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The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report – Culvert CV-3

(Cahill Drain, 9+963.74 Cousineau Road, LaSalle)

Revision History					
Revision	**Date**	**Status**	**Prepared By**	**Checked By**	**Reviewed By**
0	03/02/2012	Issued for Construction – Final	TL	DD	NSV

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Mar 22, 2012
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PROJECT DOCUMENT AND DATA MANAGEMENT

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1 Introduction

1.1 Preface

The Windsor Essex Parkway (the Parkway) was conceived to strengthen transportation and trade links between Canada and the United States, reduce road congestion, and foster economic growth. The Parkway will connect Highway 401 to a new Canadian inspection plaza and a new international crossing over the Detroit River to Interstate 75 in Michigan, USA. It will be a six-lane highway, 11 kilometres long with 15 bridges, 11 tunnels and a four-lane service road that will provide full access to schools, neighbourhoods, natural areas, and shopping. Other components of the project include community and environmental features, such as: 300+ acres of green space, 20 kilometres of recreational trails, extensive landscaping throughout the corridor, as well as noise and environmental mitigation measures. The environmental mitigation measures were based upon Permit AY-D-001-09 which was approved in February 2010.

The Parkway's strategic international importance, urban location, and unique ecological context necessitate strong design and planning principles to guide infrastructure development. The Parkway is to be a state-of-the-art facility within a contextually sensitive landscape setting that has ecological integrity, builds physical and cultural connections, and establishes a sustainable network of amenities that can be enjoyed by present and future generations.

The plans for the Parkway strive to build and strengthen linkages within and between both human and ecological communities. Over time, restored green space will evolve into a tall grass prairie and oak savannah landscape that will, through ecological succession, allow the roadway to become a 'Parkway in a Prairie'. All of the green space areas of the Parkway, (whether associated with the Roadway, the Stormwater Management Areas, the Ecological Landscape areas, or the Screening), are ecologically based areas that in their totality will represent an extensive habitat network consisting of existing, new and rehabilitated terrestrial and aquatic communities.

Natural and cultural history are proposed to be celebrated in the artful design of three Gateways, and eleven Land Bridges that support the existing municipal road system and the inter-connected multi-use pathway system. The Gateways are conceived as bold and commanding landscapes that draw on sculpted landform, strong patterning, and public art to create strong visual elements for the driving experience within themes of 'Arrival, Settlement, and Flow'.

The Land Bridges draw on natural and cultural influences to create distinct and memorable places that serve as markers, urban respite areas, and focal points to the overall green space system. Other opportunities for artistic expression include the streetscapes and urban amenity areas, trail bridges; tunnel abutments, and noise walls. These structural elements offer opportunities for simple expression of the surrounding natural environment, area history and the 'prairie' landscape in particular, through color, form, materials, and the integration of public art.

The lasting legacy of the Windsor Essex Parkway project will not only be its significant contribution as an international trade and transportation route, but rather includes the establishment of a contiguous and sustainable green space system that contributes to the quality of life in the community and supports the re-establishment of an ecologically rich Carolinian landscape.

On December 17, 2010 Infrastructure Ontario and Ministry of Transportation of Ontario (MTO) announced that the Windsor Essex Mobility Group (WEMG) reached financial close and signed a fixed-price contract with the Province to design, build, finance, and maintain the Windsor-Essex Parkway. To build the initial works, WEMG has formed a Design-Build Joint Venture – Parkway Infrastructure Constructors (PIC). This team includes Dragados Canada, Inc., Acciona Infrastructure Canada Inc., and Fluor Canada Ltd. This combination brings a wide range of local and international experience to the project.

1.2 Report Introduction

This report presents the geotechnical design of Culvert CV-3, located below Cousineau Road at Sta. 9+964, near Sta. 12+200L (Highway 401) in LaSalle sector of the proposed Windsor-Essex Parkway (WEP) project. The report includes the results of the additional geotechnical investigation carried out to support the design and other relevant background information.

The 11.2 km long proposed WEP will run generally east-west and connect the existing Highway 401 in Tecumseh to the proposed new international crossing bridge across Detroit River (near Zug Island). It will run successively along segments of Highway 3 and Huron Church Road and then adjacent to the E.C. Row Expressway to its intersection with Ojibway Parkway. It will be constructed mostly within a cut section until the intersection of Huron Church Road and E.C. Row Expressway, beyond which it will be mostly on embankments. The proposed WEP includes 15 bridges (Bridges B-1 to B-15), 11 tunnels (T-1 to T-11), 9 trail bridges, approximately 5.5 km length of retaining walls, 2 submerged culverts, 5 box culverts, and other structures.

The proposed one-span concrete box structure of the culvert will pass underneath Cousineau Road and will be used to carry the realigned Cahill Drain as shown on Drawing 285380-03-060-WIP1-5301.

The design presented in this report was generally advanced from the preliminary geotechnical design developed for the WEMG proposal in June 2010 (ref. R-26)[1]. The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines as well as the Parkway Infrastructure Constructors (PIC).

The report is organized in two parts. Part 1 is the factual information and is presented in Sections 1 to 4. Part 2 presents the geotechnical design and recommendations in Sections 5 and 6. Other information is presented in Sections 7 to 9.

The design complies with the requirements of the execution version of the Project Agreement (PA), Schedule 15-2 Part 2 Article 5.

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[1] References are listed in Section 9.

