



**Geotechnical Foundation Design
Report**

Metrolinx - Georgetown Corridor
Bridges Highway 427 Overpass

Prepared for:

Metrolinx

Prepared by:

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Project No. 113536134

May 2011

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1.0 Introduction

Stantec Consulting Ltd. (Stantec) was retained by Metrolinx to undertake the detailed design for the retaining wall required beneath the Highway 427 Overhead Bridge at Weston Subdivision Mileage 13.50. The retaining wall will truncate the south abutment foreslope of the Highway 427 bridge structure and will allow for the construction of two new Metrolinx rails between the south abutment and the south pier of the bridge structure.

The scope of services for the geotechnical foundation design report for the Highway 427 site included a geotechnical review of the existing preliminary geotechnical investigation report prepared by Peto MacCallum Ltd. (PETO). On completion of the review to prepare a geotechnical design report in accordance with CHBDC standards, summarizing the available information, including comments with respect to the geotechnical conditions and constraints associated with the study area.

2.0 Site Description and Geology

The Georgetown South Corridor runs from the Township of Georgetown (west of Brampton) to Union Station in downtown Toronto, Ontario.

At the rail crossing, Highway 427 is constructed on a raised three-span bridge that is approximately 10 m above the underlying rails. The approach embankments are approximately 50 m wide at the top and include 3H:1V side slopes and a 2H:1V fore slope which extend down towards the existing piers. The current study area is specifically between the south abutment and the south pier.

According to “The Physiography of Southern Ontario” (Chapman and Putman), the site is underlain by the general physiographic region known as the Peel Plain. The Peel Plain is generally a till containing large amounts of shale and limestone which is frequently covered by a veneer of clay.

3.0 Geotechnical Investigation

A Preliminary Geotechnical Investigation was completed in April 2010 by Peto MacCallum Ltd. (PETO) and a report (PML Ref.: 09TF014) was issued dated April 9, 2010.

The work was commissioned to investigate the subsurface soil and groundwater conditions along the Weston Rail Corridor. A total of five boreholes (BH CW-5, BH CW-6, BH CW6A, BH CW-7, and BH CW-8) were advanced in the area of the Highway 427 Overpass. The existing boreholes were drilled adjacent to the bridge approach embankments and therefore information regarding the composition of the existing approach embankments is not known.

Factual results extracted from the PETO report are included in Appendix C. These include Log of Boreholes CW-5, CW-6, CW-6A, CW-7, and CW-8 which are specific to the Highway 427 site. Reference to other borehole numbers on the laboratory summary reports are for other locations which are not discussed in this report.

4.0 Subsurface Conditions

The typical subsurface conditions encountered on the site consisted shallow fill or peat underlain by native stiff to hard clayey silt to silty clay till, dense to very dense sandy silt to silty sand till. Localized layers and zones of compact sand and silt, very stiff to hard silt clay, dense to very dense sand and gravel, and loose silty fine sand were encountered in the boreholes.

Shale bedrock was encountered in two of the boreholes at depths of 21 m and 23 m below existing grade. The upper zone of the shale was reported to be weathered. Unconfined compressive strength values of the bedrock measured on samples from below the weathered zone were reported to be in the range of 9.5 to 16.9 MPa.

Groundwater observations recorded by PETO in Borehole CW-8 are summarized as follows:

Borehole Elevation:	164.6 m
Screen Depth:	7.6 to 10.6 m
Groundwater Depth:	4.9 to 5.0 m
Groundwater Elevation:	159.6 to 159.7 m

The above range of groundwater observations were recorded by PETO on December 23, 2009, January 25, 2010 and February 4, 2010. It should be noted that groundwater levels are subject to seasonal variations and can change rapidly in response to specific precipitation events.

5.0 Discussions

5.1 GENERAL

Based on the information and drawings provided, it is our understanding that the proposed development at the location of the Highway 427 overpass will consist of two new tracks additions between the south abutment and the south pier of the Highway 427 overhead bridge. A new retaining wall will be required to accommodate the proposed additional tracks. The General Arrangement Drawing included in Appendix D indicates that the foundations for the proposed retaining wall will include H-piles within concrete filled pre-augered holes (drilled-in soldier piles) and soil or rock anchor tie-backs to be threaded between the existing abutment piles.

At the rail crossing Highway 427 is constructed on a raised three-span bridge that is approximately 10 m above the underlying rails. The approach embankments are approximately

50 m wide at the top and include 3H:1V slideslopes and 2H:1V foreslope which extends down towards the existing piers. The anchored drilled-in soldier pile retaining wall will truncate the south foreslope to allow for the construction of two new tracks between the south abutment and south pier. The retaining wall will support up to 4.5 m of ground that will slope upwards at a grade of 2H:1V.

5.2 GEOTECHNICAL MODEL

The general conditions encountered in the boreholes nearest the Highway 427 Overpass (BH CW-5, BH CW-6, BH CW-6A, BH CW-7 and BH CW-8) are summarized above under “Background”. The following soil model was generated for the analysis:

Table 5.1 – Geotechnical Model

Material	Depth Relative to Top of Pile (m)	Total Unit Weight, γ (kN/m³)	Friction Angle, ϕ (°)	Undrained Shear Strength, S_u (kN/m²)
Fill	0 – 0.6	21.0	32	---
Clayey Silt to Silty Clay Till	0.6 – 3.1	21.5	---*	75*
	3.1 – 9.4			100*
Silty Fine Sand	9.4 – 10.4	20.8	30	---
Sandy Silt Till	10.4 – 23.0	22.5	35	---
Shale Bedrock	➤ 23 m	25.8	-	N/A

Note: Where drained analysis was carried out, an effective angle of 25 degrees and an effective cohesion of 10 kPa was used for the cohesive soils between depths 0.6 m and 9.4 m.

The design groundwater table should be assumed to be at final grade. The effective unit weight of the soils below the water table is obtained by subtracting the unit weight of water (9.807 kN/m³) from the total unit weight.

5.3 FROST PENETRATION

The design frost penetration depth for Toronto is 1.2 m. Therefore the underside of the concrete wall which will cover the drilled-in soldier pile retaining wall should be 1.2 m below the ground level adjacent to the base of the retained height.

6.0 Foundation Recommendations

6.1 DEEP FOUNDATIONS (PILES)

It is understood that the proposed retaining wall will be supported drilled-in soldier piles consisting of HP 310x110 piles inserted within 760 mm diameter predrilled holes and then concreted in place. Soil anchor tie-backs will also support the retaining wall and are to be threaded between the existing abutment piles.

6.1.1 Geotechnical Axial Resistance

The axial capacity of 760 mm diameter drilled-in soldier piles was analyzed using the program APile Plus v5.0 developed by Ensoft. It is understood that the soldier H-Piles will be embedded within the concrete filled 760 mm diameter pre-augered holes.

A plot of the unfactored geotechnical axial pile capacity vs depth is shown on Figure 2 in Appendix D. Assuming a pile length of 5.0 m, the unfactored axial capacity of the pile is 760 kN.

In accordance with the CHBDC, a geotechnical resistance factor of 0.4 should be applied to the unfactored axial capacity in order to obtain the geotechnical axial resistance at ULS of 300 kN.

Settlements of less than 5 mm are anticipated; therefore geotechnical reaction at SLS is not provided.

6.1.2 Geotechnical Lateral Resistance

The factored geotechnical lateral resistance at ULS for a 3 to 5 m long, 760 mm diameter pile was calculated using the simplified method outlined in Section C6.8.7 of the CHBDC and a resistance factor, R_f , of 0.5. The geotechnical lateral resistance at ULS for a single pile is summarized as follows:

Table 6.1: Geotechnical Lateral Pile Resistance at ULS - 760 mm Drilled-In Soldier Pile

Length of Drilled-In Portion (m)	Geotechnical Resistance ULS (kN)
3.0	540
4.0	880
5.0	1070

In order to calculate appropriate linear spring constants for structural modeling purposes, the lateral soil-pile response was analyzed using the program LPile Plus v6.0 developed by Ensoft. The inputs based on the design soil profile are given in the table below. An axial load of 300 kN has been assumed. In addition, the piles were assumed to have a pinned head condition (that is, no moment at the top of pile). The anticipated lateral deflection of the piles is shown on Figure 3 in Appendix D.

Table 6.2: Design Soil Profile for Axial Pile Capacity Analysis of Concrete-Encased Piles

Depth Relative to Top of Concrete (m)	Soil Type	Total Unit Weight, γ (kN/m ³)	Angle of Internal Friction, ϕ (°)	Undrained Shear Strength (kN/m ²)	p-y Modulus, k (kN/m ³)	Strain Factor, E_{50}
0 – 0.6	Fill	21.0	32	---	6790	---
0.6 – 3.1	Clayey Silt to Silty Clay Till	21.5	---	75	---	0.007
3.1 – 9.4			---	100	---	0.006
9.4 – 10.4	Silty Fine Sand	20.8	30	---	5430	---
10.4 - 13.0	Sandy Silt Till	22.5	35	---	33900	---

Note: Groundwater water was assumed 3 m below final grade in front of the wall.

Several p-y curves were also generated by LPILE, which relate the non-linear soil response of a pile (p) with the lateral deflection (y). Separate curves were generated for the mid-point of each distinct soil layer, and are shown on Figure 4 in Appendix C. From the p-y curves, the linear spring constant, k, can be generated for the elastic region by determining the slope of each curve between the origin (0,0) and the maximum allowable deflection (in this case, 12 mm was assumed). As the piles are anticipated to be modeled in 0.3 m (1 ft) sections, the slope of the line was multiplied by this distance. Calculated linear spring constants for relevant soil layers are given below.

Table 6.3: Calculated Linear Spring Constants (k) for a HP 310x110 Pile within 760 mm Diameter Concrete Filled Hole at Selected Depths

Depth below Top of Concrete Portion (m)	Soil Type	Calculated Linear Spring Constant, k (kN/m)
0 – 0.6	Fill	600
0.6 – 3.1	Clayey Silt to Silty Clay Till	3000
3.1 – 9.4		5600

Note: The constants are applicable for springs spaced at 0.3 m (1 ft).

6.2 LATERAL EARTH PRESSURES

The proposed drilled-in soldier pile retaining wall will be retaining the soils which currently form the 2H:1V fore-slopes extending from the abutment. The retained height will be as high as 4.5 m and the back-slope will be as steep as 2H:1V.

The entire supported height will consist of embankment fill material which was placed above native ground. Therefore the following points should be considered:

- Currently no boreholes have been drilled through the embankment fill, and for this reason conservative soil parameters need to be considered in assessing lateral earth pressure.
- Groundwater will not rise within the embankment fill and therefore groundwater pressure need not be incorporated in the design. However, a prefabricated drainage layer should be incorporated between the lagging and concrete facia to ensure that infiltrated surface water does not accumulate behind the wall.

Table 6.4: Static Earth Pressure Coefficients - 2H:1V Backfill

Parameter	Assumed Embankment Foreslope Fill
Unit Weight (kN/m^3)	20.0
Angle of Internal Friction, Φ	30°
Coefficient of Passive Earth Pressure, K_p	7.48
Coefficient of Active Earth Pressure, K_a	0.54

The at-rest earth pressure coefficient was calculated to be 0.5 which is less than the active pressure coefficient; therefore, the at-rest case is not included in the above table.

Table 6.5: Seismic Earth Pressure Coefficients - 2H:1V Backfill

Parameter	Assumed Embankment Foreslope Fill
Unit Weight (kN/m^3)	20.0
Angle of Internal Friction, Φ	30°
Coefficient of Passive Earth Pressure, K_{pE}	7.42
Coefficient of Active Earth Pressure, K_{aE}	0.60

The seismic earth pressure coefficients were calculated using a zonal acceleration ratio of 0.05 which is applicable for the Toronto area.

6.3 SOIL AND ROCK ANCHORS

The anchors for the drilled-in soldier pile retaining wall should be designed and constructed in accordance with the minimum requirements of the CHBDC (CAN/CSA-S6-06) and OPSS 942 Construction Specification for Pre-Stressed Soil and Rock Anchors. Appropriate design methods are outlined in FHWA Geotechnical Circular No. 4 "Ground Anchors and Anchored Systems".

The earth pressure coefficients presented in Table 5 may be used to develop the "apparent earth pressure diagram" behind the retaining wall. It is recommended that the pressure distribution defined in FHWA Geotechnical Circular No. 4, Section 5.2.4 Recommended Apparent Earth Pressure Diagrams for Sands, be used to calculate the horizontal loads in the anchors.

The sections below provide some guidance in assessing anchor capacities. Where the contractor proposes higher bond strengths than discussed below, pre-production test anchors as per Section 942.07.12.05 of OPSS 942 should be carried out.

6.3.1 Soil Anchors

For soil anchors the top portion of the bonded length should be at least below elevation 163.5 m to ensure that they extend into the native soil, as well as below the critical failure surface. As per the CHBDC the critical failure surface may be taken at a 45° angle from the back of the wall, starting at the bottom of the excavation.

The actual soil anchor bond lengths required in the field will depend on the method of drilling, quality of drill hole cleaning, length of time the drill hole is left open, the diameter of the drill hole, the method and pressure used in grouting, and the design load required to be resisted by the anchor. Therefore, the actual depth at which the anchor should be constructed, the free-stressing length and the bond length should be left to the discretion of the specialist anchor contractor.

The anchor capacity used by the contractor for design should be confirmed by full scale testing.

For purpose of preliminary estimates, the capacity of a single soil anchor may be calculated by:

Where Q_u = Bond resistance

B = Diameter of grouted portion or drilled hole, depending if grout take is high or low, respectively

L_c = Length of grouted section in clay

L_s = Length of grouted section in sand

$C = S_u$ for undrained clays

σ_n = Overburden stress at midpoint of fixed portion

δ = Angle of internal friction, ϕ

The Maximum Pullout Resistance, R , is given by:

Where α = geotechnical resistance factor = 0.4

s = spacing

The soil properties given in the Geotechnical Model presented in Section 5.2 may be used.

When assessing the potential available soil anchor capacities, the unbonded tendon length should extend beyond the critical failure surface and should be a minimum of 4.6 m in length, and the bonded length should be assumed to be 12 m. For soil anchors, significant increases in capacity for bonded lengths greater than 12 m are generally not achievable.

Example preliminary soil anchor capacities

- for a 150 mm diameter, 12 m long bonded length embedded in the very stiff clay till, between 3 and 9 m below the base of the retained height, the geotechnical resistance at ULS for the soil anchor would about 200 kN.
- for a 150 mm diameter, 12 m long bonded length, with the midpoint embedded about 15 m below base of the retained height within the very dense silty sand till, the geotechnical resistance at ULS for the soil anchor would be about 275 kPa.

6.3.2 Rock Anchors

For rock anchors it is anticipated that the bonded length would be below elevation 140 m.

For the design of rock anchors extending into bedrock, the following design parameters may be considered for the rock mass.

- An unfactored rock to grout bond stress of 600 kPa may be used for holes grouted with non-shrink grout having a minimum compressive strength of 30 MPa. As per the CHBDC, a resistance factor of 0.4 should be applied when calculating the geotechnical resistance at ULS.
- The top of the bonded length should be at least 3.0 m below the top of rock.
- The minimum fixed anchor length (i.e. the length over which the rock to grout bond stress is developed) should be no less than 3 m.
- Regardless of the overall length of the bonded section of the anchor, the maximum bond length which should be used to calculate the anchor capacity is 8 m. For rock anchors, significant increases in capacity for bond lengths greater than 8 m are generally not achievable.

