



Stantec

**FINAL
Foundation Investigation and
Design Report**

18th Concession Drain Bridge
Replacement
Highway 40 (South of Wallaceburg),
Site 13-45
Municipality of Chatham-Kent

G.W.P. 3103-03-01

GEOCRES No. 40J9-21

Project No. 165000744
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Stantec
FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT

FOUNDATION INVESTIGATION REPORT
For
G.W.P 3103-03-01

18th Concession Drain Bridge Replacement
Highway 40 (South of Wallaceburg), Site 13-45
Municipality of Chatham-Kent

1.0 Introduction

Stantec Consulting Ltd. (Stantec) was retained by the Ministry of Transportation, Ontario (MTO) to undertake the detailed design for the replacement of the 18th Concession Drain Bridge on Highway 40 south of the community of Wallaceburg in the Municipality of Chatham-Kent, Ontario.

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

Project Number: G.W.P.: 3103-03-01

Project Location: Highway 40 (Murray Street) near Elbow Line, Chatham-Kent

The work was carried out under Agreement Number 3008-E-0039 with Stantec Consulting Ltd., the Detailed 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 site is located near the intersection of Highway 40 and Elbow Line approximately 2.5 km south of the community of Wallaceburg in the Municipality of Chatham-Kent, Ontario.

General Site Description

At the project site, Highway 40 crosses the 18th Concession Drain with a single span bridge (± 12.9 m) which was originally constructed in 1937. Highway 40 runs in the north-south direction with chainage increasing from south to north. 18th Concession Drain runs in the NE-SW direction. Highway 40 has a single lane in each direction and the bridge has a deck width of 9.1 m. Elbow line has a staggered intersection with Highway 40 to the south of the bridge. The east leg of Elbow Line is less than 10 m from the bridge while the west leg is approximately 20 m to 30 m from the bridge.

Physiographic Description and Drainage

The site is located within a physiographic region known as the St. Clair Clay Plains. The region contains extensive clay plains with little relief. Generally, drainage is towards Lake St. Clair to the south-west. Because of the faint relief, dredged ditches and drains are used to facilitate or improve drainage. The prevailing surficial soil deposit consists of Brookston clay loam, a dark-surfaced gleysolic soil developed under a swamp forest of elm. The clayey surficial deposit is underlain by limestone bedrock. In the vicinity of the project site the terrain is fairly flat.

The 18th Concession Drain is part of a local drainage canal system which ultimately discharges to Lake St. Clair via Sydenham River and the Chenal Ecarté. At the bridge site, the drainage canal is a linear feature flowing slowly towards the Townline Drain located 400 m to the west.

3.0 Investigation Procedures

3.1 DRILLING INVESTIGATION

The geotechnical investigation for the bridge foundations for the proposed replacement bridge included four boreholes in the vicinity of the proposed bridge. These boreholes are designated BH11-1 through BH11-4 and are shown on the Borehole Locations and Soil Strata drawing, Drawing No. 1 in Appendix A. One borehole was advanced for each abutment and each bridge approach. Prior to carrying out the investigation, Stantec contacted the public utility authorities to clear the borehole locations of both private and public utilities.

The field drilling program was carried out between April 18 and May 5, 2011. The boreholes were advanced with continuous flight hollow stem augers using a D120 Track-mounted drill rig equipped for soil and bedrock sampling.

The subsurface stratigraphy encountered in each borehole was recorded in the field. Standard Penetration Tests (SPT) were carried out in all holes and split spoon samples were collected. Four Shelby Tube samples were also retrieved. All SPT samples recovered were returned to our Ottawa laboratory for detailed classification and testing. The Shelby Tube samples were sent to the Golder Associates Mississauga laboratory for consolidation and unconfined strength testing.

A standpipe was installed in Borehole BH11-4 after completion of drilling. It consisted of a 50 mm PVC pipe slotted over the lower 5 m. The annulus around the pipe was backfilled with sand for 5.3 m below a clay plug. The upper portion was backfilled with auger cuttings and bentonite to the ground surface. Asphalt was used to create a surface seal.

Bedrock was confirmed by coring HQ size rock cores in Boreholes BH11-2 and BH11-3. Coring was conducted in these boreholes to depths of approximately 3.0 m and 4.2 m, respectively, beneath split-spoon refusal.

Summary information pertaining to the boreholes included in this report is given in Table 3.1.

Table 3.1: Borehole Information Summary

	Boreholes			
	BH11-1	BH11-2	BH11-3	BH11-4
Station	31+064	31+077	31+098	31+111
Offset, m	2.4 Lt	1.8 Rt	2.4 Lt	1.9 Rt
Ground Surface Elevation, m	176.6	176.7	176.9	176.9
Total Depth Drilled, m	9.6	28.0	28.6	9.6
End of Borehole Elevation, m	167.0	148.7	148.3	167.3
Depth Augered, m	9.6	25.0	24.4	9.6
Depth Cored and tri-coned, m	0	3.0	4.2	0
Number of Soil Samples	9	16	14	9

Note: The station and offset information are with respect to the centerline chainage of Highway 40

3.2 LOCATION AND ELEVATION SURVEY

The ground surface elevation at each borehole location was surveyed by Stantec personnel on April 27, 2011, with reference to the nearest geodetic benchmark. This benchmark was on a tablet in the east face of the coping on the north concrete abutment of the existing bridge (GBM 3048). The geodetic elevation of this benchmark was 176.74 m. The MTM reference coordinates for this site are with respect to Zone 11.

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	55	2 by Golder
Atterberg Limits	15	2 by Golder
Grain Size Distribution	15	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.

Two 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 four boreholes included in this report 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 in Appendix B.

In general, the subsurface stratigraphy consisted of pavement and fill material over silty sand or sandy silt overlying firm to stiff silty clay underlain by a thick deposit of soft to firm silty clay overlying a stiff gravelly clay till over shale bedrock.

A borehole location plan and a stratigraphic section of the soil encountered within the boreholes are provided on Drawing No. 1 in Appendix A.

4.1.1 Pavement and Fill Material

The pavement structure observed on site included:

Asphalt	150 mm to 380 mm
Sand/Gravel Fill	100 mm (at BH11-1 and BH11-4)
Concrete	200 to 230 mm (at BH11-1, BH11-3, BH11-4)

At Borehole BH11-2 where concrete was not observed, the asphalt is directly underlain by 300 mm of crushed sand and gravel.

4.1.2 Organic Layer / Topsoil

Within Boreholes BH11-1 and BH11-4, an organic layer was observed directly beneath the concrete; the observed layer thickness was 100 mm and 300 mm at the respective locations.

4.1.3 Silty Sand to Sand/Sandy Silt

A 400 mm silty sand to sand (SM) layer was observed in Borehole BH11-2. A 300 mm sandy silt (ML) layer was also observed in Borehole BH11-3. A 1.6 m of sandy silt (ML) layer interbedded with clayey silt (CL) was observed in Borehole BH11-4. For these layers, moisture content testing was completed on 7 samples and yielded the range of 4.4% to 22.5%.

4.1.4 Stiff Silty Clay

A stiff to very stiff layer of silty clay was observed beneath fill or sandy material in all boreholes. The layer was between 1.9 m and 3.5 m thick, with a base elevation ranging from 172.5 m to 174.7 m.

Grain size analysis and Atterberg Limit tests were completed on six samples and moisture content testing on 27 samples. The test results are summarized as follows:

- Gravel 0%
- Sand 1% to 48%
- Fines 52% to 99%
- Liquid Limit 23 to 34
- Plastic Limit 6 to 20
- Moisture Content 21% to 30%

Fines noted above represent both silt size and clay size particles.

The results of laboratory testing indicate that the silty clay layer can be classified as (CL). The grain size distribution curves and Atterberg Limits are shown on Figures 1a to 2d in Appendix C.

The layer was generally observed to be stiff based on pocket penetrometer results which ranged from 70 to 200 kPa and SPT results (typical N values from 3 to 11 blows per 0.3 m).

Two samples retrieved from this layer were analyzed for pH, water soluble sulphates and chloride concentrations, and resistivity. The analysis results are provided in Table 4.1.

Table 4.1: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH11-2	SS-2	0.76 to 1.4	7.8	661	83	8.36
BH11-3	SS-3	1.5 to 2.1	7.8	395	88	12.8

4.1.5 Soft to Firm Silty Clay

A deposit of silty clay was observed beneath the stiff silty clay layer in all boreholes. The silty clay layer was fully penetrated at Boreholes BH11-2 and BH11-3 where it was observed to be 21.4 m and 21.2 m thick, with corresponding base elevations of 152.3 m and 153.5 m.

The upper portion of the silty clay, above approximate elevation 172.5 m (4.1 to 4.4 m below ground surface), is generally dryer with a consistency of generally stiff to very stiff.

Below elevation 172.5 m, the silty clay is generally wetter with a consistency of soft to firm.

Grain size analysis and Atterberg Limit tests were completed on seven samples and moisture content testing on nineteen samples. The test results are summarized as follows:

- Gravel 0%
- Sand 0% to 1%
- Fines 99% to 100%
- Liquid Limit 33 to 41
- Plastic Limit 19 to 21
- Specific Gravity 2.74 and 2.75
- Moisture Content 21% to 50%
 - Generally below 40% above elevation 172.5 m
 - Generally above 40% below elevation 172.5 m

Fines noted above represent both silt size and clay size particles.

The results of laboratory testing indicate that the silty clay deposit can be classified as (CI). The grain size distribution curves and Atterberg Limits are shown on Figures 1a to 2d in Appendix C.

Below elevation 172.5, the layer was generally observed to be soft based field vane results which ranged from 18 to 89 kPa (with an average of 23.9 kPa) and SPT results (typical N values from 1 to 5 blows per 0.3 m). Unconfined compressive strength testing on two samples for BH11-2 yielded values of 74 kPa and 19 kPa.

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

Table 4.2: Consolidation Test Results

Sample ID	Sample el.	Moisture Content	Initial Void Ratio/Initial Unit Weight	Estimated Preconsolidation Pressure, P' _c	Recompression Index, C _r	Compression Index, C _c
BH11-2 ST-7	172.1 m	33%	0.9/18.8 kN/m ³	150 kPa	0.06	0.28
BH11-2 ST-12	164.5 m	49%	1.33/17.2 kN/m ³	75 kPa	0.13	0.40

4.1.6 Gravelly Clay Till

A layer of stiff gravelly clay till was observed beneath the silty clay deposit in boreholes BH11-2 and BH11-3. The layer was 0.6 m and 1 m thick, with a base elevation of 151.7 m to 152.5 m.

Moisture content testing was completed on two samples and yielded the values of 36% and 19%. Occasional cobbles were observed in the layer. It is noted that although not observed in the boreholes drilled at this site, boulders are frequently encountered in glacial till deposits. The material can be classified as gravelly clay (CI), till.

4.2 BEDROCK

Dark grey shale bedrock was encountered in Boreholes BH11-2 and BH11-3. The bedrock was confirmed by tri-coning and coring approximately 3 and 4.2 m, respectively, into the bedrock. Bedrock was encountered at elevations of 152.5 and 151.7 m (approximately 25.0 and 24.4 m below existing ground surface).

The rock core recovery ranged between 96 and 100%. The rock quality designation (RQD) ranged between 46 and 83%, indicating generally fair to good rock mass quality. Rock core photographs are provided in Appendix B.

Unconfined compressive strength tests were carried out on one bedrock sample from borehole BH11-2 and two bedrock samples from BH11-3. The results of these tests are summarized in Table 4.3.

Table 4.3: Unconfined Compressive Strength of Rock Cores

Borehole No	Ground Surface Elevation (m)	Test Elevation (m)	Unconfined Compressive Strength (MPa)
BH11-2	176.7	150.5	61.8
BH11-3	176.9	150.7	53.6
		149.4	65.1

4.3 GROUNDWATER

A standpipe was installed in Borehole BH11-4 after completion of drilling and the water level was measured on April 28, 2011. The measured groundwater level was at a depth of 2.7 m (Elevation of 174.2 m).

Due to the cohesive nature of the silty clay deposit, the depth to groundwater within boreholes BH11-1 to BH11-3 could not be detected during soil sampling.

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

5.0 Miscellaneous

The field work was carried out under the supervision of Mr. Dan Stunden, Technologist, under the direction of Mr. Paul Carnaffan, M.Eng., P.Eng.

The drilling equipment was owned and operated by Walker Drilling Ltd. of Utopia, Ontario.

Geotechnical laboratory testing was carried out at the Stantec Ottawa laboratory and the Golder Associates Mississauga laboratory. Chemical testing on soil samples was carried out by Paracel Laboratories in Ottawa.

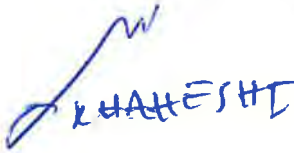
This report was prepared by Dr. Kasgin Khareshi Banab, Ph.D., and reviewed by Dr. Fred Griffiths, Ph.D., P.Eng. and Mr. Raymond Haché, M.Sc., P.Eng., MTO Designated Principal Contact.

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 and timeframe described herein. 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.

Respectively Submitted;

STANTEC CONSULTING LTD.



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FOUNDATION DESIGN REPORT

For

G.W.P. 3103-03-01

18th Concession Drain Bridge Replacement
Highway 40 (South of Wallaceburg), Site 13-45
Municipality of Chatham-Kent

7.0 Discussions

7.1 GENERAL

Project Purpose/Justification

At the project site Highway 40 passes over the 18th Concession Drain on a 12.9 m (on skew) long, single-span concrete tee beam bridge supported on pile foundations. Information on existing pile length and foundation conditions is not available. The bridge is approximately 9.1 m wide and has two lanes.

It is proposed to replace the existing bridge with a wider and longer single-span structure.

Proposed Bridge

The preliminary general arrangement of the replacement bridge indicates the following:

- The profile is anticipated to be raised by 50 mm on the south end of the bridge and by 250 mm at the north end. The finished grade at centerline will be 176.7 m and 176.8 m at the south and north ends respectively.
- Single-span bridge (20.2 m on skew) constructed on the same horizontal alignment as the existing.
- Permanent structure and approach fills to be widened from 11.48 m to 14.1 m.
- The new bridge deck will be constructed such that the increase in width will be exclusively on the east side of the existing deck footprint. The widening at the south end of the bridge will be within an area where the ground surface is already at the bridge profile grade and therefore no significant increases in profile grade is anticipated to accommodate the bridge deck widening. At the north end of the bridge, less than 4 m in length of roadway platform will need to be widened to accommodate the widened bridge deck.

The proposed abutments are located approximately 2.0 m to 5.0 m from the existing abutments. Therefore, it is anticipated that the existing piles will remain in place after removal of the existing abutments.

7.2 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions at this site generally consist of pavement and fill materials over stiff silty clay underlain by a thick deposit of soft to firm silty clay over a thin till layer and shale bedrock. The lower portion of the silty clay appears to be normally consolidated.

Table 7.1 provides an idealized soil model for the conditions observed on site.

For the cohesive silty clay (CL) of low plasticity and silty clay (CI) of intermediate plasticity, the design parameters are provided on profiles on Figures 3a and 3b in Appendix D. The design parameters provided include the following:

- Total Unit Weight (γ)
- Undrained Shear Strength (S_u)
- Preconsolidation Pressure (P'_c)
- Void Ratio (e_o)
- Compression Index (C_c)
- Recompression Index (C_r)
- Secondary Compression Index (C_{α})

Table 7.1: Representative Soil Profile (Geotechnical Model)

Elevation (m)		Soil Type	Design Parameters
176.9	176.0	Fill Material	$\gamma = 20.5 \text{ kN/m}^3$, $\phi = 30^\circ$, $E = 15 \text{ MPa}$
176.0	172.5	Silty Clay (CL)	From Figures 3a and 3b in Appendix D
172.5	153.5	Silty Clay (CI)	From Figures 3a and 3b in Appendix D
153.5	152.0	Gravelly Clay Till with Cobbles	$\gamma = 21 \text{ kN/m}^3$, $S_u = 100 \text{ kPa}$

Note: (1) γ = total unit weight (kN/m^3).
 ϕ = soil friction angle ($^\circ$).
 E = soil modulus (MPa).
 Design water elevation = 174.4 m.

It is noted that the glacial till deposit may contain boulders.

7.3 FROST PROTECTION

OPSD3090.101 indicates that the frost penetration depth at the project site is 1.0 m. Therefore, footings and pile caps should be provided with a minimum of 1.0 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

7.4.1.1 Soil Profile Type

It is recommended that a Soil Profile Type III as defined in Canadian Highway Bridge Design Code (CHBDC, 2006) Section 4.4.6 be used in the seismic design of this site.

7.4.1.2 Zonal Acceleration Ratio

Table A3.1.1 of the CHBDC indicates that the Zonal Acceleration Ratio (ZAR) for Wallaceburg, Ontario, which is approximately 2.5 km north of the site, is 0.00. A seismic hazard calculation for the site was obtained from Natural Resources Canada (copy attached in Appendix E). It indicates that for this site, the peak ground acceleration (PGA) value corresponding to 10% exceedance in 50 years is 0.04, which is greater than the ZAR for Wallaceburg. Hence, a ZAR of 0.04 should be used for this site.

7.4.1.3 Liquefaction Potential

The site is underlain by a deep deposit of soft to stiff clay overlying Shale bedrock. The depth to bedrock is in the order of 25 m beneath existing ground surface. Based on the observed site soil condition and the low zonal acceleration ratio for the site, no liquefaction of the site soil and hence no liquefaction-induced hazard to the bridge is anticipated.

7.5 FOUNDATION OPTIONS

7.5.1 General

The silty clay soils encountered at the bridge site are not considered suitable to support shallow spread footings. Deep foundations extending to bedrock are recommended to support the bridge abutments.

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 Driven to Bedrock	<ul style="list-style-type: none"> Minimize disturbance to the clay (non-displacement) Commonly used for integral abutment bridges Will penetrate till with relative ease 	<ul style="list-style-type: none"> Cannot be internally inspected Attracts larger drag loads compared to circular steel piles 	Medium	No significant risks
Closed End Pipe Piles filled with Concrete	<ul style="list-style-type: none"> Reduced drag loads where applicable Can be internally inspected for alignment and damage 	<ul style="list-style-type: none"> Significantly stiffer than H-Pile and therefore may not be suitable for integral abutments Will have more difficulty penetrating the till, therefore would need to be designed end-bearing on till More risk of overdriving /damaging of piles 	Low	<ul style="list-style-type: none"> Risk of refusal within the till would require use of till base resistance Risk of pile damage during installation would require additional piles
Drilled Caissons	<ul style="list-style-type: none"> Very high axial capacity 	<ul style="list-style-type: none"> Would require a liner due to presence of soft silty clay Very stiff cross-section may not be suitable for integral abutment 	High	<ul style="list-style-type: none"> Bedrock permeability not known, therefore, dewatering could become an issue/ Tremie concrete methods could address this issue

Given that a comparable pipe pile would be at least 3 times stiffer than an H-pile, it is recommended that H-piles be carried forward as the most suitable deep foundation option for an integral abutment..