2 Background Information

2.1 Geological Setting

The WEP project site is located within the Essex Clay Plain (a part of the St. Clair Clay Plain physiographic region) (ref. R-7, R-9 and R-15). The Essex Clay Plain was deposited during the retreat of the late Pleistocene Era ice sheets, when a series of glacial lakes inundated the area. The ice sheets generally deposited materials with a glacial till like gradation in the Windsor area. Depending on the locations of the glacial ice sheets and depths of water in the ice-contact glacial lakes, the materials may have been directly deposited at the contact between the ice sheet and bedrock or, as the lake levels rose and the ice sheets retreated and floated, the soil and rock debris within and at the base of ice may have been deposited through the lake water (i.e., lacustrine environment). It is considered that unlike typical till deposits (that have undergone consolidation and densification under the weight of the ice sheet), the majority of the “glacial till” soils in the Windsor and Detroit area were deposited through water and have a soft to firm consistency below a surficial crust layer that has become stiff to hard due to weathering and desiccation. Geologically, the deposit in the project area is considered to be slightly over-consolidated, having experienced no major overburden stresses in excess of the existing stresses.

The overburden in the St. Clair Clay Plain has variously been described as clayey silt till, silty clay till and glaciolacustrine clay. Hudec (ref. R-15) summarized the overburden geology in Windsor as consisting of the following successive strata: desiccated lacustrine clay, normally consolidated lacustrine clay, silty Tavistock till, glaciolacustrine clay and coarse Catfish Creek till. A distinct change in overburden deposits occurs in the east-west direction along a boundary located generally along the Huron-Church Road. Whereas, the eastern part of Windsor is underlain by firm to stiff glaciolacustrine silts and clays with upper deposits of stiff sandy to silty weathered clay and hard to stiff lacustrine clay-silt crust, the western part of Windsor is characterized by a thin surficial granular deposit underlain by thin crust layer underlain by soft to firm glaciolacustrine silts and clays.

At the WEP project area, the glacial till like deposit is typically 20 to 35 m thick and consists primarily of silty clay and clayey silt gradation with a random distribution of coarser particles. Random and apparently discontinuous seams / lenses of silt, sand and or gravel are present at various depths within the mass of the silty clay deposit. A stiff to hard surficial crust layer has formed due to weather and desiccation. Up to 2 m thick surficial layers of lacustrine silty clay or silt and sand are also encountered in the western sector of the project. A 1 m to 6 m thick very dense or hard basal glacial till or dense silty sand may be found directly overlying the bedrock surface. The bedrock at the project area comprises the Devonian Dundee Formation of the Hamilton Group and the underlying Devonian Lucas Formation of the Detroit River Group.

The Windsor area, referred to as the Essex Domain (with respect to bedrock geology), is located in the Grenville Front Tectonic Zone (GFTZ). The bedrock geology within the Essex Domain was formed as part of the midcontinent rift south-eastern extension. The midcontinent rift south-eastern extension is composed of Paleozoic cover rocks which form the bedrock foundation of the Essex Domain. The bedrock was deposited in the Paleozoic Era during the Middle Devonian period. Within the Essex Domain the following strata were deposited: the Hamilton Group, Dundee Formation, and Detroit River Group Onondaga Formation all consisting of Limestone, Dolostone, and Shale.

2.2 Site Seismic Background

Windsor-Tecumseh area is described in Canadian Highway Bridge Design Code (CHBDC) by a seismic hazard associated to a Velocity Zone Zv = 0 and Acceleration seismic zone Za = 0. Zonal Velocity ratio, V, and Zonal Acceleration ratio, A, are both 0.

In accordance with the CHBDC, and based on a series of cross-hole tests completed during the background investigation program (ref. R-12), the soil profile at the site of the project generally meets the description for Soil Profile Type III (soft clay and silts greater than 12 m in depth). The above noted cross-hole tests were completed during the background investigation program at locations distributed along the project alignment between Howard Avenue (east end) and Matchette Road (west end). The measured velocities of the shear waves were consistently over 200 m/s, with the bulk of results ranging between 200 and 300 m/s.

2.3 Site Conditions

Culvert CV-3 site is situated near the middle part of the LaSalle segment of the Parkway, just north of Tunnel T-9. The structures at this site will be constructed under Phase II of WEP mostly within the realigned Cahill Drain. As shown on Drawing 285380-03-060-WIP1-5301, the culvert will pass underneath Cousineau Road and will be used to carry the realigned Cahill Drain.

The topography of the lands immediately adjacent the Culvert CV-3 is essentially flat with ground surface elevations ranging from 184m to 185m. Adjacent land use is typically urban residential, parkland and light commercial.

2.4 Frost depth

In accordance with MTO–SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-23) and OPSD 3090.101 the frost depth below the ground surface in Windsor area is estimated at 1.0 m. This estimate is considered applicable to natural soils and / or conventional pavement materials where the ground surface is usually cleaned from the snow cover. Considering the variability of the near surface materials and of the degree of exposure to elements, geotechnical recommendations for the frost depth penetration in the Windsor region vary between 1.0 m and 1.2 m. A frost penetration depth of 1.0 m may be utilized for design purposes.

In the case of rip/rap, or otherwise coarse rockfill cover, the insulation effects of such materials are considered to be one half of the insulation offered by soil deposits /cover, and the depth of frost penetration will have to be increased proportionally.