To ensure against the possibility of a rock mass failure, the following design parameters should be used:

- Submerged unit weight of bedrock = 16 kN/m³
- A 90° (apex angle) failure cone with the apex located at the midpoint of the bonded.
- The effective overburden weight provides additional resistance to rock cone pull-out failure and should be applied to the cone base area at the top of rock; the effective pressure at the top of rock associated with the overburden can be assumed as 250 kPa.

The bond stress used by the contractor for design should be confirmed by full scale testing of anchors.

7.0 Construction Considerations

The soil or rock anchors for the drilled-in soldier pile retaining wall should be constructed in accordance with OPSS 942 Construction Specification for Pre-stressed Soil and Rock Anchors.

The embankment fills have not been investigated. Therefore, the soil conditions that are within the embankment foreslope are not known. Generally, it is anticipated to consist of a granular soil.

The embankment foreslope is above the water table and the underlying soils are cohesive. Therefore, a liner is not anticipated to be required within the augered holes which will be drilled to construct the drilled-in soldier piles.

It is recommended that full time geotechnical inspection services be provided during construction of the drilled-in soldier piles.

8.0 Closure

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of Metrolinx who is identified as "the Client" within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying or unexpected site conditions
- Planning, design or construction

Respectfully submitted,

STANTEC CONSULTING LTD.

fer.
7.5.13

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APPENDIX A

Statement of General Conditions

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec Consulting Ltd.'s present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec Consulting Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd. will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd. that differing site or subsurface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec Consulting Ltd., sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec Consulting Ltd. cannot be responsible for site work carried out without being present.

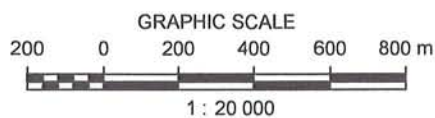
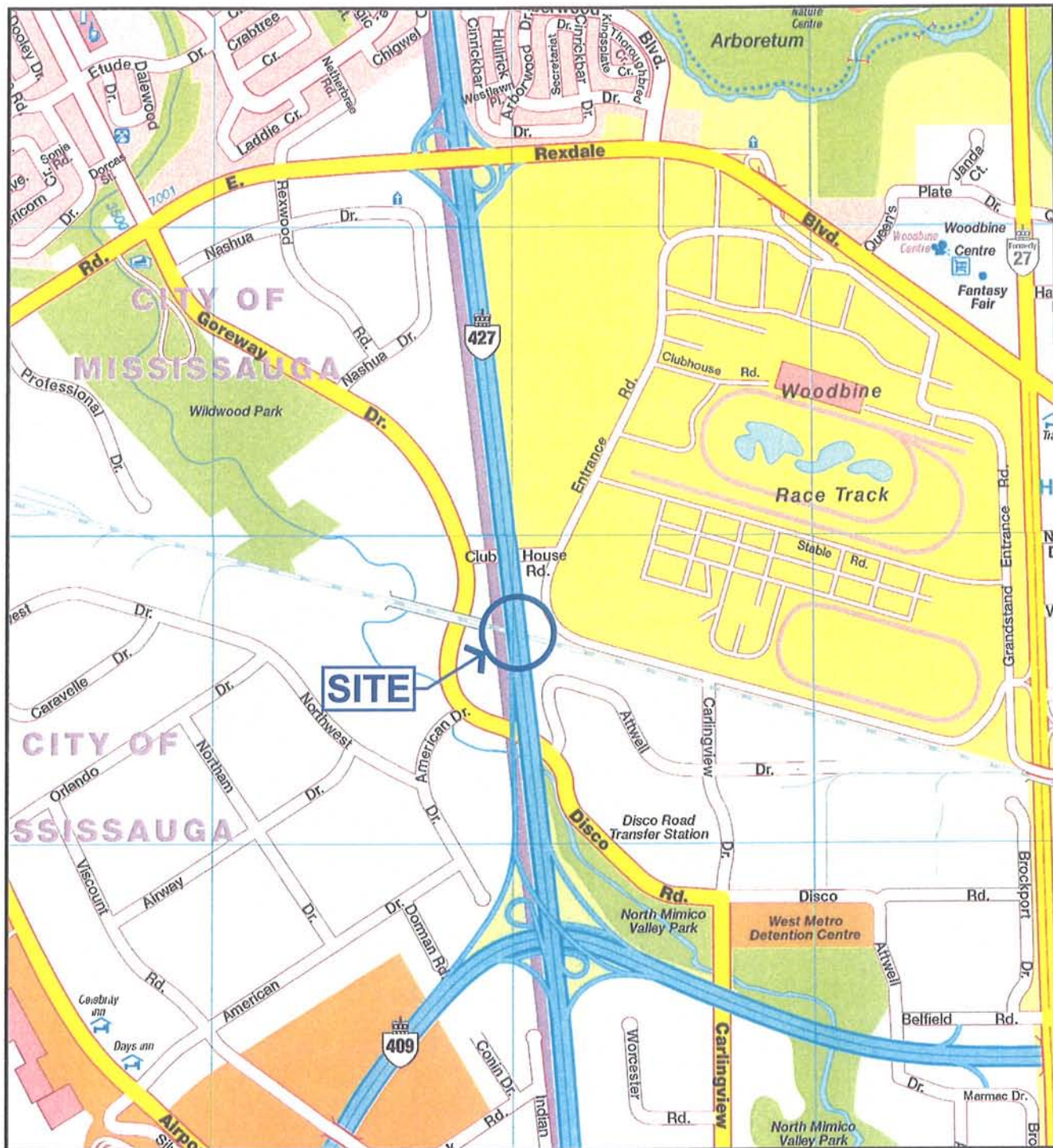


APPENDIX B

Drawing No. 1 – Key Plan

Drawing No. 2 – Borehole Location Plan

Drawing No. 3 – Section 1



REFERENCE: MAPART

NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A STANTEC CONSULTING LTD. REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

KEY PLAN

GEOTECHNICAL INVESTIGATION
GEORGETOWN CORRIDOR BRIDGES, HWY 427, TORONTO, ONTARIO

Job No.: 113536134

Scale: 1 : 20 000

Date: 11/03/30

Dwn. By: GBB

App'd By:

Dwg. No.:

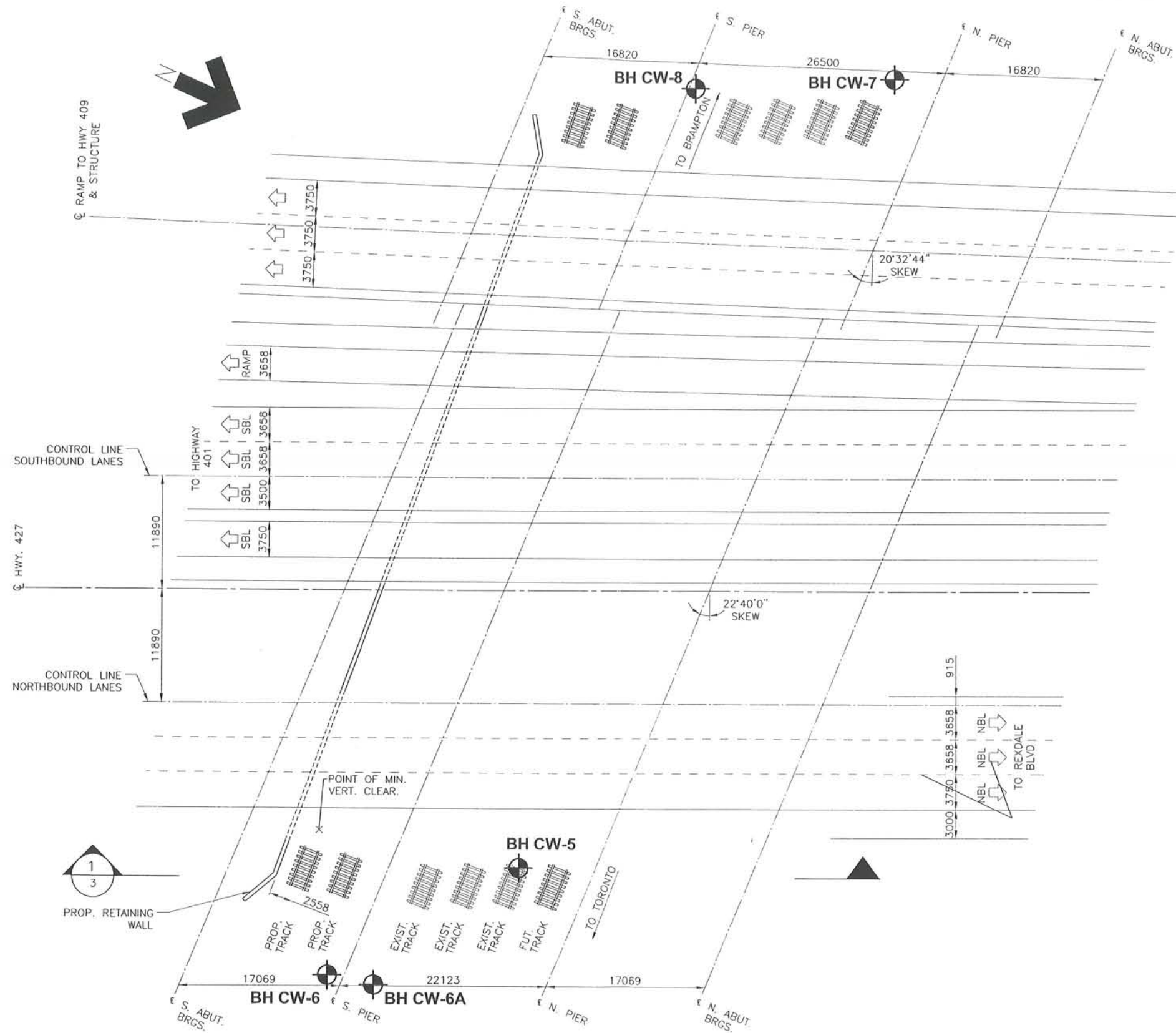
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Stantec



Client: METROLINX

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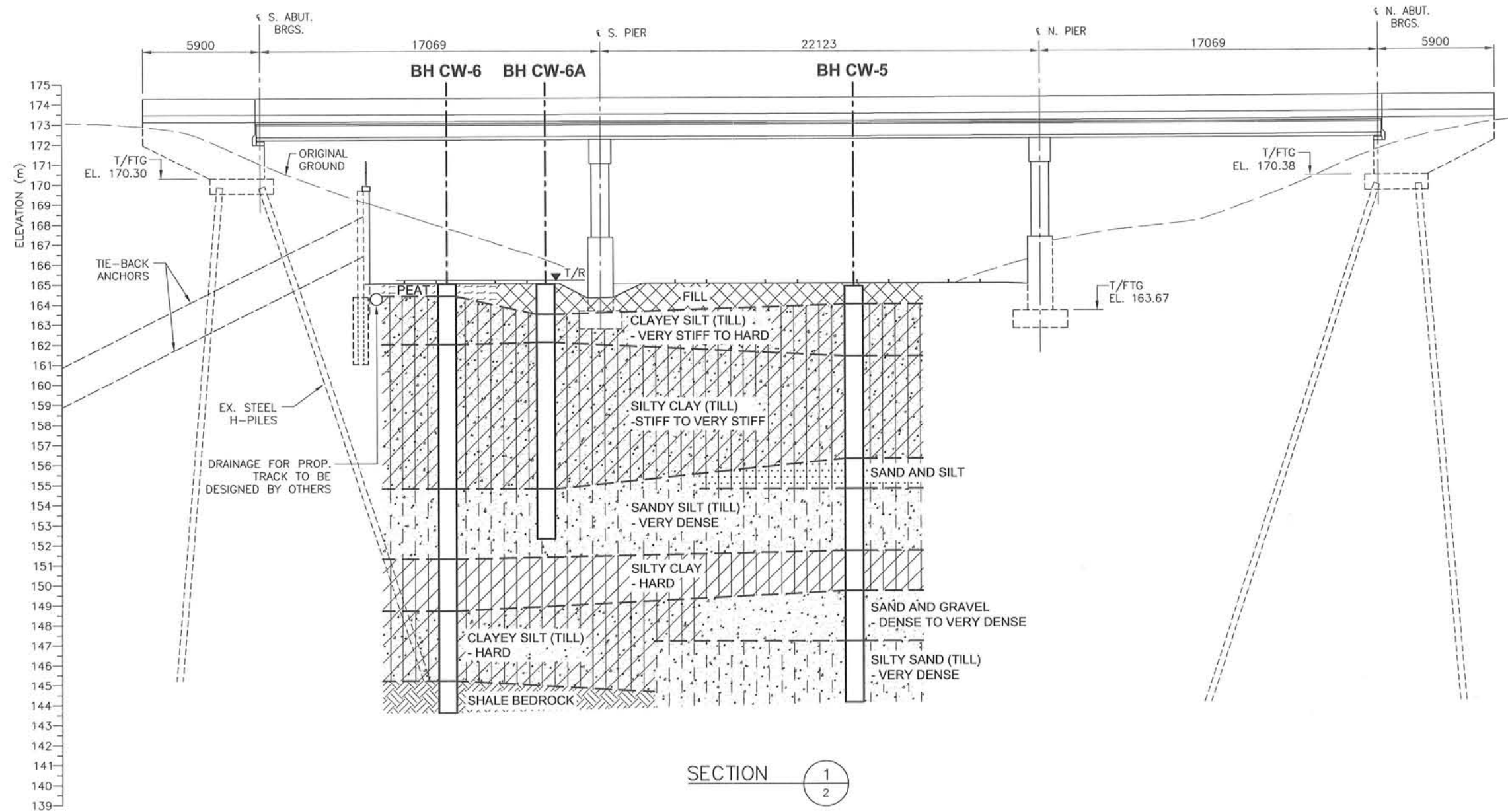


LEGEND:
 BOREHOLE

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Reference:	Job No.:	113536134	Client:	GEOTECHNICAL INVESTIGATION	BOREHOLE LOCATION PLAN	Dwg. No.:	2	
	Scale:	1 : 500						
	Date:	11/04/08	Site Address					
	Dwn. By:	GBB						
	App'd By:							
			METROLINX					
			GEORGETOWN CORRIDOR BRIDGES HWY 427, TORONTO, ONTARIO					

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NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A STANTEC CONSULTING LTD. REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

Reference:	Job No.:	113536134	Client:	METROLINX	GEOTECHNICAL INVESTIGATION	SECTION 1	Dwg. No.:	3	
	Scale:	1 : 250							
	Date:	11/05/27	Site Address						
	Dwn. By:	GBB							
	App'd By:								
			GEORGETOWN CORRIDOR BRIDGES HWY 427, TORONTO, ONTARIO						

APPENDIX C

Factual Information Extracted from Peto MacCallum Ltd. Report No. 09TF014 dated
April 19, 2010

LIST OF ABBREVIATIONS



PENETRATION RESISTANCE

Standard Penetration Resistance N: - The number of blows required to advance a standard split spoon sampler 0.3 m into the subsoil. Driven by means of a 63.5 kg hammer falling freely a distance of 0.76 m.

