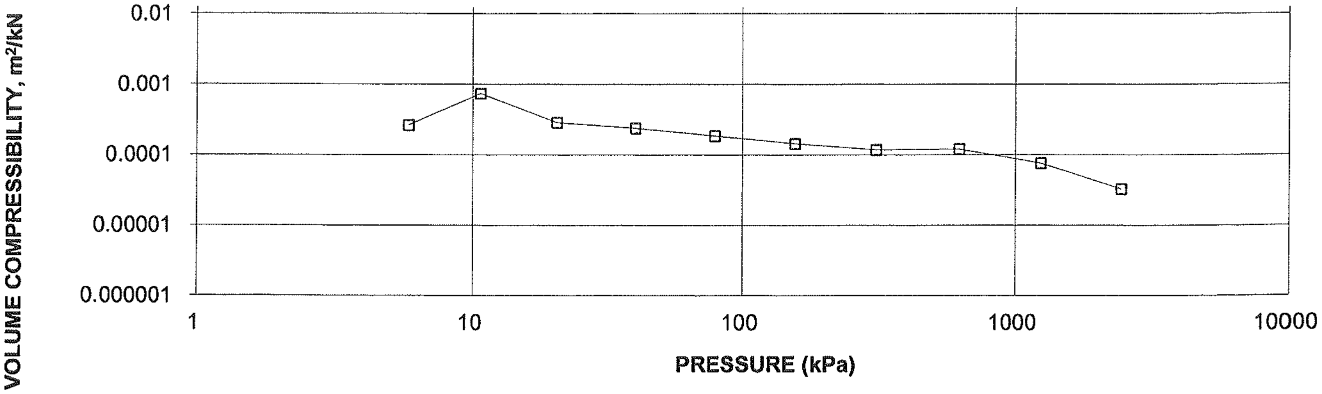
Checked By:

CONSOLIDATION TEST SUMMARY	FIGURE

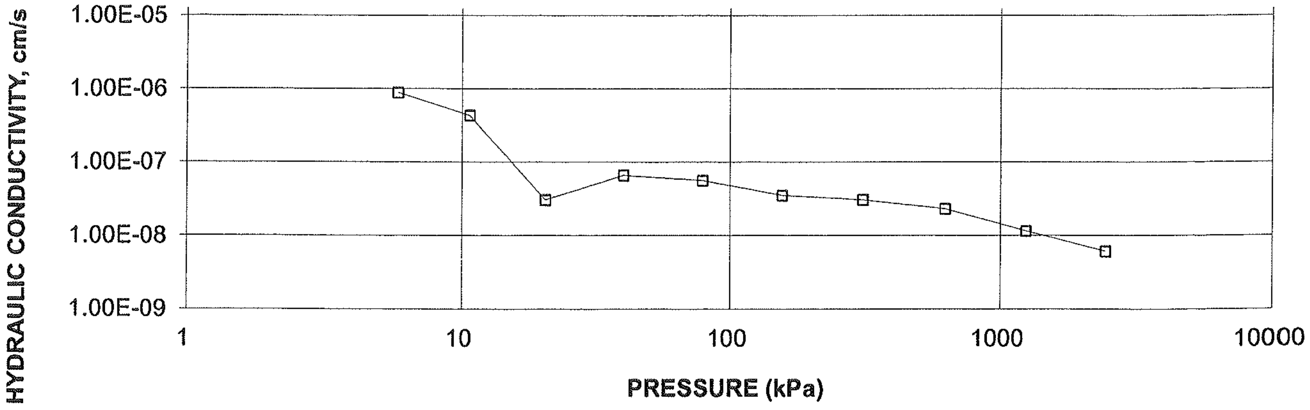
CONSOLIDATION TEST
C_V cm²/s VS PRESSURE (kPa)
BH 12-3 ST-22



CONSOLIDATION TEST
M_V m²/kN vs PRESSURE (kPa)
BH 12-3 ST-22



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 12-3 ST-22



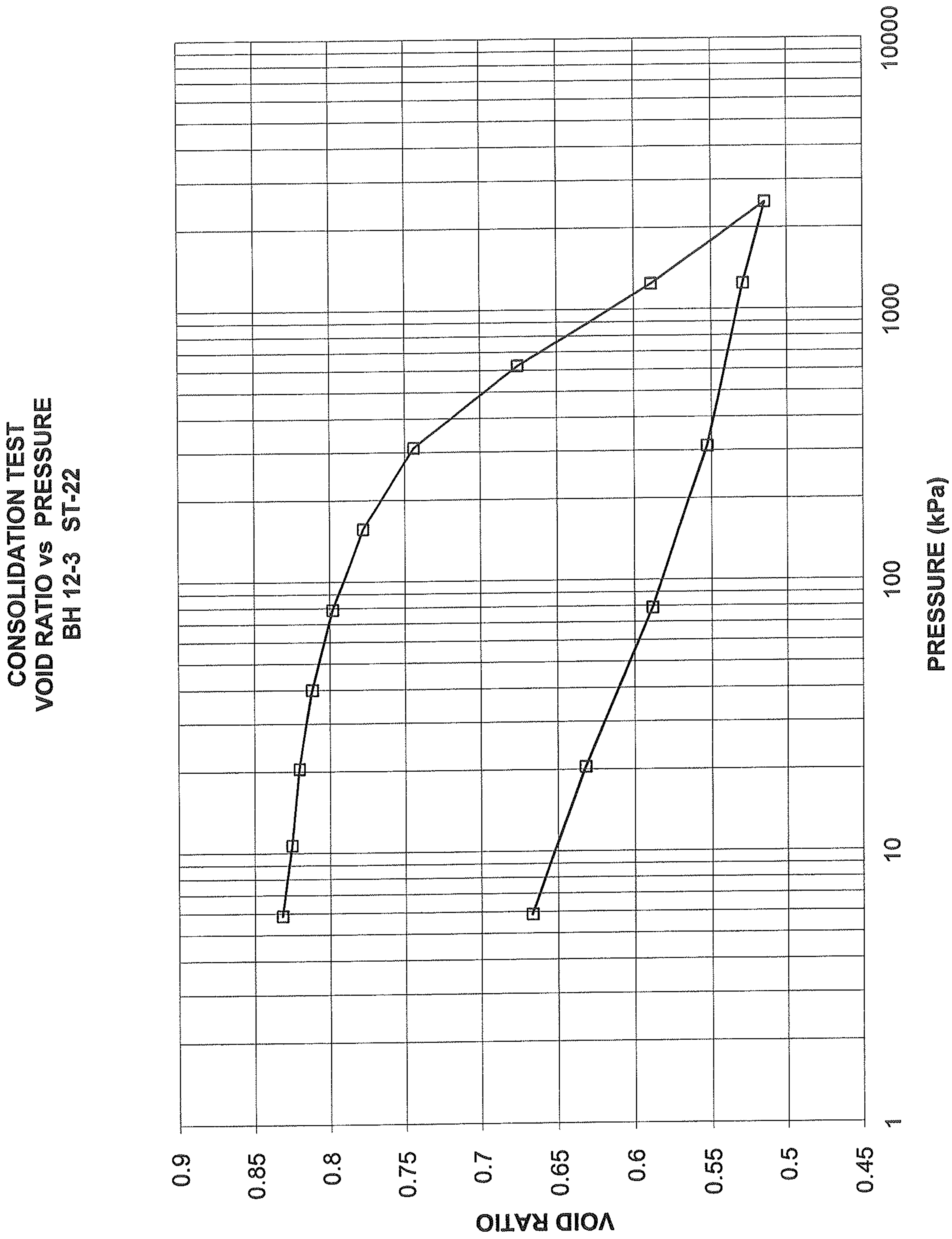
Project No. 12-1183-0048
Prepared By: LFG

Golder Associates

Checked By:

CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE



CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 12-3 ST-22

Project No. 12-1183-0048

Prepared By: LFG

Golder Associates

Checked By:

UNCONFINED COMPRESSION TEST (UC)

ASTM D 2166 - 06

SAMPLE IDENTIFICATION

PROJECT NUMBER	12-1183-0048	SAMPLE NUMBER	ST-22
BOREHOLE NUMBER	12-3	SAMPLE DEPTH, m	15.85-16.46

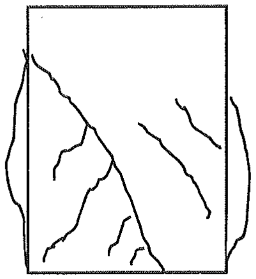
TEST CONDITIONS

MACHINE SPEED, mm/min	1.42	TYPE OF SPECIMEN	Thin wall tube sample
RATE OF AXIAL STRAIN, %/min	1.00	L/D	2.07

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	14.21	WATER CONTENT, (specimen) %	19.75
SAMPLE DIAMETER, cm	6.87	UNIT WEIGHT, kN/m^3	21.17
SAMPLE AREA, cm^2	37.11	DRY UNIT WT., kN/m^3	17.68
SAMPLE VOLUME, cm^3	527.36	SPECIFIC GRAVITY, measured	2.77
WET WEIGHT, g	1138.90	VOID RATIO	0.54
DRY WEIGHT, g	951.06		

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	11.3	COMPRESSIVE STRESS, kPa	140

REMARKS: Specimen taken from bottom of the tube. DATE: 04/24/2012

Checked By:

Golder Associates

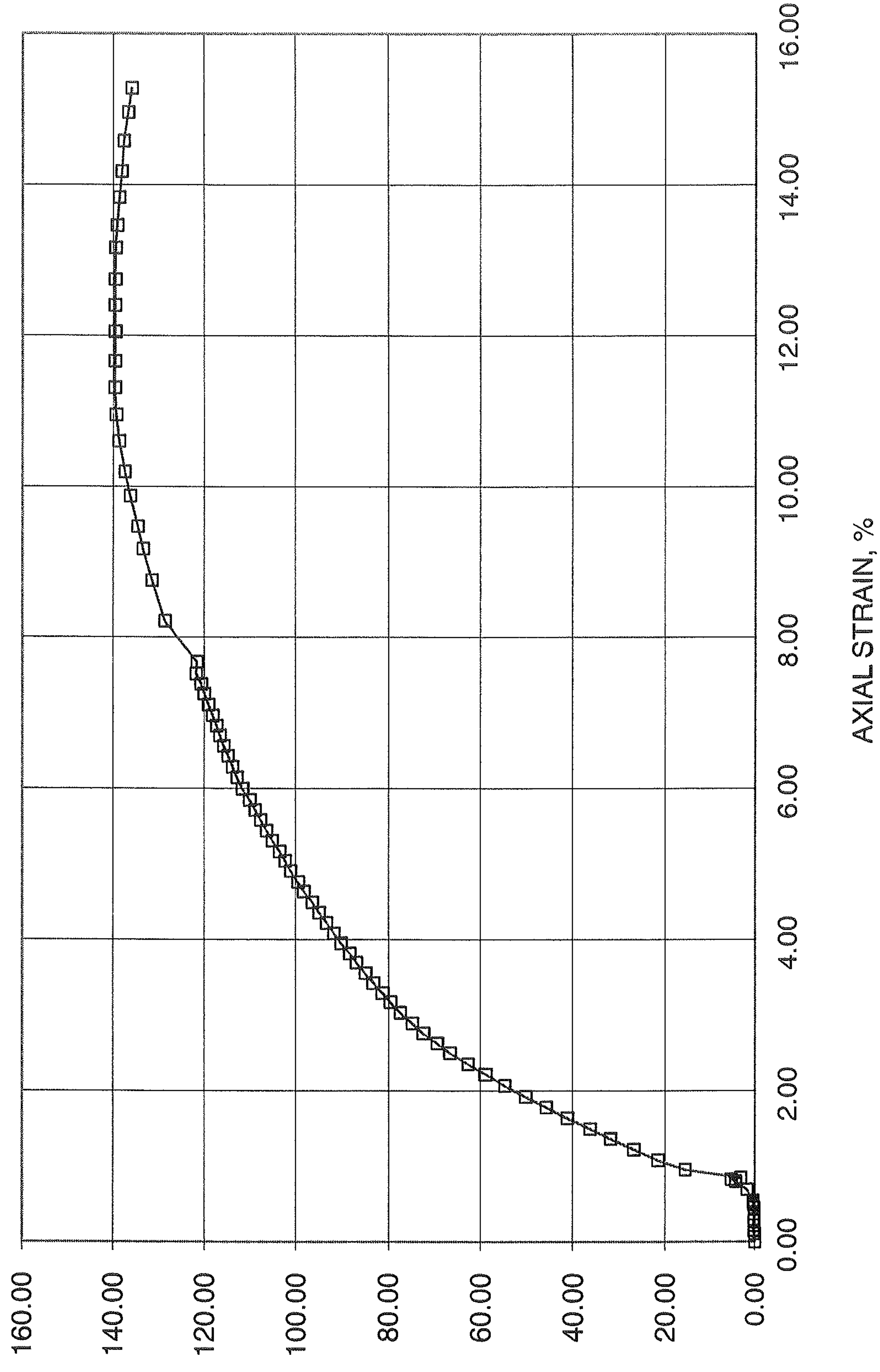
UNCONFINED COMPRESSION TEST (UC)