3 Geotechnical Investigation

3.1 Scope and Procedures of Geotechnical Investigations

Geotechnical investigations involving boreholes, cone penetration tests (CPT) and Nilcon vane tests had been carried out between 2006 and 2009 by Golder Associates (ref. R-7 to R-14) as part of background information for development of the WEP proposal designs. Additional geotechnical investigation was carried out to supplement the previously obtained (pre-bid) subsurface soil data, as required to support the detailed design development of the WEP embankment and structures. One borehole (CV3-1) was put down within the footprint area of the proposed culvert. Additional boreholes, Flat Blade Dilatometer (DMT) and CPT were carried out for the nearby structure (Tunnel T-9) and other structures in close proximity (e.g., Tunnels T-8 and T-10) and highway design components (slopes, retaining structures). One of the main objectives of Borehole CV3-1 was to examine the site specific subsurface conditions and confirm that the soil and groundwater conditions at the culvert site were comparable to those indicated by the nearby tests and investigations. Table 3.1 lists the test holes at or in close proximity to the culvert site during both the previous and the current geotechnical investigations.

Table 3-1: Test Holes at and around Culvert CV-3 Site

Reference	Boreholes	Nilcon Vane Tests	CPT	DMT
This Investigation (2011)	CV3-1	Nil T9-1	CPT 46-RW	DMT T9-1
	BH15-RW			
	T9-1			
	TB7A-1			
	TB7-2			
	TB7-3			
Previous Studies (2007-09)	BH 115 (2008)		CPT-114	
	BH 115A		CPT-6	
	BH 116 (2008)			
	BH 116A			

Legend: N/A = Not Applicable

The locations of boreholes, Nilcon tests, CPT and DMT executed during the pre-bid and additional investigations, and the inferred soil profile in the general area of the culvert are shown on Drawing 285380-04-090-WIP1-5301. Borehole and DMT logs from the additional investigation are included in Appendix A. Relevant borehole logs from the previous investigation are included in Appendix B.

Drawings 285380-04-090-WIP1-5301 and 285380-04-090-WIP1-5302 show the location of the test holes and an interpreted soil stratigraphic profile at Culvert CV-3 site.

3.2 Additional Investigation at the Culvert Site

This section presents the exploration procedure and results of the investigation carried out at the culvert site.

3.2.1 Fieldwork at Culvert Site

Borehole CV3-1 was advanced on July 12, 2011 utilizing a track-mounted CME 75 auger rig owned and operated by Marathon Drilling Co. Ltd. under contract to AMICO and under full-time technical observation by AMEC engineers and technicians. The borehole was advanced to a maximum depth of 9.8 m below grade using 200 mm diameter hollow stem augers.

Soil sampling was carried out using a 50 mm diameter split spoon sampler. Soil sampling was carried out at 0.75 m depth intervals to the depths explored. All samples were visually classified and placed in airtight containers and transported to AMEC’s Tecumseh (Windsor) laboratories for further examination and testing. Standard Penetration Tests (SPT, ASTM D1586) were carried out in conjunction with split spoon sampling. The borehole was decommissioned using a bentonite-cement grout following completion of sampling and testing.

3.2.2 Laboratory and Analytical Testing

All recovered soil samples were examined in the field and the AMEC geotechnical laboratory. Natural moisture content measurements were carried out on most of the recovered samples from Borehole CV3-1 as well as Atterberg limit tests on selected samples. The results are presented on the borehole log in Appendix A.

Analytical testing consisting of pH, redox potential, resistivity, sulphide and sulphate contents were carried out on one sample collected from Borehole CV3-1. The results from these chemical tests are presented in Appendix C.

3.2.3 Data Interpretation – General Discussion

Field Vane Test Data Correction: The chart shown in Figure 3.1[2] initially developed by Bjerrum (ref. R-3) and updated subsequently by Ladd et al (ref. R-20) based on circular arc failure analyses of embankment failures suggests correction by multiplying the field vane data by 1.05 to 1.10 for soils with plasticity index of about 15. However, based on re-evaluation of the Bjerrum chart by Aas et al. (ref. R-1), the Canadian Foundations Manual suggests that the vane test data for clays with PI<20 should not be corrected (ref. R-1 and R-4). The field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI. The undrained shear strength (S_u) profiles inferred from the DMTs and the S_u values obtained from the conventional field vane tests in boreholes were consistently higher than the Nilcon vane test values.

Strength Profiles from Cone Penetration Tests: The undrained shear strength of the silty clay deposit was estimated using the CPT tip resistance, Q_t, as follows:

$$S_{u\,CPT} = \frac{Q_t - \sigma_{vo}}{N_{kt}} \qquad \text{(Eq. 3.1)}$$

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[2] All figures are included at the end of the report text.

Where:

$S_{u\,CPT}$	is the undrained shear strength estimated from the CPT test;
Q_t	is the corrected total cone tip resistance;
σ_{vo}	is the total vertical stress at the corresponding depth of measurement of the Q_t value; and
N_{kt}	is an empirical factor that varies, depending on soil type and test arrangement, typically between 8 and 20.

The CPT based S_u profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles by modifying the N_{kt} factor values used to calibrate the CPT strength profiles varied for different segments of the WEP and the soil strata. Thus, a N_{kt} factor of 14 was used to estimate the undrained shear strength of the clay crust and transition layers. The N_{kt} factors used for the underlying grey silty clay to clayey silt stratum and the lower clayey silt stratum were 15 and 13, respectively. Figure 3.3 presents the undrained shear strength profiles for WEP segment between Sta. 11+500L and Sta. 12+300L.

Pre-Consolidation Pressures from Cone Penetration Tests: The approach used for estimating the pre-consolidation pressures from the estimated S_u profiles follows the Stress History and Normalized Soil Engineering Properties (SHANSEP) method developed at MIT (ref. R-19). The following relationship was used to compute the pre-consolidation pressures:

$$OCR = \frac{\sigma'_p}{\sigma'_{vo}} = \left[\frac{S_u / \sigma'_{vo}}{S} \right]^{1/m} \qquad \text{(Eq. 3.2)}$$

Where:

S_u	is the undrained shear strength,
σ'_{vo}	is the vertical effective stress,
σ'_p	is the pre-consolidation pressure (also referred as maximum past pressure),
S	is the normalized strength ratio, S_u/σ'_{vo}, of normally consolidated soil,
OCR	is the overconsolidation ratio, and
m	is an empirically determined exponent, typically varying between 0.7 and 1.0.