Dynamic Penetration Resistance: - The number of blows required to advance a 51 mm, 60 degree cone, fitted to the end of drill rods, 0.3 m into the subsoil. The driving energy being 475 J 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:

<u>CONSISTENCY</u>	<u>N (blows/0.3 m)</u>	<u>c (kPa)</u>	<u>DENSENESS</u>	<u>N (blows/0.3 m)</u>
Very Soft	0 - 2	0 - 12	Very Loose	0 - 4
Soft	2 - 4	12 - 25	Loose	4 - 10
Firm	4 - 8	25 - 50	Compact	10 - 30
Stiff	8 - 15	50 - 100	Dense	30 - 50
Very Stiff	15 - 30	100 - 200	Very Dense	> 50
Hard	> 30	> 200		
WTPL	Wetter Than Plastic Limit			
APL	About Plastic Limit			
DTPL	Drier Than Plastic Limit			

TYPE OF SAMPLE

SS	Split Spoon	TW	Thinwall Open
WS	Washed Sample	TP	Thinwall Piston
SB	Scraper Bucket Sample	OS	Oosterberg Sample
AS	Auger Sample	FS	Foil Sample
CS	Chunk Sample	RC	Rock Core
ST	Slotted Tube Sample		
	PH	Sample Advanced Hydraulically	
	PM	Sample Advanced Manually	

SOIL TESTS

Qu	Unconfined Compression	LV	Laboratory Vane
Q	Undrained Triaxial	FV	Field Vane
Qcu	Consolidated Undrained Triaxial	C	Consolidation
Qd	Drained Triaxial		

LOG OF BOREHOLE NO. CW-5

PROJECT Go Transit - Weston Rail Corridor

Coords: 4 840 092.1 N; 811 824.8 E

OUR PROJECT NO. 09TF014

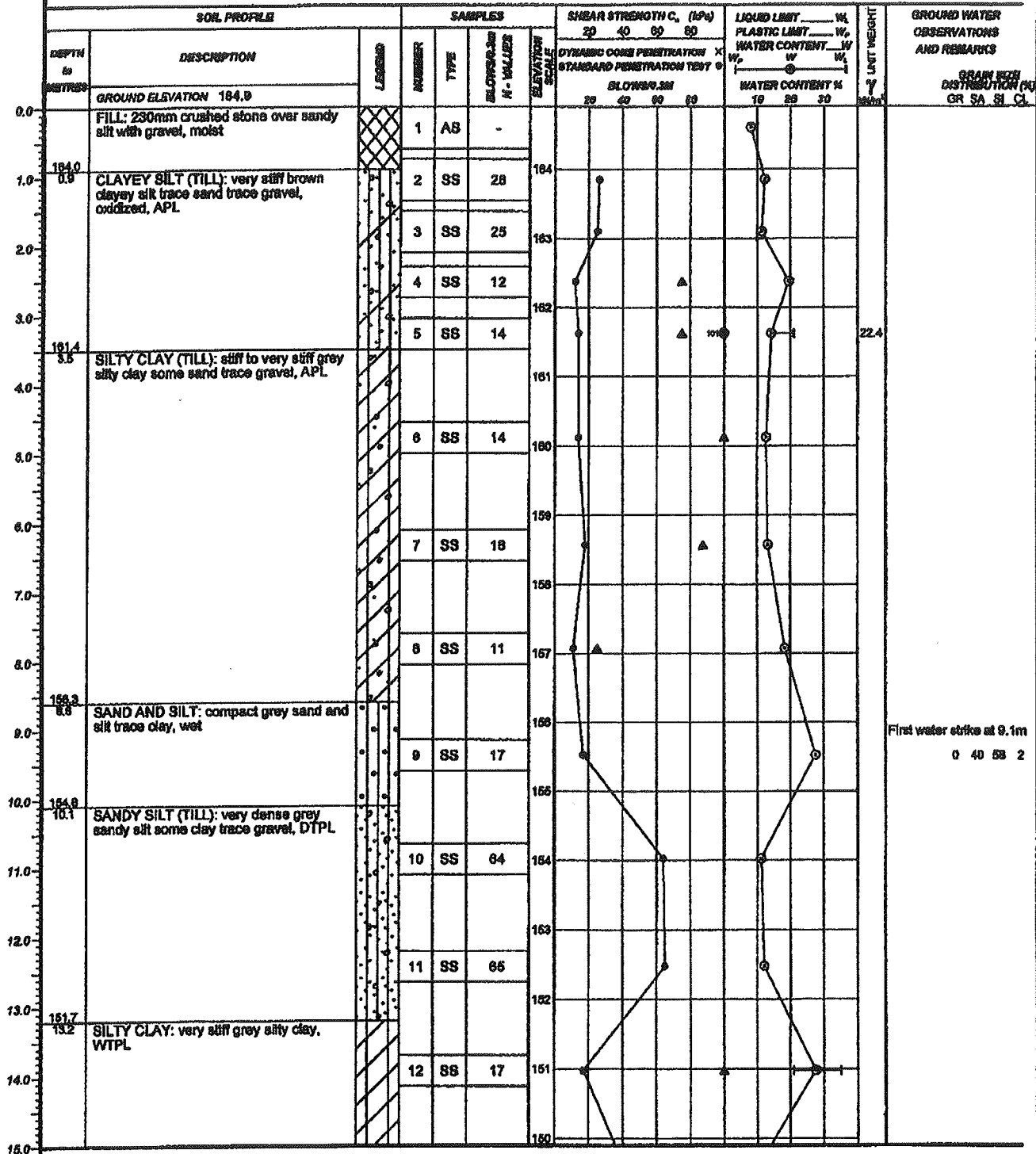
LOCATION Carlingview Drive to Mimico Creek (M13.0 to 13.8), Toronto, Ontario

ENGINEER T.X.

BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE Jan. 18 to 20, 2010

TECHNICIAN S.A.



NOTES:

+ UNDISTURBED FIELD VANE
⊗ REMOLDED FIELD VANE
⊙ LAB SHEAR TEST
△ POCKET PENETROMETER

CHECKED BY T.X.

LOG OF BOREHOLE NO. CW-5

PROJECT Go Transit - Weston Rail Corridor

Coords: 4 840 092.1 N ; 611 824.8 E

OUR PROJECT NO. 09TF014

LOCATION Carlingview Drive to Mimico Creek (M13.0 to 13.8), Toronto, Ontario

ENGINEER T.X.

BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE Jan. 18 to 20, 2010

TECHNICIAN S.A.

SOIL PROFILE			SAMPLES			SHEAR STRENGTH C_u (kPa)				LIQUID LIMIT W_L				UNIT WEIGHT γ	GROUND WATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	NUMBER	TYPE	BLOWN/BL N - VALUES	ELEVATION SCALE	20 40 60 80				PLASTIC LIMIT W_P					
							DYNAMIC CONE PENETRATION X				WATER CONTENT W					
							STANDARD PENETRATION TEST 0				WATER CONTENT %					
CONTINUED FROM PREVIOUS PAGE						BLOWN/BL				10 20 30					GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
148.7 15.2	SAND AND GRAVEL: dense to very dense gray medium sand and gravel trace silt, wet		13	SS	44	149								13 43 33 11		
17.0			14	SS	78/0.25	148										
147.3 17.7	SILTY SAND (TILL): very dense gray silty sand some clay some gravel, wet		15	SS	60/0.10	147										
19.0						146										
20.0			16	SS	58/0.15	145										
144.1 20.8	BOREHOLE TERMINATED AT 20.8 m		17	SS	60/0.03									Upon completion of augering, free water at 10.7m, cave-in at 12.2m		
21.0																
22.0																
23.0																
24.0																
25.0																
26.0																
27.0																
28.0																
29.0																
30.0																

NOTES:

+ UNDISTURBED FIELD VANE
⊗ RECONSOLIDATED FIELD VANE
⊙ LAB SHEAR TEST
Δ POCKET PENETROMETER
CHECKED BY *TX*

LOG OF BOREHOLE NO. CW-6

PROJECT Go Transit - Weston Rail Corridor

Coords: 4 840 078.4 N; 811 843.5 E

OUR PROJECT NO. 09TF014

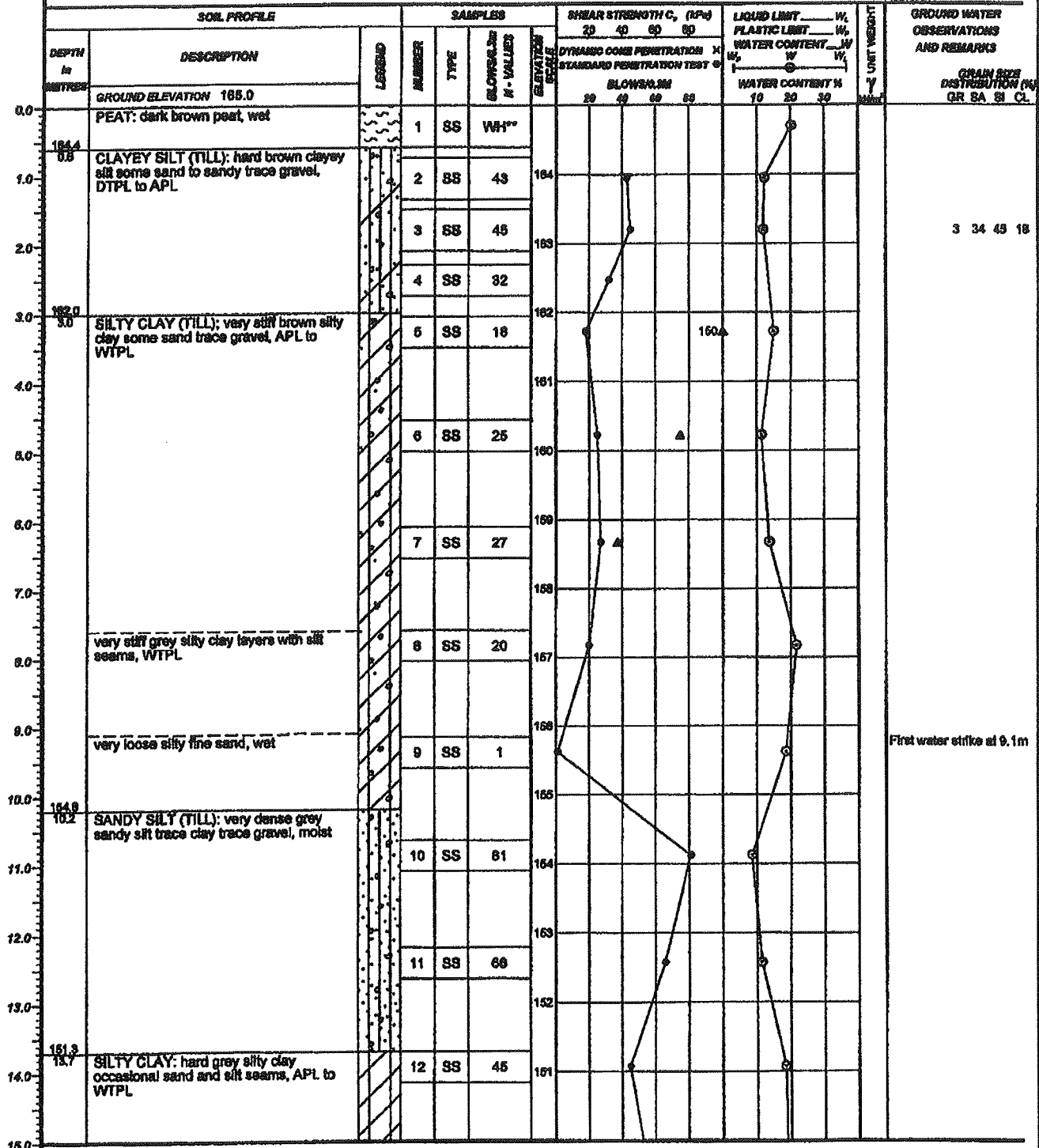
LOCATION Carlingview Drive to Mimico Creek (M13.0 to 13.8), Toronto, Ontario

ENGINEER T.X.

BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE Nov. 30 & Dec. 01, 2009

TECHNICIAN S.A.



LOG OF BOREHOLE NO. CW-6

PROJECT Go Transit - Weston Rail Corridor

Coords: 4 840 078.4 N ; 811 843.5 E

OUR PROJECT NO. 09TF014

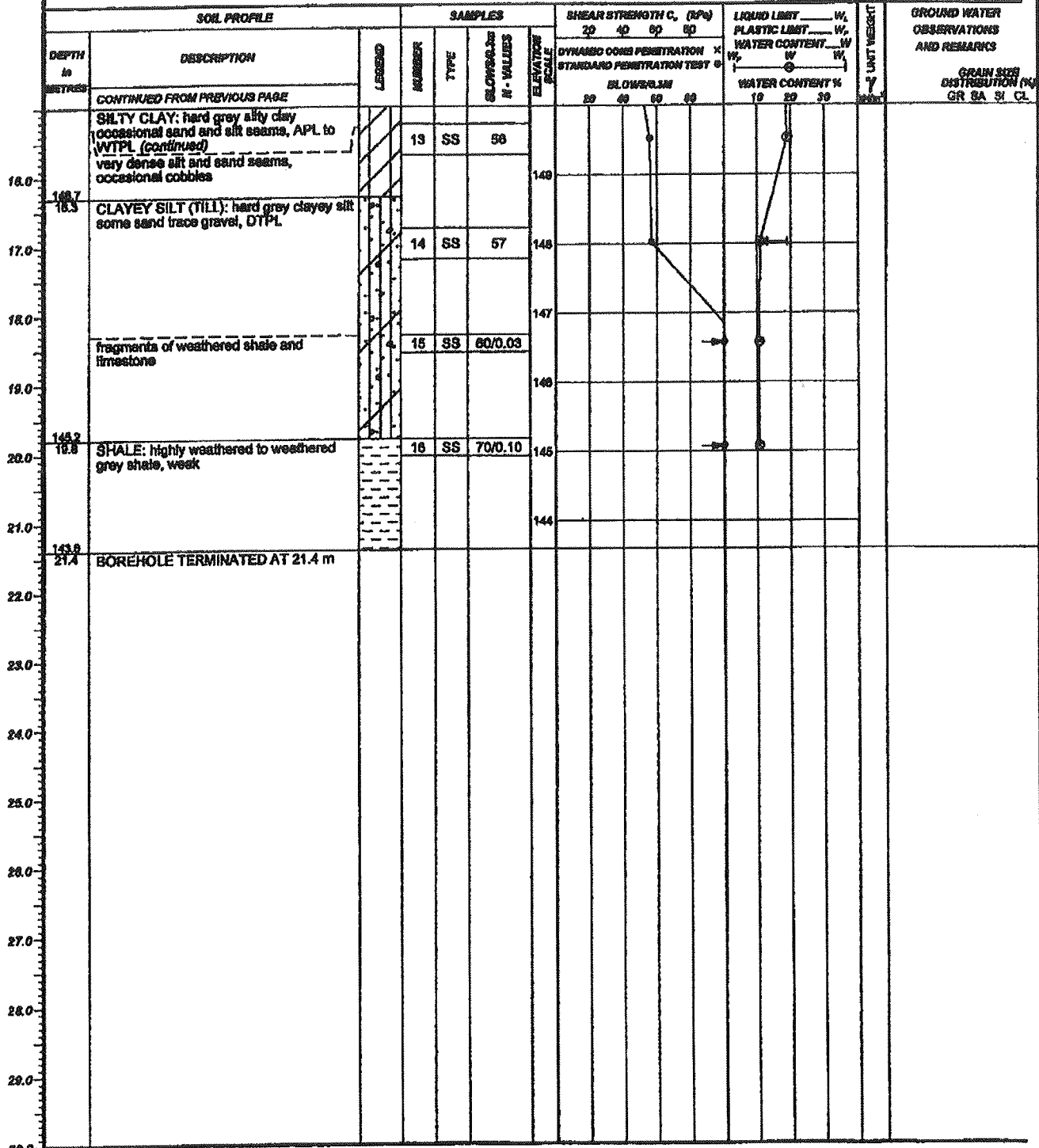
LOCATION Carlingview Drive to Mimico Creek (M13.0 to 13.8), Toronto, Ontario

ENGINEER T.X.

BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE Nov. 30 & Dec. 01, 2009

TECHNICIAN S.A.



NOTES: Auger was grinding and advancing slow from 21.5 to 21.5m depth
WTPL denotes penetration due to weight of hammer and rods.

+ UNDISTURBED FIELD VANE
⊗ REBOLDED FIELD VANE
⊙ LAB SHEAR TEST
△ POCKET PENETROMETER

CHECKED BY T.X.

LOG OF BOREHOLE NO. CW-6A

PROJECT Go Transit - Weston Rail Corridor

Coords: 4 840 083.3 N ; 811 842.3 E

OUR PROJECT NO. 09TF014

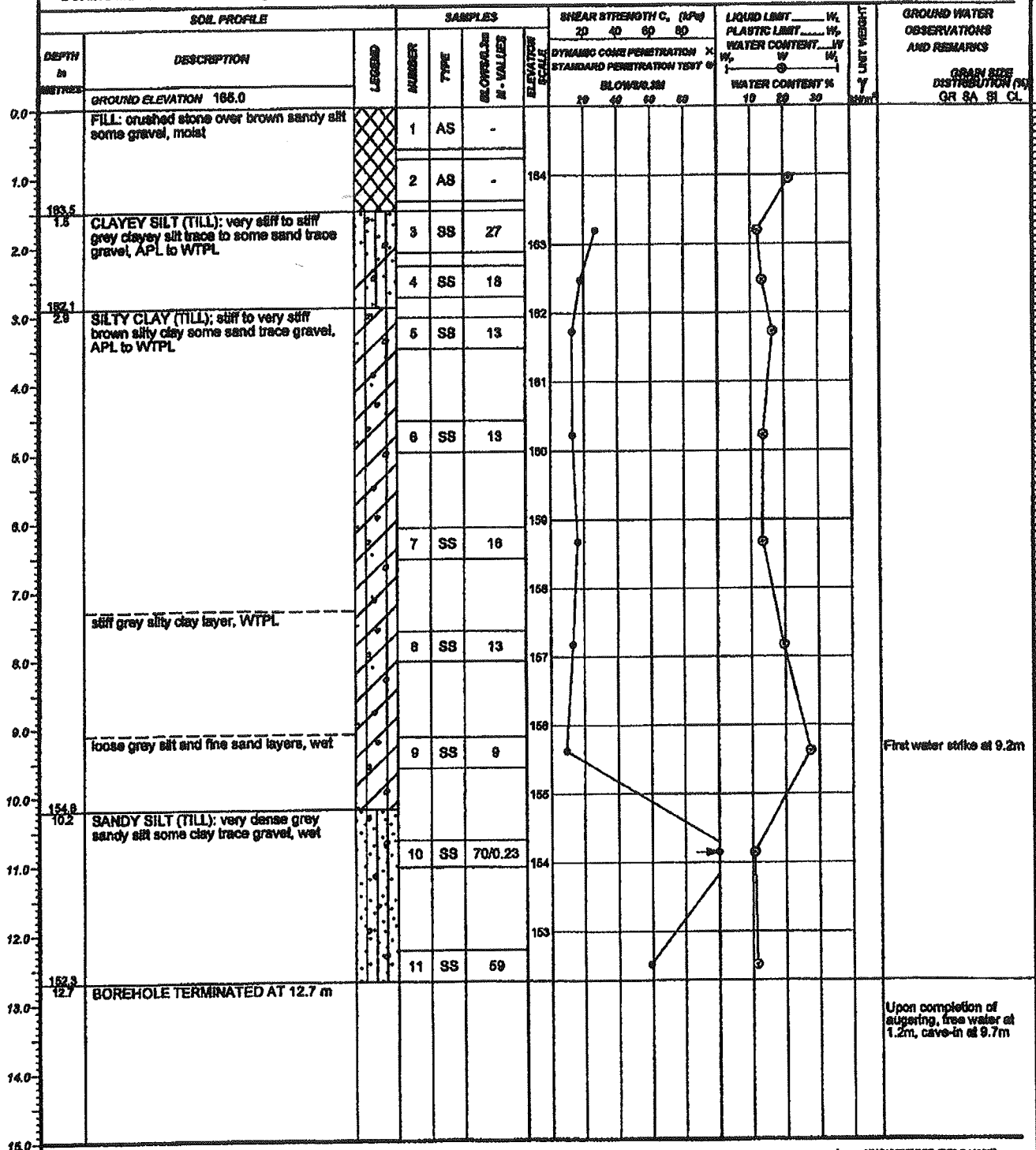
LOCATION Carlingview Drive to Mimico Creek (M13.0 to 13.8), Toronto, Ontario

ENGINEER T.X.

BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE December 15, 2009

TECHNICIAN S.A.



NOTES:

+ UNDISTURBED FIELD VANE
⊗ REMOLDED FIELD VANE
⊙ LAB SHEAR TEST
△ POCKET PENETROMETER

CHECKED BY *T.X.*

LOG OF BOREHOLE NO. CW-7

PROJECT Go Transit - Weston Rail Corridor

Coords: 4 840 093.1 N ; 811 732.2 E

OUR PROJECT NO. 09TF014

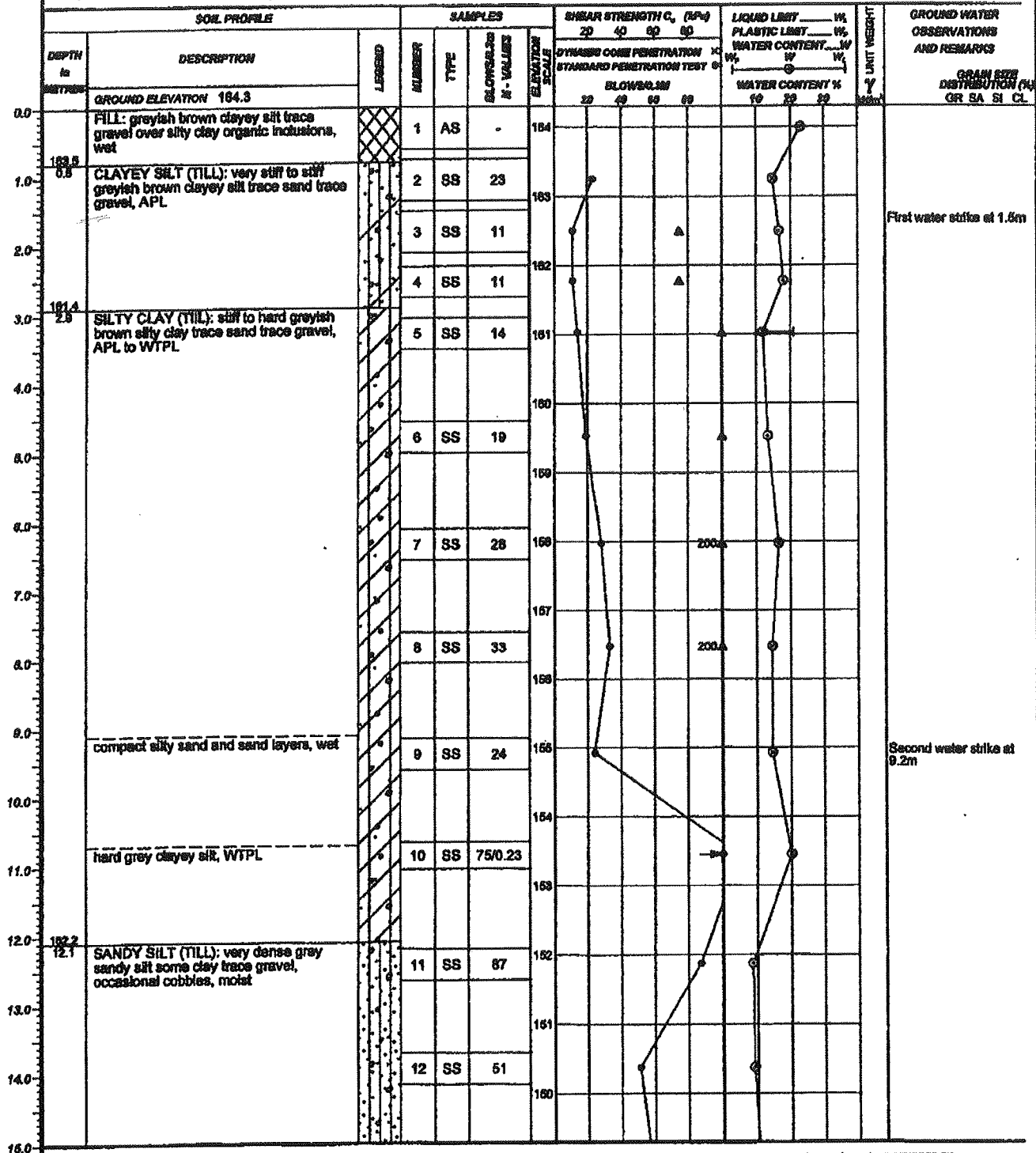
LOCATION Carlingview Drive to Mimico Creek (M13.0 to 13.8), Toronto, Ontario

ENGINEER T.X.

BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE Nov. 09-11, 2009

TECHNICIAN S.A.



NOTES: * R.Q.D. (Rock Quality Designation) is the total length of those pieces of sound core which are 10 cm or greater in length in a core run expressed as a percentage of the total length of that core run. RQD is measured on minimum HQ size (63 mm diameter) cores.

+ UNDISTURBED FIELD VANE
⊙ REMOLDED FIELD VANE
⊙ LAB SHEAR TEST
Δ POCKET PENETROMETER

CHECKED BY *TX*

LOG OF BOREHOLE NO. CW-7

PROJECT Go Transit - Weston Rail Corridor

Coords: 4 840 083.1 N ; 811 732.2 E

OUR PROJECT NO. 09TF014

LOCATION Carlingview Drive to Mimico Creek (M13.0 to 13.8), Toronto, Ontario

ENGINEER T.X.

BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE Nov. 09-11, 2009

TECHNICIAN S.A.

SOIL PROFILE			SAMPLES				SHEAR STRENGTH C, (kPa)		LIQUID LIMIT W_L		PLASTIC LIMIT W_P		WATER CONTENT W		UNIT WEIGHT γ	GROUND WATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LOG	NUMBER	TYPE	BLOW COUNTS N - VALUES	ELEVATION m	20	40	60	60	60	60	60	60		
CONTINUED FROM PREVIOUS PAGE								DYNAMIC CONE PENETRATION		STANDARD PENETRATION TEST		WATER CONTENT %				
								BLOW/CM		BLOW/CM		WATER CONTENT %				
								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
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								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
								20	40	60	60	60	60			
</																

NOTES: * R.Q.D. (Rock Quality Designation) is the total length of those pieces of sound core which are 10 cm or greater in length in a core run expressed as a percentage of the total length of that core run. R.Q.D. is measured on minimum HQ size (63 mm diameter) cores.

+ UNDISTURBED FIELD VANE
⊕ REMOLDED FIELD VANE
⊙ LAB SHEAR TEST
△ POCKET PENETROMETER

CHECKED BY *T.K.*

LOG OF BOREHOLE NO. CW-8

PROJECT Go Transit - Weston Rail Corridor

Coords: 4 840 074.2 N ; 811 742.2 E

OUR PROJECT NO. 09TF014

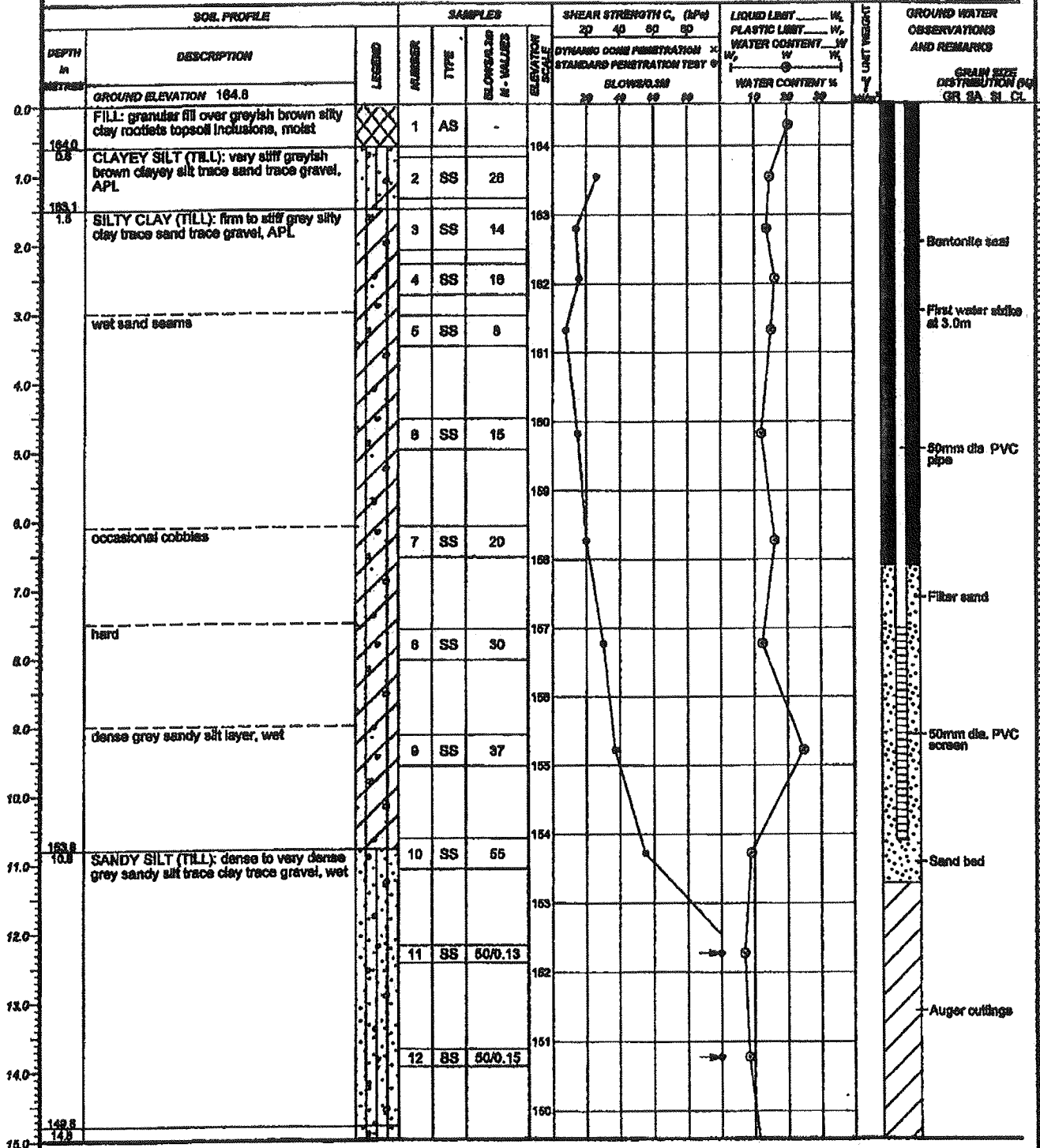
LOCATION Carlingview Drive to Mimico Creek (M13.0 to 13.8), Toronto, Ontario

ENGINEER T.X.

BORING METHOD Continuous Flight Solid Stem Augers

BORING DATE November 23, 2009

TECHNICIAN S.A.