7.5.2 Deep Foundations

7.5.2.1 Geotechnical Axial Resistance

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 foundation consisting of HP310x110 piles are recommended to support the proposed integral abutments 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 172.5 m. The piles would extend approximately 20.5 m to bedrock at approximate elevation of 152.0 m).

A factored axial resistance in compression at ULS for an HP310x110 pile of 2000 kN may be used. This resistance at ULS assumes that the piles are successfully driven to competent bedrock. Due to the piles being driven to bedrock the axial resistance at SLS is not applicable.

For this project, axial geotechnical resistance in tension or pull-out capacities of the piles is not anticipated to be required for design purposes.

7.5.2.2 Potential Drag Loads Associated with the Settlement of Soft Clay

The silty clay soils which underlie the site has been interpreted as being normally consolidated below el. 168 m and therefore the potential for soil settlement induced drag loads has been considered.

As part of evaluating the potential for drag loads, the following was considered.

- The thickness of the asphalt observed in the foundation boreholes was initially considered an indication that settlement of the approach fills may have occurred in the past. Asphalt thicknesses along Highway 40 from the 2011 pavement investigation by Stantec are summarized below.

31+040	355 mm
31+060	205 mm
31+069	240 mm
31+077	200 mm (BH11-2)
31+084	Existing South Abutment
31+092	Existing North Abutment
31+098	380 mm (BH11-3)
31+107	350 mm
31+116	370 mm
31+136	330 mm

Based on the above summary, it is likely that the asphalt thickness observed near the existing bridge is not associated with historic settlements.

- The proposed modifications to the cross-section and profile at the bridge site includes approximately 2 m of platform widening for a limited length of 4 m on the east side of Highway 40 and a grade raise of between 25 to 250 mm.
- The maximum calculated settlements associated with the proposed grade raise was estimated to be less than 20 mm at approximately 20 m north of the north abutment face. At the north abutment the anticipated settlements are anticipated to be less than 10 mm. Settlement estimates were carried out using the geotechnical model presented in Appendix D and Settle3D software by Rocscience Inc.

On the basis of the above information, drag loads due to soil settlement do not need to be incorporated in the pile design.

7.5.2.3 Geotechnical Lateral Resistance

The geotechnical resistance of the pile against lateral loads is mobilized due to the passive resistance of the surrounding soil. Assessed values for horizontal passive resistance and geotechnical resistances at SLS for the proposed pile can be generated from information provided in Table C6.4 of the Commentaries to the Canadian Highway Bridge Design Code (CHBDC, 2006) for firm cohesive material.

7.5.2.3.1 ULS Resistance

For cohesive soils, passive soil resistance should be limited to $2c_u$ (twice the undrained shear strength) at the surface and increase linearly to $9c_u$ at the depth of three pile diameter (section C6.8.7.1 of CHBDC, 2006). The geotechnical resistance factor would be 0.5 at ULS. By using bearing width equal to the pile width over a length of six pile widths, the factored passive resistance at ULS is calculated to be approximately 150 kN.

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

7.5.2.3.2 SLS Resistance

The lateral geotechnical resistance at SLS was evaluated using the program LPILE Plus v6.0 developed by Ensoft, Inc. (Ensoft, 2010). The input parameters used are given in Table 7.3. A moment of inertia of $22.8 \times 10^7 \text{ mm}^4$ was used for a 310x110 pile section. A modulus of elasticity of 200 GPa was used for the pile material (steel). The pile was modeled with a total length of 20.5 m and driven to competent bedrock. The p-y modulus values were based on values suggested by Ensoft, Inc. (Ensoft, 2010).

Table 7.3: Parameters Used for Lateral Resistance at ULS and SLS for Piles

Soil Layer	Depth Range (m)		Unit weight, γ	Undrained shear strength, S_u	p-y Modulus, k
	From	To	kN/m ³	kPa	kN/m ³
Stiff silty clay	172.5	171	18.5	75	135,000
Soft silty clay	171	152	18	30	8140
Bedrock		<152	24	NA	-----

Notes: Base of pile cap was placed at elevation of 172.5 m.

Top 3 m of pile is surrounded by CSP filled with loose sand.

Two plots from LPILE are presented in Figures 4 and 5 in Appendix D. Figure 4 shows the deformed shape of the pile for lateral (shear) force ranging between 15 and 135 kN. This plot indicates that the pile would undergo negligible deflection below a depth of approximately 8.0 m from the underside of the pile cap.

Figure 4 in Appendix D illustrates the displacement of the pile in depth for different lateral loads. According to this Figure, a lateral load of 75 kN would correspond to a pile top displacement of 10 mm. Therefore, the SLS geotechnical Resistance of an HP 310x110 has been calculated as 75 kN.

In the case where the structural designers wish to model the soil using linear elastic springs, the spring stiffness provided in Table 7.4 may be used in the case where:

- i) Spring are spaced 250 mm apart and
- ii) Where the pile consists of a HP 310x110.

Table 7.4: Recommended Stiffness of Soil Springs in Different Depth of Pile at SLS

Soil Layer	Elevation Range (m)		Spring Spacing (m)	Target Deflection (mm)	Stiffness of Soil Spring, k (kN/m)
	From	To			
Stiff silty clay	172.5	172	0.25	10	1360
Stiff silty clay	172	171	0.25	6	1700
Firm silty clay	171	170	0.25	3	1335
Firm silty clay	170	168	0.25	1	2400
Soft to firm silty clay		<168	0.25	1	2400

7.6 LATERAL EARTH PRESSURES

7.6.1 Static Lateral Earth Pressures

Earth pressures will need to be considered in the design of abutments and retained soil systems (if applicable). 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 CHBDC. For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. The unfactored soil parameters provided in Table 7.5 may be used for design of walls with a horizontal backfill. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC.

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

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

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

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

Table 7.5: Recommended Lateral Earth Pressure Parameters

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II
Bulk Unit Weight, γ (kN/m ³)	21.2	22.8
Effective Friction Angle	32°	35°
Coefficient of Active Earth Pressure (K_a)	0.31	0.27
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43
Coefficient of Passive Earth Pressure (K_p)	3.25	3.69

7.6.2 Seismic Lateral Earth Pressures

The abutments and the retained soil systems (if applicable) 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 Table 7.6.

The total active (P_{AE}) and passive (P_{PE}) thrusts under seismic loading can be calculated using the following equations:

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

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

where:

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

Seismic earth pressure coefficients are given for non-yielding retaining structures

Table 7.6: Recommended Seismic Earth Pressure Parameters (non-yielding structures)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II
Bulk Unit Weight, γ (kN/m ³)	21.2	22.8
Effective Friction Angle	32°	35°
Coefficient of Active Earth Pressure (K_{AE})	0.34	0.31
Coefficient of Passive Earth Pressure (K_{PE})	3.14	3.57
Height of Application of P_{AE} from base as a ratio of wall height, (H)	0.352	0.353
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.313	0.312

7.7 STABILITY OF SLOPES

A slope stability evaluation was carried out using commercially available limit equilibrium based software called SLOPE/W (GEO-SLOPE, 2007). The analyses considered seismic loading using one-half of the ZAR. Long-term soil parameters ($\phi' = 30^\circ$, $c' = 2 \text{ kPa}$) were used in the analysis (see Soderman et al., 1961).

A 2H:1V slope is considered for the slope extending from the ground surface to the drain canal bed. Slope stability analysis results are presented in Figure 6 Appendix D. The slope stability evaluation results indicate that the failure planes generally tend to be relatively shallow. Based on this preliminary slope stability analysis, the factor of safety against the shallow critical failure plane is obtained 1.59 which exceeds the target value of 1.3 for highway embankments.

7.8 TEMPORARY ROADWAY PROTECTION

Temporary roadway protection at the locations shown on Drawing No. 2 in Appendix D is anticipated to form part of the staged construction approach that will be required to maintain traffic flow. Provided an appropriate unwatering system is in place both sheet-pile and soldier pile and lagging systems are feasible. The sheet-pile likely can be designed to provide support by cantilevering. The soldier pile and lagging would likely require bracing for support.

Computation of earth pressures should be in accordance with Section 6.9 of the Canadian Highway Bridge Design Code (CHBDC). For roadway protection with a horizontal backfill, the unfactored soil parameters provided in Table 7.7 may be used for design. For the second stage of excavation the effects of backfill compaction should be accounted for by estimating the compaction-induced load in accordance with the methodology described in Figure 6.6 of the CHBDC.

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

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

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

Where H is the height of the wall and the values for K_a , K_p , and γ are provided below. 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.

Table 7.7: Recommended Lateral Earth Pressure Parameters

Parameter	Pavement Structure Materials	Sandy Silt	Silty Clay
Total Unit Weight, γ (kN/m ³)	23	21	18
Effective Friction Angle, ϕ	35°	29°	30°
Effective Cohesion (kPa)	0	0	2
Coefficient of Active Earth Pressure (k_a)	0.27	0.35	0.33
Coefficient of Earth Pressure at Rest (K_o)	0.43	0.51	0.5
Coefficient of Passive Earth Pressure (K_p)	3.69	2.88	3.00

Shoring design should meet the requirements of Performance Level 1b as per OPSS 539 and should consider traffic loading. Performance Level 1b specified a Maximum Angular Distortion of 1:1000 and a Maximum Horizontal Displacement of 10 mm.

7.9 CONSTRUCTION CONSIDERATIONS

7.9.1 Construction Staging

The replacement of the 18th Concession Drain Bridge is anticipated to involve a staged construction. This will involve closure of one lane of the road by using appropriate traffic control and would likely require the use of temporary roadway protection near the centerline of the highway (see Section 7.8 above).

7.9.2 Excavation and Backfilling

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

Any vegetation, fill, organic soils and other deleterious materials must be removed from beneath the abutments of the bridge and the embankments. Where deleterious materials are encountered, the material should be excavated, wasted and replaced. The lateral extent of such excavation should include all deleterious material within the influence zone of the embankments.

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

Any side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects (OHSA). Within the anticipated excavation depths, the soils encountered in the boreholes would be considered a Type 3 soil.

7.9.3 Temporary Construction Unwatering

It is anticipated that the abutment excavations are going to extend below the water level in the 18th Concession Drain. The following should be considered:

- The excavation will require isolation from the drain water by using either a sheet-pile coffer dam or alternatively an aqua-dam. An aqua-dam is suggested as a possible alternative to a cofferdam given that the native soils upon which it would rest is a silty clay of low permeability (less than 10^{-6} cm/sec) and the expected water depth to be retained is less than 2.0 m.
- The native soils are of low permeability and therefore unwatering within a cofferdam or an aqua-dam is anticipated to be handled using conventional sump and pump techniques.

7.9.4 Cement Type and Corrosion Potential

Two samples of the soil in the vicinity of the anticipated foundation for the bridge were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations for the bridge structure and any buried infrastructure. The analysis results are summarized in the Table 4.1.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected on concrete in contact with soil and groundwater at the site. The maximum concentration of soluble sulphate was 88 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected on 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 maximum reported soil pH was 7.8 which is 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 maximum reported resistivity of 12.8 (ohm-m) indicates that the soil is mildly corrosive. The test results provided in the Table 4.1 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

7.9.5 Pile Installation

The Ontario Provincial Standard Specification (OPSS 903) requirements for the supply and installation of deep foundation units should be followed. 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 Standard Detail, OPSD 3000.100 Type I.

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

Pile Driving Note 5 should be referenced in the contract drawings : "Piles to be driven to bedrock".

During both stages of construction, vibration monitoring is needed to ensure that the foundations of the other half of the bridge are not adversely affected. The maximum allowable peak particle velocity should be 100 mm/sec at the adjacent abutment for both stages of the construction.

8.0 Specifications

The following specifications are referenced in this report:

Table 8.1: Specifications Referenced in Report

Document	Title
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD.3000.100	Foundation, Piles, Steel H-Pile Driving Shoe Details
OPSD 3101.150	Walls – Abutment, Backfill – Minimum Granular Requirement
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 206	Construction Specification for Grading
SP 206S03	Earth Excavation, Grading
OPSS 539	Construction Specification for Temporary Protection System

9.0 References

- Becker, D.E. and K.Y. Lo, 1979, Settlement and Load Transfer of Ring Foundation for Tower Silos, Canadian Agricultural Engineering, Vol. 21, No. 2
- Canadian Foundation Engineering Manual, 4th Edition, 2006, Canadian Geotechnical Society, Richmond, British Columbia
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- CHBDC, 2006. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.
- Ensoft, 2010. User's Manual for Computer Program LPILE Plus Version 6.0. Ensoft, Inc., Austin, Texas.
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- Skempton, A.W. ,1957, Discussion on the planning and design of the new Hongkong airport, Proc. Inst. Civil Engrs., 7, pp 305-307

Soderman L. G., T. C. Kenney and A. K. Loh, 1961, Geotechnical Properties of Glacial Clays in Lake St. Clair Region of Ontario, Proceedings of Fourteenth Canadian Soil Mechanics Conference, Published by National Research Council of Canada

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FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT

10.0 Closure

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete.


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


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


This report has been prepared by Kasgin Khareshi Banab and reviewed by Fred Griffiths and Raymond Haché.

Respectfully submitted,

STANTEC CONSULTING LTD.


Kasgin Khareshi Banab Ph.D., E.I.T.


Fred J. Griffiths, Ph.D., P.Eng.
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Designated Principal MTO Foundation Contact



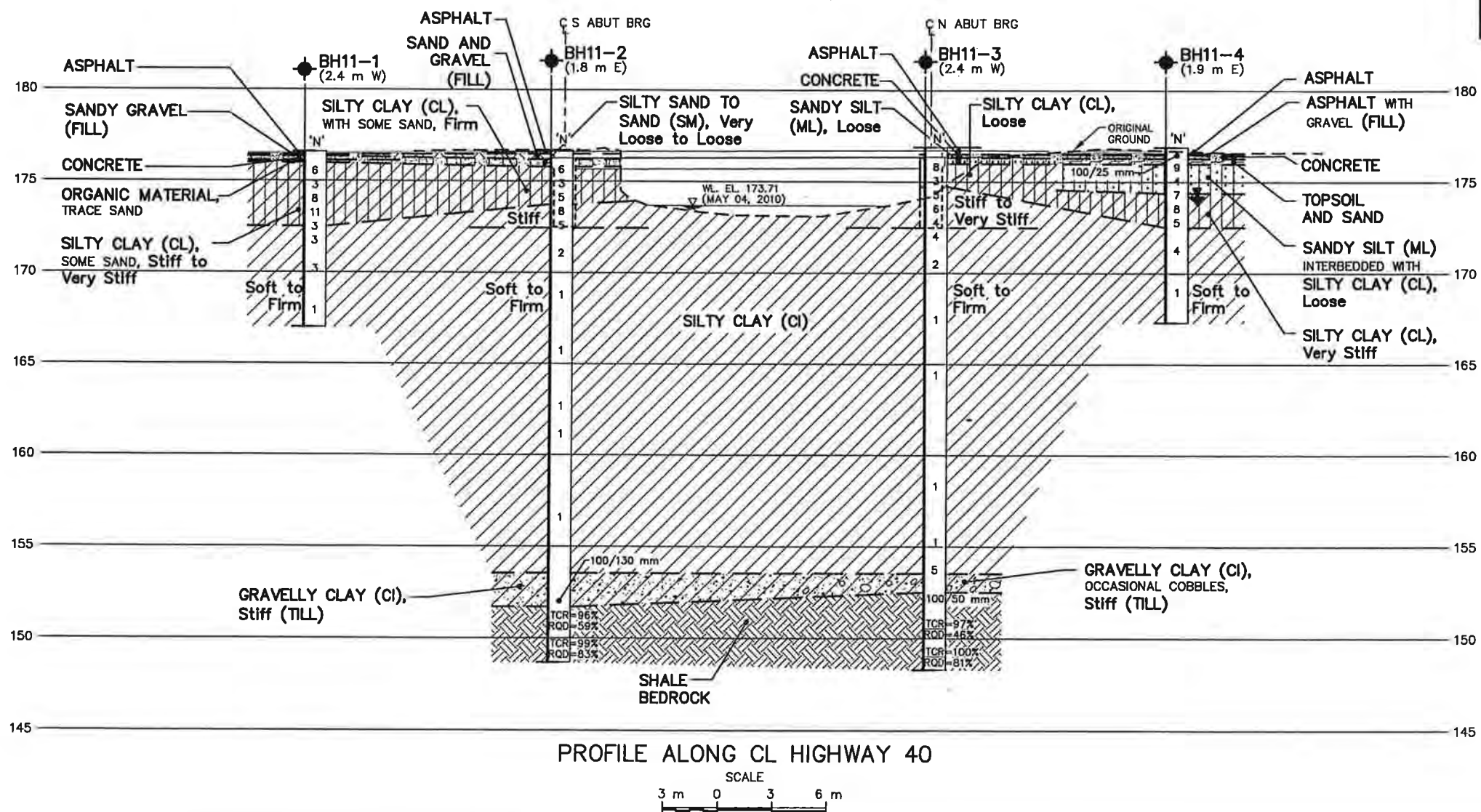
APPENDIX A

Drawings No. 1 – Borehole Location Plan and Soil Strata Plot

Site Photographs

DRAWING NAME: 185000744-1_Aug 3_04.dwg
CREATED BY: GBB
T: Autocad\Drawings\Project Drawings\2011\185000744\185000744-1_Aug 3_04.dwg (BORHOLES)
Printed: Aug 09, 2011
2011-08-09
MODIFIED:

MINISTRY OF TRANSPORTATION, ONTARIO
PR-D-707
88-13



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

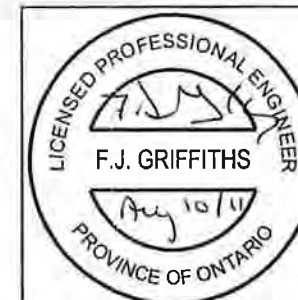


PLATE No
CONT
WP 3103-03-01

HIGHWAY 40
18th CONC DRAIN BRIDGE
BOREHOLE LOCATIONS & SOIL STRATA



KEY PLAN
1 km 0 1 2 km

LEGEND

- Bore Hole
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- W.L. at Time of Investigation May 2011
- (1.8 m E) Offset from Road CL

No	ELEVATION	MTM ZONE 11 COORDINATES NORTH	COORDINATES EAST
BH11-1	176.6	4 714 168.2	314 583.8
BH11-2	176.7	4 714 181.5	314 588.3
BH11-3	176.9	4 714 202.2	314 584.5
BH11-4	176.9	4 714 215.4	314 589.1

NOTES

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

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

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

REVISIONS	DATE	BY	DESCRIPTION

GEORES No	40J9-21
HWY No	40
SUB'D KB	CHECKED
DRAWN GBB	CHECKED
DATE	2011-07-07
SITE	13-045
DWG	1



Photo No. 1a: Concession #18 Drain Bridge at the intersection of HWY 40 and Elbow Line



Photo No. 1b: East Elevation of Concession #18 Drain Bridge





Photo No. 1c: Barrier Railing System for Concession #18 Drain Bridge



Photo No. 1d: Concession #18 Drain Bridge in Murray Street (HWY 40), Chatham-Kent