Based on plasticity index of the clayey silt to silty clay deposit, values of $S = 0.18$ and $m = 0.95$ were chosen to estimate the maximum past pressures from the inferred undrained shear strength profile. The maximum past pressure, σ'_p can then be estimated as:

$$\sigma'_p = \sigma'_{vo} \times \left[\frac{\frac{S_{uCPT}}{\sigma'_{vo}}}{0.18} \right]^{1.05} \qquad \text{(Eq. 3.3)}$$

Flat Blade Dilatometer (DMT) Test Data:

DMT tests along WEP were conducted in general accordance with ASTM D6635-01 (2007). The soil properties from the results of these tests were developed using guidelines in “The Flat Dilatometer tests (DMT) in soil investigations” Report by the ISSMGE Committee TC16, International Conference on In-Situ Measurements of Soil Properties, Bali, Indonesia, 2001 (ref. R-16). The undrained shear strength values for the clay deposits were estimated using the relationship $S_u = 0.18\ \sigma'_{vo}\ (0.5\ K_d)^{1.25}$. K_d is the horizontal stress index obtained from DMT reading and is defined by:

$K_d = (p_0 - u_0) / \sigma'_{vo}$

Where:

p_0 is the corrected instrument lateral pressure reading at zero membrane deformation (‘null method”)

u_0 is the pore water pressure in the soil prior to the blade insertion

The undrained shear strength (S_u), pre-consolidation pressure (σ_p'), natural water content (w_N) and compression index (C_c) profiles based on field and laboratory testing from boreholes, CPTs and DMT carried out between Sta. 11+500L and Sta. 12+300L are presented on Figure 3.3. Also included on the figure are $0.18x\sigma_{vo}'$ curve (representing OCR=1) and simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units. The constant 0.18 for S_u/σ'_{vo} for OCR=1 curve is based on average plasticity index of the silty clay to clayey silt stratum and published relationships (refs. R-6 and 12).

4 Subsurface Conditions

The subsurface conditions described below are based on data gathered in the historic investigations and the current investigation. The general soil stratigraphy at the site consists of the following successive strata: topsoil, fills and upper granular deposit, an extensive clayey silt to silty clay deposit below about elevation 183.5[3], and a possible discontinuous lower granular deposit below about elevation 156 (BH-115), overlying limestone and dolostone bedrock below about elevation 151.5 m. The thickness of the Clayey Silt to Sandy/Silty Clay deposit based on the available nearby boreholes is about 31.5 m. The bedrock was encountered at depth approximately 33 m below the ground surface.

4.1 Pavement, Topsoil, and Surficial Fills

A layer of topsoil was encountered at the ground surface in Boreholes TB7-1, TB7-2, TB7-3, DMT T9-1, BH-115, BH-115A, BH-116, and BH 116-A. The thickness of the topsoil was about 0.2 to 0.5 m at the borehole locations.

Asphalt was encountered surficially in Borehole T9-1. The asphalt was about 0.28 m thick at the borehole location. Wet granular fill layer consisting of crushed limestone, sand, and gravel was encountered beneath this asphalt layer. The granular fill layer was about 0.7 m thick.

Surficial fills were encountered in Boreholes CV3-1and BH15-RW. The fills were variable and consisted of silty clay to sand to silty sand and gravel. The fill thickness was around 1.0 m at the borehole locations.

Fill material consisting of silty clay and organics (topsoil) was encountered in TB-7-3 underneath the surficial topsoil layer, The thickness of this fill layer was approximately 0.9 m.

4.2 Silty Clay to Clayey Silt Stratum

An extensive cohesive silty clay to clayey silt stratum, was encountered directly underlying the surficial topsoil and/or fill layer. The encountered depth below existing ground surface varied from 0.3 to 1.5 m corresponding to elevation 182.6 to 183.8. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be subdivided into four layers as follows: brown desiccated stiff to hard clay crust, transition zone, upper grey silty clay to clayey silt deposit (referred to hereafter as silty clay), and then a generally coarser lower grey clayey silt deposit (referred to as lower clayey silt). The natural water content, Atterberg limits, and total unit weights of the clay sub-strata encountered in boreholes are summarized in Table 4-1 and illustrated in Figure 3.3.

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[3] Elevations are in metres and are referred to geodetic datum.

Table 4-1: Summary of Silty Clay Index Properties (Based on CV3-1 and Nearby Boreholes)

Property[1]	Clay Crust	Transition	Grey Silty Clay	Clayey Silt
Elevation Range (m)	184[2] to 178	178 to 175	175 to 163	163 to 151[2]
Natural Water Content, w_N, %	10 to 22	10 to 18	11 to 27	11 to 35
Liquid Limit, w_L, %	N/A	23 to 26	23 to 36	28 to 31
Plastic Limit, w_P, %	N/A	13	13 to 20	15 to 16
Plasticity Index, PI	N/A	10 to 13	9.5 to 16	13 to 15
Liquidity Index, LI	N/A	0.20 to 0.23	0 to 1.25	0 to 0.3
Unit Weight, γ, kN/m^3	N/A	21.6	21.0-21.5	21.8

1 – Index Properties are bases on laboratory results from Boreholes CV3-1, TB7-1, TB7-2, TB7-3, BH15-RW, T9-1, BH-115, BH-116.
2 – Ground surface elevations vary
3- N/A: Not Available.