FIGURE

BOREHOLE NUMBER 12-3

SAMPLE NUMBER ST-22

SAMPLE DEPTH, m 15.85-16.46



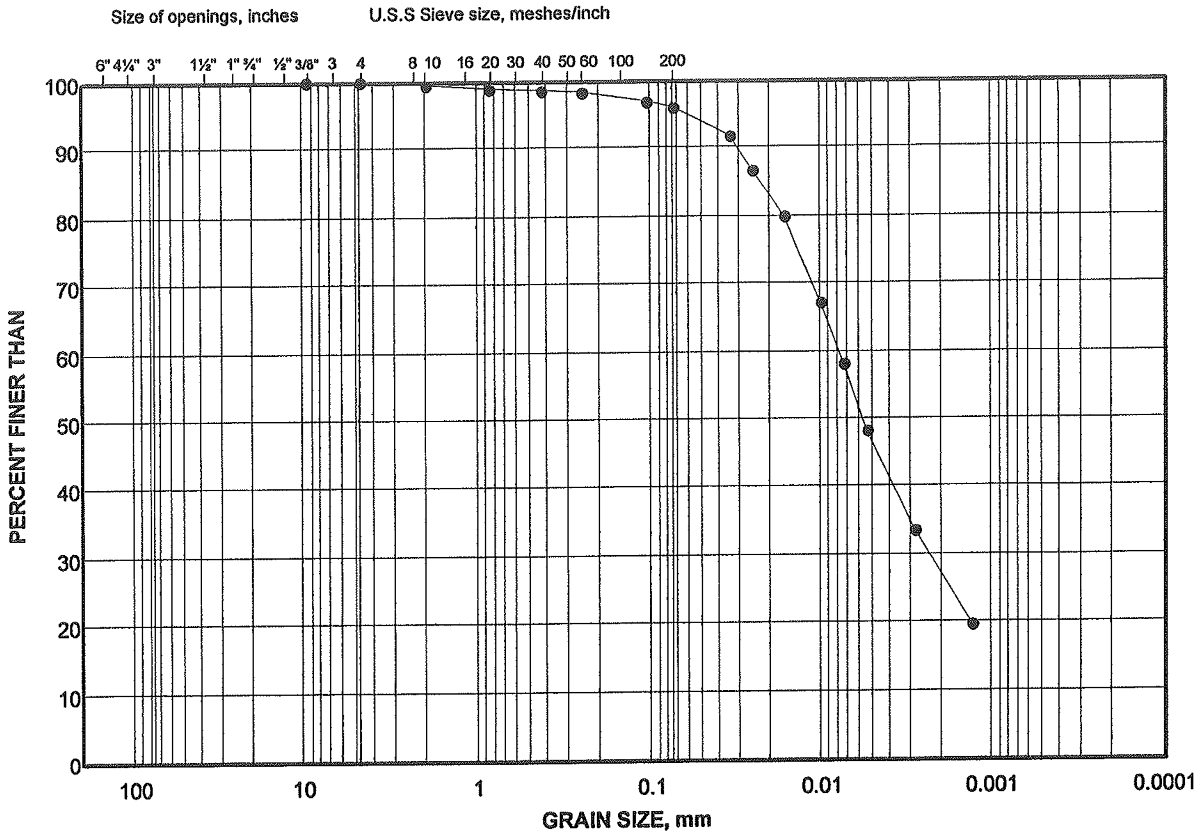
DEVIATOR STRESS, kPa

Project No.12-1183-0048

Checked By:

GRAIN SIZE DISTRIBUTION

FIGURE



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	12-3	ST-22	15.80 - 16.50

Project Number: 12-1183-0048

Checked By: ____________

Golder Associates

Date: 09-May-12

CONSOLIDATION TEST SUMMARY

FIGURE

SAMPLE IDENTIFICATION

Project Number	12-1183-0048	Sample Number	ST-7
Borehole Number	12-5	Sample Depth, m	4.4-5.0

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	4/23/2012		
Date Completed	5/08/2012		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m^3	18.45
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	13.66
Area, cm^2	31.48	Specific Gravity, measured	2.75
Volume, cm^3	59.59	Solids Height, cm	0.959
Water Content, %	35.04	Volume of Solids, cm^3	30.19
Wet Mass, g	112.10	Volume of Voids, cm^3	29.41
Dry Mass, g	83.01	Degree of Saturation, %	98.9

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	c_v cm^2/s	m_v m^2/kN	k cm/s
0.00	1.893	0.974	1.893				
5.94	1.887	0.968	1.890	375	2.02E-03	5.25E-04	1.04E-07
10.65	1.881	0.962	1.884	227	3.32E-03	6.62E-04	2.15E-07
20.60	1.862	0.942	1.872	360	2.06E-03	1.02E-03	2.06E-07
39.95	1.835	0.914	1.849	359	2.02E-03	7.29E-04	1.44E-07
78.94	1.781	0.857	1.808	577	1.20E-03	7.42E-04	8.74E-08
156.70	1.695	0.767	1.738	634	1.01E-03	5.83E-04	5.77E-08
312.34	1.604	0.672	1.649	581	9.92E-04	3.09E-04	3.00E-08
623.38	1.518	0.583	1.561	894	5.78E-04	1.46E-04	8.28E-09
1246.39	1.438	0.499	1.478	427	1.08E-03	6.77E-05	7.19E-09
2490.46	1.364	0.423	1.401	409	1.02E-03	3.12E-05	3.11E-09
1246.39	1.373	0.432	1.369				
312.34	1.384	0.443	1.379				
78.94	1.406	0.467	1.395				
20.60	1.445	0.507	1.426				
5.94	1.473	0.537	1.459				

Note:
k calculated using cv based on t_{90} values.
Specimen taken 28cm from bottom of the tube.
Loading stages assigned by the client.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.47	Unit Weight, kN/m^3	21.30
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	17.55
Area, cm^2	31.48	Specific Gravity, measured	2.75
Volume, cm^3	46.38	Solids Height, cm	0.959
Water Content, %	21.38	Volume of Solids, cm^3	30.19
Wet Mass, g	100.76	Volume of Voids, cm^3	16.20
Dry Mass, g	83.01		

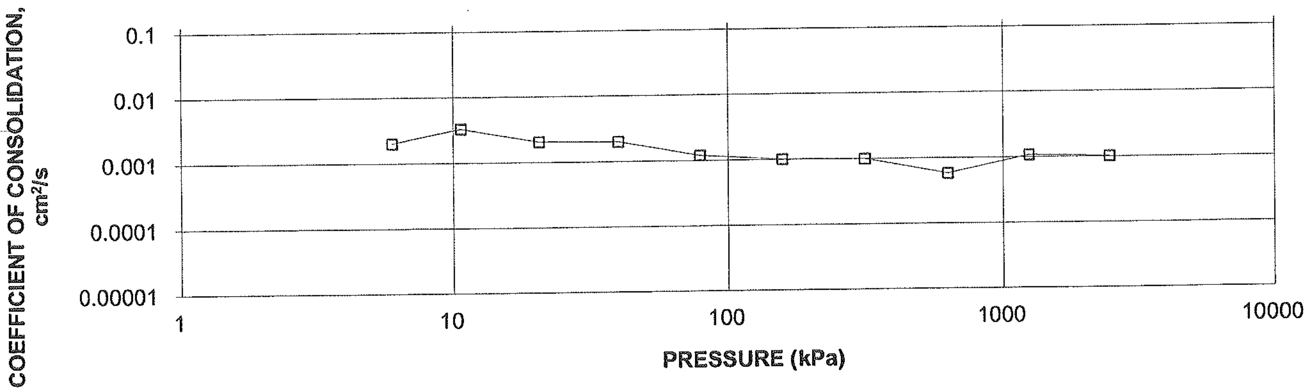
Prepared By: LFG

Golder Associates

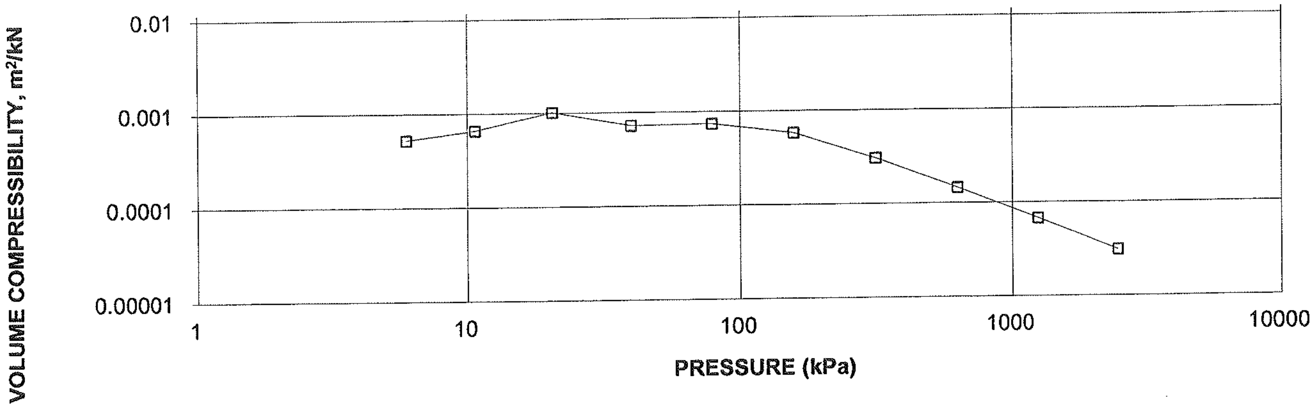
Checked By:

CONSOLIDATION TEST SUMMARY

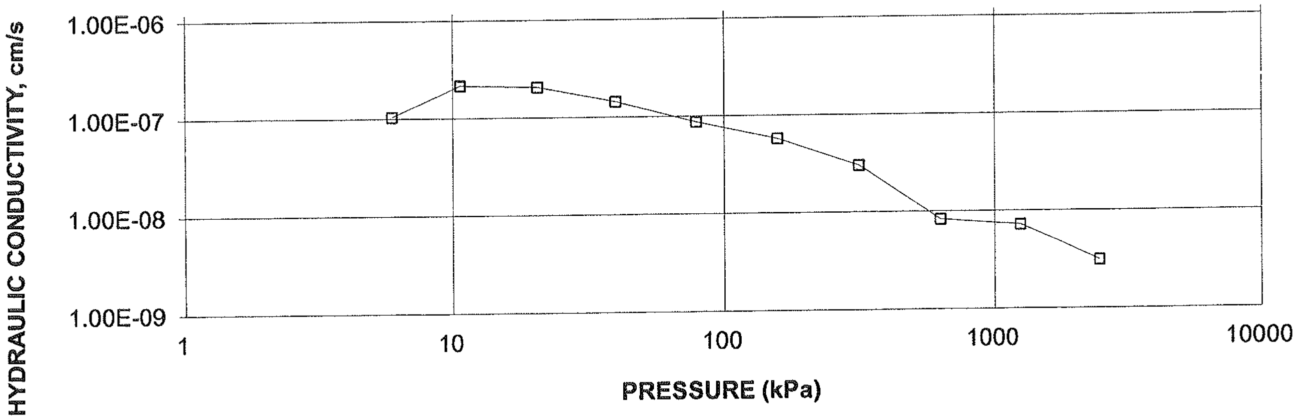
FIGURE



CONSOLIDATION TEST
C_V cm^2/s VS PRESSURE (kPa)
BH 12-5 ST-7



CONSOLIDATION TEST
M_V m^2/kN vs PRESSURE (kPa)
BH 12-5 ST-7



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 12-5 ST-7

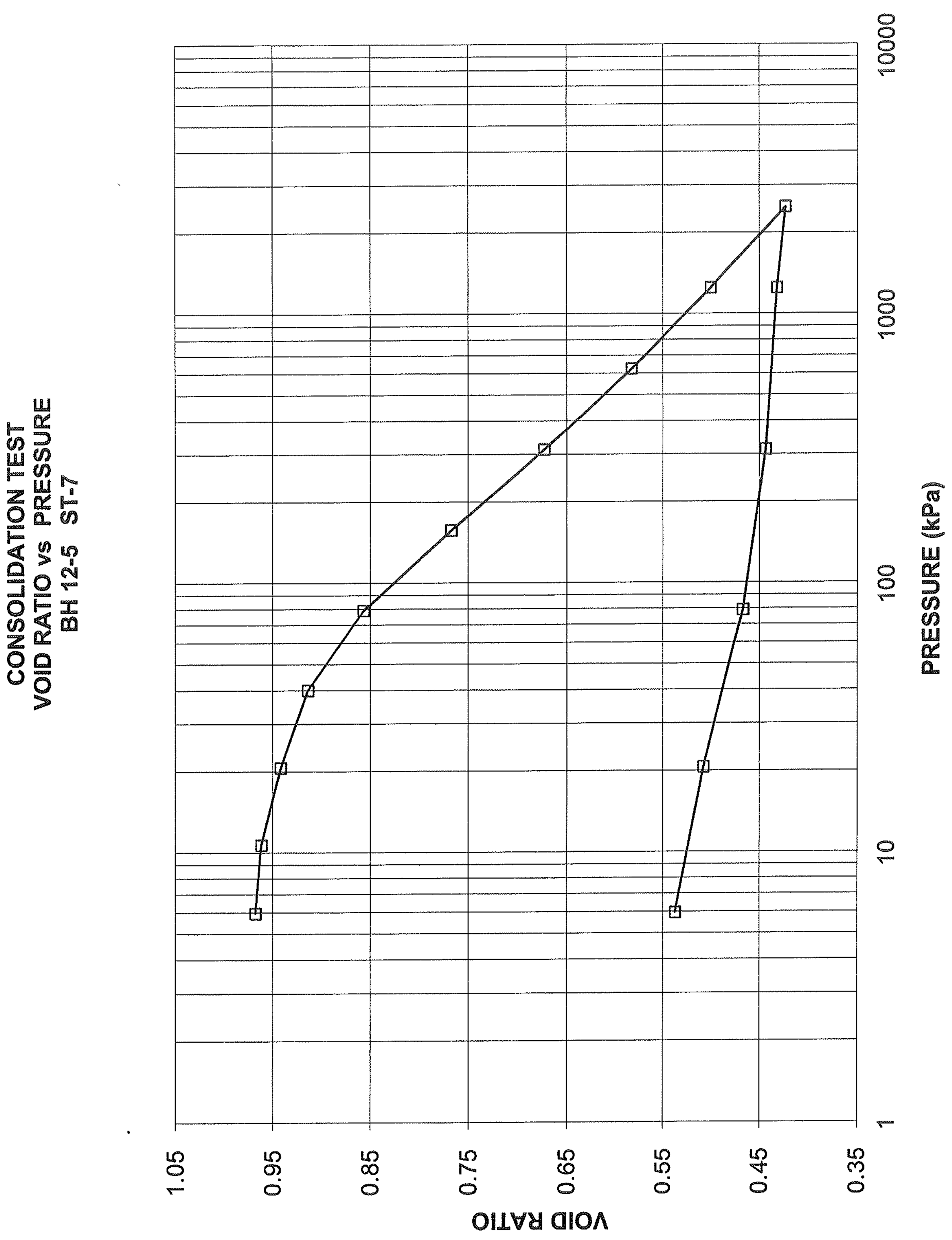
Project No. 12-1183-0048
Prepared By: LFG

Golder Associates

Checked By:

CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE



Project No. 12-1183-0048

Prepared By: LFG

Golder Associates

Checked By:

UNCONFINED COMPRESSION TEST (UC)

ASTM D 2166 - 06

SAMPLE IDENTIFICATION

PROJECT NUMBER	12-1183-0048	SAMPLE NUMBER	ST-7
BOREHOLE NUMBER	12-5	SAMPLE DEPTH, m	4.40-5.00

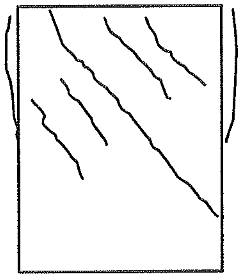
TEST CONDITIONS

MACHINE SPEED, mm/min	1.42	TYPE OF SPECIMEN	Thin wall tube sample
RATE OF AXIAL STRAIN, %/min	1.02	L/D	2.03

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.93	WATER CONTENT, (specimen) %	35.67
SAMPLE DIAMETER, cm	6.87	UNIT WEIGHT, kN/m^3	18.77
SAMPLE AREA, cm^2	37.09	DRY UNIT WT., kN/m^3	13.84
SAMPLE VOLUME, cm^3	516.66	SPECIFIC GRAVITY, measured	2.75
WET WEIGHT, g	989.30	VOID RATIO	0.95
DRY WEIGHT, g	729.18		

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	14.3	COMPRESSIVE STRESS, kPa	29

REMARKS: Specimen taken 33cm from bottom of the tube. DATE: 04/24/2012

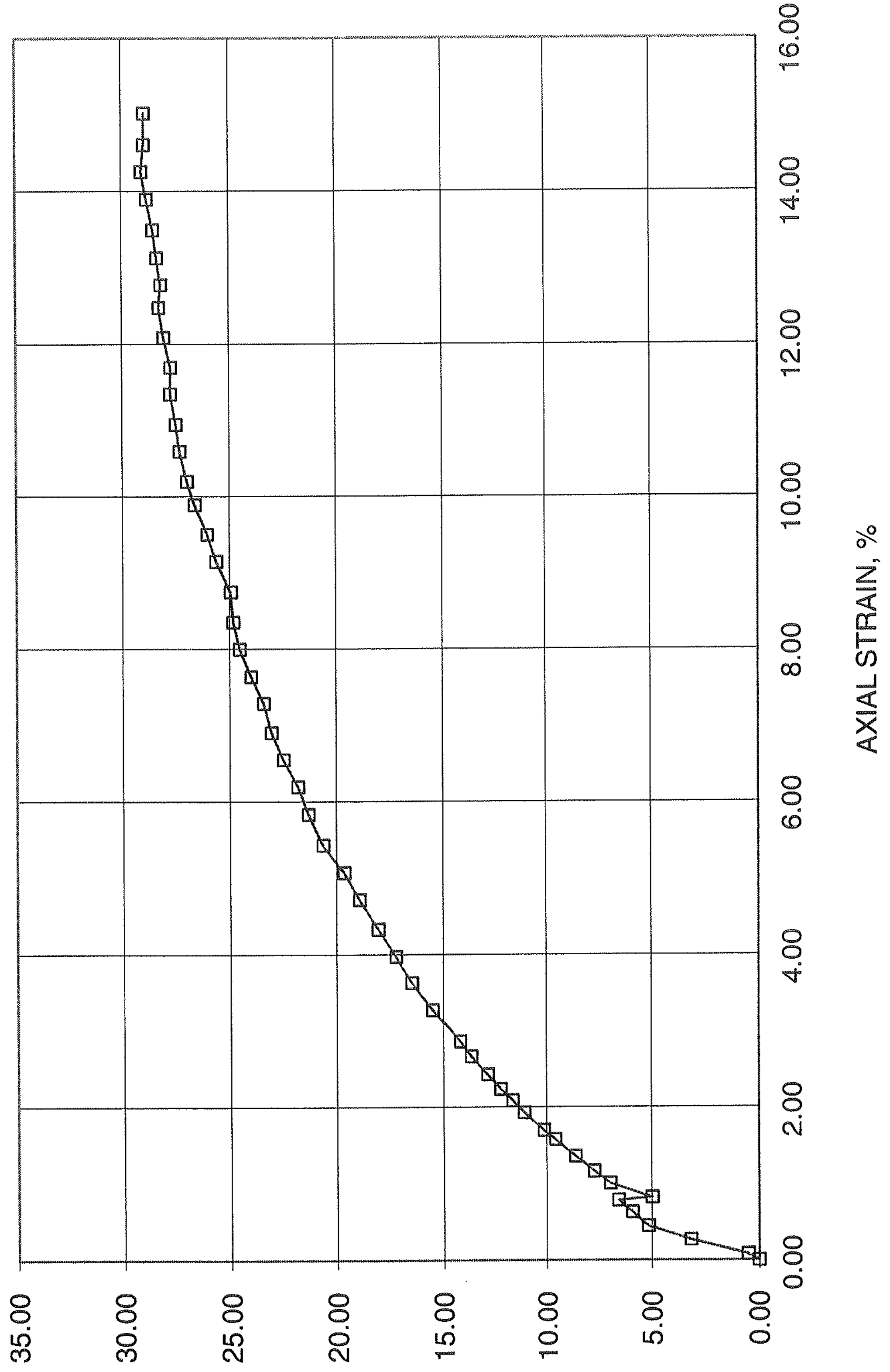
Checked By:

Golder Associates

UNCONFINED COMPRESSION TEST (UC)

FIGURE

BOREHOLE NUMBER 12-5 SAMPLE NUMBER ST-7 SAMPLE DEPTH, m 4.40-5.00

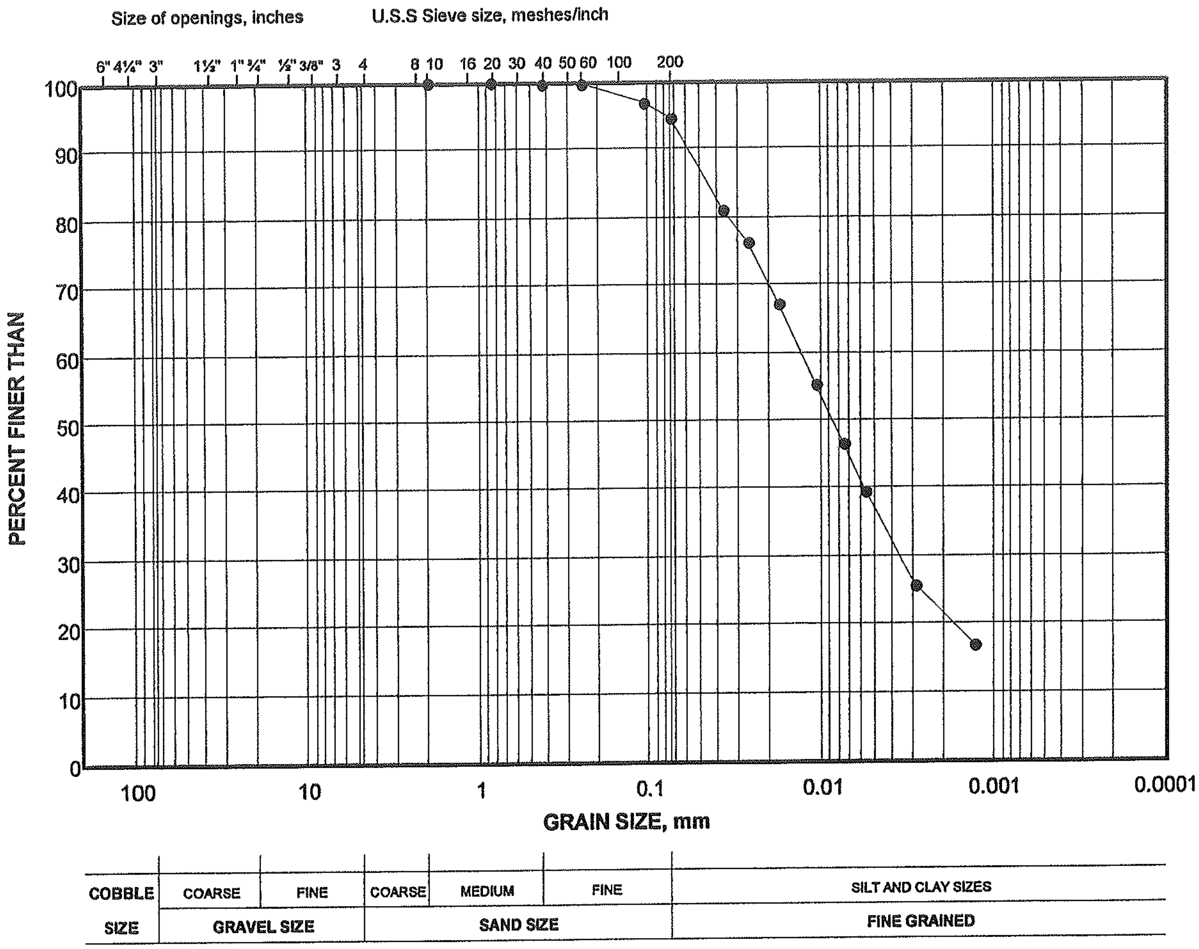


Project No.12-1183-0048

Checked By:

GRAIN SIZE DISTRIBUTION

FIGURE



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	12-5	ST-7	4.40 - 5.00

Project Number: 12-1183-0048

Checked By: ____________

Golder Associates

Date: 09-May-12

SPECIFIC GRAVITY TEST RESULTS

ASTM D 854-06 TEST METHOD A

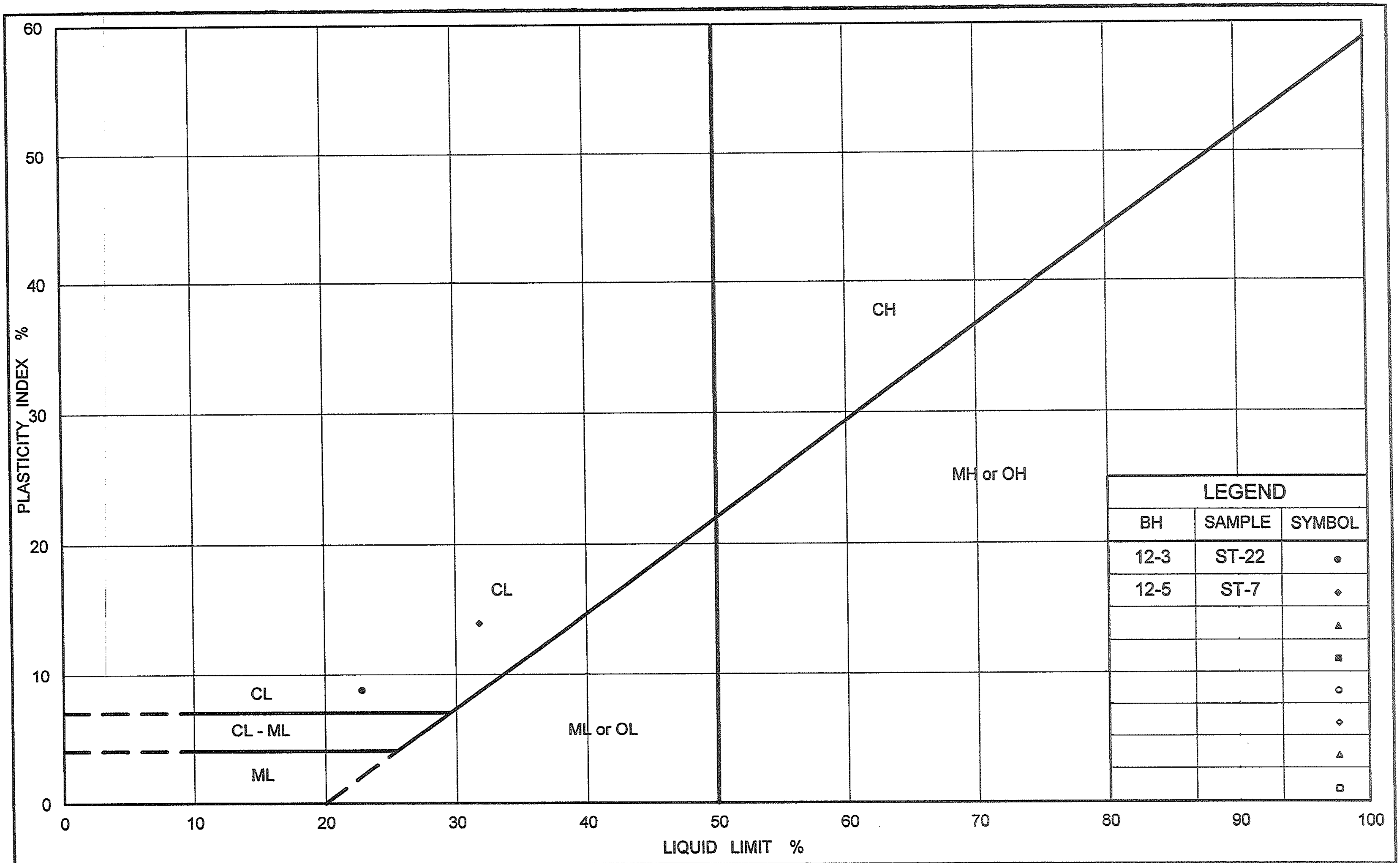
PROJECT NUMBER	12-1183-0048
PROJECT NAME	Stantec / Testing / 165000801
DATE TESTED	May, 2012

Borehole No.	Sample No.	Specific Gravity
12-3	ST-22	2.77
12-5	ST-7	2.75

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

Checked By:

Golder Associates



LEGEND		
BH	SAMPLE	SYMBOL
12-3	ST-22	●
12-5	ST-7	◆
		▲
		■
		○
		◇
		△
		□

Golder Associates

PLASTICITY CHART

Figure No.
Project No. 12-1183-0048
Checked By:

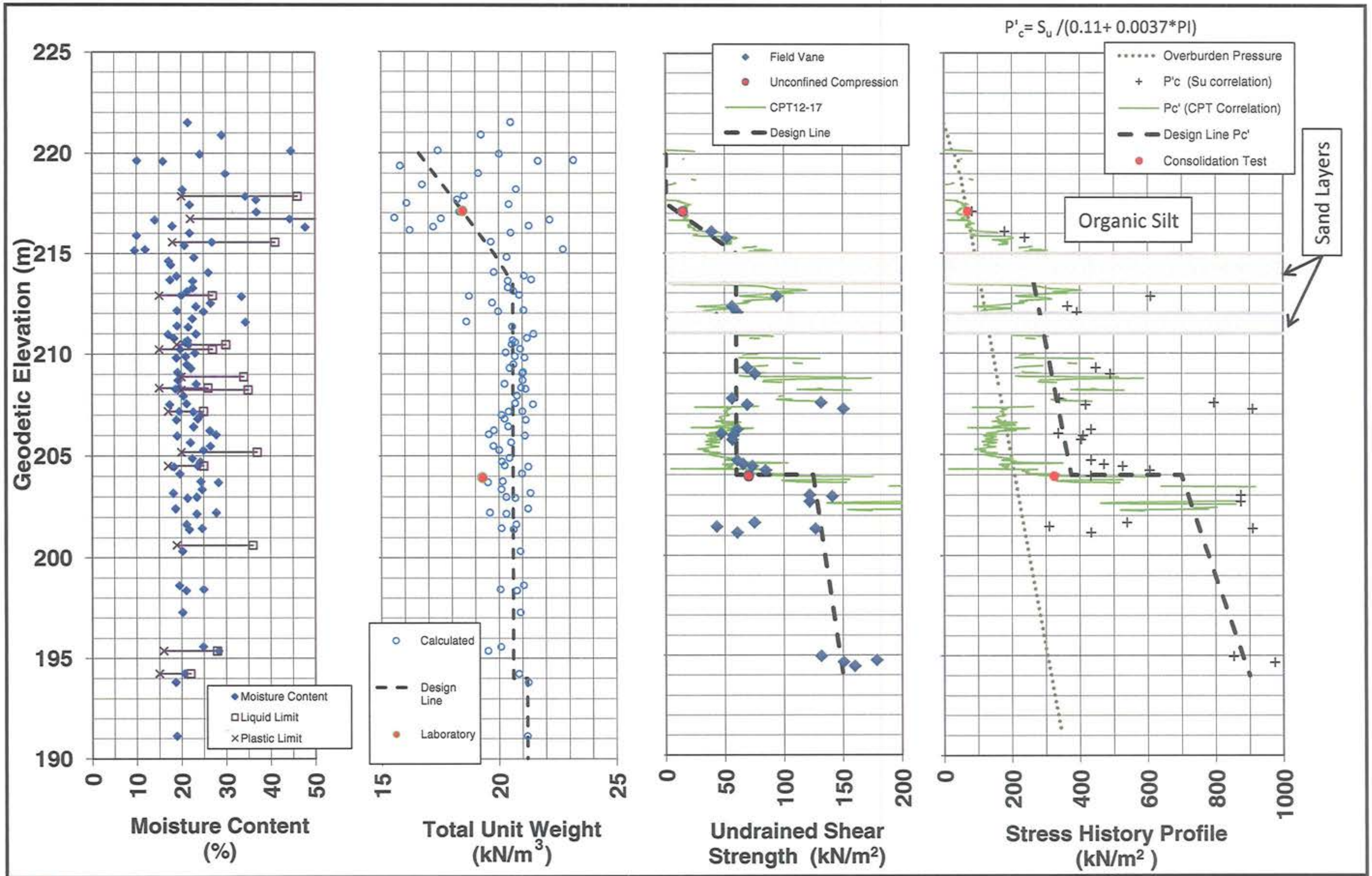
APPENDIX D

Design Parameters

Plots from LPile Analysis Results

Slope Stability Analysis Results

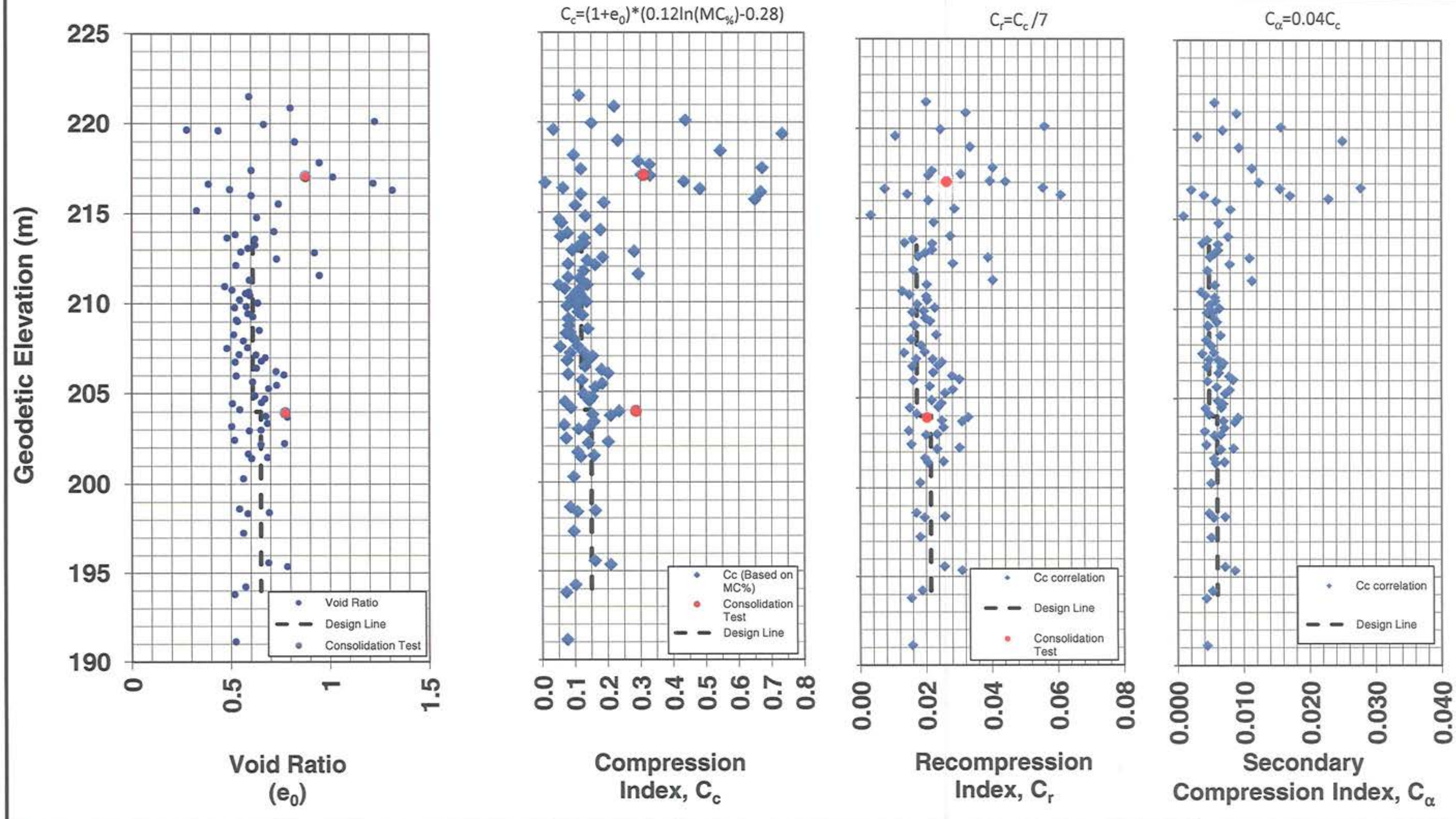
Settle3D Results



Stantec Consulting Ltd.

Highway 9
Holland Drainage Canal
Replacement

Figure 12a



Stantec

Stantec Consulting Ltd.