APPENDIX B

Symbols and Terms Used on Borehole Records

Borehole Records

Rock Core Photographs

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

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



ROCK DESCRIPTION

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	<i>Very Poor</i>
25-50	<i>Poor</i>
50-75	<i>Fair</i>
75-90	<i>Good</i>
90-100	<i>Excellent</i>

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

Terminology describing rock strength:

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

Terminology describing rock weathering:

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



STRATA PLOT

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



Boulders
Cobbles
Gravel



Sand



Silt



Clay



Organics



Asphalt



Concrete



Fill



Bedrock

SAMPLE TYPE

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

WATER LEVEL MEASUREMENT



measured in standpipe, piezometer, or well



inferred

RECOVERY

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

N-VALUE

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

DYNAMIC CONE PENETRATION TEST (DCPT)

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

OTHER TESTS

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

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



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RECORD OF BOREHOLE No BH 11-1

1 OF 1

METRIC

W.P. 3103-03-01 LOCATION 18th Concession Drain Bridge, Chatham-Kent N: 4 714 168 E: 314 584 ORIGINATED BY DS
 DIST HWY 40 BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler COMPILED BY JF
 DATUM Geodetic DATE 2011 04 19 - 2011 04 19 CHECKED BY SG,KB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	✕ FIELD VANE ✕ LAB VANE						
176.6	Asphalt						20 40 60 80 100	20 40 60 80 100	10 20 30	kN/m ³	GR	SA	SI	CL	
0.0	150 mm ASPHALT														
176.4															
176.3	Sandy gravel, blackish brown, FILL														
0.3	230 mm CONCRETE														
176.1	Organic material, trace sand, moist		1	BS			176								
176.0	SILTY CLAY (CL), some sand														
0.6	Dark grey		2	SS	6										
	Stiff to very stiff														
			3	SS	3		175							0 11 66 23	
			4	SS	8		174								
			5	SS	11		173								
			6	SS	3		172							0 0 66 34	
172.5	SILTY CLAY (CI)													pp = 125 kPa	
4.1	Grey, moist to wet		7	SS	3		171								
	Soft to firm														
			8	SS	3		170								
			9	SS	1		169								
							168							0 0 58 42	
							167								
167.0	End of Borehole													pp = pocket penetrometer	
9.6															

✕ 3.3 : Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 11-2

1 OF 3

METRIC

W.P. 3103-03-01 LOCATION 18th Concession Drain Bridge, Chatham-Kent N: 4 714 182 E: 314 588 ORIGINATED BY DS
 DIST HWY 40 BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler, HQ Rock Core COMPILED BY JF
 DATUM Geodetic DATE 2011 05 02 - 2011 05 05 CHECKED BY SG,KB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
176.7	Asphalt													
0.0	200 mm ASPHALT													
176.5														
0.2	FILL: brown crushed sand and gravel													
176.2														
0.5	SILTY SAND to SAND (SM) Grey to brown, black from 0.9 m to 1.1 m Very loose to loose		1	BS										
175.8														
0.9	SILTY CLAY (CL) with some sand Dark grey to black Stiff		2	SS	6									
			3	SS	3									0 48 40 12
			4	SS	5									
173.7														
3.0	SILTY CLAY (CI) Grey to olive Stiff		5	SS	8									0 2 57 41 pp = 70 kPa
			6	SS	5									
172.4														
4.3	SILTY CLAY (CI) Grey Soft to firm		7	ST										0 0 68 32 G _s = 2.74
			8	SS	2									
			9	SS	1									0 0 49 51

Continued Next Page

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

METRIC

W.P.	3103-03-01	LOCATION	18 th Consession Drain Bridge, Chatham-Kent	N: 4 714 182 E: 314 588	ORIGINATED BY	DS
DIST	HWY 40	BOREHOLE TYPE	Hollow Stem Augers, Spillspoon Sampler, HQ Rock Core		COMPILED BY	JF
DATUM	Geodetic	DATE	2011 05 02 - 2011 05 05		CHECKED BY	SG,KB

[illegible]

Continued Next Page

x³, X³: Numbers refer to Sensitivity **○^{3%}** STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 11-3

1 OF 3

METRIC

W.P. 3103-03-01 LOCATION 18th Concession Drain Bridge, Chatham-Kent N: 4 714 202 E: 314 585 ORIGINATED BY DS
DIST HWY 40 BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler, HQ Rock Core COMPILED BY JF
DATUM Geodetic DATE 2011 04 20 - 2011 04 26 CHECKED BY SG,KB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
176.9	Asphalt							20 40 60 80 100						
0.0	380 mm ASPHALT							20 40 60 80 100						
176.5														
0.4	200 mm CONCRETE													
176.3														
0.6	SANDY SILT (ML) Brown, moist Loose		1	BS										
176.0														
0.9	SILTY CLAY (CL) Dark grey to black Stiff		2	SS	8		176							
			3	SS	3		175							
174.7	SILTY CLAY (CL) Grey to olive Stiff to very stiff		4	SS	5		174							0 1 73 26 pp = 100 kPa
2.2														
			5	SS	6									pp = 200 kPa
							173							
			6	SS	4									pp = 75 kPa
172.5	SILTY CLAY (CI) Brown to grey, moist to wet Soft to firm		7	SS	4		172							0 1 62 37
4.4														
							171							
			8	SS	2									
							170							
							169							
							168							
			9	SS	1		167							

Continued Next Page

× 3, × 3

Numbers refer to
Sensitivity

○ 3%

STRAIN AT FAILURE

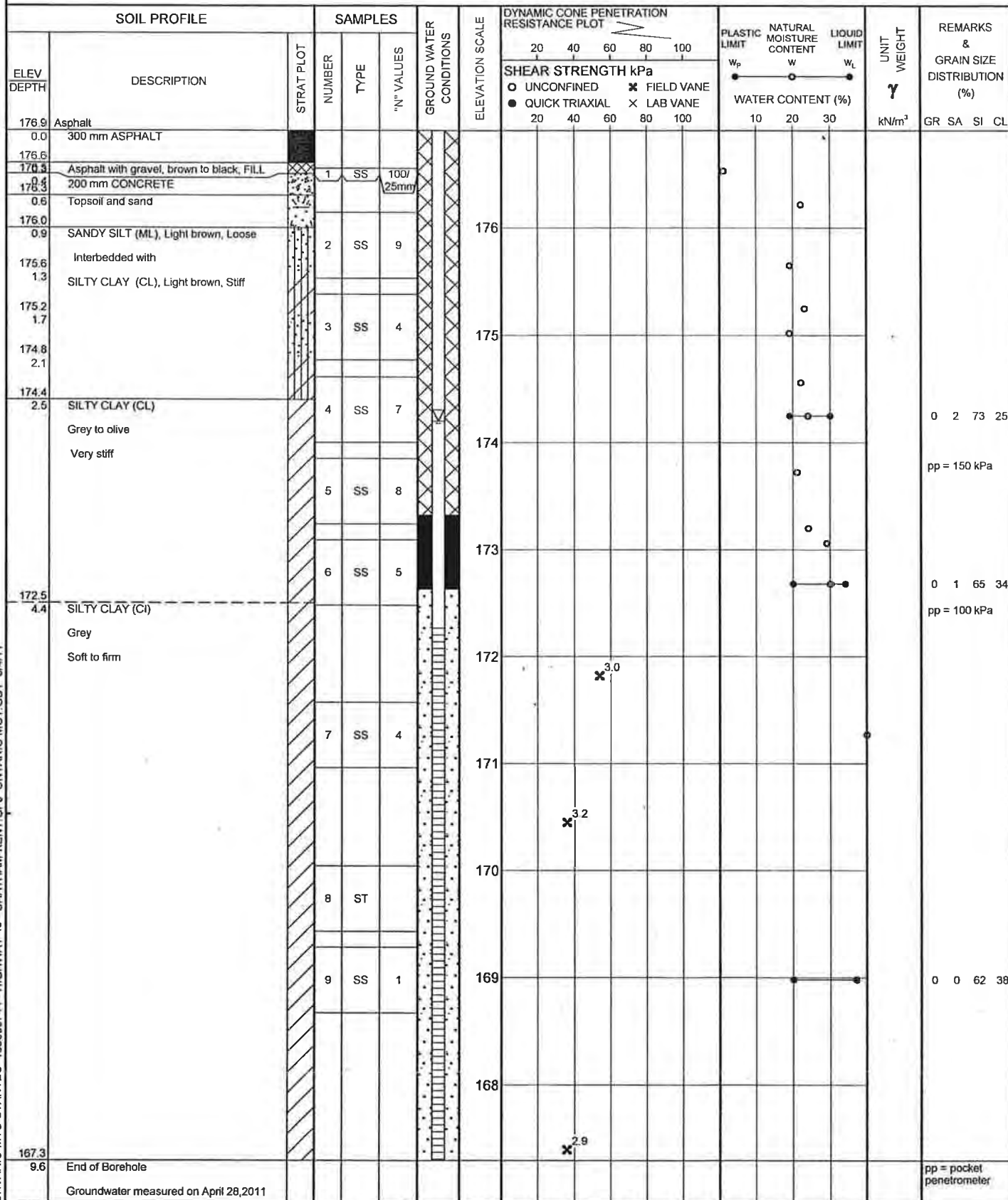
x³, X³: Numbers refer to Sensitivity **O^{3%}** STRAIN AT FAILURE

RECORD OF BOREHOLE No BH 11-4

1 OF 1

METRIC

W.P. 3103-03-01 LOCATION 18th Concession Drain Bridge, Chatham-Kent N: 4 714 215 E: 314 589 ORIGINATED BY DS
 DIST HWY 40 BOREHOLE TYPE Hollow Stem Augers, Splitspoon Sampler COMPILED BY JF
 DATUM Geodetic DATE 2011 04 18 - 2011 04 18 CHECKED BY SG,KB



ONTARIO MTO STANTEC 165000744 - HIGHWAY 40 - CHATHAM-KENT GPJ ONTARIO MOT.GDT 8/4/11

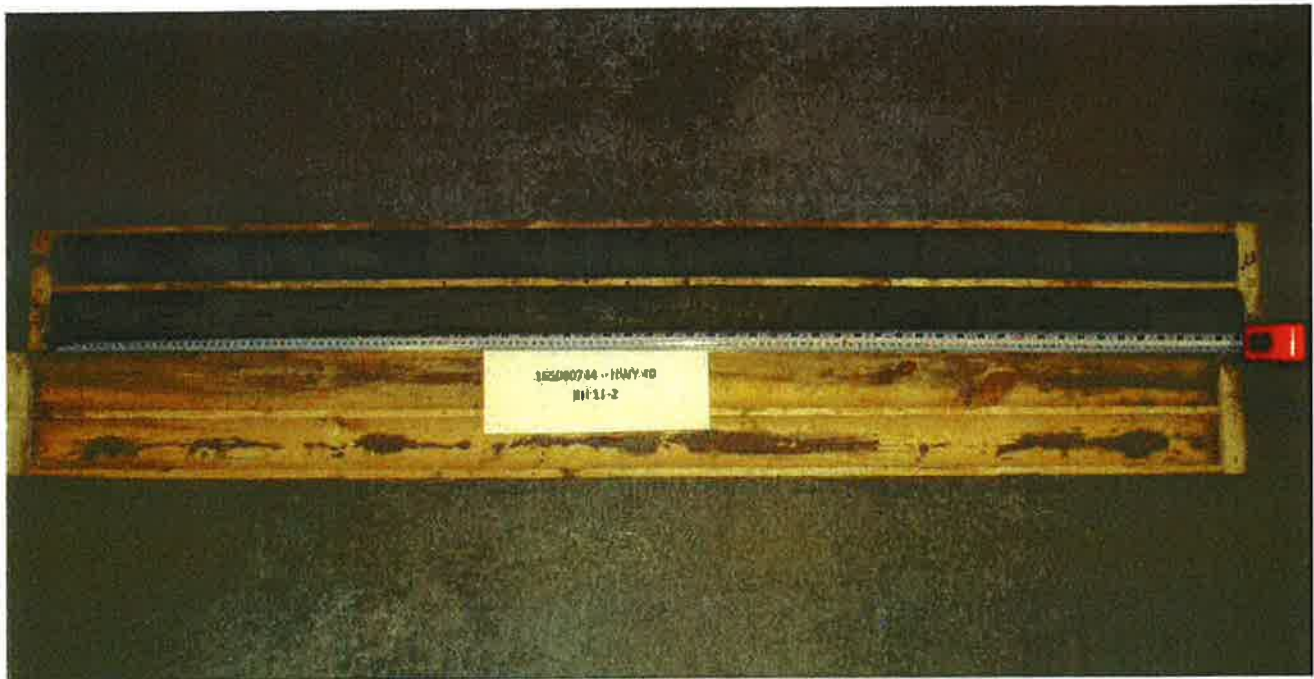


Photo No. 1: BH 11-2 – Elev. 151.7 – 148.7 m



Photo No. 2: BH 11-3 – Elev. 151.3 – 148.3 m

v:\01224\active\other_pc_projects\165000744\photos\rock photos\final\165000744_photo_pages.doc

APPENDIX C

Laboratory Test Results

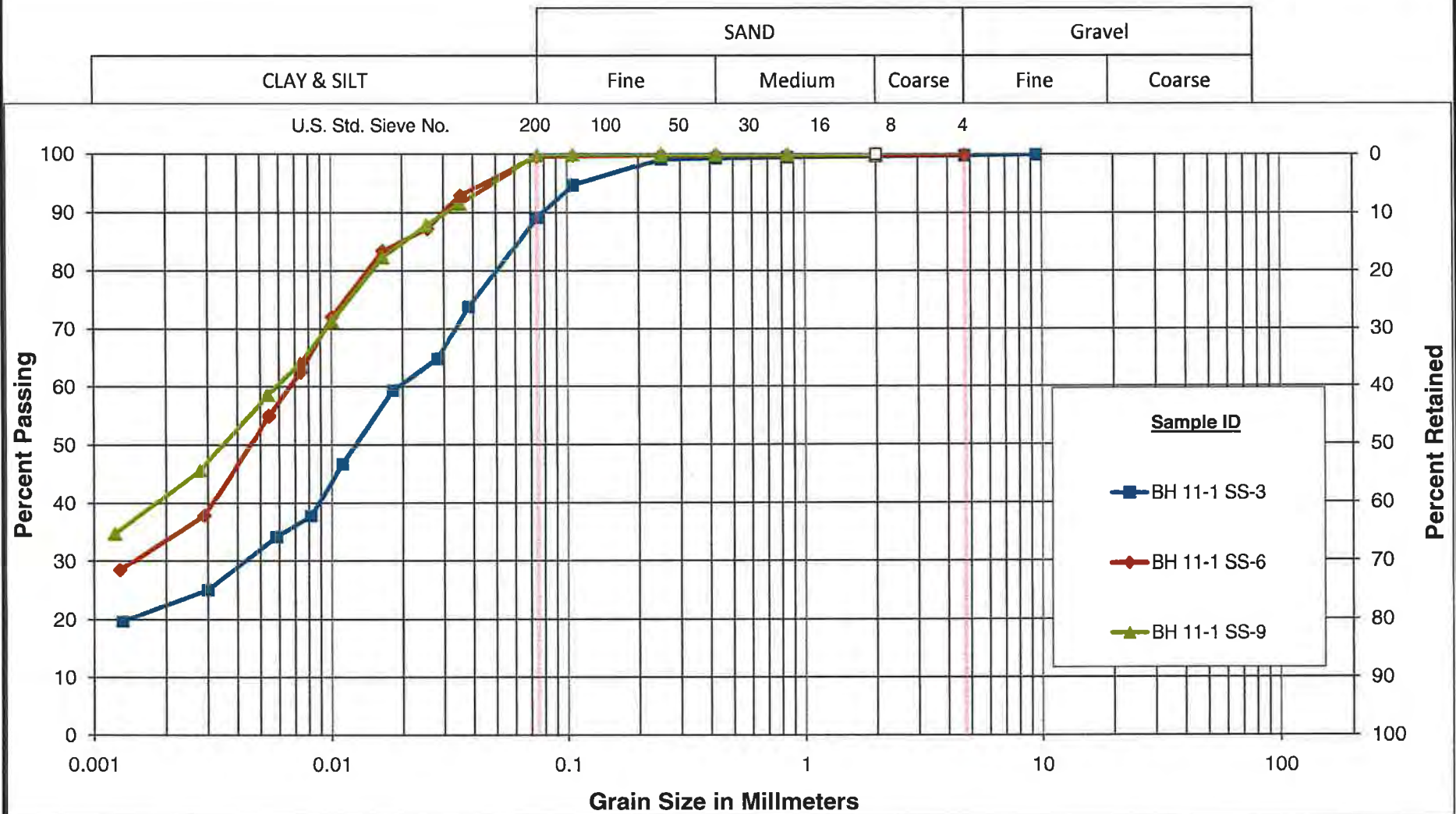
Figures 1a – 1d: Grain Size Distribution Plots

Figure 2a-2d: Plasticity Chart

Laboratory Testing by Golder

- Grain Size Distribution
- Plasticity
- Specific Gravity
- Consolidation
- Unified Compression (soil)

Unified Soil Classification System



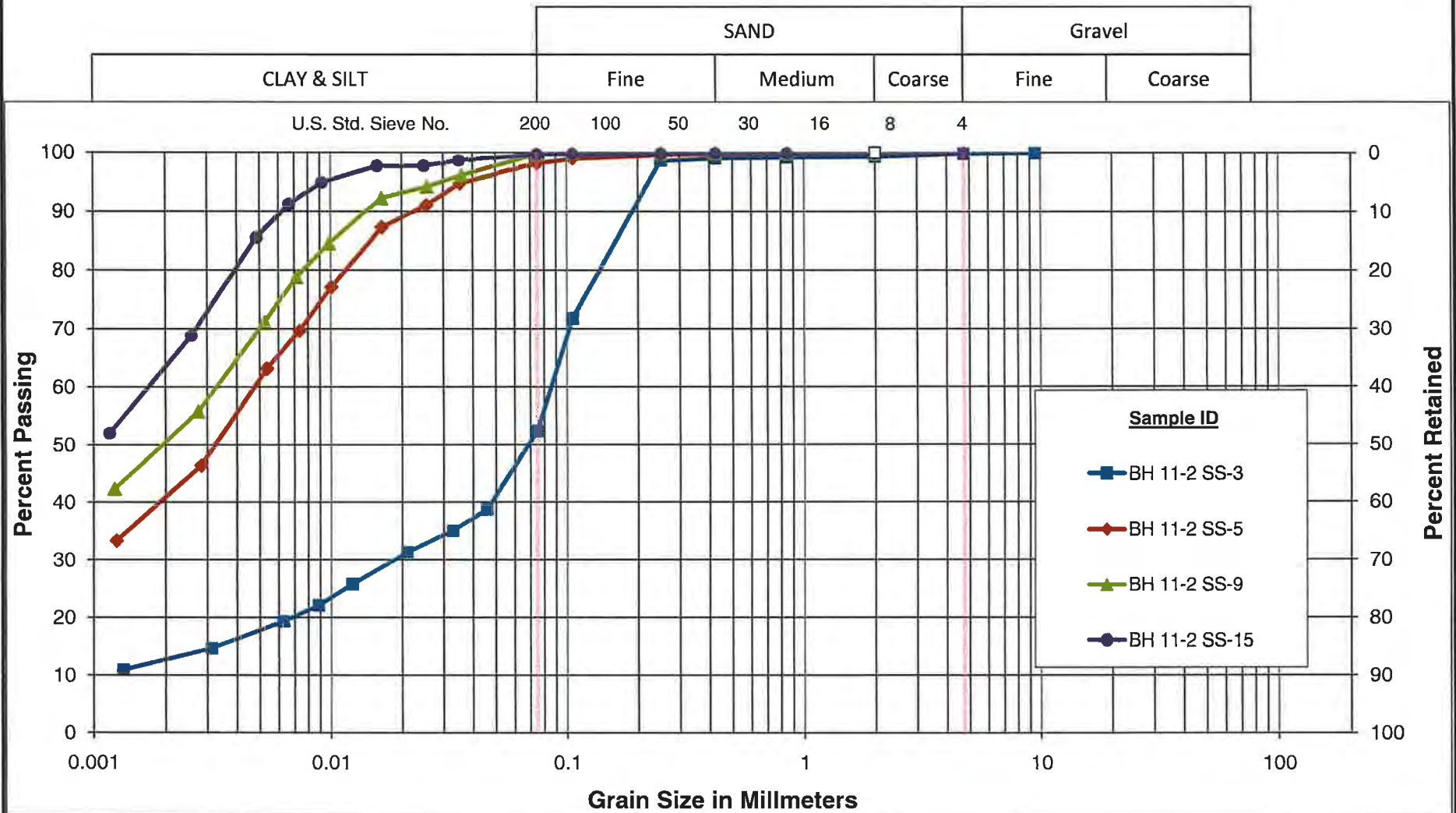
GRAIN SIZE DISTRIBUTION

Silty Clay (CI, CL)