As illustrated on Figure 3.3, the undrained shear strength of the silty clay stratum varied with depth generally as follows:

- Crust layer: > 100±20 kPa
- Transition layer: 80±20 kPa to 65±10 kPa
- Upper silty clay: 65±10 kPa to 45±10 kPa
- Lower clayey silt: ±100 kPa

4.3 Lower Granular Deposit

Beneath the silty clay to clayey silt, a deposit of layered sequences of silty sand, clayey silt and silty sand and gravel were encountered in the nearest deep boreholes (BH-115 and T9-1). This deposit is referred to as lower granular deposit, and is essentially a non-cohesive material comprising silty sand and gravel and varying amount of clay fraction. This layer was encountered around elevation 156.2 in BH-115 and 152.0 in T9-1. The thickness of the lower granular deposit varied from approximately 0.3 to 4.7 m at the borehole locations. The lower granular deposit had SPT 'N' values ranging from 36 to 48 indicating a dense to very dense state of compactness.

4.4 Bedrock

Boreholes CV3-1, TB7-1, TB7-2, TB7-3, and BH15-RW were terminated within the overburden deposits. Boreholes T9-1, BH-115, and BH-116 refused on material considered to be bedrock beneath the lower granular deposit or below the silty clay to clayey silt stratum at about elevation 151.7 to 151.5. The bedrock was light grey, fairly porous, and fine grained limestone bedrock. The Rock Quality Designation (RQD) of the recovered rock cores ranged from 33 to 100 percent, indicating a poor to excellent quality.

4.5 Groundwater Conditions

Shallow and deep standpipes were installed in selected boreholes to measure the water levels within overburden and bedrock, respectively (Table 4-2).

The observed piezometric levels within the overburden and the bedrock varied generally from 182.2 to 184.0 and 177.3 to 177.7, respectively (Table 4-2). These observations suggest a slight downward gradient between the overburden and the bedrock. Nevertheless, given the general prevalence in the Windsor area, local occurrence of artesian condition in bedrock cannot be completely ruled out.

Perched groundwater is known to accumulate seasonally within the upper deposits of fill, topsoil and granular layers, and within the fissures in the silty clay crust. In periods of wet weather, the perched groundwater levels can rise to near the ground surface.

Table 4-2: Summary of Measured Water Levels

Borehole	Ground Surface El, m	Piezometer Type	Screen El, m	Strata Type at Screen Depth	Measured Water level	
					Date	**El, m**
T9-1	184.0	VWP	174.9 151.4	Silty Clay Limestone	Aug. 29, 2011 Aug. 29, 2011	184.0 177.7
BH-115	183.8	S-Piez.	146.2 – 147.6	Limestone	Jan. 28, 2009	177.4
BH-115A	183.8	S-Piez.	173.0 – 173.3	Clayey Silt	Jan. 28, 2009	182.2
BH-116	183.6	S-Piez.	151.9 – 154.0	Clayey Silt	Jan. 28, 2009	177.5
BH-116A	183.6	S-Piez.	174.7 – 175.0	Clayey Silt	Jan. 28, 2009	182.7

Legend:	S-Piez.	Standpipe Piezometer
	VWP	Vibrating Wire Piezometer

4.6 Subsurface Gases

The groundwater in the project area, especially within the lower granular deposit and bedrock, is known to contain dissolved hydrogen sulphide (H_2S) and methane (CH_4) gases that are liberated from the water on exposure to atmospheric pressure.

The H_2S gas can frequently be detected by odour at concentrations in the order of 0.5 ppm and can be corrosive at concentrations of about 2 ppm to 3 ppm in the groundwater.

A summary of sampling and testing of the groundwater by Golder (ref. R-13), in the boreholes near Culvert CV-3 is presented in Table 4-3.

Table 4-3: Summary of Natural Groundwater Chemistry

Borehole	Surface El, m	Sample El, m	Strata Type at Screen / Sensor Depth	H_2S	CH_4
				mg/L	µg/L
BH-115	183.8	146.3	Limestone	<0.02	5
BH-116	183.6	151.7	Clayey Silt	<0.02	15

Although the presence of the H_2S and CH_4 gases was not was not observed during the 2011 geotechnical investigation at CV-3 site, their presence cannot be entirely ruled out. Pumping tests were conducted at three locations across the proposed parkway to determine concentration levels of hydrogen sulphide gas in the groundwater of the area. A summary of the results of these tests is provided in Table 4.6

Table 4-4: Pumping Tests Data

Test #	Approximate Location	H_2S Gas Concentration (mg/L)
TOW-1	East of Tunnel T-10A	<0.2
TOW-2	North of Tunnel T-7	20.0
TOW-3	South of Tunnel T-4	7.0

Dissolved methane was also sampled by Golder (ref. R-13) with most samples below detection (<5 µg/L) with the largest values generally measured where artesian conditions across the length of the WEP occur (up to 485 µg/L). These data are consistent with general water chemistry sampling taken at the end of the pumping tests

In this regard, it is recommended that the design and construction should address the potential presence of these gases. Air monitoring should be considered during construction. In general, it is recommended that equipment operating in confined spaces be selected to safely operate in a potentially gaseous environment.

5 Development of Geotechnical Designs

It is understood that the proposed box culvert will be a cast-in-place, one-span, rigid-frame box with 6 m inside width and 2.4 m inside height. The invert elevation (i.e., top of the bottom slab) will vary between elevations 181.292 and 181.132 m at the inlet and outlet, respectively. The general arrangement is shown on Drawing 285380-03-060-WIP1-5301.