Highway 9
Holland Drainage Canal Bridge
Replacement

Figure 12b

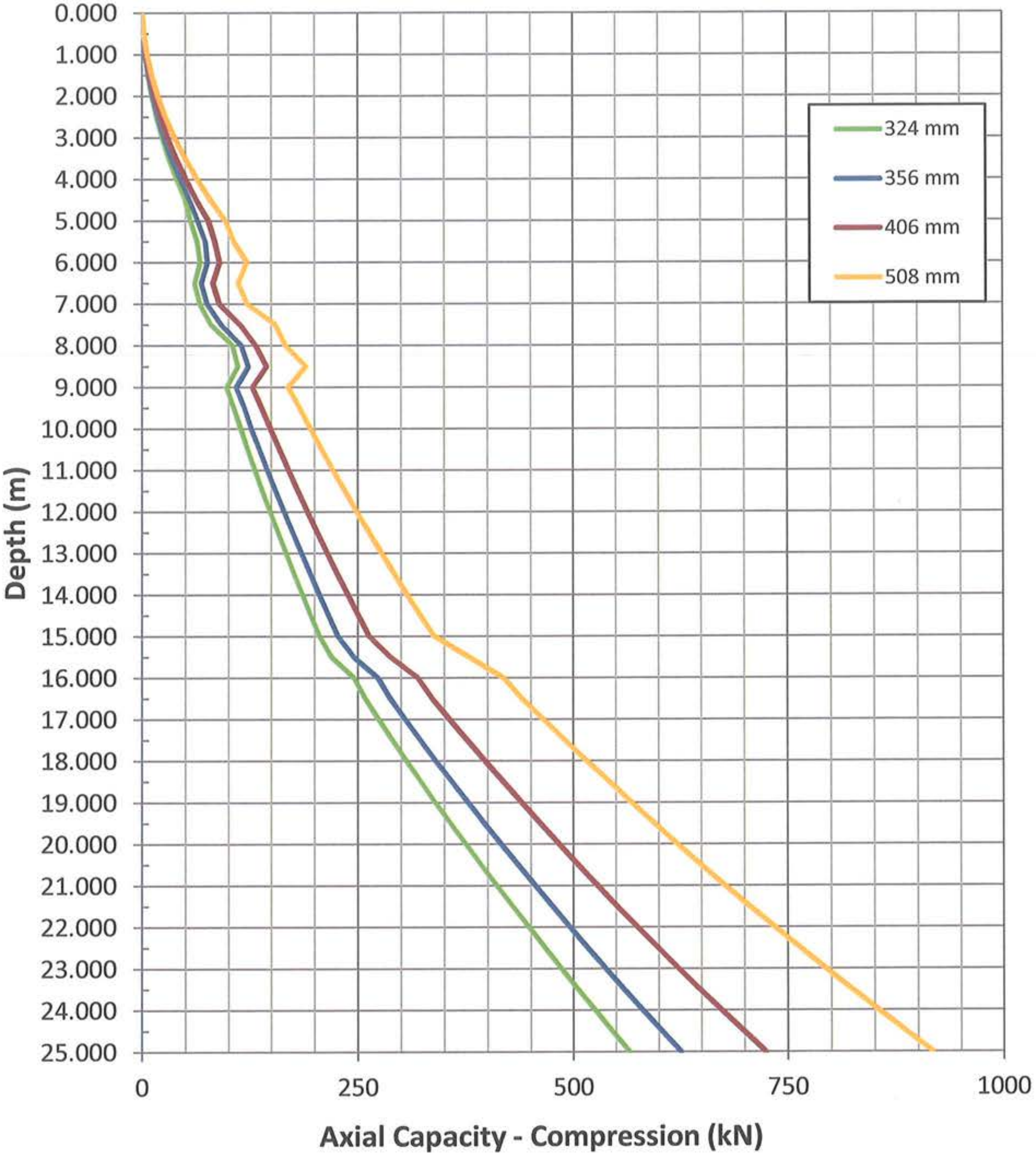


Figure 13a
Factored Axial Capacity (Compression) of Pipe Piles

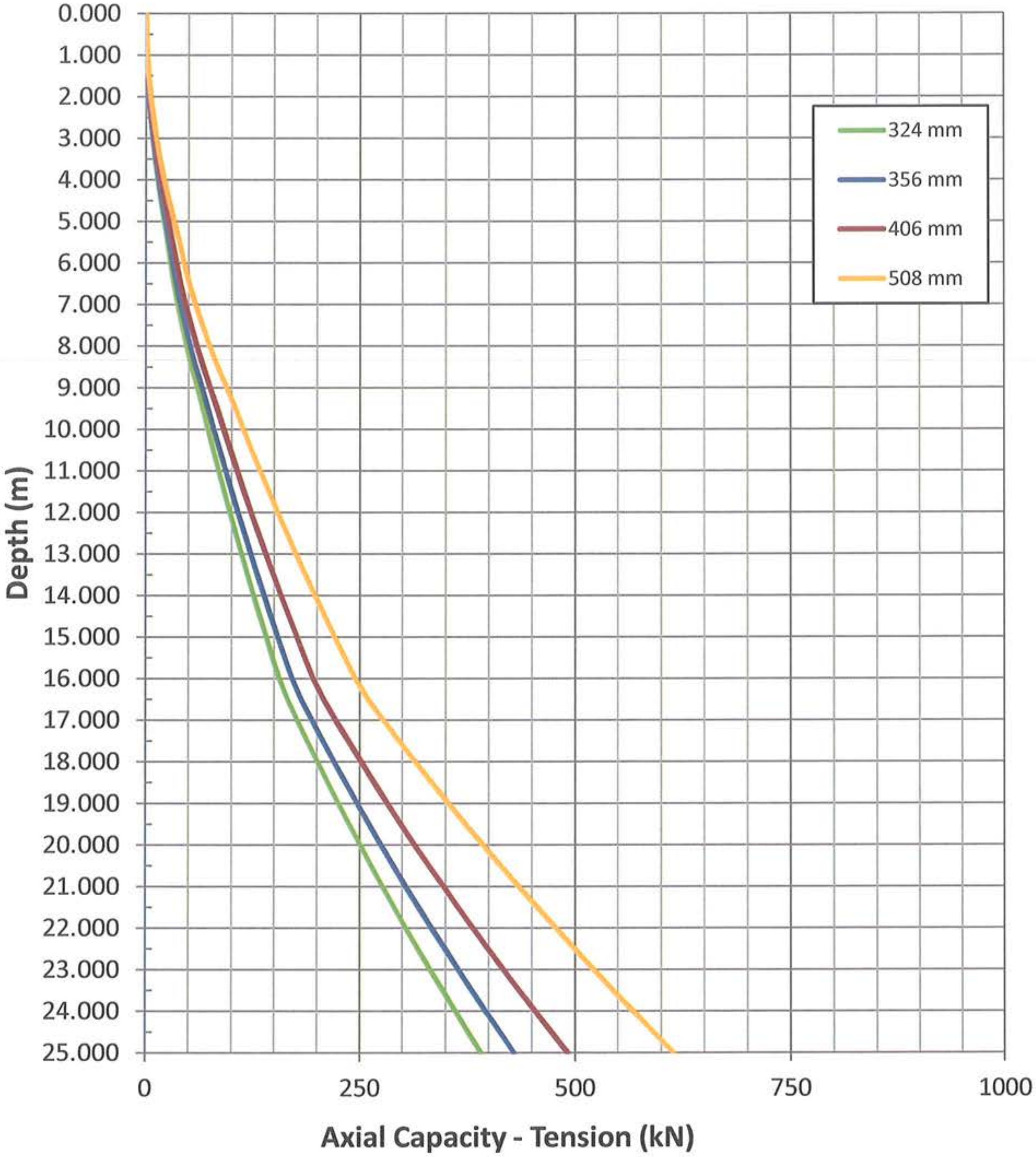


Figure 13b
Factored Axial Capacity (Tension) of Pipe Piles

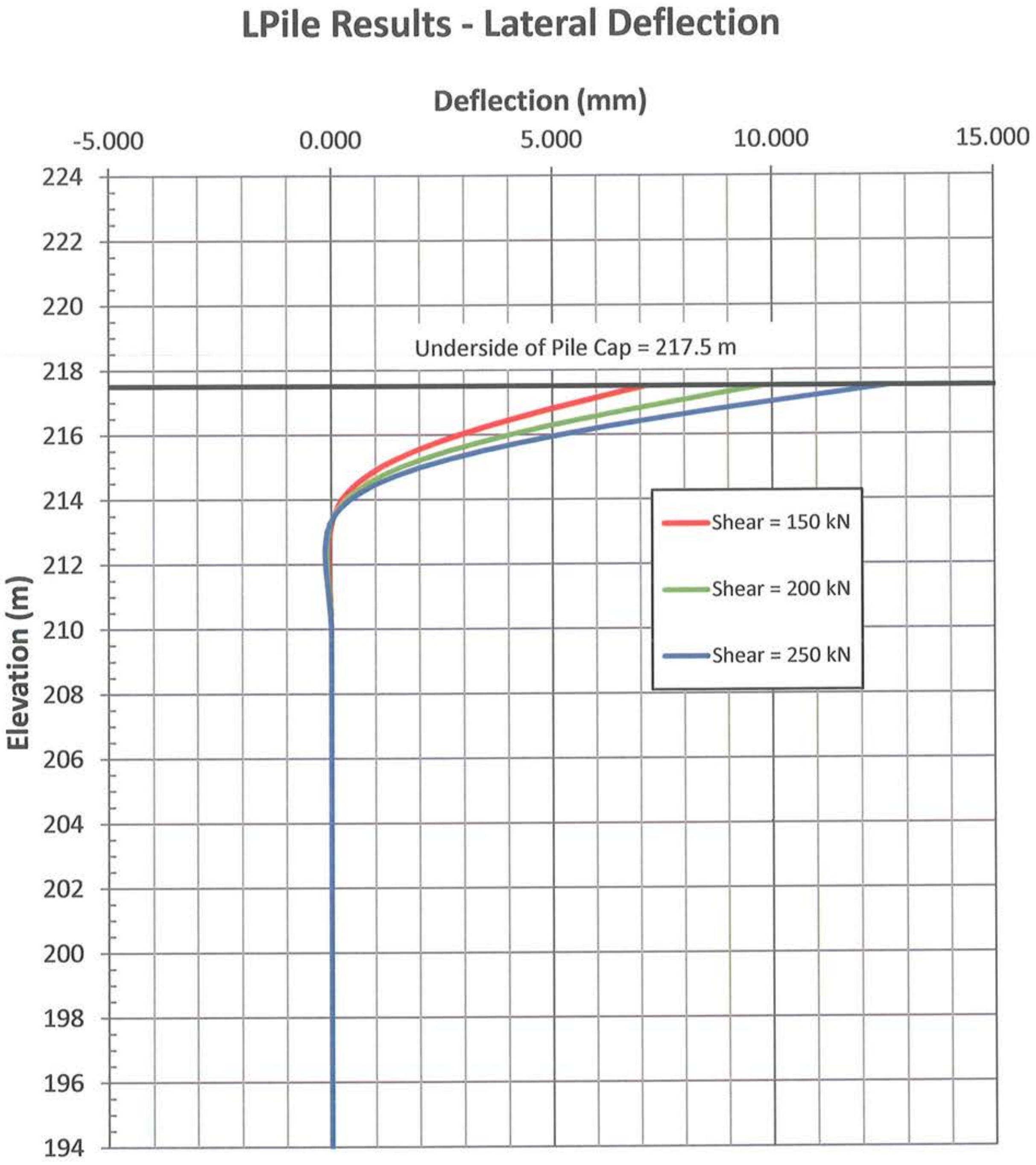


Figure 14a
Lateral Deflection of 508 mm Pipe Piles

Stantec

Project No. 165000801
Holland Drainage Canal
Bridge Replacement

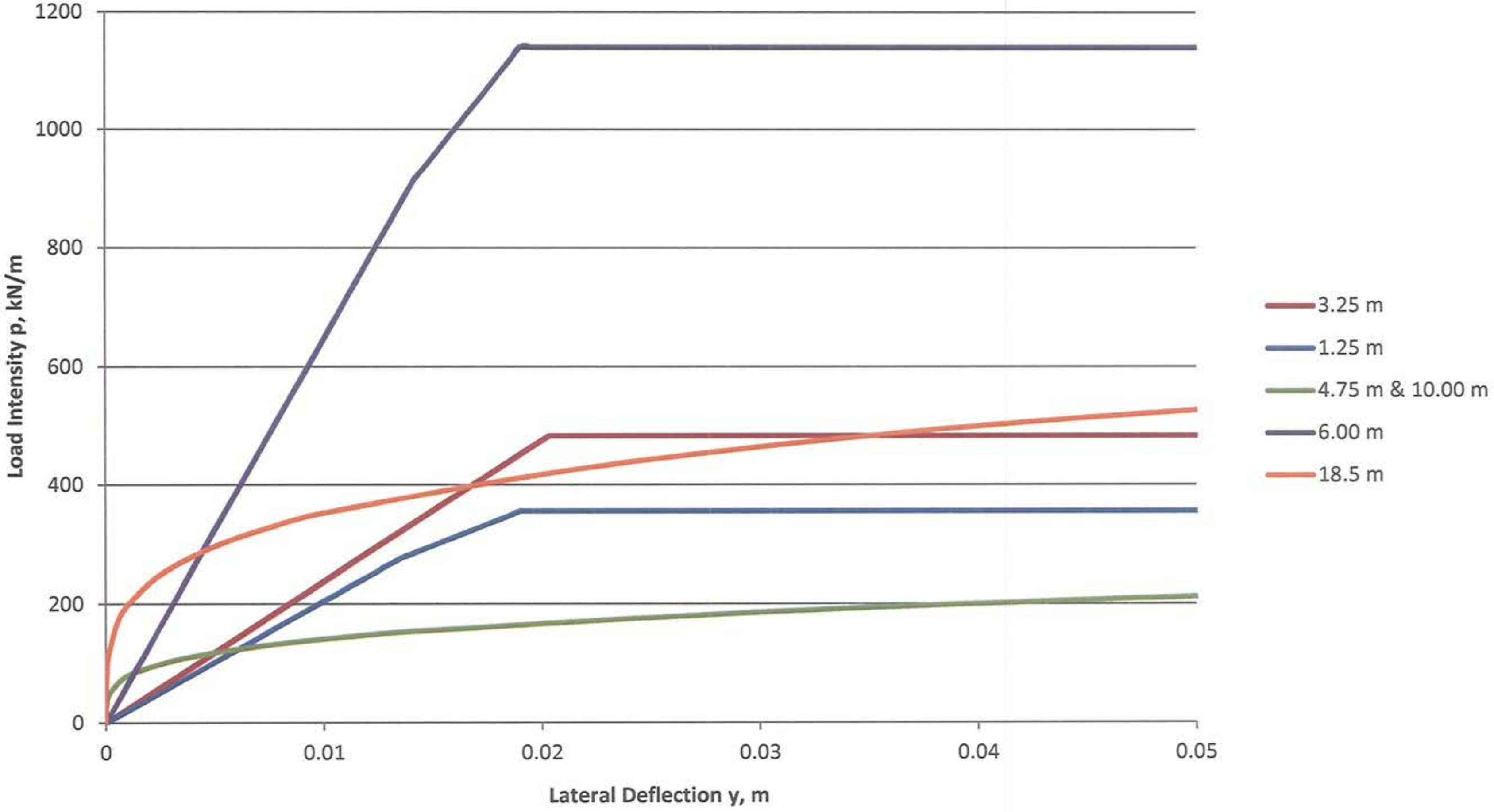
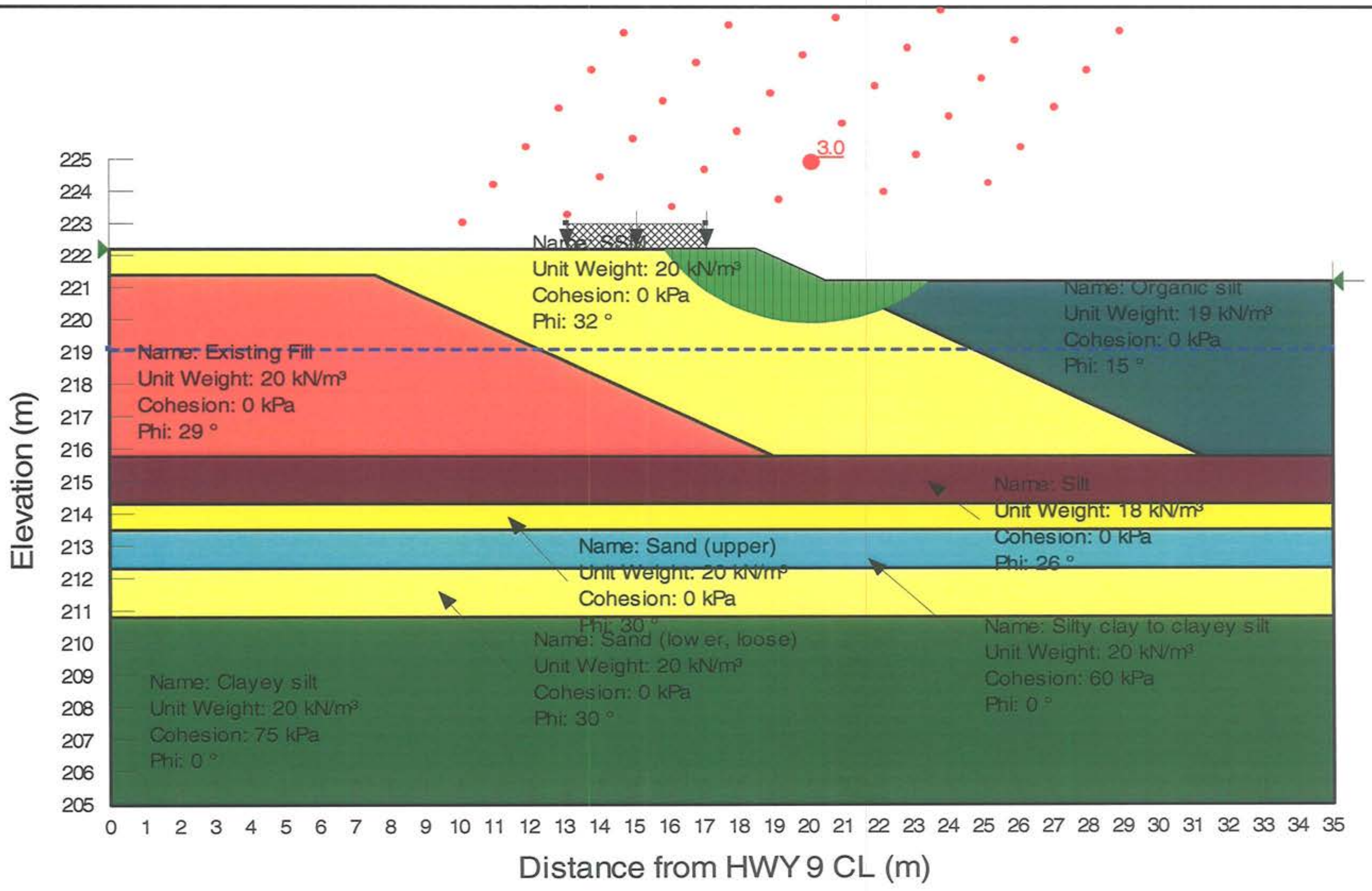