Figure No. 1a

Project No. 165000744

Unified Soil Classification System

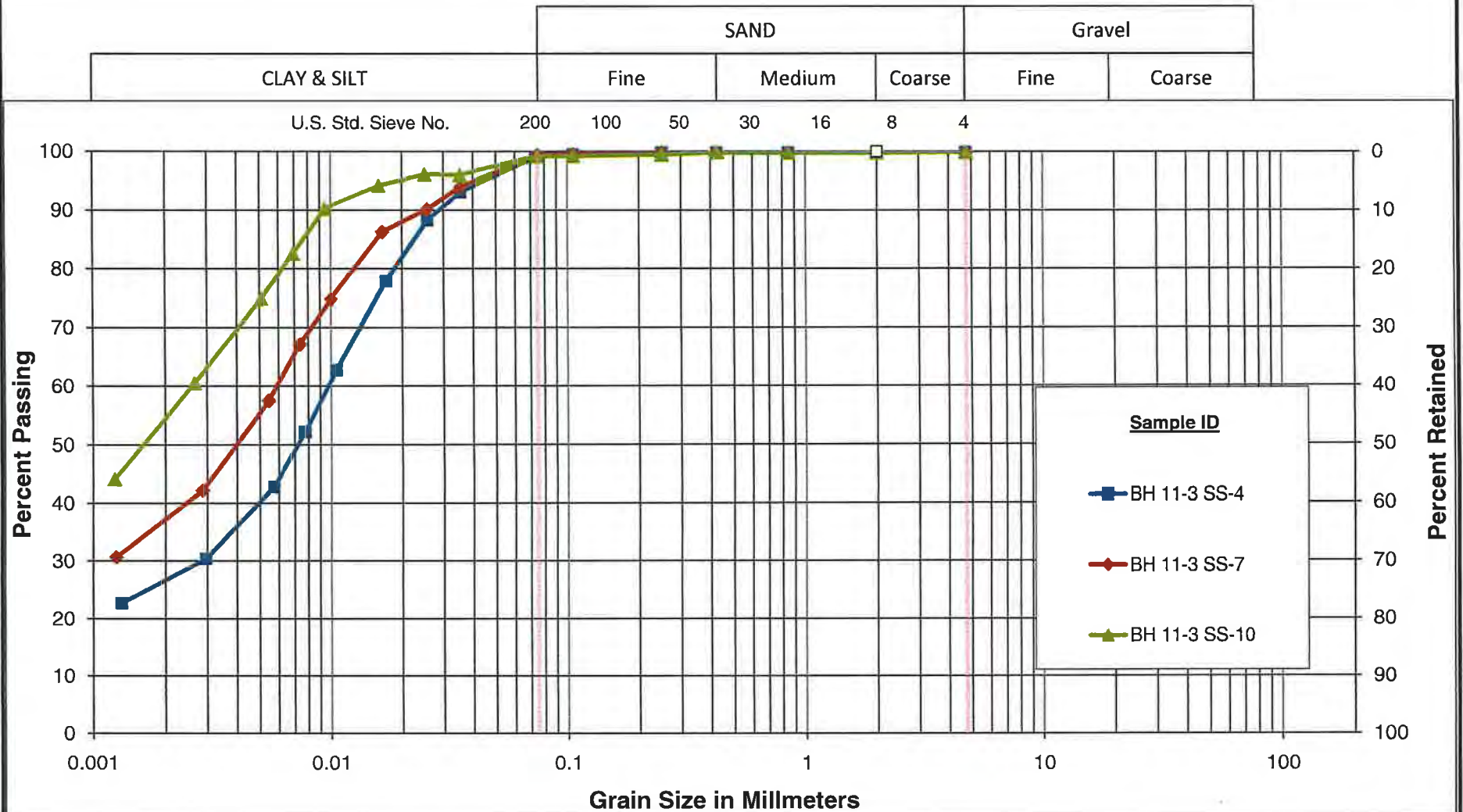


GRAIN SIZE DISTRIBUTION
Silty Clay (CI, CL)

Figure No. 1b

Project No. 165000744

Unified Soil Classification System

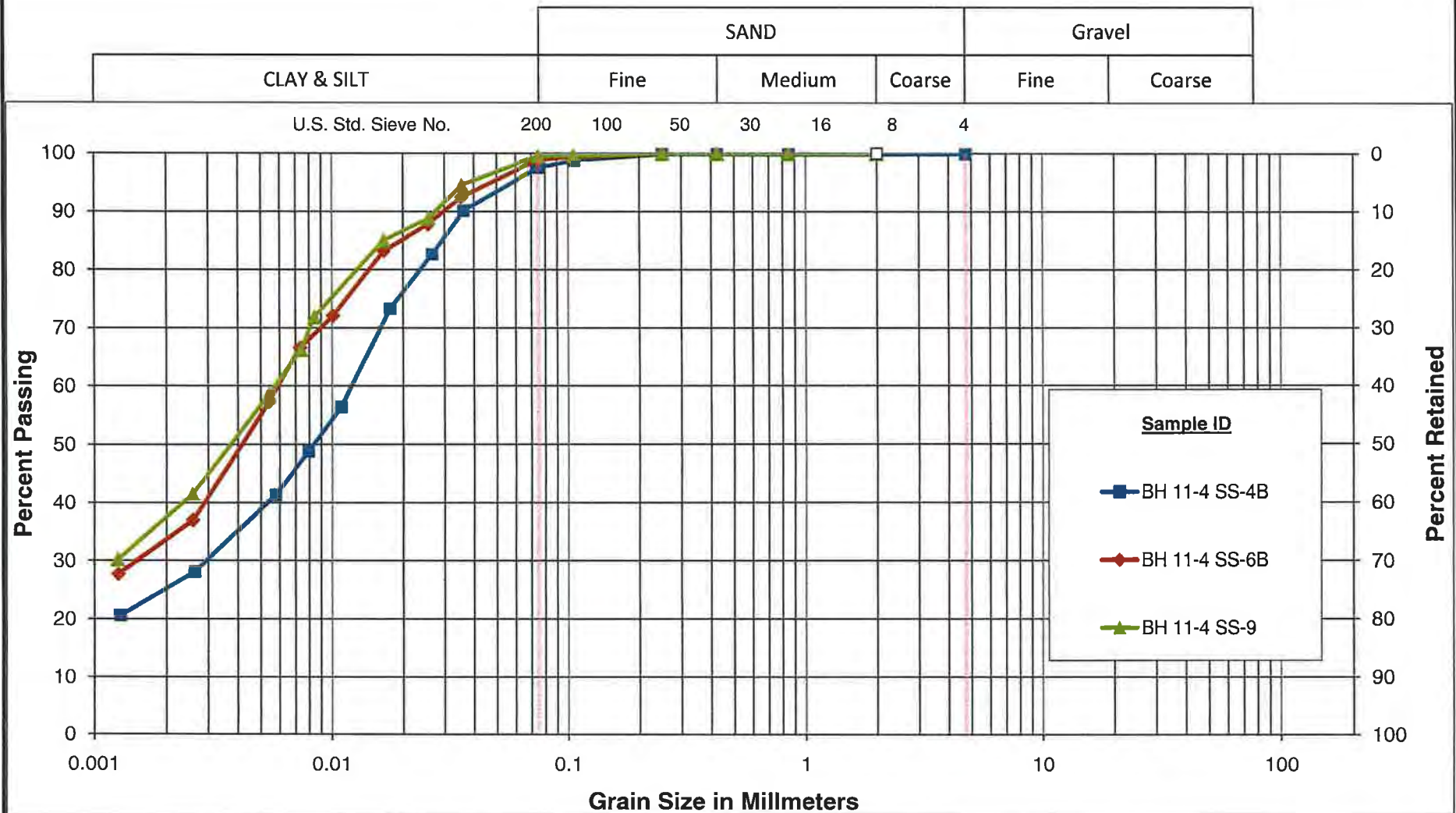


GRAIN SIZE DISTRIBUTION
Silty Clay (CI, CL)

Figure No. 1c

Project No. 165000744

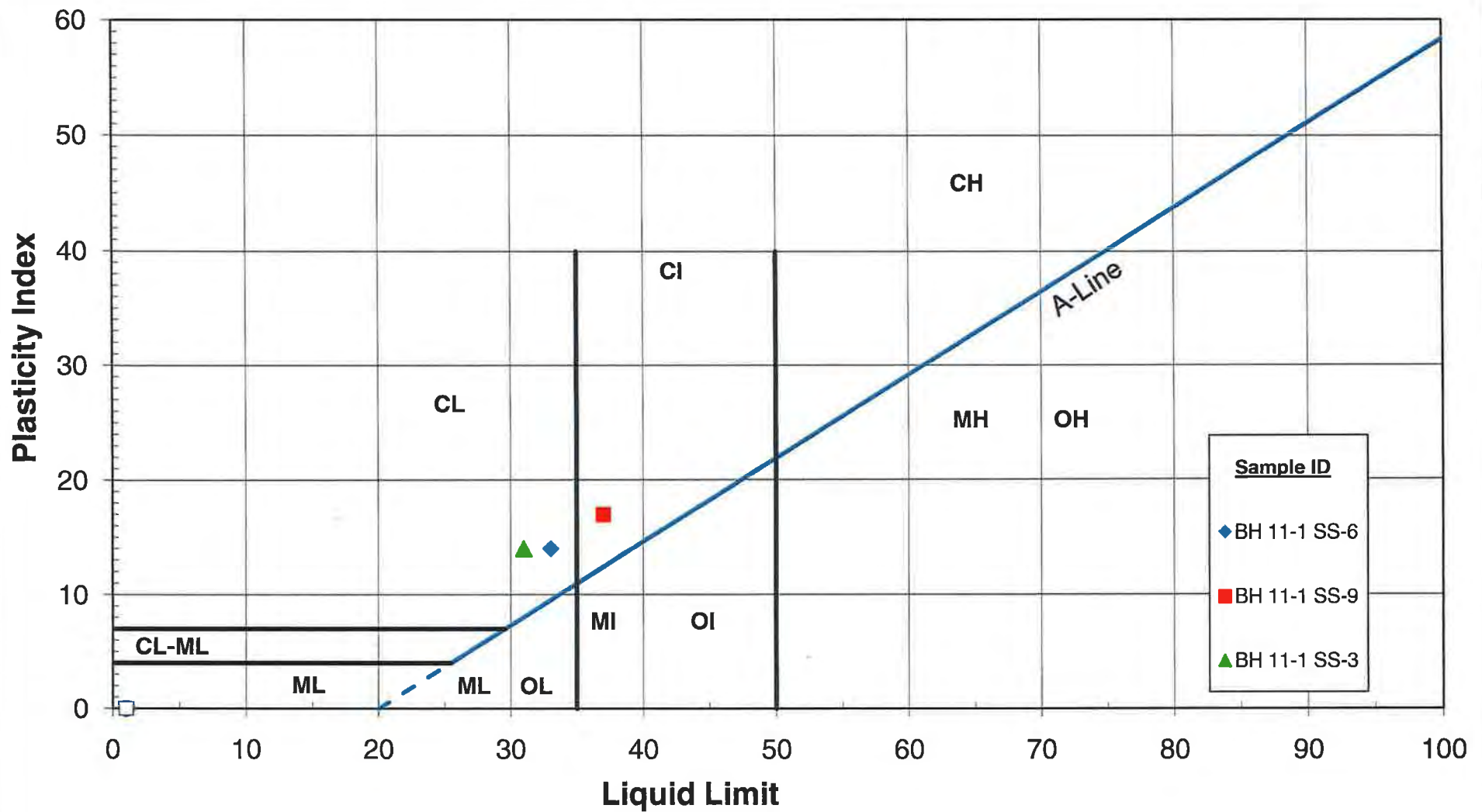
Unified Soil Classification System



GRAIN SIZE DISTRIBUTION
Silty Clay (CI, CL)