Retaining walls will be constructed at both ends of the box culvert structure along the sides of Cousineau Road embankment. It is understood the retaining walls will be cast-in-place, and founded on the native soils at a maximum depth of approximately 4 below existing grades. The wall heights vary with heights up to approximately 4.5 m anticipated.

The drain slopes are proposed to be 2.4H:1V.

5.1 Geotechnical Design Criteria and Considerations

The geotechnical design has been completed in compliance with the requirements of the execution version of the Project Agreement Schedule 15-2 Part 2, Article 5 (PA) for the Windsor-Essex Parkway Project. The foundations' design was carried out following the Limit States Design (LS method) based on Load and Resistance Factors (CHBDC and Canadian Foundation Engineering Manual).

Working Stress Design (WS Method) was employed for global stability of the earthworks and soil mass containing earth retaining structures, such as the wing-walls. The stability of the soil mass containing the wing-wall was checked for all potential surfaces of sliding.

5.2 Design Soil Properties

The undrained shear strength design soil properties for the silty clay to clayey silt deposit were interpreted from the CPT and Nilcon vane test profiles and the laboratory test results from the old and new investigations for the segment of WEP between Sta. 11+500L and Sta. 12+300L. The S_u profiles inferred from the CPTs advanced around Culvert CV-3 are shown in Figure 3.3. Selected typical design values obtained from these profiles and the trends in the east part of the WEP project are summarized in Table 5-1.

Table 5-1: Summary of Interpreted Design Properties of Clay Strata

Clay Substratum	Elevation Range, m	Undrained Shear Strength (S_u), kPa (*)	Effective Strength Parameters***	Preconsolidation Pressure (σ_p'), kPa (*)	OCR Range
Clay Crust	184 to 178	75 (**)	c’ = 0 kPa, ǿ = 30°	600	>7
Transition	178 to 175	75 to 60	c’ = 0 kPa, ǿ = 30°	600 to 400	7 to 2
Silty Clay (I)	175 to 166	60 to 50	c’= 0 kPa, ǿ = 30°	400 to 280	7 to 2
Silty Clay (II)	166 to 163	50 to 57	c’= 0 kPa, ǿ = 30°	280 to 310	2 to 1.2
Clayey Silt (I)	163 to 161	57 to 80	c’ = 0 kPa, ǿ = 30°	310 to 450	2 to 1.2
Clayey Silt (II)	161 to 151	80	c’ = 0 kPa, ǿ = 30°	450	2 to 1.2

(*) Varies with depth as illustrated in Figure 3.3
(**) Lower limit from CPT tests to be used in global stability only.
(***)Apparent cohesion, c’, Angle of internal friction, ǿ

The stress-strain properties and the effective shear strength properties of the silty clay to clayey silt soils were based on published correlations (ref. R-17, ref. R-21, and ref. R-25), the tests reported in Golder’s Subsurface Condition Interpretation Report (ref. R-10), and the tests performed during the additional geotechnical investigation carried out as part of the detailed design development for the entire WEP length.

The compressibility indexes are correlated to natural water content (w_N, expressed as percent) as illustrated in Figures 5.1 and 5.2 and summarized as follows:

$$C_c = 0.0086w_N - 0.0086$$

$$C_r = 0.11C_c$$

The interpreted representative values used for the silty clay/ clayey silt substrata for the Culvert CV-3 sites are summarized as follows:

Table 5-2: Summary of Compressibility Properties

Property	Clay Crust	Transition	Grey Silty Clay (“Upper Clay I”)	Grey Silty Clay (“Upper Clay II”)	Clayey Silt (Lower Clay I)	Clayey Silt (Lower Clay II)
Average Natural Water Content, w_N, %	13	15	20	16	16	20
Virgin Compression Index, C_c	0.10	0.12	0.16	0.13	0.13	0.16
Recompression Index, C_r	0.011	0.013	0.018	0.014	0.014	0.018

The modulus of elasticity has been correlated with the average undrained shear strength of the material, published information (ref. R-25) and local experience (ref. R-10). For the unweathered portion of the silty clay stratum the empirical relationship were used based on average shear strength profiles for the material, as follows:

Undrained Elastic Modulus $E_u = 300\ S_u$

Drained Elastic Modulus $E' = 0.9E_u$

Table 5-3: Summary of Interpreted Elastic Moduli Properties

Soils Stratigraphy	Elastic Modulus (Undrained), MPa	Poisson's Ratio (Undrained) (*)	Elastic Modulus (Drained), MPa	Poisson's Ratio (Drained) (*)
Clay Crust	23	0.49	20	0.35
Transition	20	0.49	18	0.35
Silty Clay I	17	0.49	15	0.35
Silty Clay II	16	0.49	14	0.35
Clayey Silt I	21	0.49	18	0.35
Clayey Silt II	24	0.49	22	0.35

* Assumed value

The effective shear strength properties applicable to the silty clay to clayey silt stratum were determined from triaxial and direct shear tests performed during the pre-bid and additional geotechnical investigations (and supported by published PI versus ø' relationships in reference R-21 and R-26. The effective strength parameters are summarized in Table 5-1.

5.3 Excavation and Temporary Cut Slopes

The discussion of the temporary slopes in this report relates only to the anticipated subsurface conditions to assist the designer of temporary works and as they affect the design of the culvert structure foundation. The shapes and slopes of the temporary excavations shown do not constitute the recommended design of the temporary slopes. The Contractors are fully responsible for the design, construction methods and performance (stability, deformability and deterioration) of the temporary slopes. The Contractors also must ensure that the temporary slopes meet the Project Agreement criteria and the needs to accommodate the construction of the structure as per design.