Figure 14b
p-y Curves for Proposed 508 mm Pipe Piles



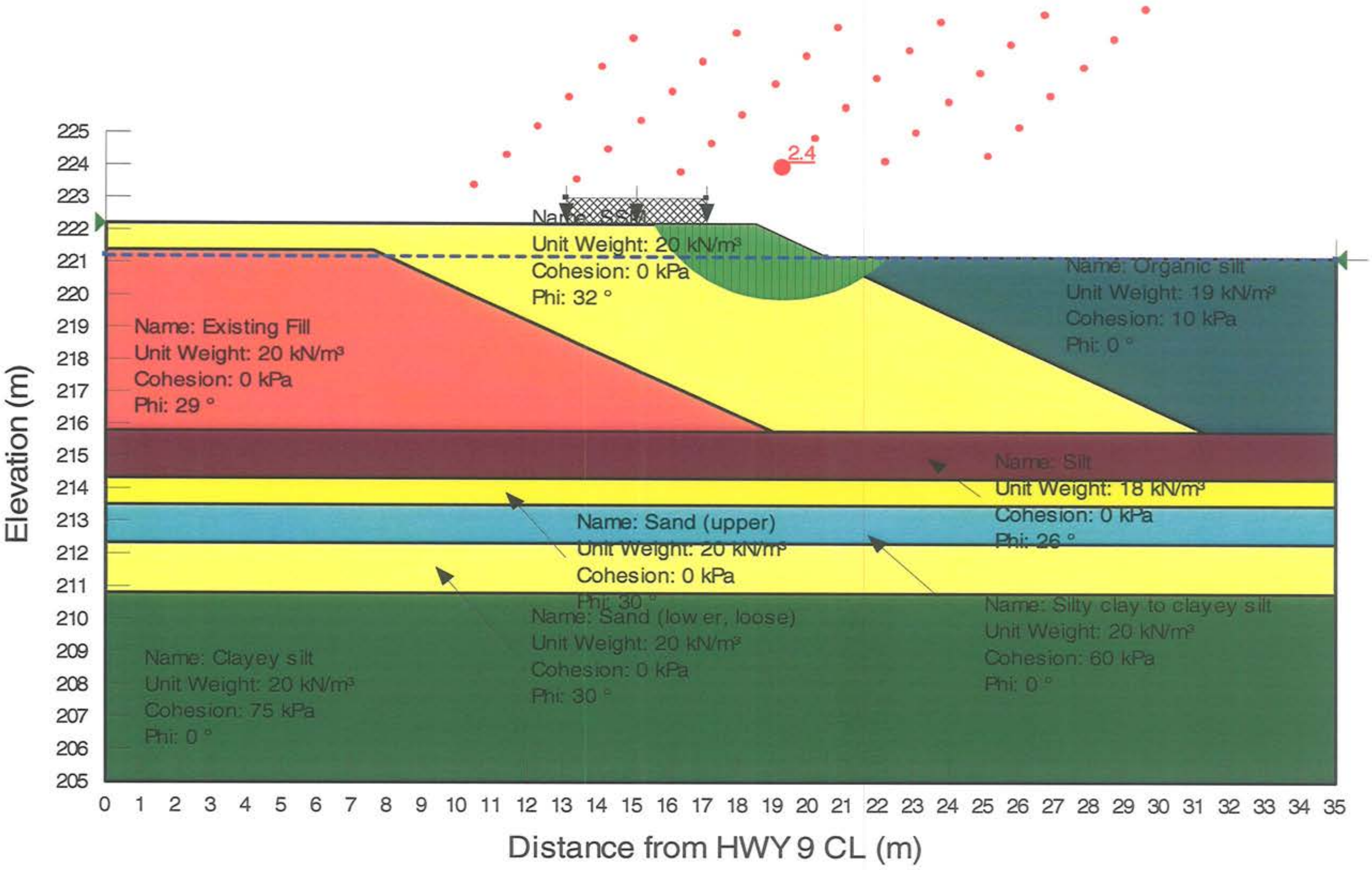
Stantec

Static Stability Analysis - Long-term

Highway 9 Holland Drainage Canal Bridge Replacement

Figure 15a

Project No. 165000801
GWP No. 2188-08-00



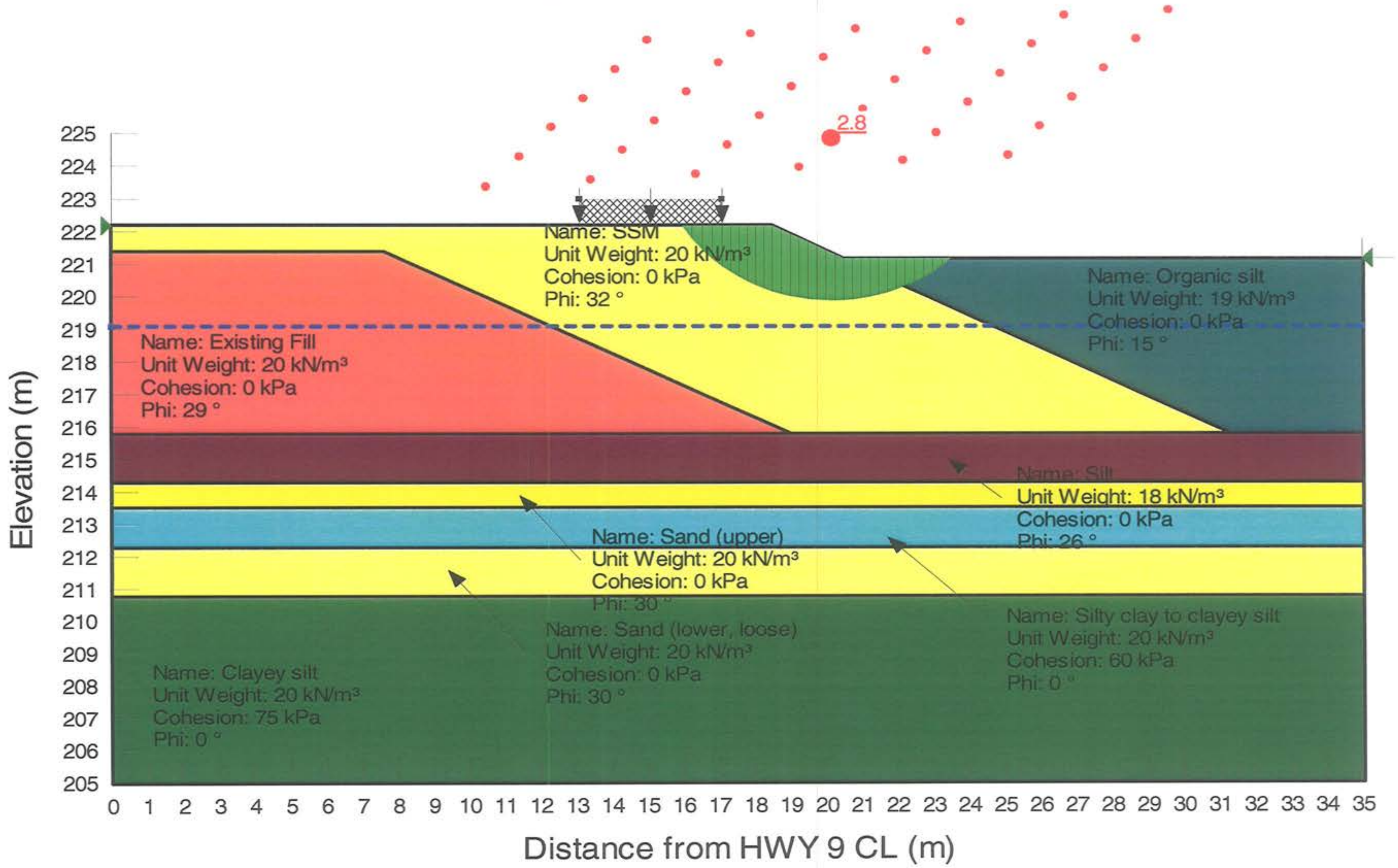
Stantec

Static Stability Analysis - Short-term

Highway 9 Holland Drainage Canal Bridge Replacement

Figure 15b

Project No. 165000801
GWP No. 2188-08-00



Stantec

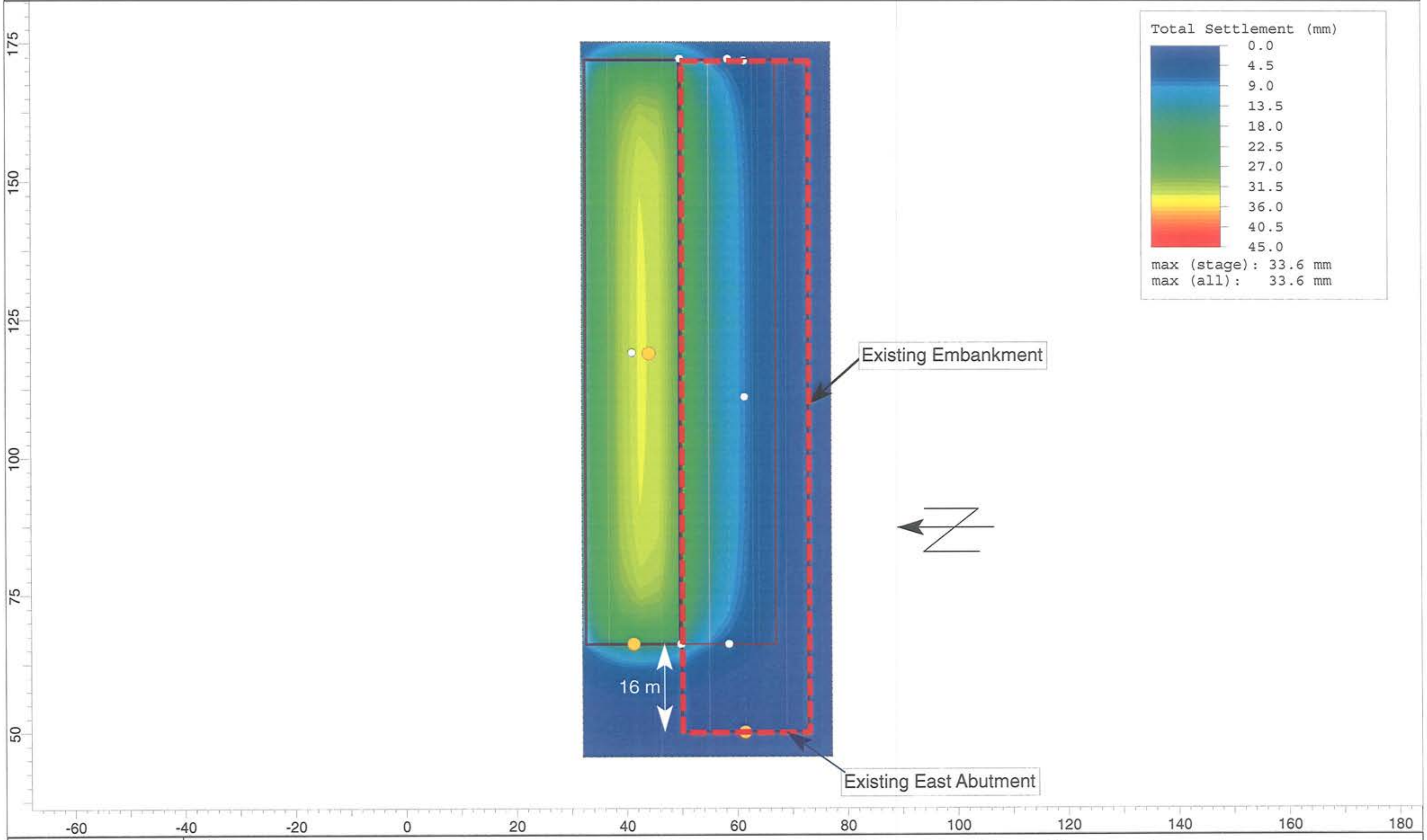
Seismic Stability Analysis

Highway 9 Holland Drainage Canal Bridge Replacement

Figure 15c

Project No. 165000801
GWP No. 2188-08-00