Figure No. 1d

Project No. 165000744

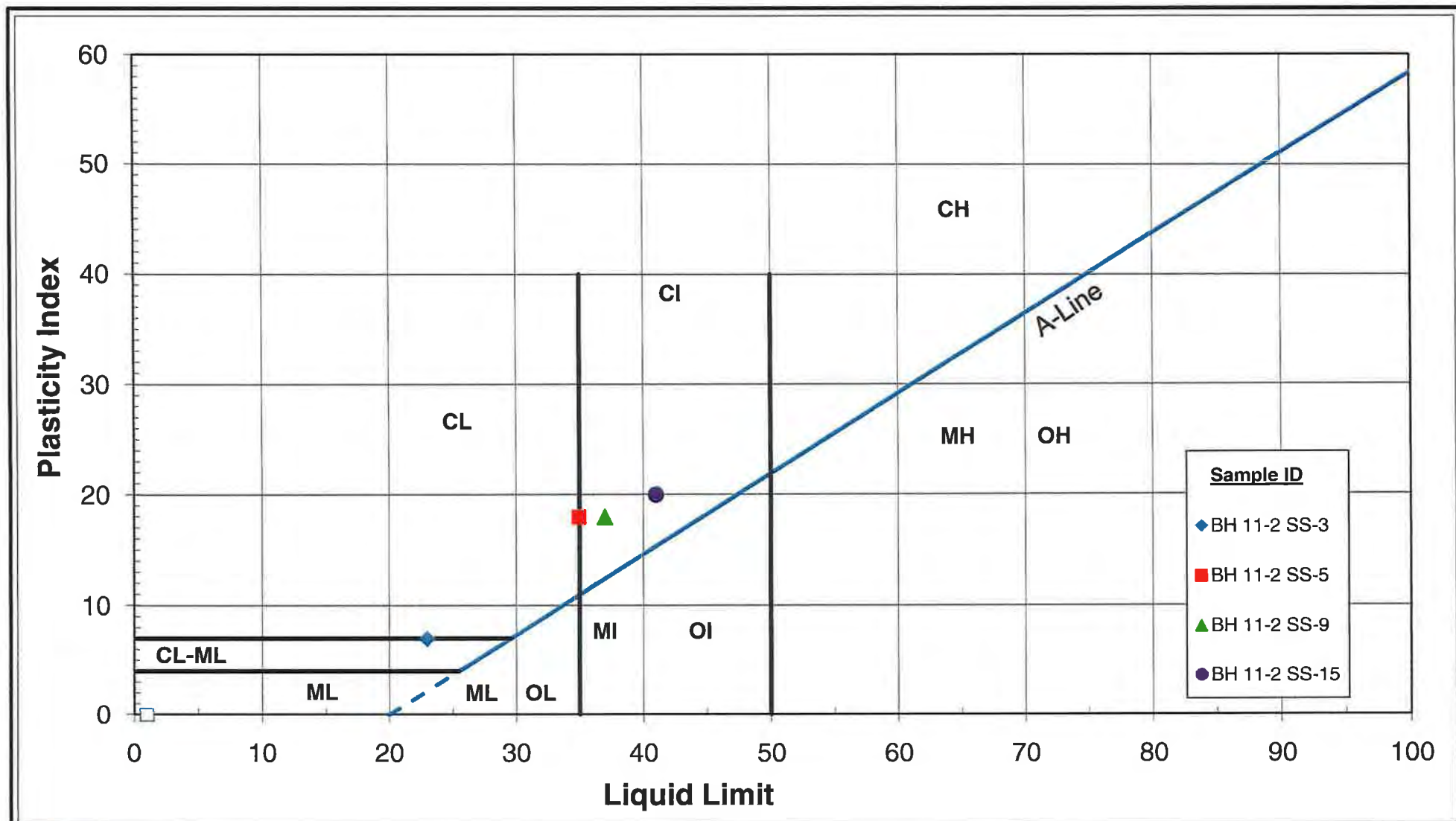


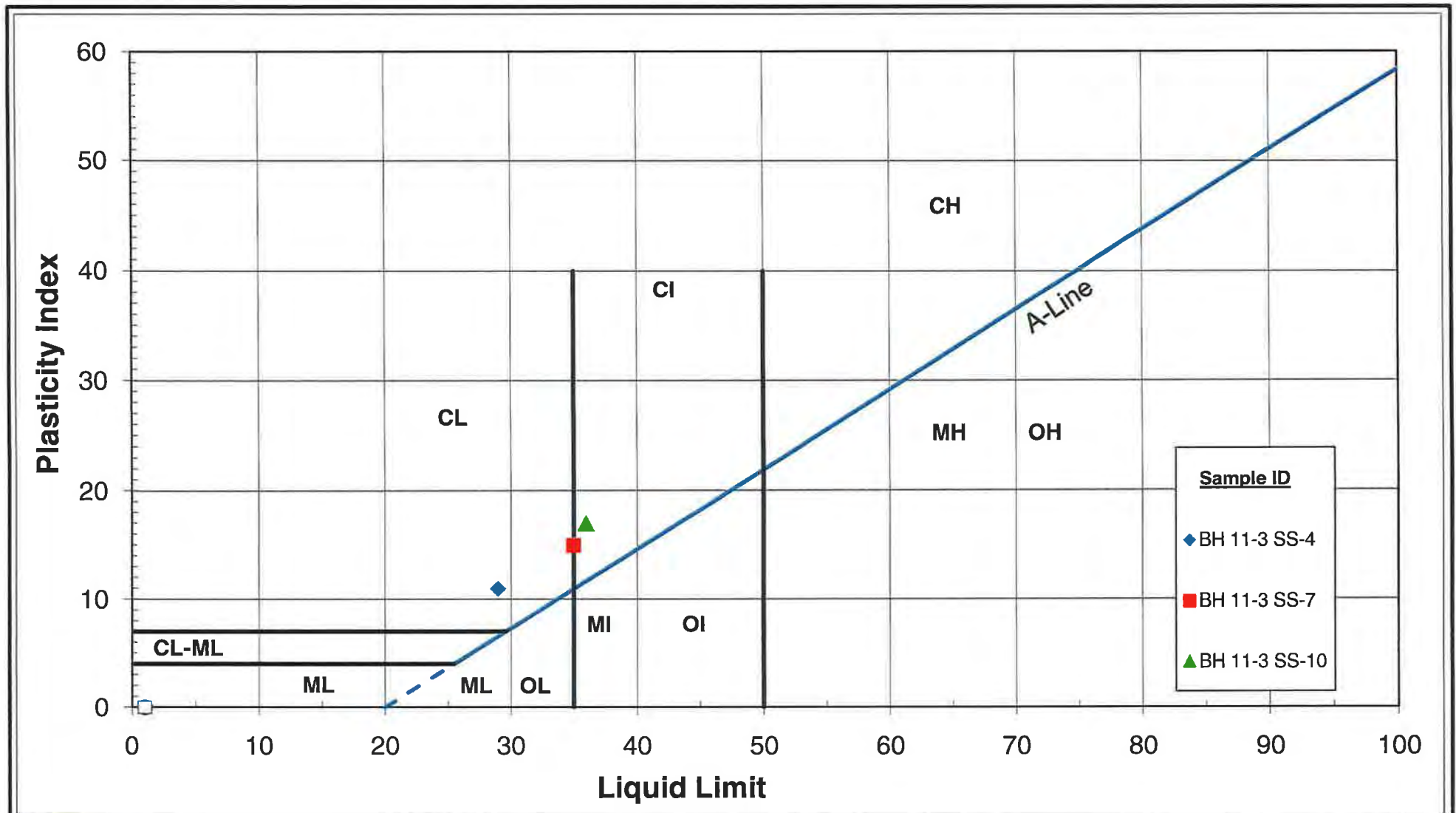
Stantec

PLASTICITY CHART

Figure No. 2a

Project No. 165000744



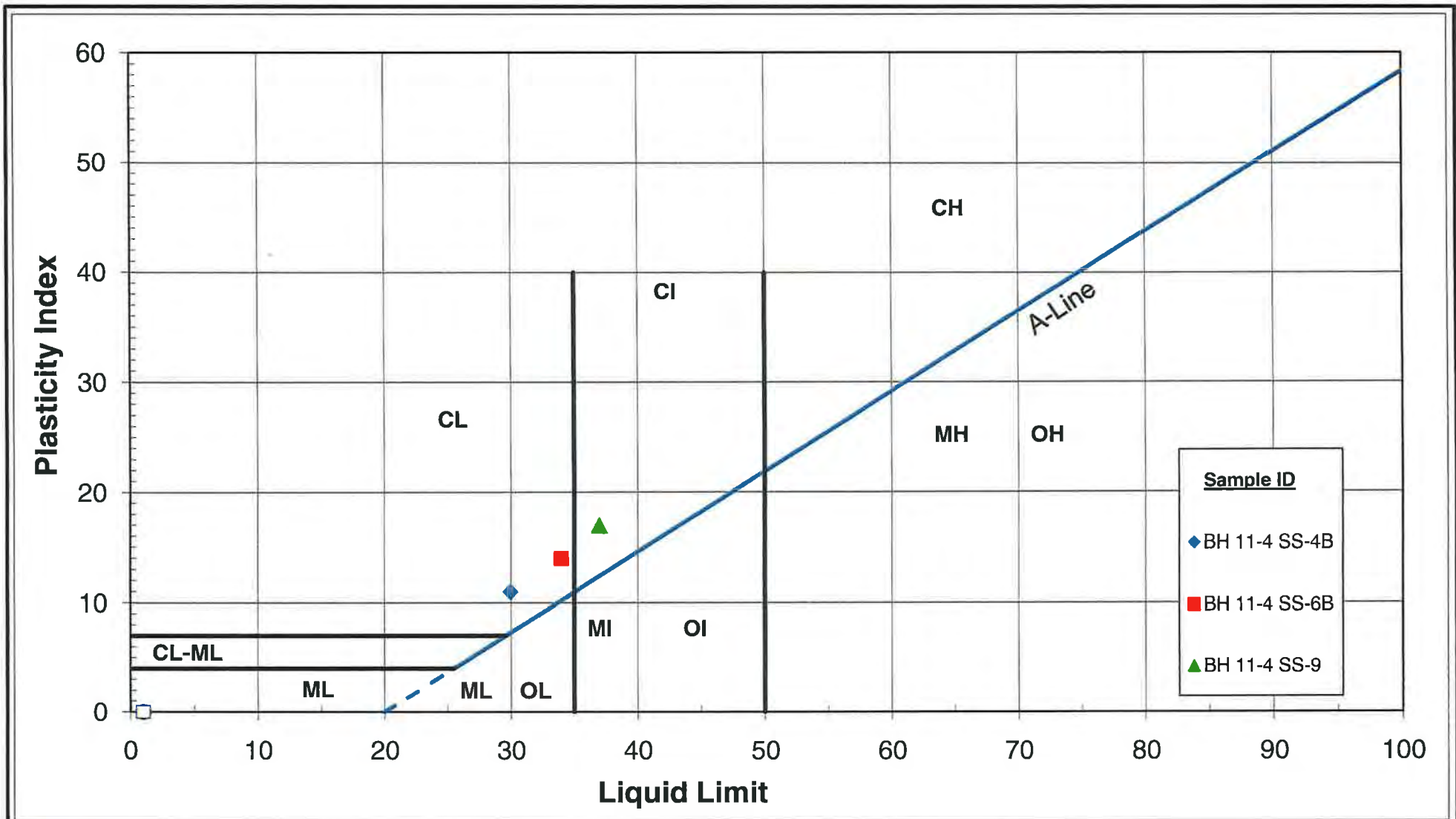


Stantec

PLASTICITY CHART

Figure No. 2c

Project No. 165000744



PLASTICITY CHART

Figure No. 2d

Project No. 165000744

May 31, 2011

Project No. 11-1183-0023

165000744

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

GEOTECHNICAL LABORATORY TESTING

Dear Sir

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

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

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

GOLDER ASSOCIATES LTD.



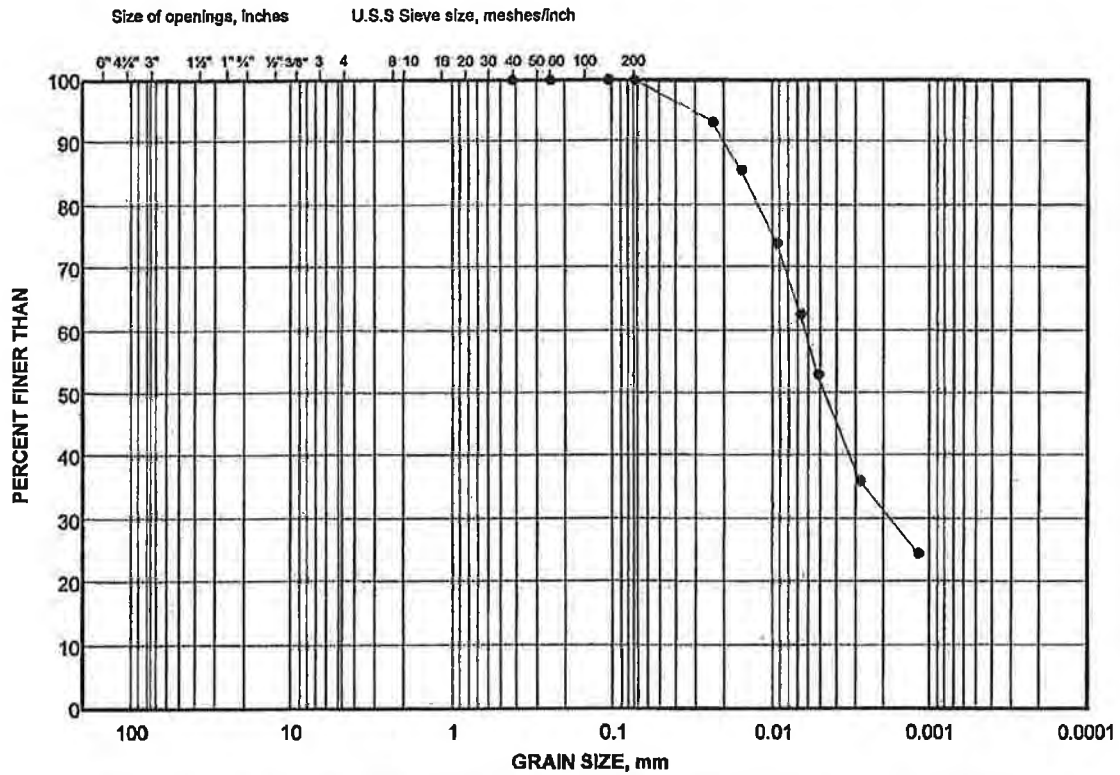
Marijana Manojlovic
Laboratory Manager

MM/lg



GRAIN SIZE DISTRIBUTION

FIGURE



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	11-2	ST-7	4.6

Project Number: 11-1183-0023

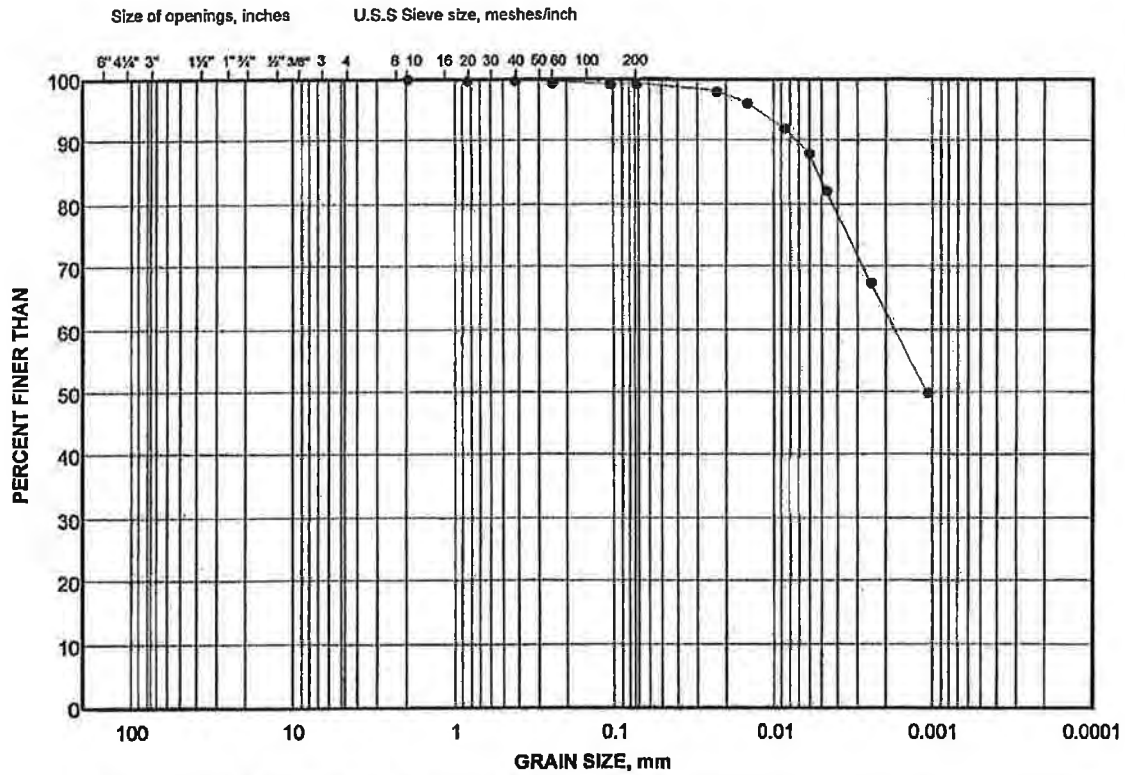
Checked By: _____

Golder Associates

Date: 28-May-11

GRAIN SIZE DISTRIBUTION

FIGURE



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

LEGEND

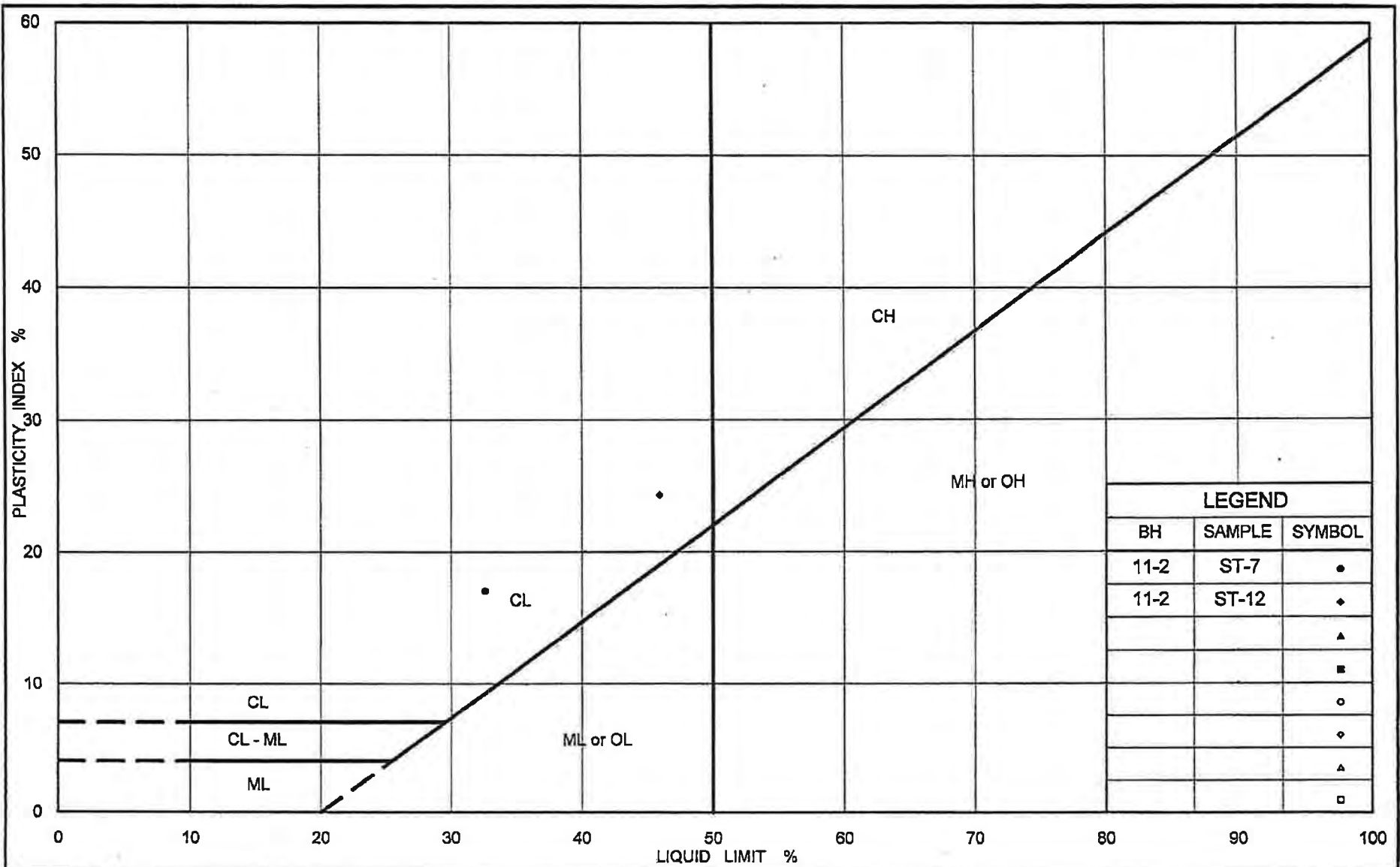
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	11-2	ST-12	12.2

Project Number: 11-1183-0023

Checked By:

Golder Associates

Date: 26-May-11



PLASTICITY CHART

Figure No.

Project No. 11-1183-0023

Checked By:

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SPECIFIC GRAVITY TEST RESULTS

ASTM D 854-06 TEST METHOD A

PROJECT NUMBER	11-1183-0023		
PROJECT NAME	Stantec / Lab Testing / 165000744		
DATE TESTED	May, 2011		
Borehole No.	Sample No.	Specific Gravity	
11-2	ST-7	2.74	
11-2	ST-12	2.75	

Note: Test carried out on soil particles <4.75mm using distilled water.