NOTES:

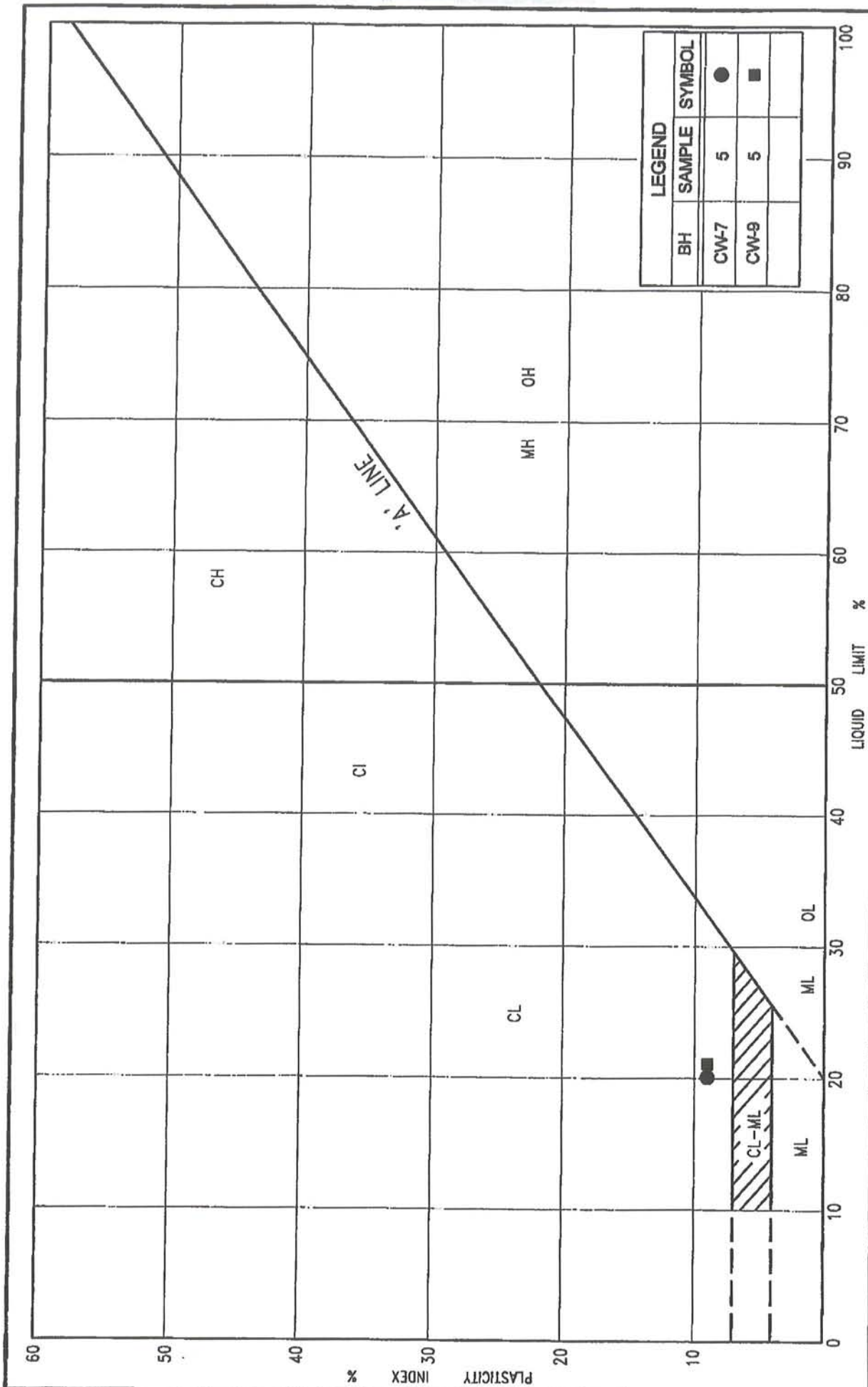
+ UNDISTURBED FIELD VANE
⊕ REMOLDED FIELD VANE
⊙ LAB SHEAR TEST
▲ POCKET PERMEAMETER
CHECKED BY *TX*



Photo 1: Rock cores retrieved from borehole CW-7, Core Runs 1 to 3 from 24.4 to 27.8 m depth. RQD of the rock cores ranges from 65 to 83%. The rock mass quality varies from fair to good.



Photo 2: Rock cores retrieved from borehole CW-9, Core Runs 1 to 4 from 27.6 to 32.3 m depth. RQD of the rock cores ranges from 0 to 99%. The rock mass quality varies from very poor to excellent.