Stage 3 | Data Type: Total Settlement

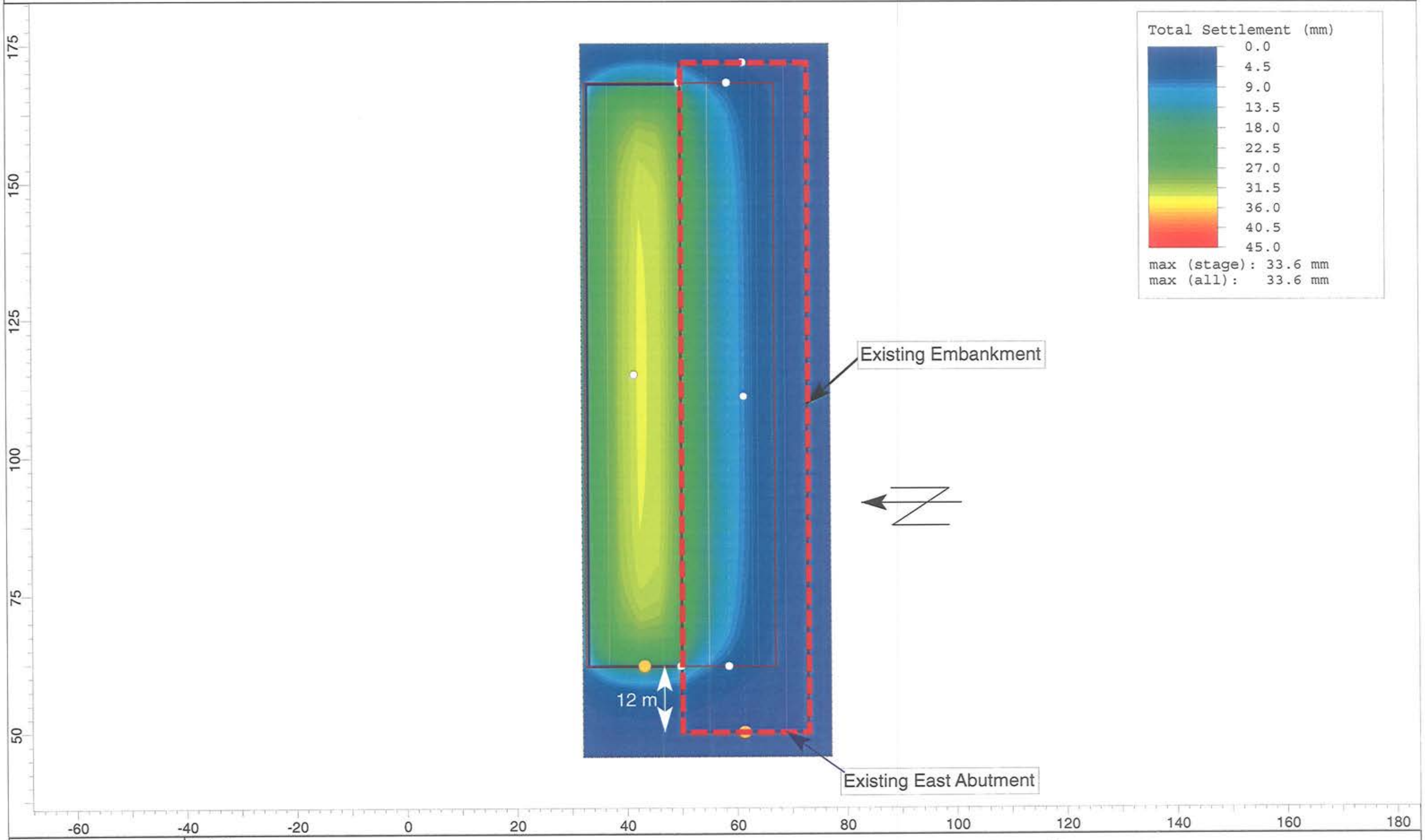


Stantec

Project	Holland Drainage Canal Bridge Replacement - East Abutment		
Project No.	165000801	Figure No.	Figure 16a
Analyzed By	LB	Date	11/6/2012
File	V:\01224\active\other_pc_projects\165000801\Analysis\Settle3D\east_abutment_16m_offset.s3z		

Stage 3

Data Type: Total Settlement



Stantec

Project	Holland Drainage Canal Bridge Replacement - East Abutment		
Project No.	165000801	Figure No.	Figure 16b
Analyzed By	LB	Date	11/6/2012
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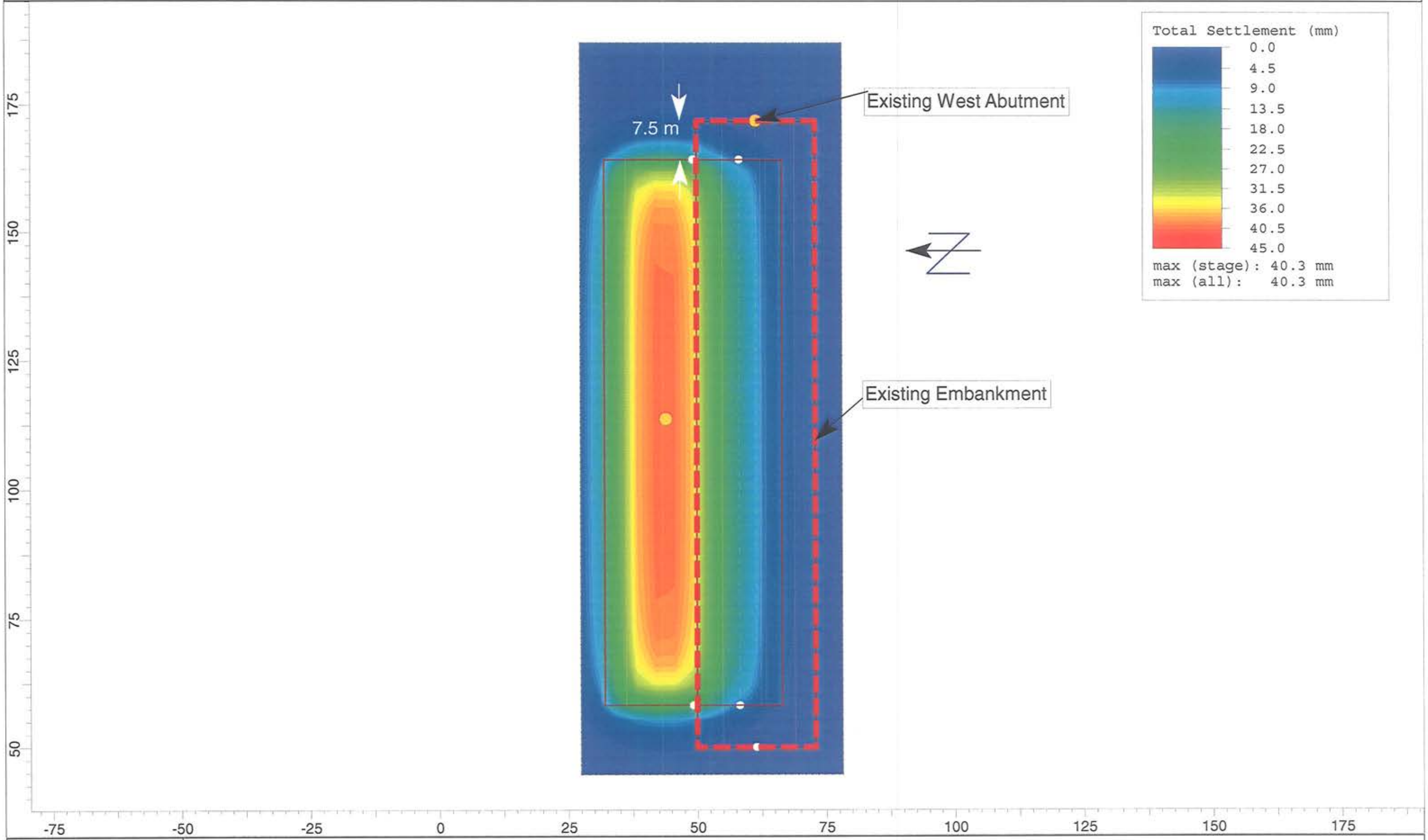
Stage 2