Checked By: 

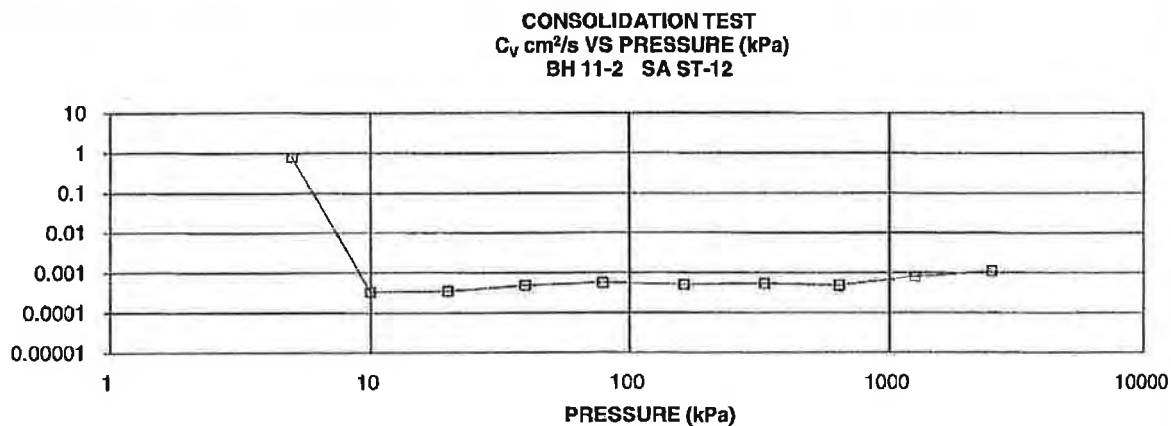
Golder Associates

CONSOLIDATION TEST SUMMARY					FIGURE		
SAMPLE IDENTIFICATION							
Project Number	11-1183-0023	Sample Number	ST-12				
Borehole Number	11-2	Sample Depth, m	12.2				
TEST CONDITIONS							
Test Type	Standard	Load Duration, hr	24				
Oedometer Number	9						
Date Started	5/09/2011						
Date Completed	5/23/2011						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.90	Unit Weight, kN/m ³	17.20				
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m ³	11.58				
Area, cm ²	31.47	Specific Gravity, measured	2.75				
Volume, cm ³	59.79	Solids Height, cm	0.816				
Water Content, %	48.51	Volume of Solids, cm ³	25.68				
Wet Mass, g	104.88	Volume of Voids, cm ³	34.11				
Dry Mass, g	70.62	Degree of Saturation, %	100.4				
TEST COMPUTATIONS							
Pressure	Corr. Height	Void Ratio	Average Height	t ₉₀ sec	c _v cm ² /s	m _v m ² /kN	k cm/s
kPa	cm		cm				
0.00	1.900	1.328	1.900				
5.00	1.890	1.316	1.895	1	7.61E-01	1.05E-03	7.85E-05
9.96	1.878	1.301	1.884	2306	3.26E-04	1.27E-03	4.07E-08
19.91	1.853	1.271	1.866	2160	3.42E-04	1.32E-03	4.43E-08
40.00	1.814	1.223	1.834	1500	4.75E-04	1.02E-03	4.76E-08
79.47	1.759	1.156	1.787	1215	5.57E-04	7.33E-04	4.00E-08
160.63	1.681	1.060	1.720	1245	5.04E-04	5.06E-04	2.50E-08
328.63	1.582	0.939	1.631	1070	5.27E-04	3.10E-04	1.60E-08
641.54	1.483	0.817	1.532	1033	4.82E-04	1.66E-04	7.86E-09
1262.67	1.389	0.702	1.436	560	7.81E-04	7.97E-05	6.09E-09
2507.36	1.296	0.588	1.342	346	1.10E-03	3.94E-05	4.26E-09
1262.67	1.316	0.612	1.308				
328.63	1.373	0.683	1.344				
79.47	1.443	0.769	1.408				
19.91	1.513	0.854	1.478				
5.00	1.581	0.937	1.547				
Note: k calculated using cv based on t ₉₀ values. Specimen taken 7cm from the bottom of sample							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.58	Unit Weight, kN/m ³	18.94				
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m ³	13.92				
Area, cm ²	31.47	Specific Gravity, measured	2.75				
Volume, cm ³	49.75	Solids Height, cm	0.816				
Water Content, %	36.09	Volume of Solids, cm ³	25.68				
Wet Mass, g	96.11	Volume of Voids, cm ³	24.07				
Dry Mass, g	70.62						
<div style="display: flex; justify-content: space-between;"> Prepared By: LFG Golder Associates Checked By: </div>							

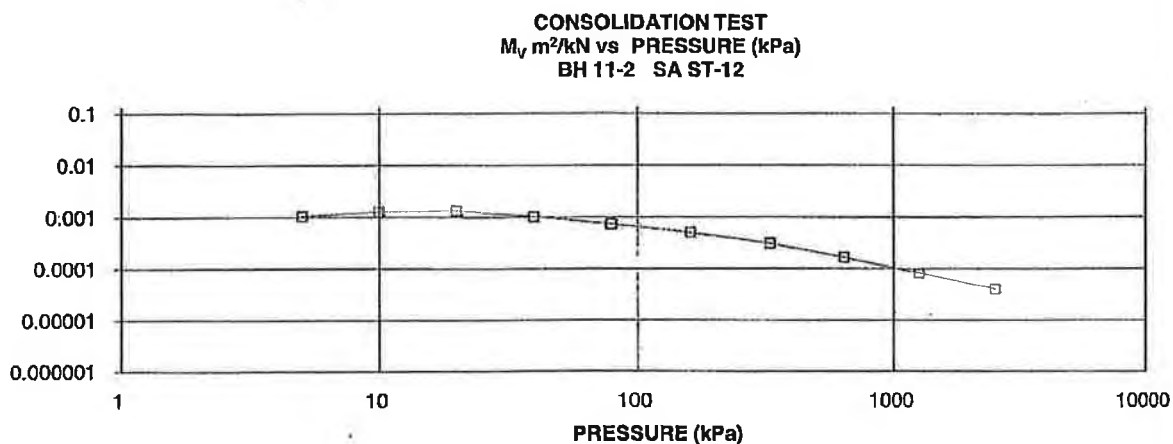
CONSOLIDATION TEST SUMMARY

FIGURE

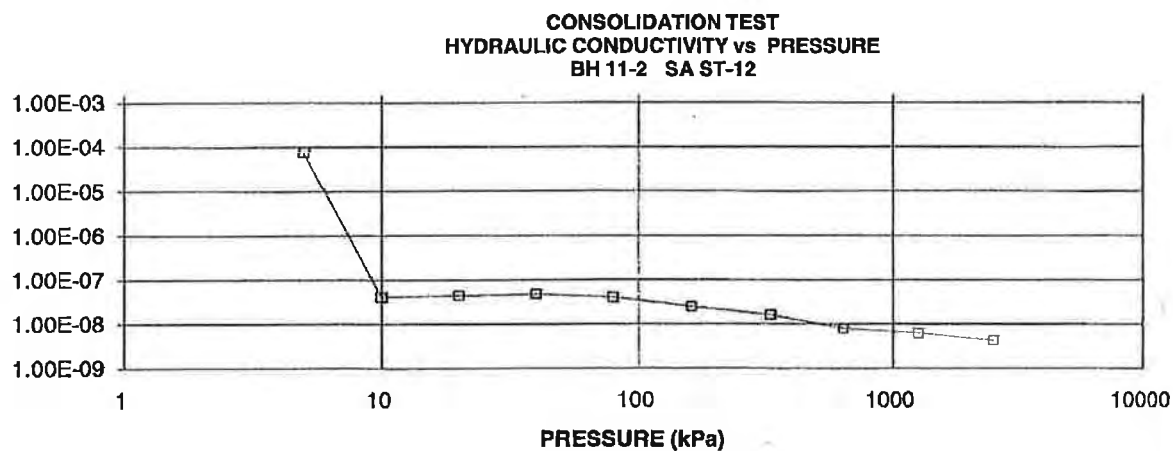
COEFFICIENT OF CONSOLIDATION,
cm²/s



VOLUME COMPRESSIBILITY, m²/kN



HYDRAULIC CONDUCTIVITY,
cm/s



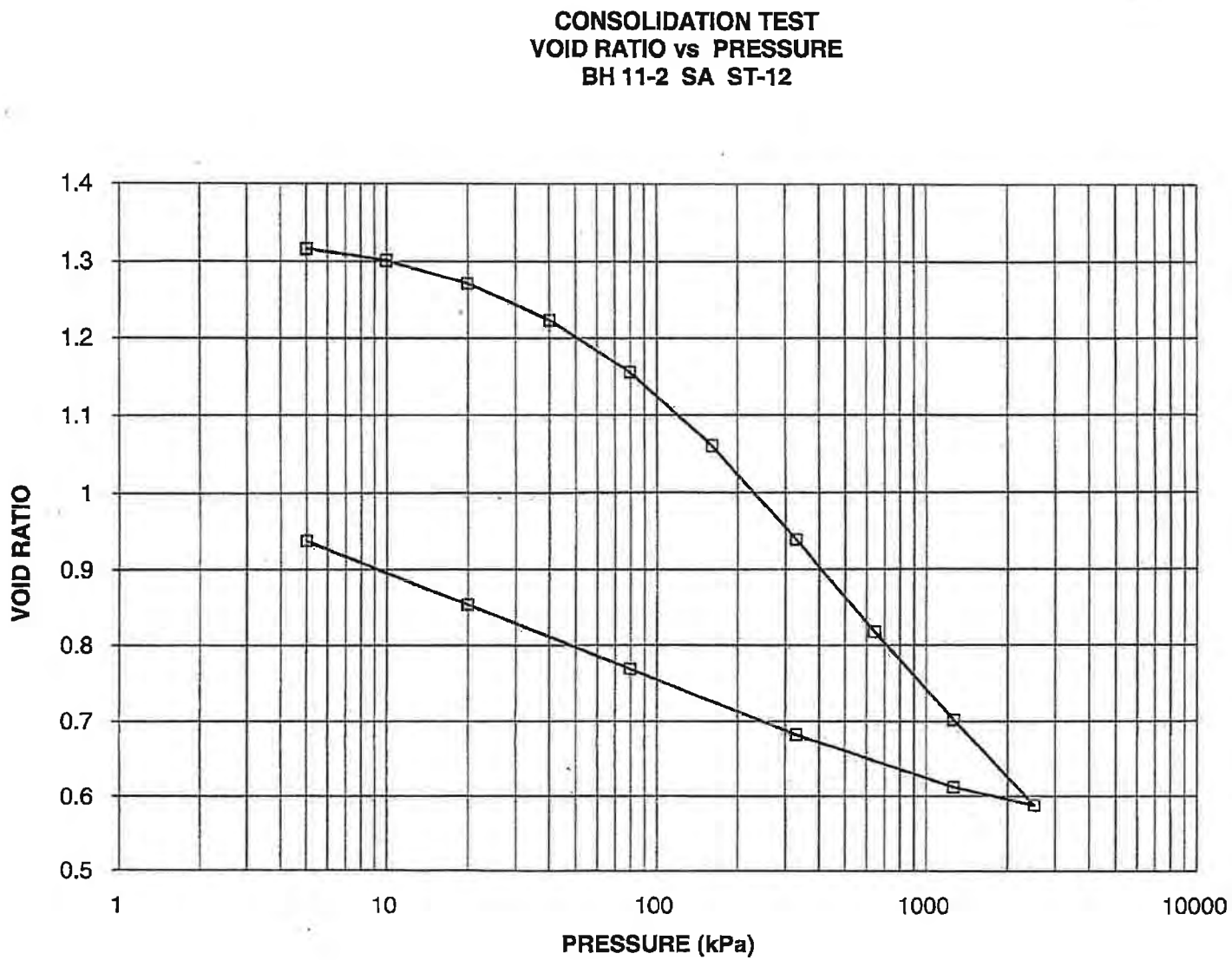
Project No. 11-1183-0023
Prepared By: LFG

Golder Associates

Checked By: *[Signature]*

CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE



Project No. 11-1183-0023
Prepared By: LFG

Golder Associates

Checked By: *W*

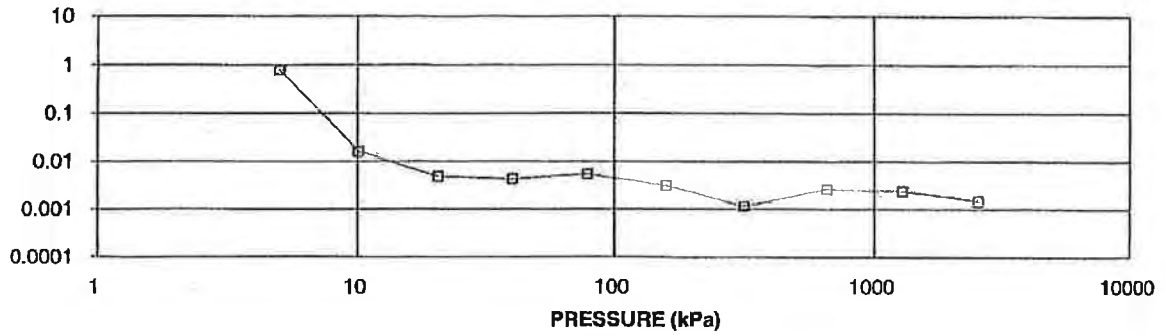
CONSOLIDATION TEST SUMMARY					FIGURE		
SAMPLE IDENTIFICATION							
Project Number	11-1183-0023			Sample Number	ST-7		
Borehole Number	11-2			Sample Depth, m	4.6		
TEST CONDITIONS							
Test Type	Standard			Load Duration, hr	24		
Oedometer Number	7						
Date Started	5/09/2011						
Date Completed	5/23/2011						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.89			Unit Weight, kN/m ³	18.78		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m ³	14.12		
Area, cm ²	31.48			Specific Gravity, measured	2.74		
Volume, cm ³	59.62			Solids Height, cm	0.995		
Water Content, %	33.07			Volume of Solids, cm ³	31.32		
Wet Mass, g	114.21			Volume of Voids, cm ³	28.30		
Dry Mass, g	85.83			Degree of Saturation, %	100.3		
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	1.894	0.903	1.894				
5.03	1.893	0.902	1.894	1	7.60E-01	9.45E-05	7.04E-06
10.06	1.892	0.902	1.893	47	1.62E-02	9.45E-05	1.50E-07
20.49	1.879	0.888	1.886	154	4.89E-03	6.68E-04	3.21E-07
40.01	1.868	0.878	1.874	173	4.30E-03	2.89E-04	1.22E-07
78.95	1.853	0.862	1.860	135	5.44E-03	2.14E-04	1.14E-07
156.68	1.822	0.831	1.837	228	3.14E-03	2.09E-04	6.43E-08
312.29	1.752	0.760	1.787	581	1.16E-03	2.38E-04	2.71E-08
655.96	1.657	0.665	1.704	235	2.62E-03	1.46E-04	3.75E-08
1278.12	1.582	0.590	1.619	240	2.32E-03	6.35E-05	1.44E-08
2522.42	1.497	0.505	1.540	346	1.45E-03	3.58E-05	5.10E-09
1278.12	1.508	0.515	1.503				
312.29	1.534	0.542	1.521				
78.95	1.569	0.577	1.552				
20.49	1.607	0.615	1.588				
5.03	1.642	0.651	1.625				
Note: k calculated using cv based on t ₉₀ values. Specimen swelled under 10kPa							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.64			Unit Weight, kN/m ³	20.29		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m ³	16.28		
Area, cm ²	31.48			Specific Gravity, measured	2.74		
Volume, cm ³	51.70			Solids Height, cm	0.995		
Water Content, %	24.64			Volume of Solids, cm ³	31.32		
Wet Mass, g	106.98			Volume of Voids, cm ³	20.38		
Dry Mass, g	85.83						
<div style="display: flex; justify-content: space-between; align-items: flex-end;"> <div>Prepared By: LFG</div> <div style="text-align: center;">Golder Associates</div> <div>Checked By: </div> </div>							

CONSOLIDATION TEST SUMMARY

FIGURE

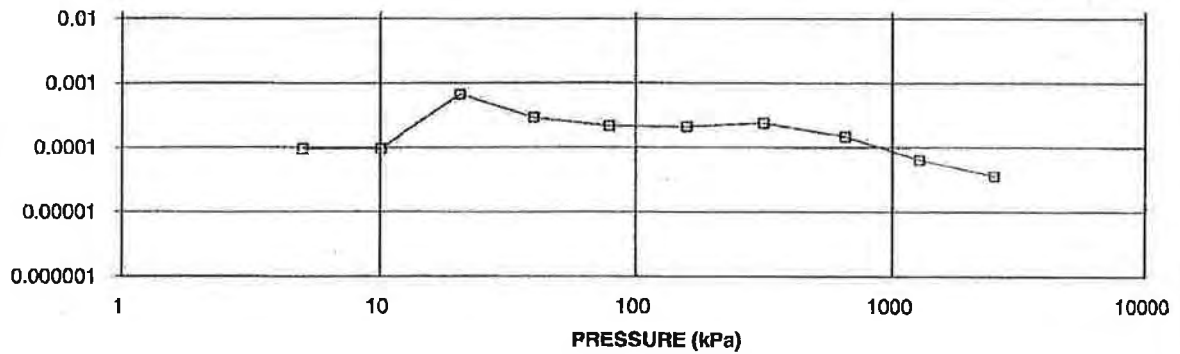
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
C_v cm²/s VS PRESSURE (kPa)
BH 11-2 SA ST-7



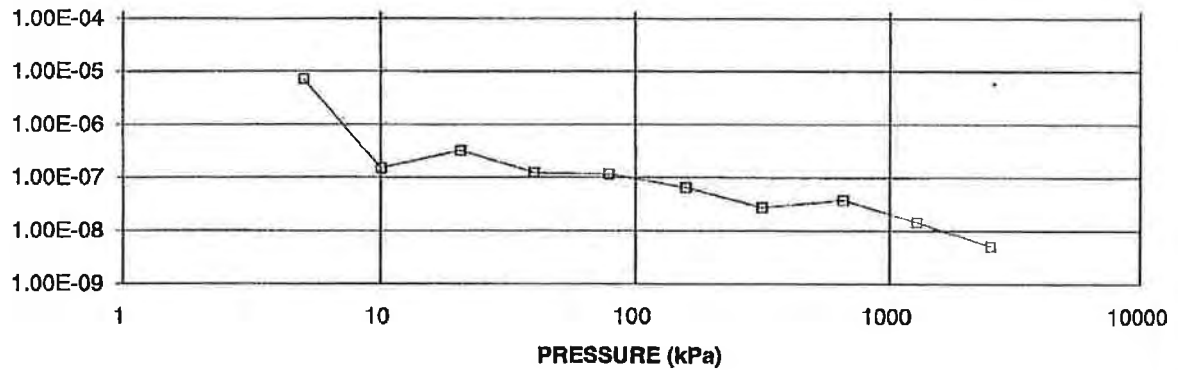
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
M_v m²/kN vs PRESSURE (kPa)
BH 11-2 SA ST-7



HYDRAULIC CONDUCTIVITY,
cm/s

CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 11-2 SA ST-7



Project No. 11-1183-0023

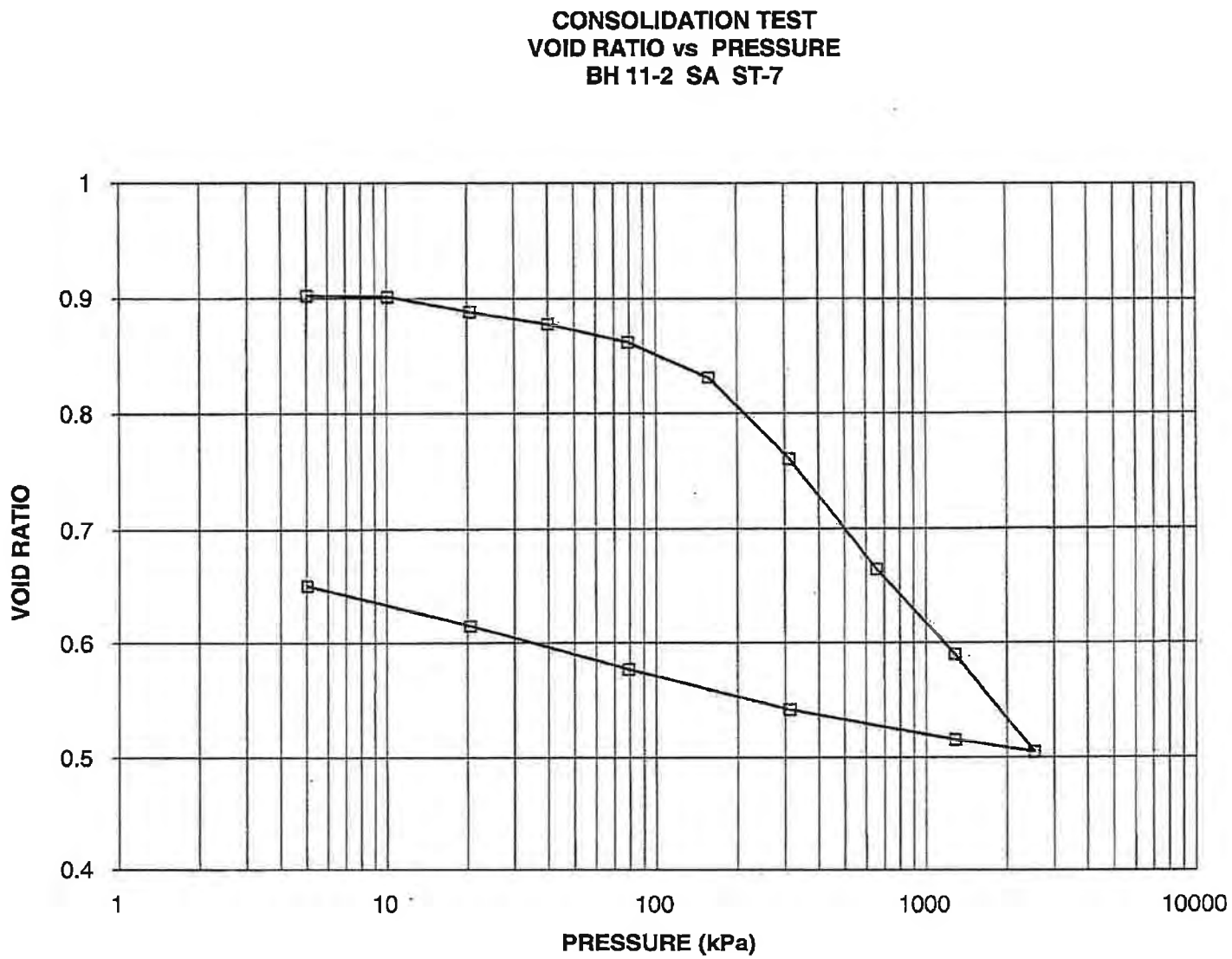
Prepared By: LFG

Golder Associates

Checked By: *[Signature]*

**CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE**

FIGURE



Project No. 11-1183-0023
Prepared By: LFG

Golder Associates

Checked By:

WJ

UNCONFINED COMPRESSION TEST (UC)

ASTM D 2166 - 06

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1183-0023	SAMPLE NUMBER	ST-7
BOREHOLE NUMBER	11-2	SAMPLE DEPTH, m	4.6

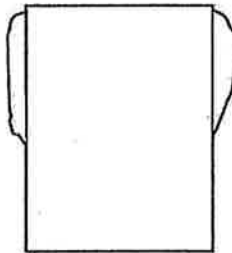
TEST CONDITIONS

MACHINE SPEED, mm/min	2.80	TYPE OF SPECIMEN	Thin wall tube sample
RATE OF AXIAL STRAIN, %/min	2.00	L/D	2.04

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	14.01	WATER CONTENT, (specimen) %	31.10
SAMPLE DIAMETER, cm	6.87	UNIT WEIGHT, kN/m ³	19.19
SAMPLE AREA, cm ²	37.07	DRY UNIT WT., kN/m ³	14.64
SAMPLE VOLUME, cm ³	519.33	SPECIFIC GRAVITY, measured	2.74
WET WEIGHT, g	1016.70	VOID RATIO	0.83
DRY WEIGHT, g	775.50		

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	6.6	COMPRESSIVE STRESS, kPa	74
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REMARKS: Specimen taken 30cm from top of the sample. DATE: 5/9/2011

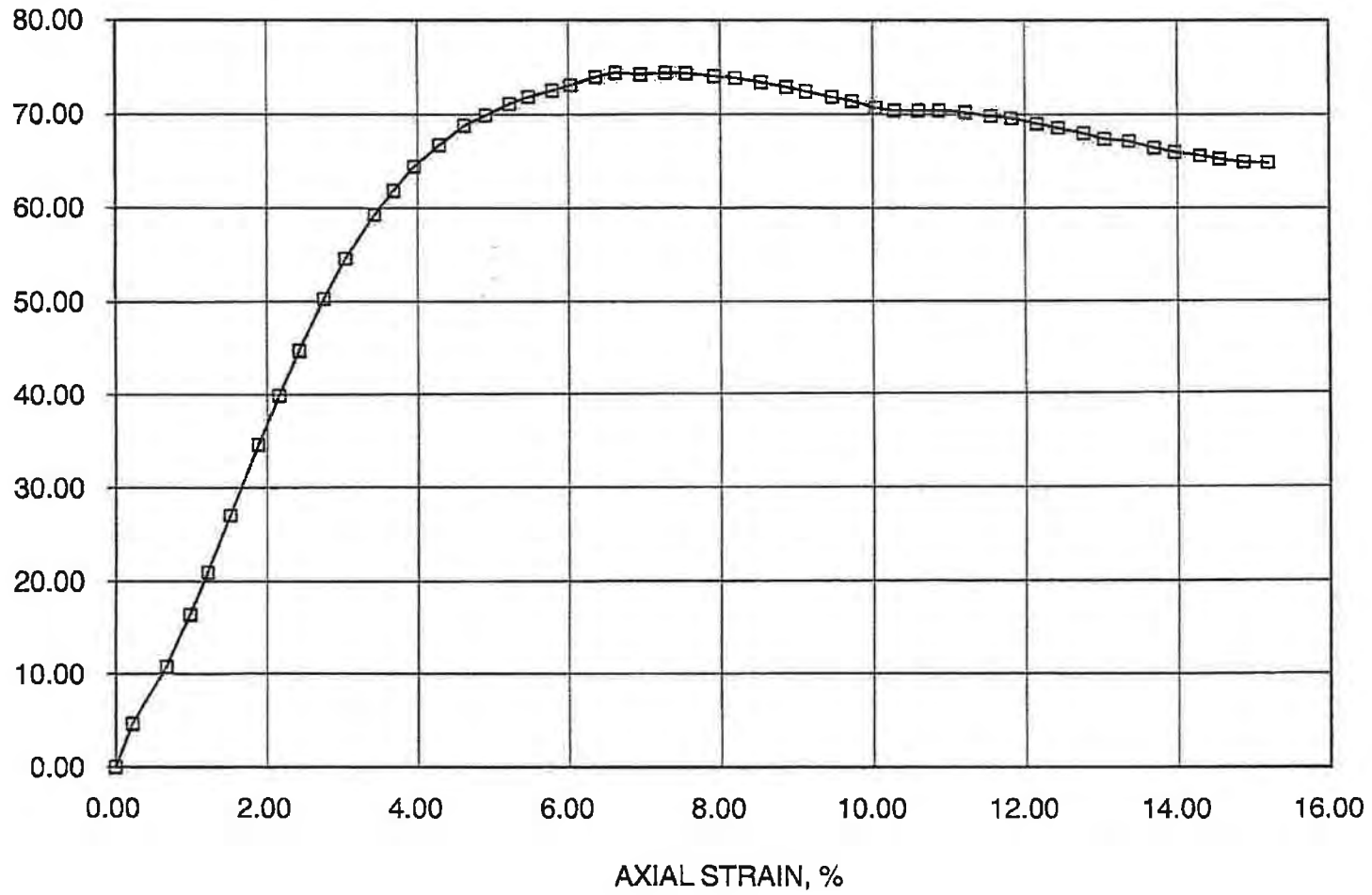
Checked By: *ME*

Golder Associates

UNCONFINED COMPRESSION TEST (UC)

FIGURE

Borehole 11-2 Sample ST-7 Depth 4.6m



Project No. 11-1183-0023

DEVIATOR STRESS, kPa

Checked By: *Mu*

AXIAL STRAIN, %

UNCONFINED COMPRESSION TEST (UC)

ASTM D 2166 - 06

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1183-0023	SAMPLE NUMBER	ST-12
BOREHOLE NUMBER	11-2	SAMPLE DEPTH, m	12.2

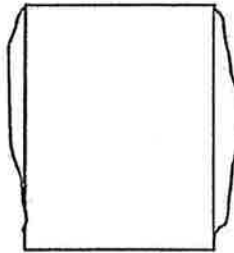
TEST CONDITIONS

MACHINE SPEED, mm/min	2.78	TYPE OF SPECIMEN	Thin wall tube sample
RATE OF AXIAL STRAIN, %/min	2.00	L/D	2.01

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.90	WATER CONTENT, (specimen) %	44.70
SAMPLE DIAMETER, cm	6.90	UNIT WEIGHT, kN/m ³	17.51
SAMPLE AREA, cm ²	37.39	DRY UNIT WT., kN/m ³	12.10
SAMPLE VOLUME, cm ³	519.76	SPECIFIC GRAVITY, measured	2.75
WET WEIGHT, g	928.20	VOID RATIO	1.23
DRY WEIGHT, g	641.47		

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	14.9	COMPRESSIVE STRESS, kPa	19
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REMARKS: Specimen taken 27cm from top of the sample. DATE: 5/9/2011

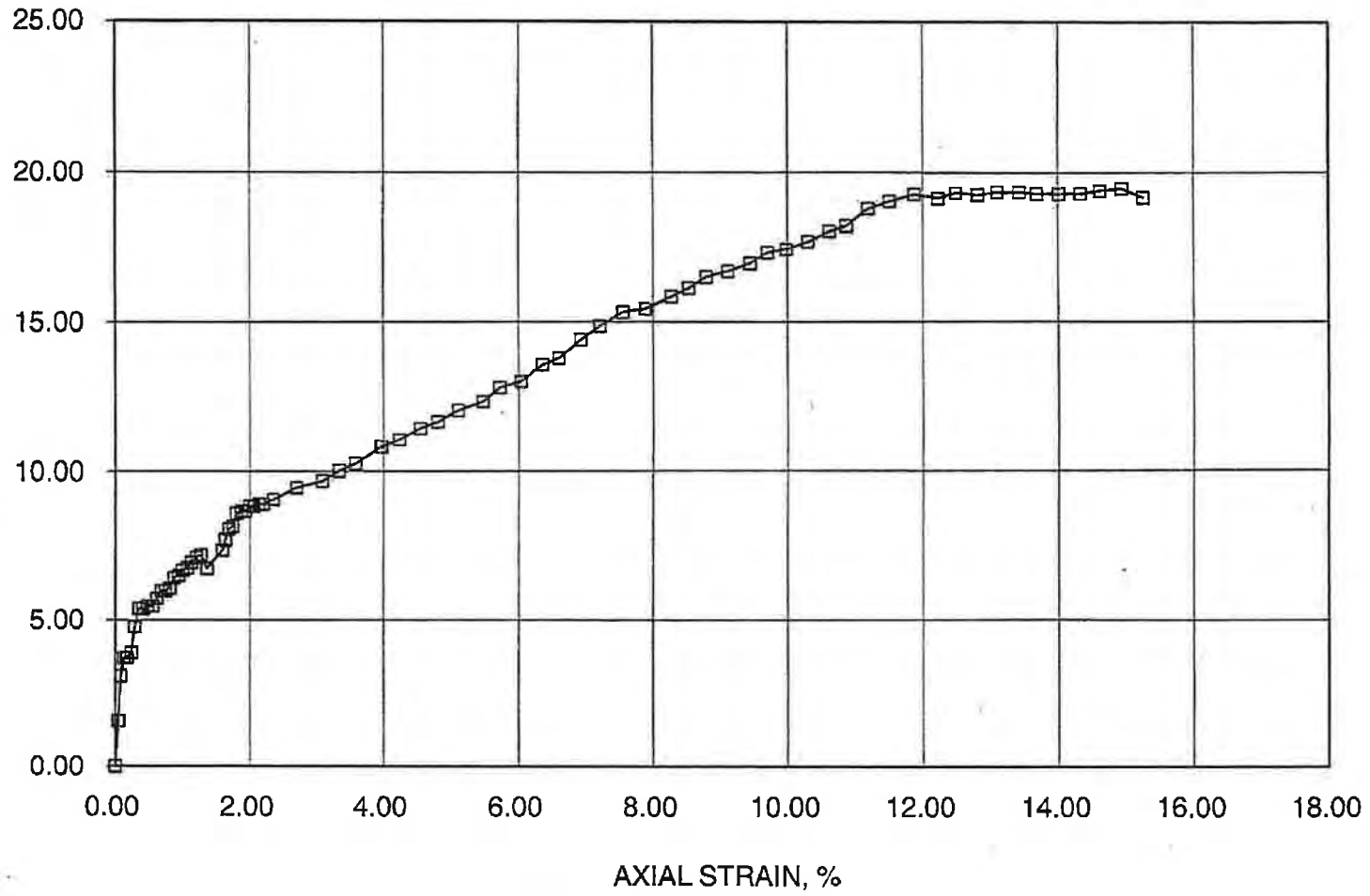
Checked By: *ll*

Golder Associates

UNCONFINED COMPRESSION TEST (UC)

FIGURE

Borehole 11-2 Sample ST-12 Depth 12.2m



Project No. 11-1183-0023

DEVIATOR STRESS, kPa

Checked By:

MH

APPENDIX D

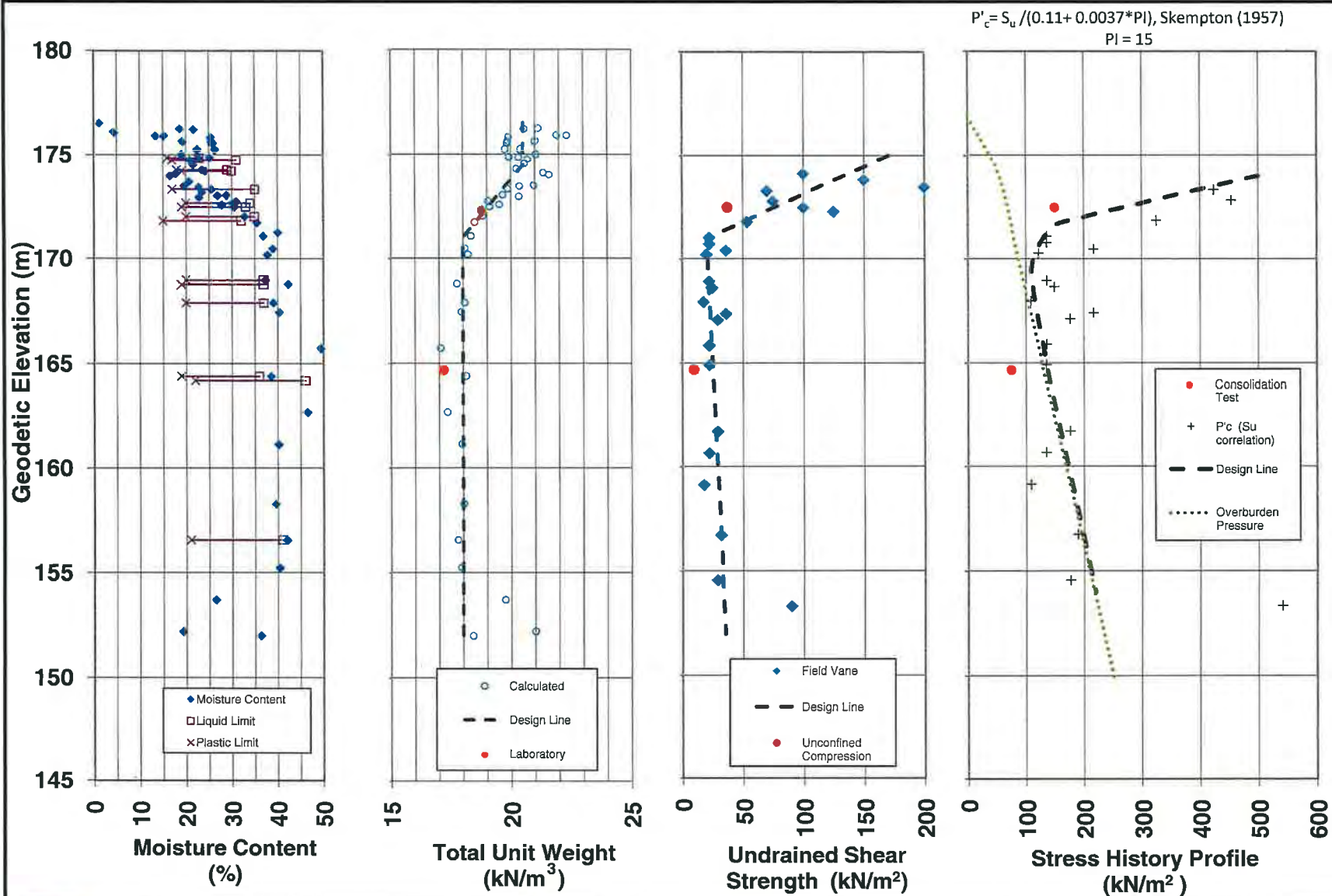
Figure 3a and 3b: Design Parameters

Figure 4: Lateral Deflection for HP310x110 (LPILE)

Figure 5: p-y Curves for HP310x110 (LPILE)

Figure 6: Typical Slope Stability Evaluation Results (Slope/W)

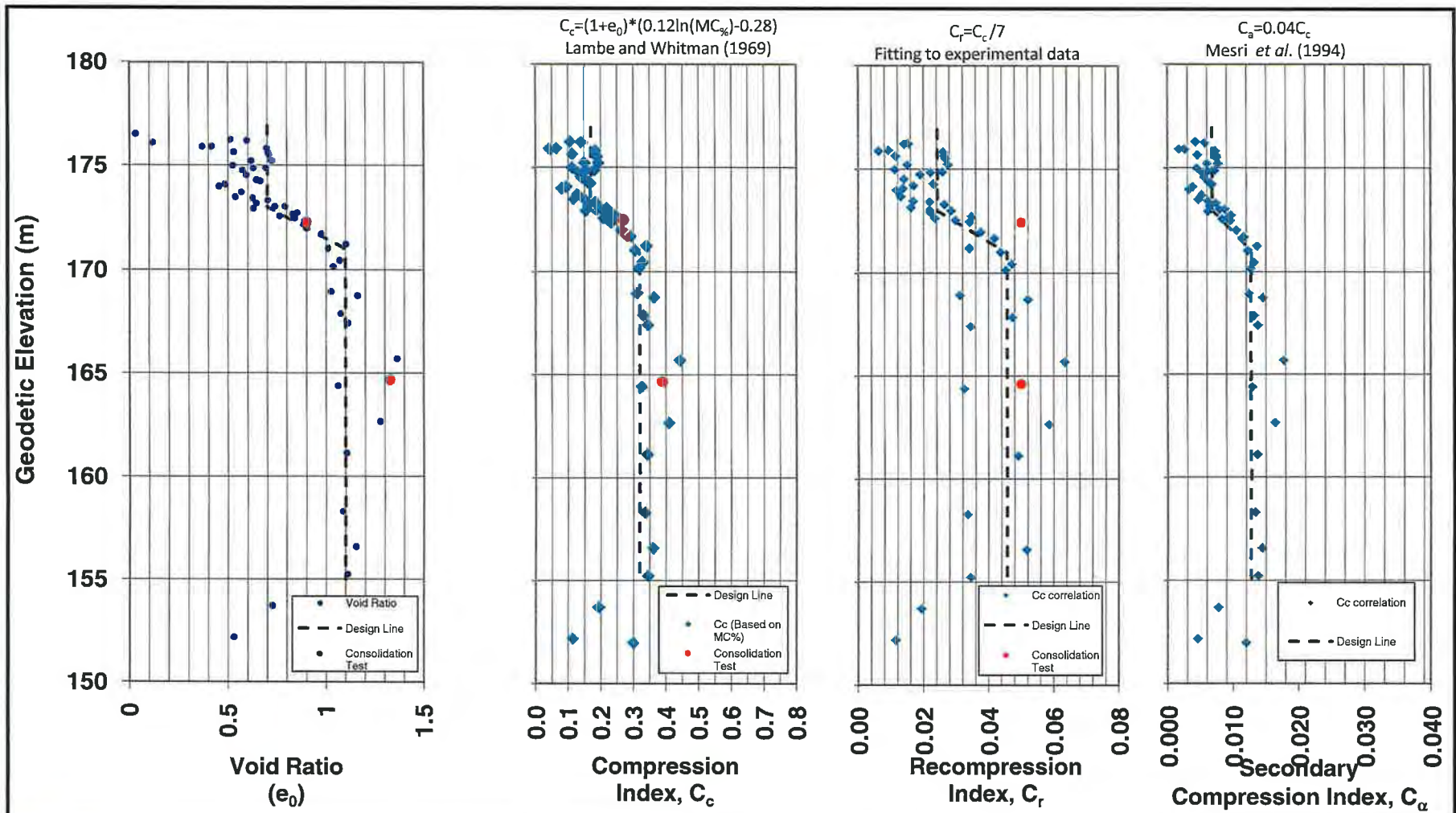
Drawing No. 2: Roadway Protection System



Stantec Consulting Ltd.