The excavations are expected to encounter surficial fills, topsoil and water bearing upper granular soils and will be extended into the native stiff clayey silt to silty clay. The excavation for the culvert and associated retaining walls will be between 180.7 and 180.0 i.e. about 3.5 to 4.0 m below grade.

Basal hydrostatic uplift stability was calculated based on the highest measured water level (182.8) measured in the silty sand deposit encountered in BH-116A below elevation 175 and the anticipated deepest excavation depth (elevation 180.7). Accordingly, the minimum thickness of the silt-clay layer above the silty sand deposit would be 24.1 m. The calculated factor of safety against hydrostatic uplift was greater than 2.0 based on the weight of the silty clay cap only.

5.4 Concrete Box Culvert

5.4.1 General

All topsoil, disturbed soils and other deleterious materials must be completely removed from the footprint area of the structure foundation. The exposed subgrade should be inspected and upon approval, a subgrade protection layer of at least 75 mm of lean concrete should be placed same day as excavated.

The excavations and foundation grades should be inspected in accordance with OPSS 902. Any low areas should be brought to grade with lean concrete fill, or approved soil backfill, as directed by the engineer. Depending on the site conditions, the use of geofabric may be required where soil backfill is approved for subgrade corrections.

5.4.2 ULS Bearing Resistance

A factored net geotechnical resistance of 200 kPa at Ultimate Limit States (ULS) was determined for the native undisturbed silty clay subgrade soils supporting the box culvert structure near elevation 180.9 and higher.

Due to the culvert embedment after construction, the ULS resistance will increase with the completion of the compacted backfill along the culvert walls at an approximate rate of 20 kPa for every 1 meter of embedment below the finished grade.

5.4.3 SLS Resistance and Performance

A net serviceability limit states (SLS) resistance (soil stress increase) of 135 kPa was determined on the basis of a maximum of 25 mm post-construction settlement.

Since the construction of the culvert involves ground unloading (associated with removal of the existing culvert and backfill) followed by reloading (new construction), the net soil stress increase is expected to be minimal (current and proposed finished grades are similar with a depth of excavation of approximately 3.5 m) and time-dependent settlement should be less than 25 mm. Assuming the load distribution along the culvert is relatively uniform, differential settlement between the centre and the ends of the culvert is expected to be less than 15 mm.

All the ground movement and deformations discussed above are estimates based on soil deformation/compressibility properties interpreted from laboratory tests and empirical correlations. Therefore, the reported values are approximate and should be considered only as an indication of the magnitude of the soil response. These estimates will need to be verified and refined based on performance monitoring in the field.

The settlements discussed above do not include deformations caused by seasonal temperature and moisture variations or related to compression of the backfill, which for well compacted backfill should be small. In this regard, stringent compaction specifications and control should be implemented to minimize the magnitude of backfill compression. In order to minimize the long-term effects, the soil backfill materials must be compacted according to the recommendations in Section 5.6.

5.5 Retaining / Head Walls

5.5.1 General

Cast-in-place retaining walls are proposed at this culvert structure. The following general recommendations are considered applicable:

- All topsoil and other deleterious materials are to be completely removed from the footprint area of the structure so that it is founded directly on the competent native soils.
- The retaining wall structure should be founded on undisturbed firm to stiff grey silty clay at/near elevation 180.0.
- The exposed subgrade should be inspected and upon approval, a subgrade protection layer of at least 75 mm of lean concrete should be placed same day as excavated.
- Any low areas should be brought to grade using lean concrete fill. The footing excavations should be inspected in accordance with OPSS 902.
- The retaining wall footings should be stepped up in a manner that ensures proper frost cover.

5.5.2 Global Stability

The global stability analyses were carried out for both short-term and long-term loading conditions using the design soil properties discussed in Section 5.2 for the highest wall section from Drawing 285380-03-060-WIP1-5301.

The global stability analyses were carried out for short-term during construction, short-term end of construction (EOC), and long-term steady state (LT) loading conditions using the design soil properties discussed in Section 5.2. The analysis models are presented in Appendix D and the results are summarized as follows:

Table 5-4: Results of Global Stability Analyses

Model and Loading Condition	Soil Properties	Figure No.	Factor of Safety*
Culvert Wall – End of Construction	Undrained	D.1	2.63 (2.02)
Culvert Wall – Long-term Steady State	Drained	D.2	1.88 (1.65)

(*) Values in parentheses refer to factor of safety for non-circular failure surface.

5.5.3 ULS Bearing Resistance

A net factored bearing resistance of 200 kPa at Ultimate Limit States (ULS) was determined for the native stiff clay crust subgrade soils supporting the headwalls. The above resistance assumes that the wall foundation bears on stiff silty clay subgrade at/near elevation 180.0 or higher. The factored bearing resistance increases by 20 kPa for every 1 m of footing embedment below finished grade.

In the case of concrete retaining walls, the above bearing resistance will decrease by a factor depending on the load inclination at the base of the wall foundation as indicated in the CHBDC (ref. R-5).

5.5.4 ULS at Sliding

ULS at Sliding: The factored geotechnical resistance can be determined with the following expression (ref. R-5):

$$H_{ri} = 0.8\ A'c' + 0.8V\tan\delta > H_f$$

Where: A' = effective contact area of the base (m^2).

c' = cohesion = 0 (long-term loading condition)

c' = undrained shear strength = 65 kPa (short-term loading condition)

$\delta = 30^0$ for the silty clay stratum for foundations cast directly on the native soil. (long-term loading condition)

$\delta = 0^0$ (short term loading condition)

V = unfactored vertical force (kN)

H_f = factored horizontal load (kN)

Allowance for buoyancy should be made, where applicable.