Data Type: Total Settlement



Stantec	Project	Holland Drainage Canal Bridge Replacement - West Abutment		
	Project No.	165000801	Figure No.	Figure 17a
	Analyzed By	LB	Date	11/6/2012
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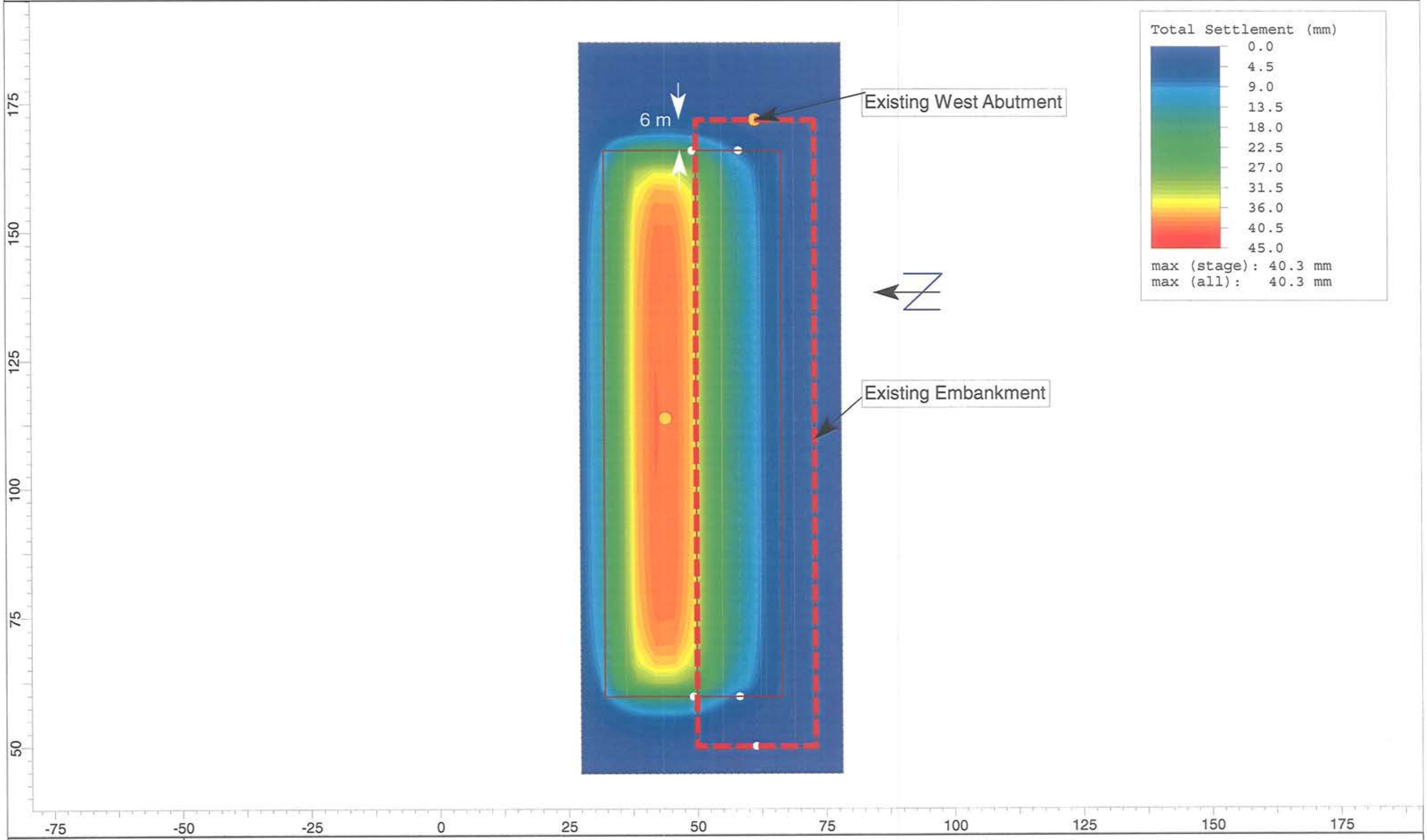
Stage 2	Data Type: Total Settlement



Stantec				
	Project	Holland Drainage Canal Bridge Replacement - West Abutment		
	Project No.	165000801	Figure No.	Figure 17b
	Analyzed By	LB	Date	11/6/2012
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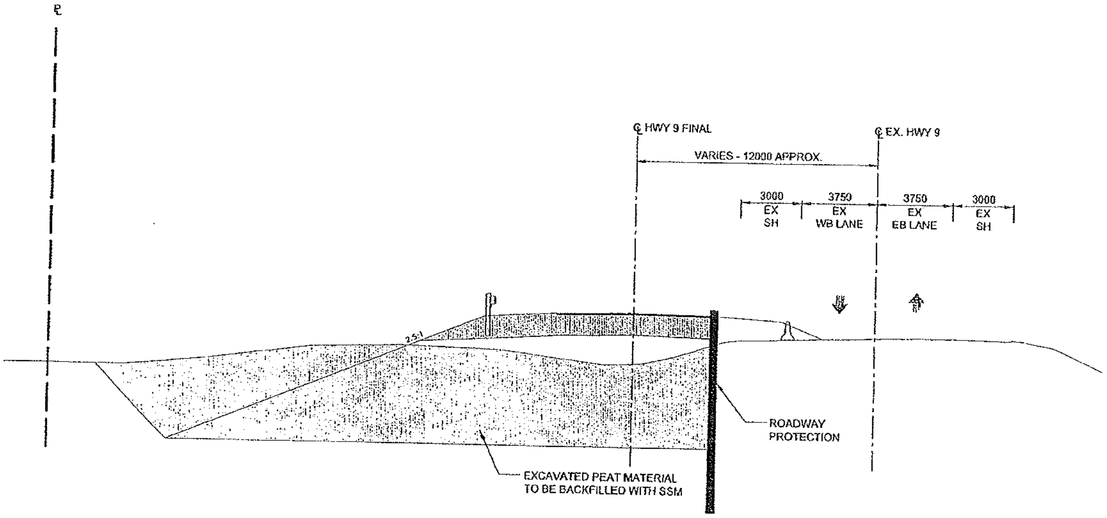
APPENDIX E

Schematic of Roadway Protection System Location

OPSD 203.030 Embankments Over Swamp

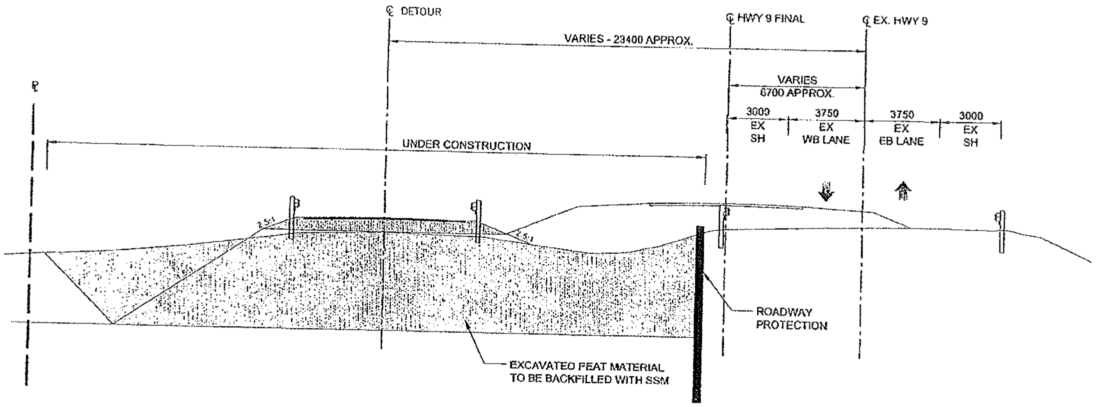
Dewatering NSSP

PRELIMINARY ALTERNATIVES



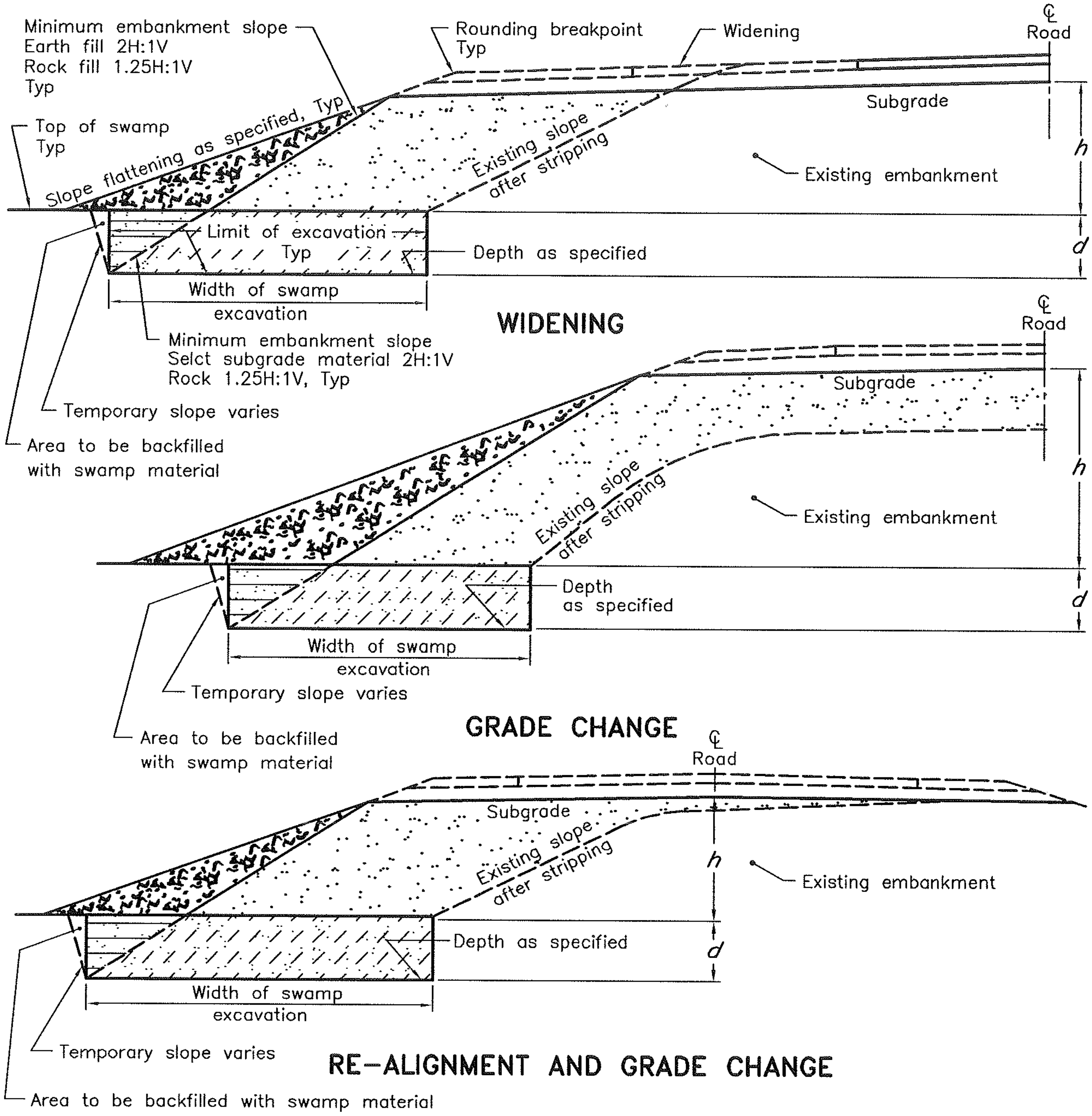
ALTERNATIVE 1 - STAGE 1

STA 20+650



ALTERNATIVE 2 - STAGE 1

STA 20+650



WIDENING

GRADE CHANGE

RE-ALIGNMENT AND GRADE CHANGE

NOTES:

A For this OPSD, $h \leq 4.5$m and $d \leq 6.0$m.

B Topsoil shall be stripped from existing slopes.

C Height of fill is the vertical difference between subgrade and top of swamp measured at new road centreline.

D Widening of existing earth embankments shall be benched according to OPSD 208.010.

LEGEND:

h – Height of fill

d – Depth of sub-excavation

⁘	Embankment materials as specified
▩	Excavated swamp material
▨	Excavate and backfill as specified
▤	Excavate and backfill with swamp material

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 Rev 3

EMBANKMENTS OVER SWAMP

EXISTING SLOPES MAINTAINED

OPSD 203.030

DEWATERING—Item No.

Special Provision

SCOPE

The work required for the above tender item shall include consideration of dewatering to provide stable earth platform.

CONSTRUCTION

The contractor is advised of the following:

- Excavation is required to remove the existing organic silt and for the construction of the approach embankments and the roadway widening.
- The groundwater level was measured at elevation 219.2 m in Borehole BH12-3 on April 18, 2012.
- The water level in the Holland Drainage Canal was measured at elevation 218.72 m in November 2011 and 219.2 m in April 2012.
- The contractor shall consider that seasonal groundwater fluctuations may result in higher groundwater levels than observed and that higher groundwater levels may result in an unstable working earth platform.
- The anticipated excavation level for organic silt removal is approximately 215.5 m which is approximately 3 m below the water level in the canal noted above.
- The presence of cohesionless sand near the excavation bottom can render the soils susceptible to unbalanced hydrostatic head, soil sloughing and cave-in.
- The contractor shall consider the site conditions, sequence of work and schedule when assessing requirements for dewatering.

Requirements for dewatering are contained in OPSS 517.

Requirements for the control of water during construction are contained in OPSS 518.

BASIS OF PAYMENT

Payment at the Contract price for the appropriate tender items associated with dewatering shall include full compensation for all labour, Equipment and Material to do the work.