Highway 40
18th Concession Drain Bridge

Figure 3a



Stantec Consulting Ltd.

Highway 40
18th Concession Drain Bridge

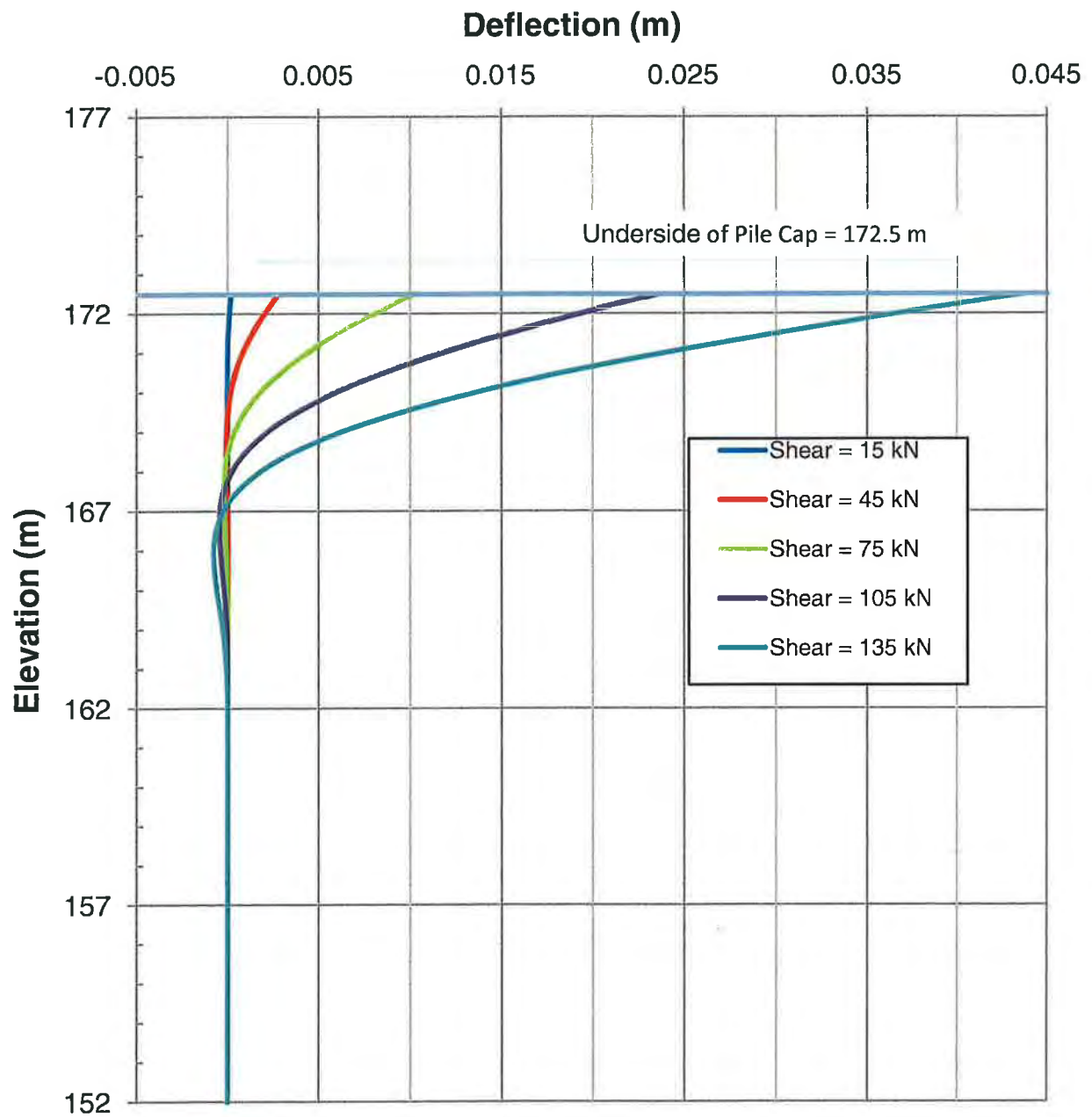
Figure 3b



Stantec

**Project No. 165000744
Concession # 18 Drain Bridge**

LPile Results - Lateral Deflection



**Figure 4
Lateral Deflection of HP 310x110 Piles**



Stantec

Project No. 165000744
Concession # 18 Drain Bridge

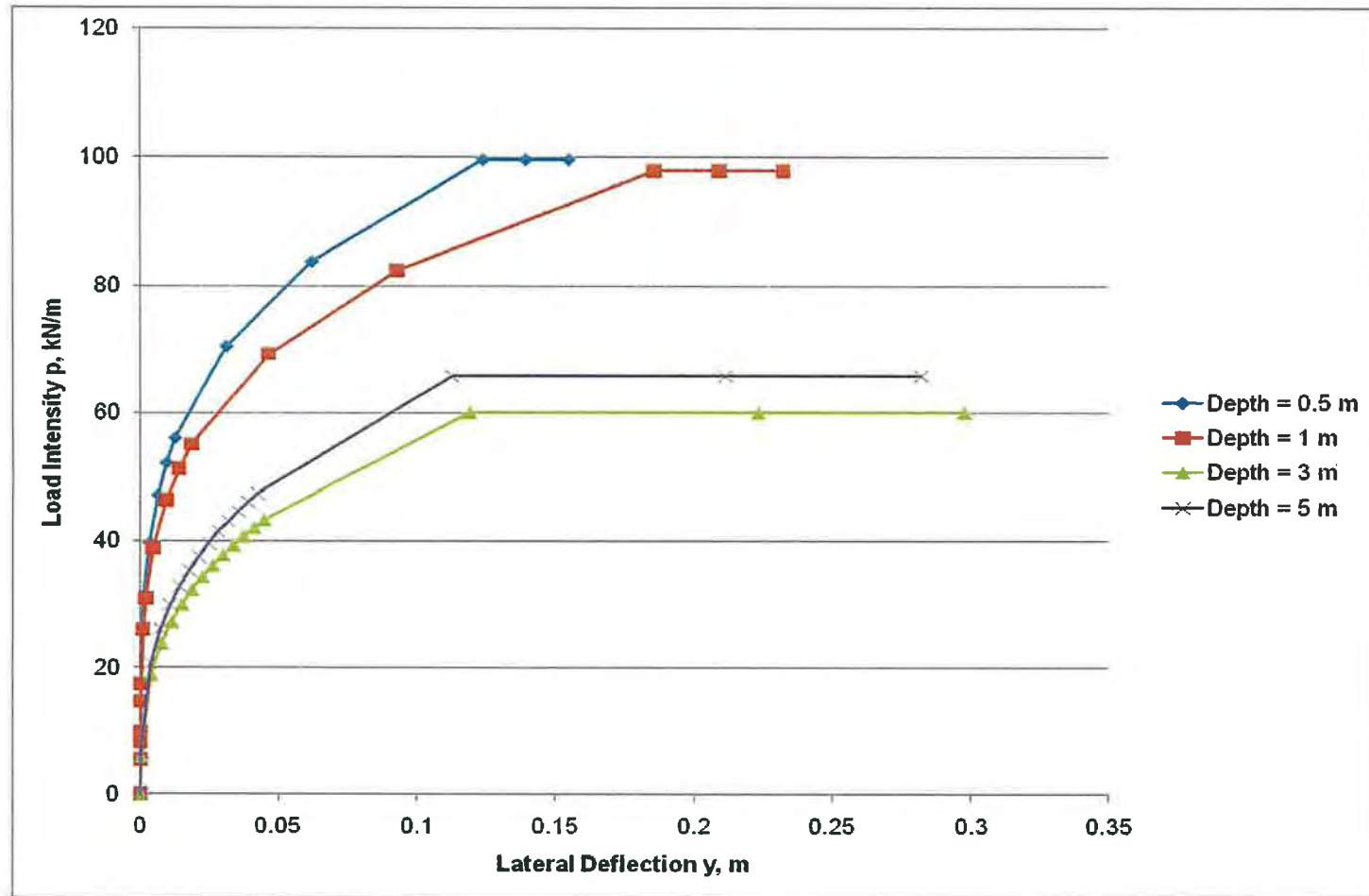


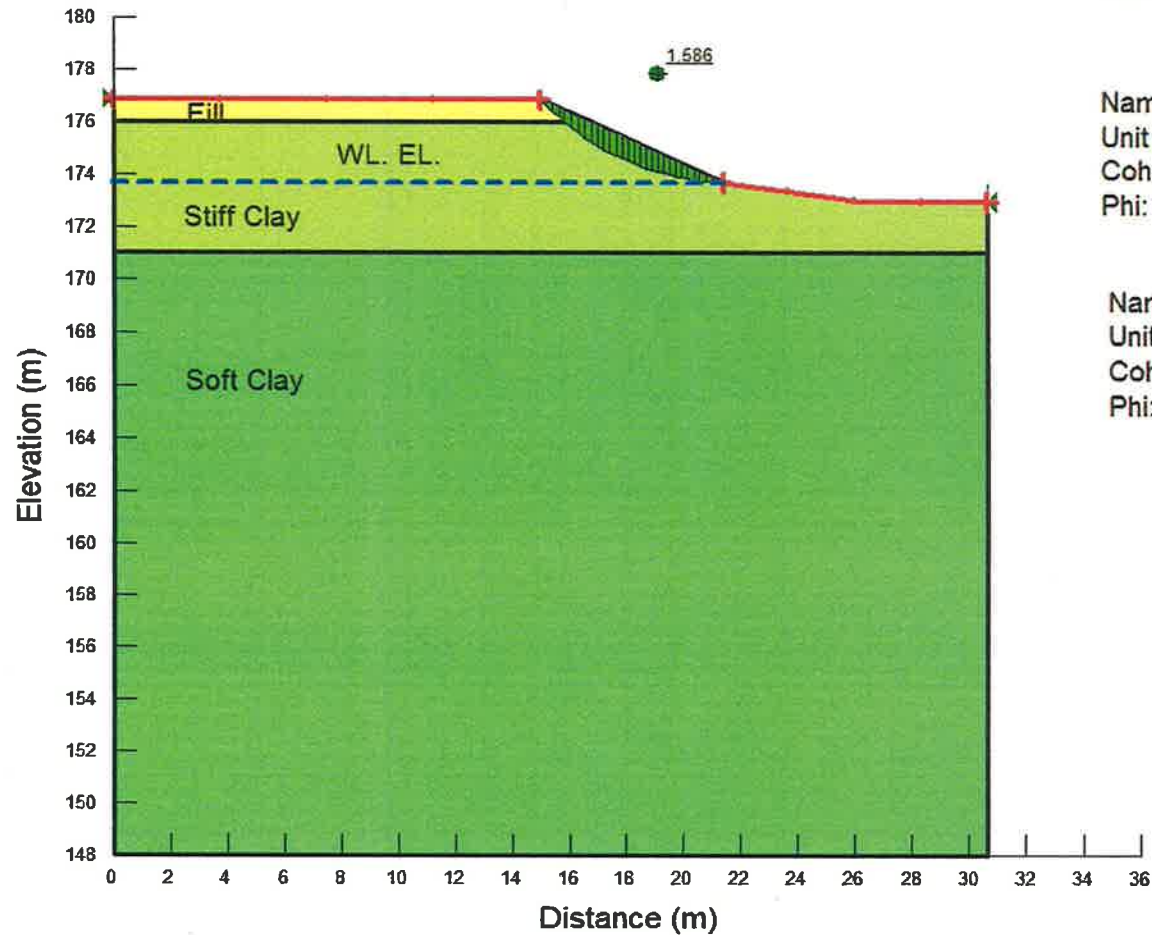
Figure 5
p-y Curves for Proposed HP 310x110 Piles

18th Concession Drain Bridge
 2H:1V Slope extending from Ground Surface
 to Drain Canal Bed
 Long-Term Soil Parameters

Name: Fill
 Unit Weight: 20.5 kN/m³
 Cohesion: 0 kPa
 Phi: 30 °

Name: Stiff Clay
 Unit Weight: 19.5 kN/m³
 Cohesion: 2 kPa
 Phi: 30 °

Name: Soft Clay
 Unit Weight: 18 kN/m³
 Cohesion: 2 kPa
 Phi: 30 °



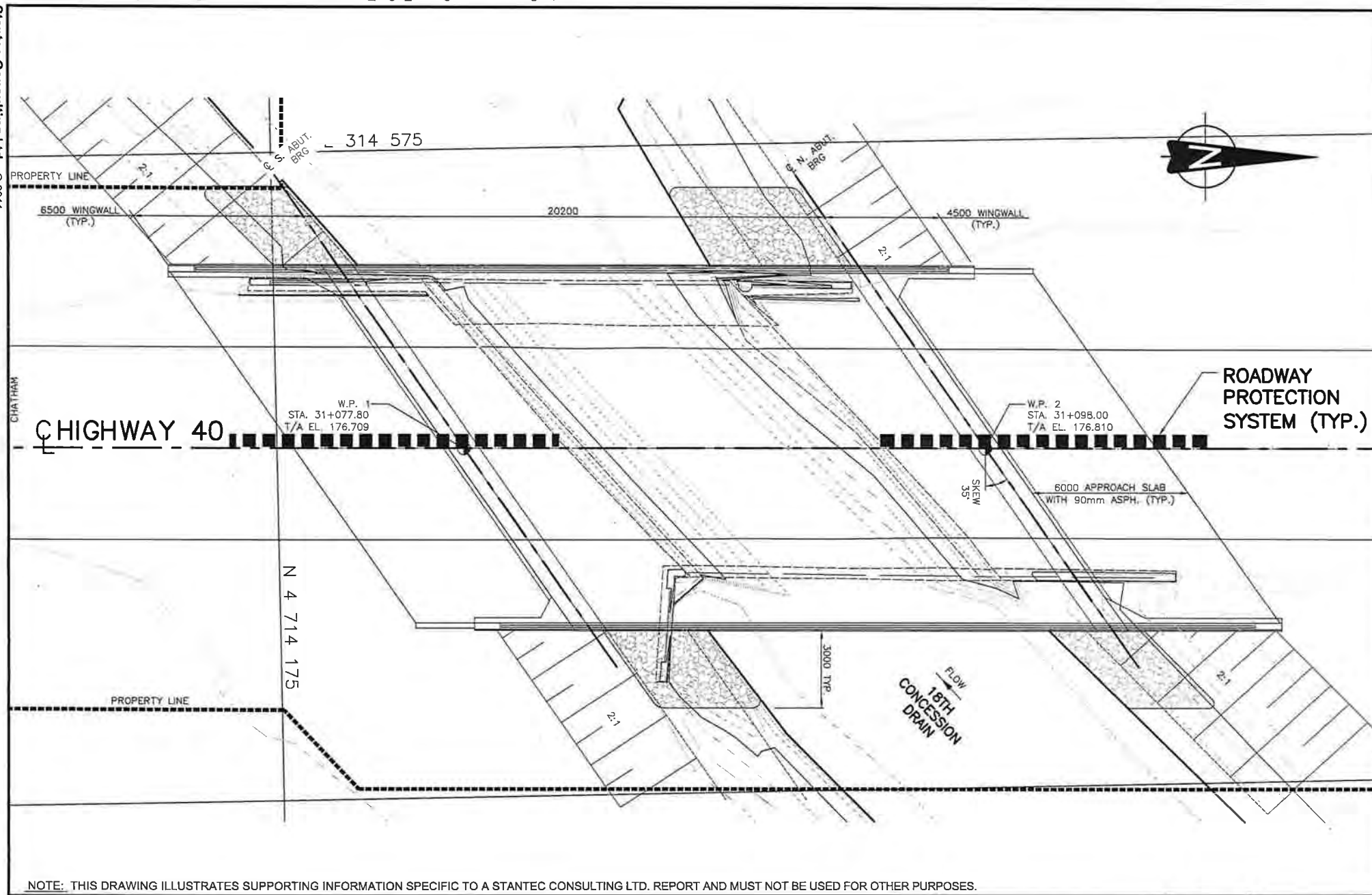
Stantec

Typical Slope Stability Analysis

18th Concession Drain Bridge
 Ground Slope to Canal Bed at 2:1

Figure 6a

Project No. 165000744



ROADWAY PROTECTION SYSTEM, CONCEPTUAL LOCATION

WP 3103-03-01, HIGHWAY 40
18TH CONCESSION DRAIN BRIDGE, TOWNSHIP OF CHATHAM, ONTARIO

Client:

MTO

Job No.: 165000744

Scale: 1 : 200

Date: 11/08/04

Dwn. By: GBB

App'd By: 7.12.1

Dwg. No.:

2



Stantec

APPENDIX E

Geological Survey of Canada Seismic Hazard Calculation

2005 National Building Code Seismic Hazard Calculation

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

Requested by: SG, Stantec

April 29, 2011

Site Coordinates: 42.567 North 82.3808 West

User File Reference: Hwy 40 & Elbow Line, Near Wallaceburg

National Building Code ground motions:

2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.179	0.088	0.042	0.012	0.116

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

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.023	0.069	0.113
Sa(0.5)	0.012	0.035	0.054
Sa(1.0)	0.005	0.015	0.024
Sa(2.0)	0.002	0.005	0.007
PGA	0.011	0.040	0.069

References

National Building Code of Canada 2005 NRCC no. 47666; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3

Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2005, Structural Commentaries NRCC no. 48192

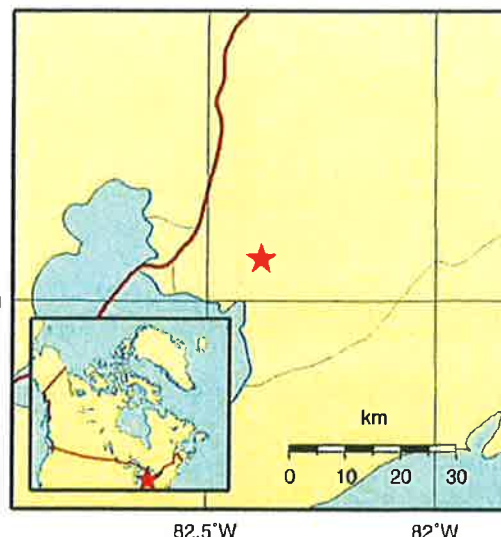
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx

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

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

Aussi disponible en français



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