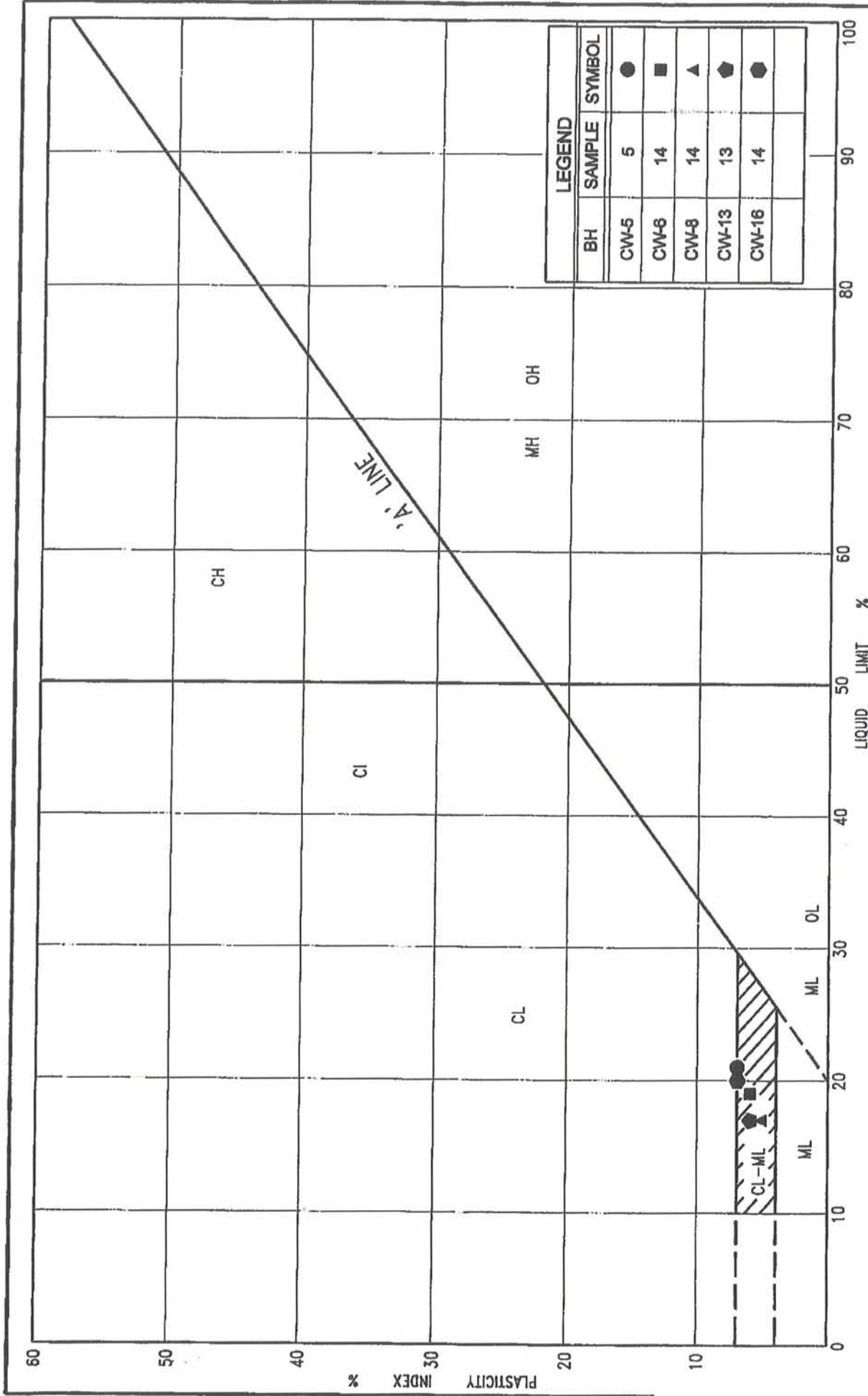


PLASTICITY CHART
SILTY CLAY (TILL):
 Silty clay trace sand, trace gravel

Figure No. CW-PC-1
 Project No. 09TF014



Paul Pato MacCallum Ltd.
 CONSULTING ENGINEERS



PLASTICITY CHART
CLAYEY SILT (TILL):
 Clayey silt, trace sand, trace gravel

Figure No. CW-PC-2
 Project No. 09TF014



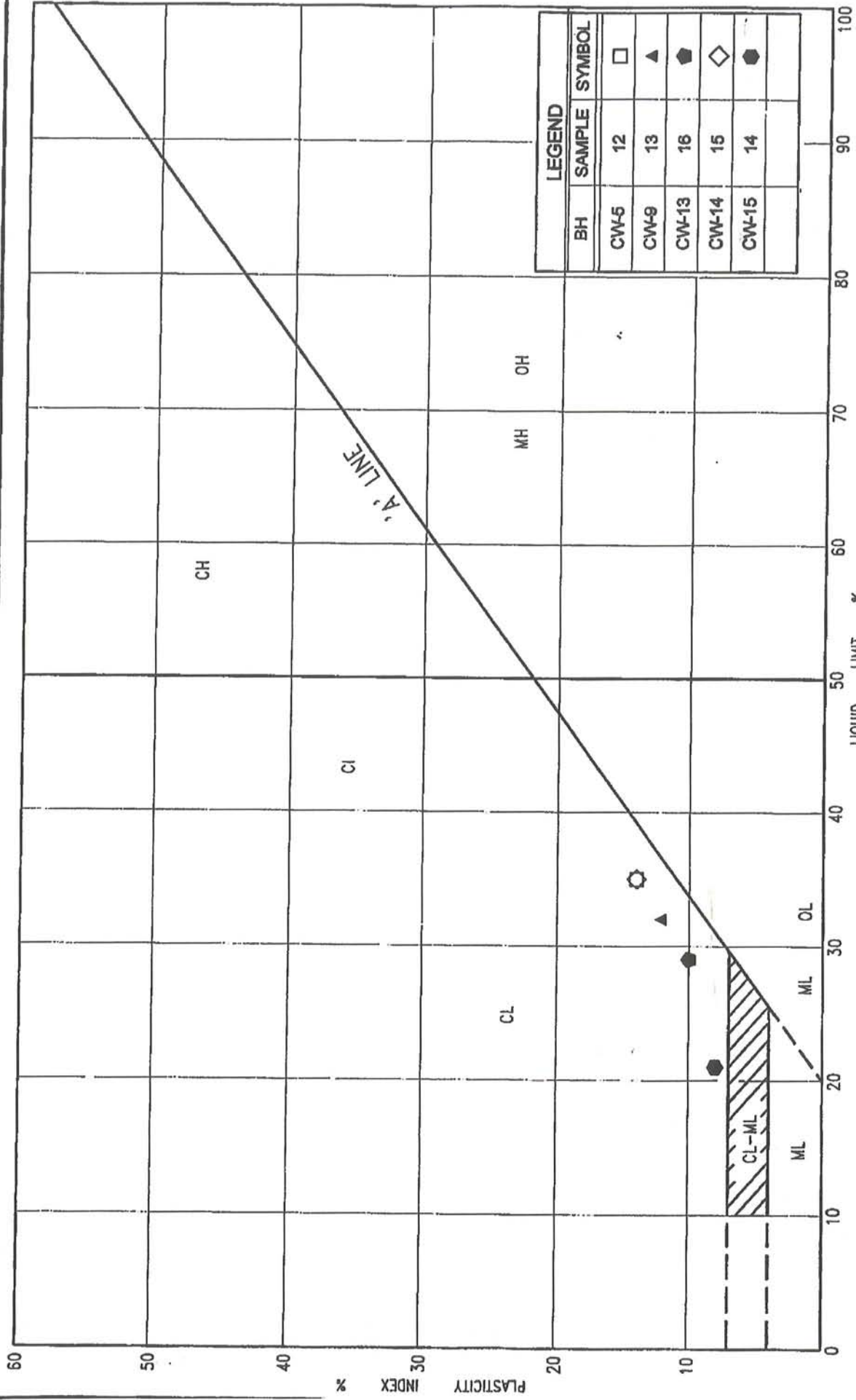


Figure No. CW-PC-3

Project No. 08TF014

PLASTICITY CHART

SILTY CLAY

Silty clay, trace sand, trace gravel

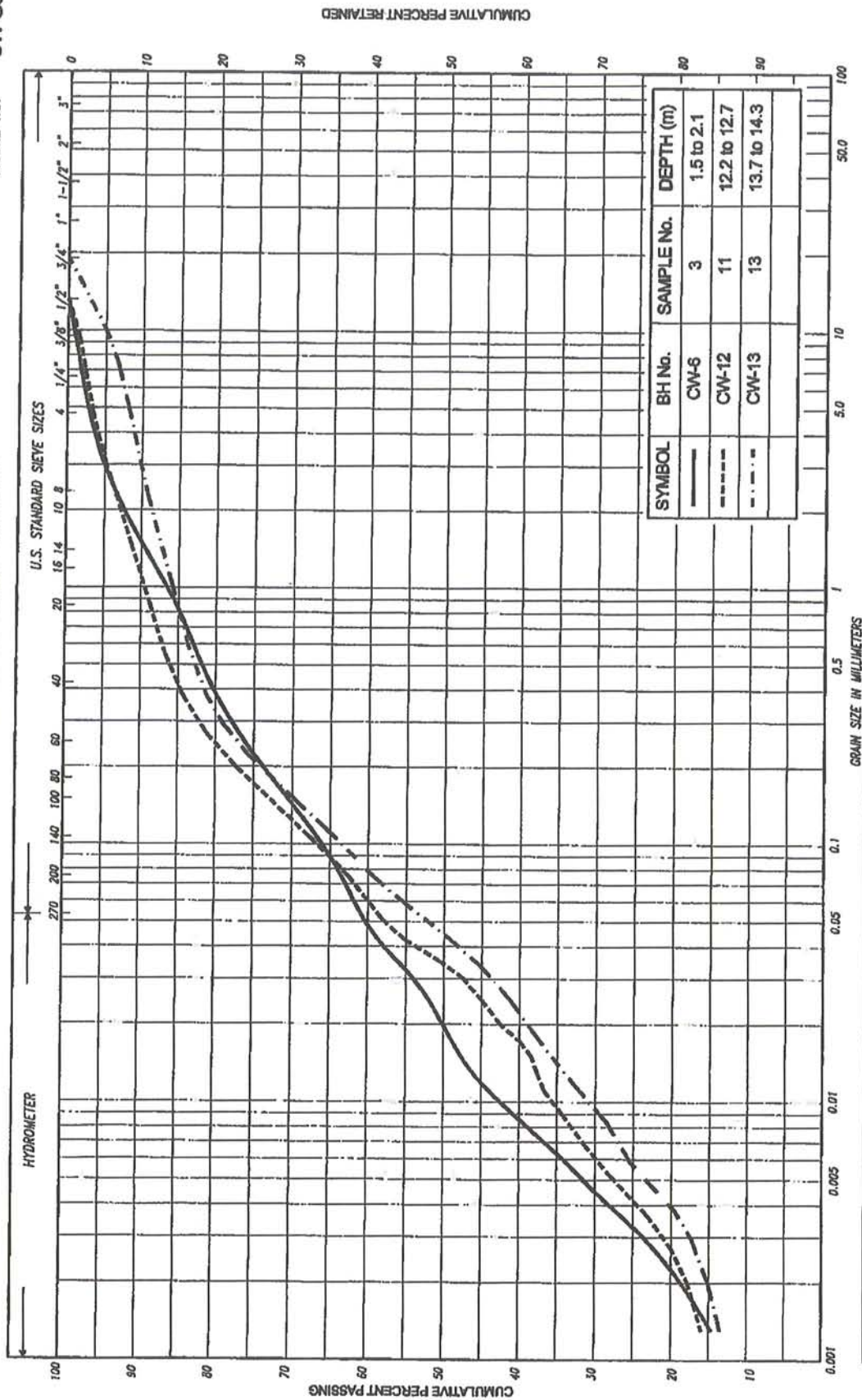


Pato MacCullum Ltd.
CONSULTING ENGINEERS



PROJECT NO. 09TF014
FIGURE NO. CW-GS-1

PARTICLE SIZE DISTRIBUTION CHART



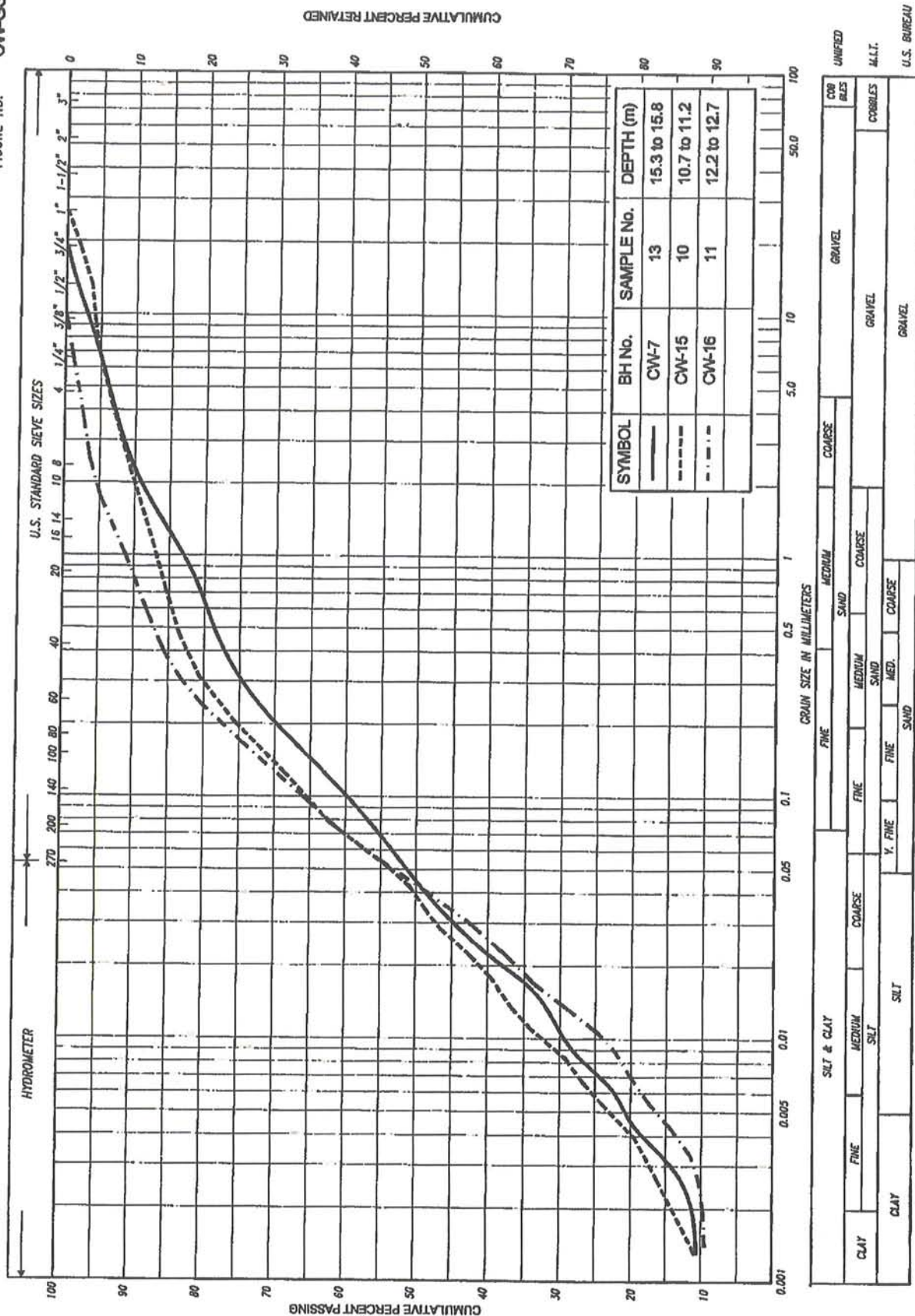
CLAY		SILT & CLAY		FINE		COARSE		MEDIUM SAND		COARSE SAND		GRAVEL		COBLES		U.S. BUREAU	
CLAY		SILT		FINE		COARSE		MEDIUM SAND		COARSE SAND		GRAVEL		COBLES		U.S. BUREAU	
CLAY		SILT		FINE		COARSE		MEDIUM SAND		COARSE SAND		GRAVEL		COBLES		U.S. BUREAU	
CLAY		SILT		FINE		COARSE		MEDIUM SAND		COARSE SAND		GRAVEL		COBLES		U.S. BUREAU	
CLAY		SILT		FINE		COARSE		MEDIUM SAND		COARSE SAND		GRAVEL		COBLES		U.S. BUREAU	

REMARKS: CLAYEY SILT (TILL); Clayey silt sandy, trace gravel



PARTICLE SIZE DISTRIBUTION CHART

PROJECT NO.	09TF014
FIGURE NO.	CW-GS-3

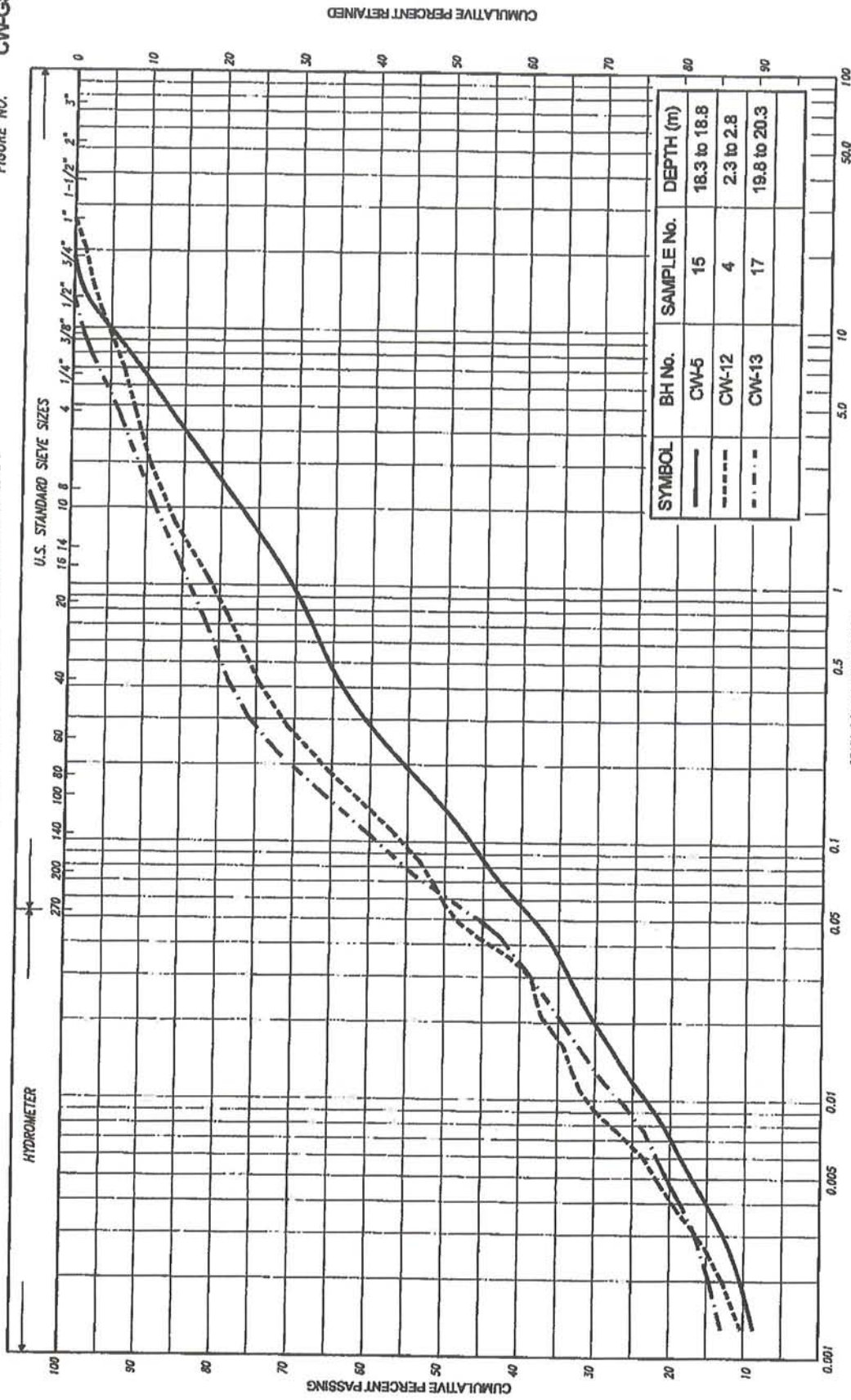


SANDY SILT (TILL): Sandy silt trace to some clay, trace gravel



PROJECT NO. 09TF014
FIGURE NO. CW-GS-4

PARTICLE SIZE DISTRIBUTION CHART

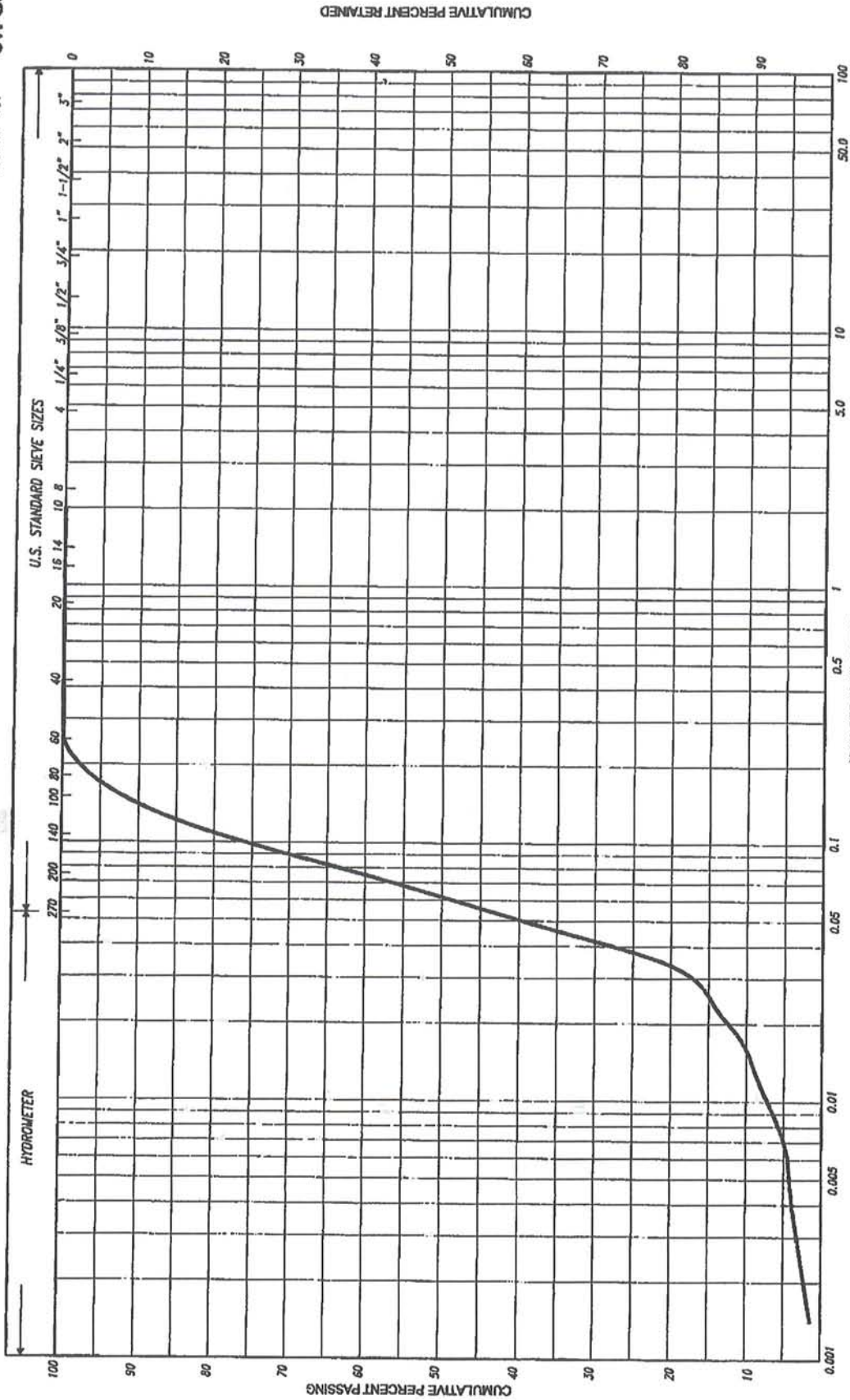


GRAIN SIZE IN MILLIMETERS												UNIFIED	C.B. & M.C.S.				
CLAY		FINE		MEDIUM SILT		COARSE		FINE		MEDIUM SAND				COARSE		GRAVEL	
CLAY		FINE		MEDIUM SILT		COARSE		FINE		MEDIUM SAND		COARSE		GRAVEL		M.I.T.	U.S. BUREAU
CLAY		FINE		MEDIUM SILT		COARSE		FINE		MEDIUM SAND		COARSE		GRAVEL			



PROJECT NO. 09TF014
FIGURE NO. CW-GS-6

PARTICLE SIZE DISTRIBUTION CHART



SILT & CLAY				GRAVEL				COBBLES		UNITED STATES BUREAU OF MINES	
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL	COBBLES	M.I.T.	U.S. BUREAU	

REMARKS: SAND AND SILT: Sand and silt, trace clay
Borehole CW-5, Sample 9, Depth: 9.1 to 9.5m

QUICK UNDRAINED TRIAXIAL COMPRESSION TEST

CLIENT:
PROJECT:

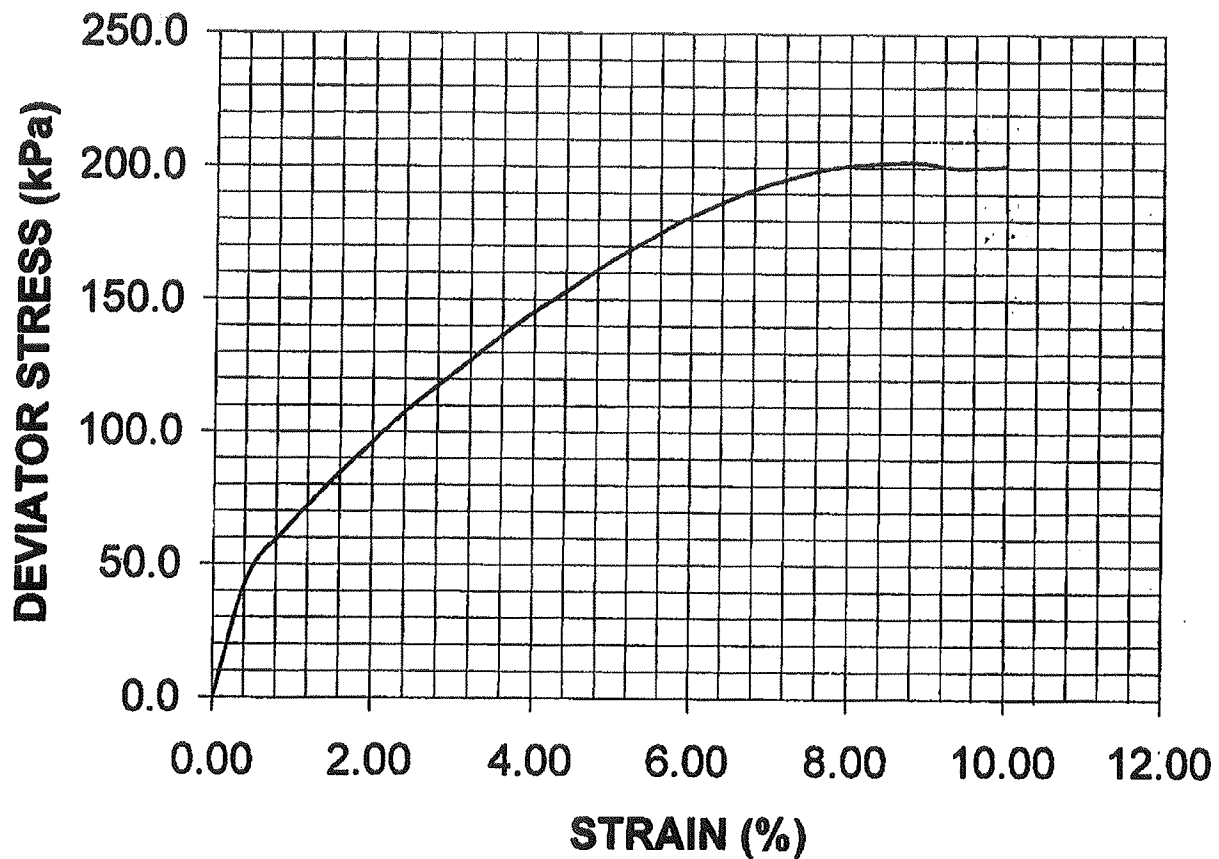
GO TRANSIT
Go Weston Rail Corridor Expansion

OUR REF.: 09TF014
DATE: 2010-02-04

SAMPLE IDENTIFICATION:

BH: CW-5 SA: 5

DEPTH:



Peto MacCallum Ltd
CONSULTING ENGINEERS
ROCK CORE TESTING

CLIENT Metrolinks
PROJECT Geotechnical Investigations Contract ITC 2009-GT-008
SAMPLE IDENTIFICATION CW7, 87'7" - 88'1"

PML REF 09TF014
LAB NO. 37975 A
DATE SAMPLED

DATE TESTED
TESTED BY

2009-11-17
BM

UNCONFINED COMPRESSIVE STRENGTH

CORE DIMENSIONS		COMPRESSIVE STRENGTH	
SPECIMEN DIAMETER (in.)	1.8640	TEST TIME (min) (spec. 2 to 15)	2:30
SPECIMEN LENGTH (in.)	5.378	MAXIMUM LOAD APPLIED (kN)	25.58
	5.374		
	5.381	COMPRESSIVE STRENGTH (MPa)	14.5
	AVE. 5.378	TYPE OF FAILURE	B
SURFACE AREA (sq mm)	1761	LENGTH TO DIAMETER RATIO (spec 2-2.5)	2.89

MOISTURE CONTENT

UNIT WEIGHT

WEIGHT OF WET SAMPLE + TARE (g)	694.90	WEIGHT OF DRY SAMPLE IN AIR (g)	622.95
WEIGHT OF DRY SAMPLE + TARE (g)	674.20	VOLUME OF SAMPLE (cu m)	0.000240
WEIGHT OF WATER (g)	20.70	UNIT WEIGHT (kg/cu m)	2590
WEIGHT OF TARE (g)	72.91		
WEIGHT OF DRY SAMPLE (g)	601.29		
MOISTURE CONTENT (%)	3.4		

REMARKS

Peto MacCallum Ltd

CONSULTING ENGINEERS

ROCK CORE DIMENSIONS

CLIENT: Metrolinks
 PROJECT: Geotechnical Investigations Contract ITC 2009-GT-008
 SAMPLE IDENTIFICATION: CW7, 87'7" - 88'1"

PML REF
 LAB NO.
 DATE SAMPLED
 DATE TESTED
 TESTED BY

09TF014
 37975-A
 2009-11-17
 BM

DEVIATION FROM STRAIGHTNESS

DIAL READING (IN)	TRIAL		
	1	2	3
MINIMUM	0.0850	0.0870	0.0860
MAXIMUM	0.0930	0.0960	0.0940
DIFFERENCE	0.0080	0.0090	0.0080
MAX DIFF.	0.009	SPEC.	0.020 max.

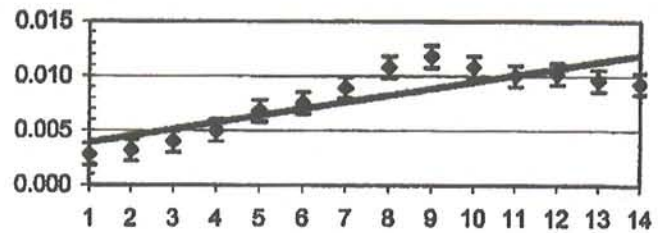
FLATNESS TOLERANCE

DIAL READING (IN)	END 1		END 2	
	SET 1	SET 2	SET 1	SET 2
RDG 1	0.0028	0.0082	0.0138	0.0020
RDG 2	0.0032	0.0080	0.0130	0.0024
RDG 3	0.0040	0.0076	0.0125	0.0038
RDG 4	0.0050	0.0072	0.0117	0.0040
RDG 5	0.0068	0.0065	0.0109	0.0051
RDG 6	0.0075	0.0061	0.0102	0.0061
RDG 7	0.0089	0.0058	0.0098	0.0068
RDG 8	0.0108	0.0053	0.0091	0.0073
RDG 9	0.0118	0.0052	0.0084	0.0079
RDG 10	0.0108	0.0051	0.0077	0.0085
RDG 11	0.0100	0.0051	0.0081	0.0089
RDG 12	0.0102	0.0048	0.0064	0.0093
RDG 13	0.0096	0.0042	0.0058	0.0097
RDG 14	0.0093	0.0037	0.0053	0.0109
RDG 15				
RDG 16				
RDG 17				
RDG 18				
RDG 19				
RDG 20				

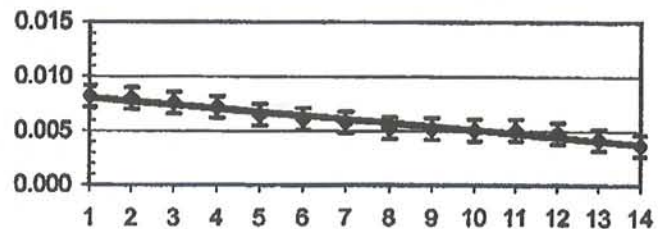
FLATNESS TOLERANCE= .001 in.

CORE DIAMETER (in.)	1.8670	1.8630	1.8620
AVE:	1.8640	PERPENDICULARITY RATIO (Specified .0043 max.)	
SLOPE OF BEST FIT LINE			
	MINIMUM	MAXIMUM	
END 1A	0.0040	0.0120	0.0043
END 2B	0.0040	0.0080	0.0021
END 2A	0.0050	0.0140	0.0048
END 2B	0.0020	0.0110	0.0048

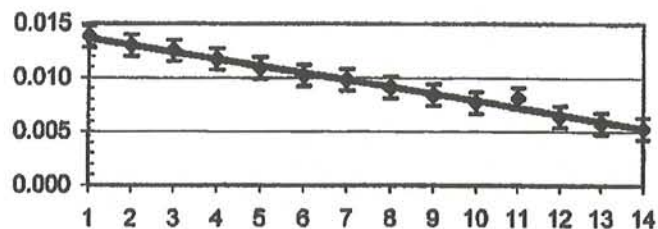
END 1a



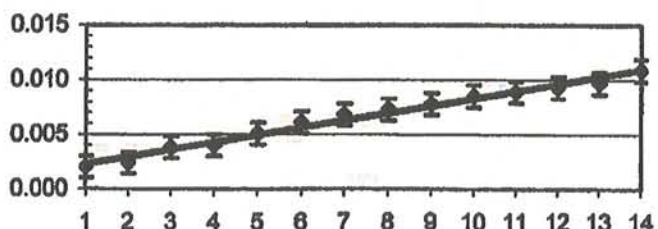
END 1b



END 2a



END 2b



APPENDIX D

General Arrangement Drawing

Figure 1 – Design Soil Profile

Figure 2 – Axial Capacity of Proposed Concrete-Encased Pile

Figure 3 – Lateral Deflection of Concrete-Encased Pile

Figure 4 – P-Y Curves for Proposed Concrete-Encased Piles

ALL DIMENSIONS SHOWN ARE
IN METRES AND/OR MILLIMETRES
UNLESS OTHERWISE NOTED.

IT IS PROPOSED TO CONSTRUCT A RETAINING WALL BENEATH THE EXISTING BRIDGES TO ACCOMMODATE TWO ADDITIONAL RAILWAY TRACKS.

EXISTING TRACKS ARE UNCHANGED.

ALL DIMENSIONS TO BE VERIFIED IN THE FIELD BY SURVEY TO
CONFIRM THE LOCATION OF THE EXISTING STRUCTURES.

NEAREST STATION: KIPLING, Mi 11.07

SPECIFICATIONS: CANADIAN HIGHWAY BRIDGE DESIGN
CODE CAN/CSA S6-06

LATERAL SOIL PRESSURE: $K=0.5$

BACKFILL UNIT WEIGHT: 21 kN/m³

CONCRETE: ACI 318 OR CSA A23.1 AND A23.2-2005
MINIMUM COMPRESSIVE STRENGTH SHALL
BE 30 MPa AT 28 DAYS.
ALL EXPOSED CONCRETE EDGES SHALL
BE GIVEN A 20mm x 20mm CHAMFER.

REINFORCING STEEL: CSA G30.18-09, GRADE 400 MPa.
MINIMUM CONCRETE COVER TO
REINFORCING STEEL SHALL BE
50mm UNLESS OTHERWISE NOTED.

STRUCTURAL STEEL: CSA G40.21-04 (R2009), GRADE 350W

WELDING: CSA W59-03 (R2008)

GALVANIZING: CAN/CSA-G164-M92 (R2003)

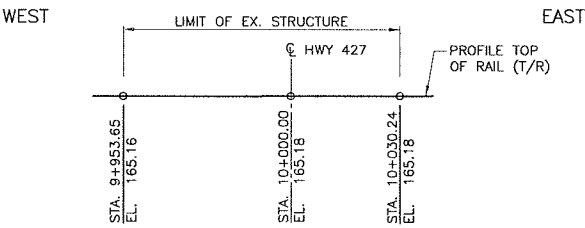
TECHNICAL SURVEY PERFORMED BY STANTEC, DATED APRIL 01, 2011

GEOTECHNICAL RECOMMENDATIONS PROVIDED BY STANTEC

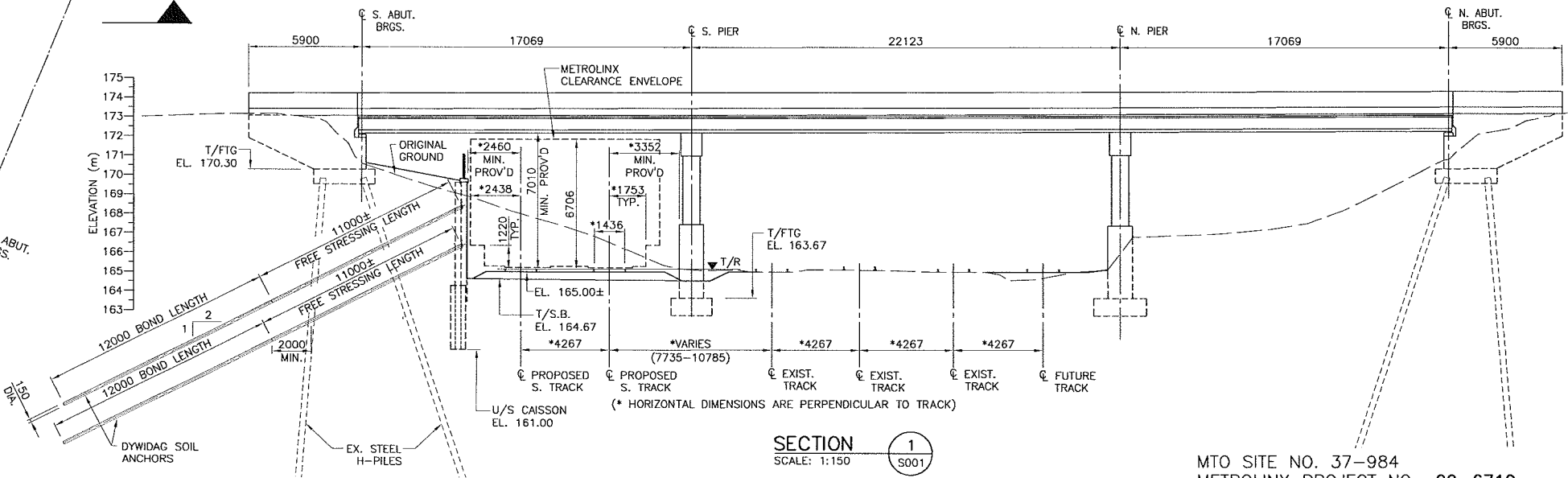
REFERENCE DWGS:

CNR OVERHEAD AT HIGHWAY 427, MTO CONTRACT 78-111
HIGHWAY 427 WIDENING, MTO CONTRACT 96-12
BRIDGE No. 706, GTAA CONTRACT TP-H03-002

T/R - DENOTES TOP OF RAIL
T/FTG - DENOTES TOP FOOTING
T/S.B. - DENOTES TOP OF SUB BALLAST
O.G. - DENOTES ORIGINAL GROUND
U/S - DENOTES UNDERSIDE



PROFILE OF METROLINX
EX. SOUTH TRACK
N.T.S.



SECTION 1
SCALE: 1:150 S001

REFERENCE DRAWINGS		ISSUE			REVISIONS		
DWG NO.	TITLE	NO.	DATE	ISSUED FOR	REV.	DATE	

DRAWN BY: B.H. 11/03/18	DESIGNED BY: M.T. YY/MM/DD
CHECKED BY: M.D. YY/MM/DD	APPROVED BY: YY/MM/DD
SCALE: AS SHOWN FULL SIZE ONLY	



95% SUBMISSION
DATE: 30 MAY 2011



A Division of Metrolinx

GEORGETOWN SOUTH RETAINING WALL

HIGHWAY 427 OVERHEAD - WESTON Mi 13.50
GENERAL ARRANGEMENT

CONTRACT NO. 2010-CIG-065	DWG. NO. S--001	REV. 0	SHEET 5/9
------------------------------	--------------------	-----------	--------------

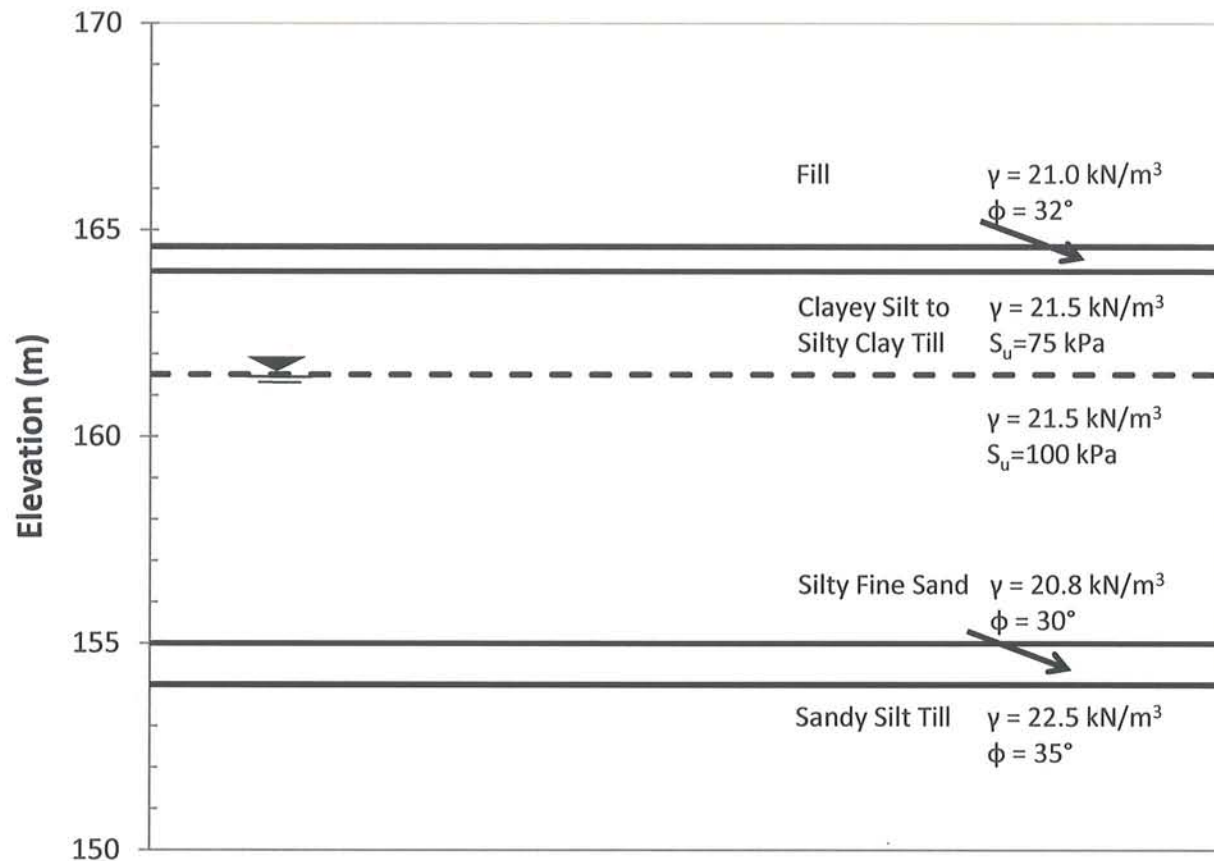
\\01135\active\13538134\design\drawing\hwy_427\contract_dwgs\13538134--hwy_427--motorize_overhead--5-001.dwg



Stantec

Project No. 113536134
Metrolinx - Georgetown Corridor Bridges
Hwy 427

Geotechnical Model



Notation :

ϕ' = effective angle of internal friction

γ = total unit weight

S_u = undrained shear strength

Figure 1
Design Soil Profile



Stantec

Project No. 113536134
Metrolinx - Georgetown Corridor Bridges
Hwy 427

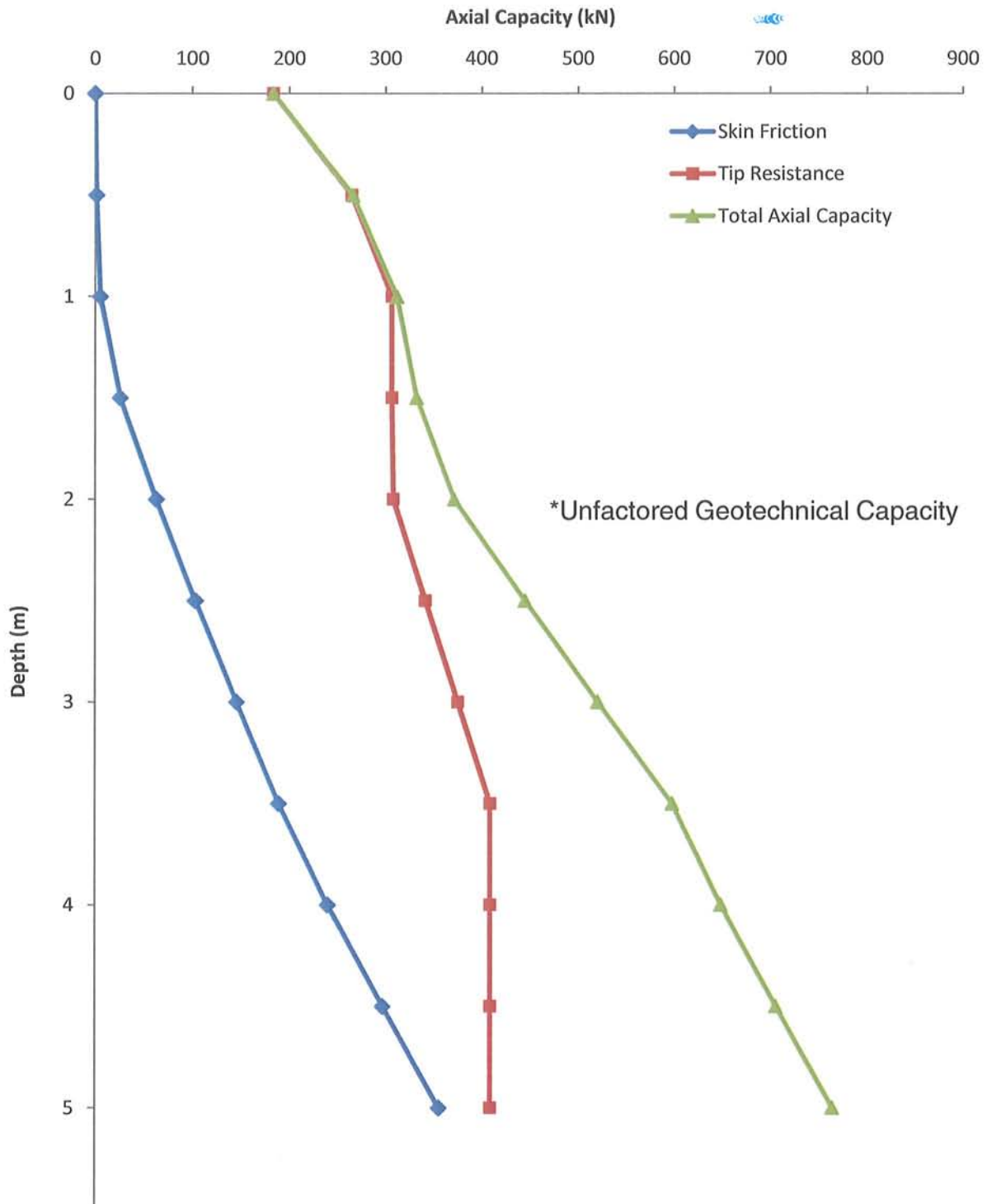


Figure 2
Axial Capacity of Proposed Concrete-Encased Piles

L-Pile Results - Lateral Deflection

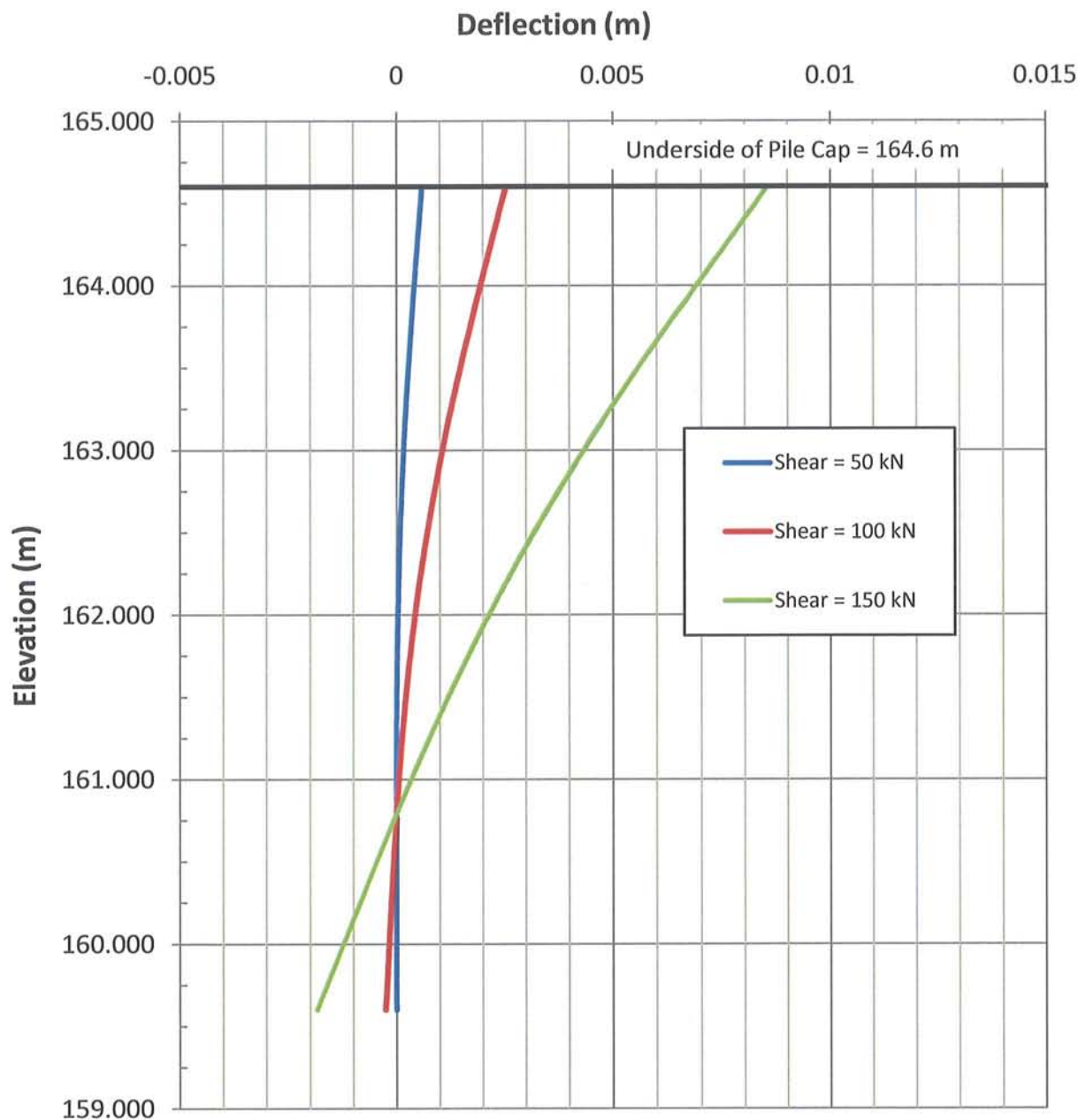


Figure 3
Lateral Deflection of Concrete-Encased Piles



Stantec

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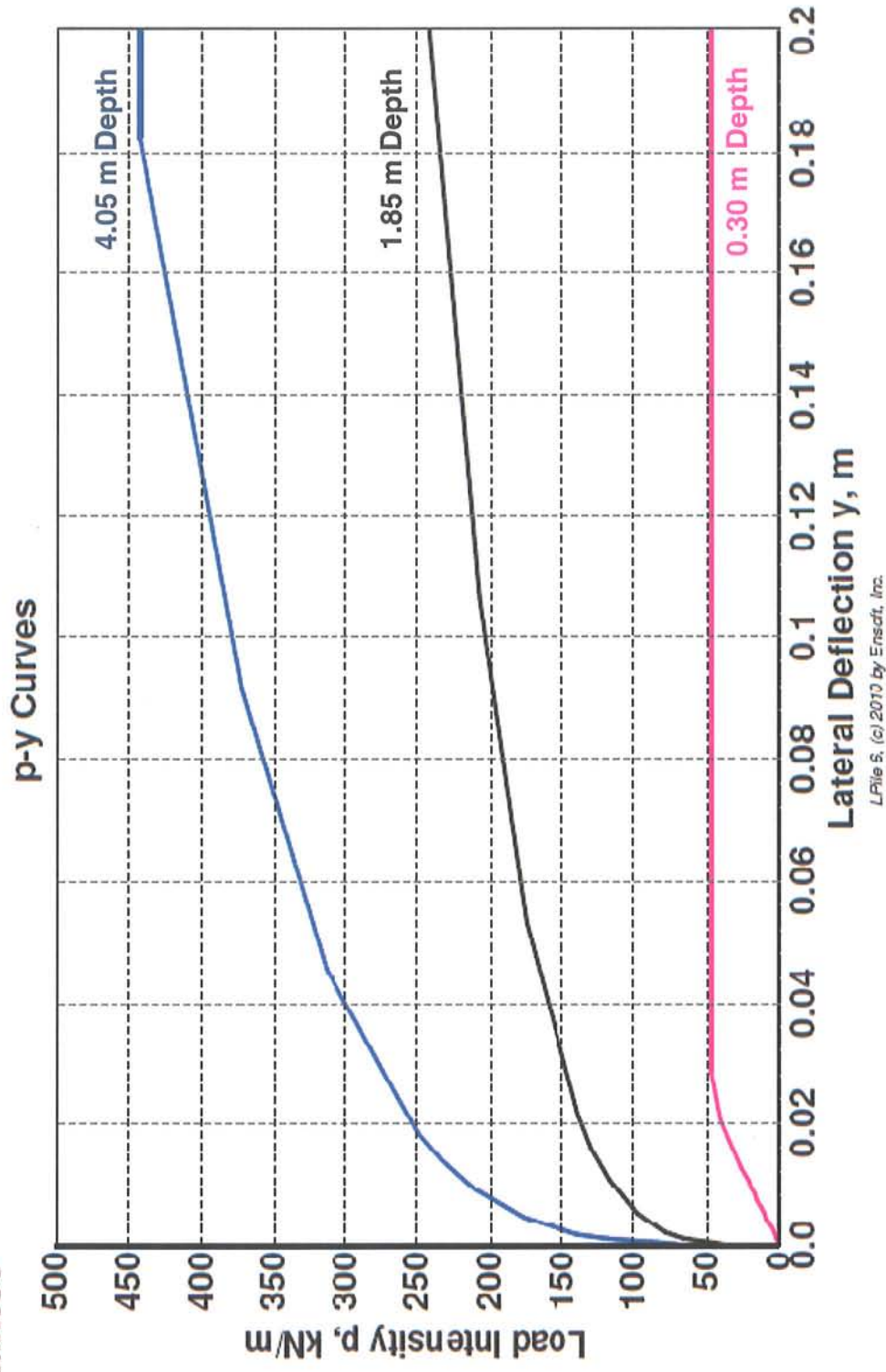


Figure 4
p-y Curves for Proposed Concrete-Encased Piles