5.5.5 SLS Resistance

A net SLS resistance (soil stress increase) of 145 kPa is estimated for a retaining wall founded on stiff silty clay at, or above elevation 180.0 on the basis of a 25 mm maximum post-construction settlement. The footing width considered is not wider than 5.0 m.

Assuming that the maximum unfactored bearing pressure at the edge of the footing is limited to 1.4 times the average bearing pressure, the estimated minimum and maximum footing settlements would vary between 18 mm and less than 35 mm (less than 0.34% maximum inclination for a 5 m wide foundation). The above discussion is applicable to a retaining wall of height and width that can be accommodated to meet the ULS resistance criteria.

The settlements discussed above do not include deformations caused by seasonal temperature and moisture variations or related to compression of the backfill, which for well compacted backfill should be small. In order to minimize the long-term effects, the soil backfill materials must be compacted according to the recommendations in Section 5.6.

5.6 Backfilling

Behind the concrete box culvert and the associated retaining walls, bedding and backfill materials should meet the requirements of OPSS 902 and the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC). Appropriate frost tapers will need to be provided if the associated backfill materials are not compatible with the native soils.

The backfill should be compacted in maximum 200 mm thick loose lifts in accordance with SP105S10. Longitudinal drains should be installed to provide positive drainage of the backfill. Other aspects of the backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 and 3190.100.

Behind the retaining/wing wall well graded sand and gravel fill (Granular B Type I, or approved equivalent) should be used and placed as per OPSD 3101.150 requirements for minimum granular.

Heavy compaction equipment should not be used immediately adjacent to the walls of the structure as per the CHBDC and OPSS 501. Effects of backfill compaction activities should be simulated as live load over and above the static lateral earth pressure for structural design in accordance with Section 6.9.3 in the CHBDC.

Earth pressures on retaining/wing walls may be calculated on the basis of the parameters given in Table 5-4. In the case of sloping backfill surface, the coefficients in this table should be modified based on the following equations:

$$K_a = \left(\frac{\cos\emptyset}{1 + \sqrt{\frac{\sin\emptyset . \sin(\emptyset - \beta)}{\cos\beta}}} \right)^2 \qquad \text{(Eq. 5.7)}$$

$$K_0 = (1 - \sin\emptyset)(1 + \sin\beta) \qquad \text{(Eq. 5.8)}$$

$$K_p = \left(\frac{\cos\emptyset}{1 - \sqrt{\frac{\sin\emptyset . \sin(\emptyset + \beta)}{\cos\beta}}} \right)^2 \qquad \text{(Eq. 5.9)}$$

Where: ϕ= Friction angle of backfill material,

β= Slope of the backfill surface.

Table 5-5: Soil Parameters for Earth Pressure Calculations

Soil Parameter	Group I Soils	Group II Soils	Group III Soils
Fill Unit Weight, kN/m^3	22	21	20.5
Friction angle, (degrees)	33 to 35	29 to 32	22 to 30
Coefficients of Static Lateral Earth Pressure:			
'Active' or Unrestrained, $K_a^{(*)}$	0.27 to 0.30	0.31 to 0.35	0.33 to 0.45
'At Rest' or Restrained, $K_o^{(*)}$	0.43 to 0.46	0.47 to 0.52	0.50 to 0.62
‘Passive’, $K_p^{(*)}$	3.3 to 3.7	2.9 to 3.2	2.2 to 3.0

(*)Values are given for level backfill and ground surface behind the wall. The coefficients of lateral earth pressure should be adjusted if there is sloping ground at the back of the wall.
Note: Compacted to > 95% Standard Proctor maximum dry density.
Legend:

- Group I Soils: Coarse grained soils (e.g. Granular A and B Type 2).
- Group II Soils: Finer grained than Group I noncohesive soils (e.g. Granular B Type1, pit run, etc).
- Group III Soils: Finer grained soils (e.g. approved site generated silty clay)

Group III soils may be used as general backfill within approved areas.

5.7 Drain Slope Stability

The slope stability analysis for the drain sides near the inlet and outlet areas of the culvert with slopes of 2.4H:1V are illustrated on Figures D-3 and D-4 in Appendix D, and the results are summarized as follows:

Table 5-6: Results of Global Stability Analyses

Model and Loading Condition	Soil Properties	Figure No.	Factor of Safety*
Trench – During Construction	Undrained	D.3	3.71 (3.25)
Trench – Long-term Drained Slope	Drained	D.4	1.35 (1.31)

(*) Values in parentheses refer to factor of safety for non-circular failure surface.

The 2.4H:1V open slopes are susceptible to surficial sloughing and erosion. In this regard, suitable slope protection (rip-rap blankets over filter fabric, or equivalent) should be considered to prevent slope surface sloughing.

6 Other Geotechnical Recommendations

6.1 Construction Dewatering

The design of the dewatering system should comply with the OPSS 517 and 518 provisions.

Due to the relatively low permeability of the silty clay deposit, minor groundwater seepage is anticipated, which should be controllable by conventional temporary dewatering methods. Runoff and seepage into the excavations from perched groundwater from the fill and upper granular layers should also be anticipated. In adverse conditions, the runoff and seepage from perched groundwater can be significant and accompanied by piping and wash-outs of the fines causing sloughing of the slopes.

Accordingly, provision should be made to prevent runoff and piping erosion of the slope surface by cutoff drains and / or blanketing of the excavation slopes with a geotextile and free draining granular material. The seepage flow should be directed to collection sumps by temporary drainage ditches properly sized, filtered and lined to accommodate the flow rates.

All surface water should be directed away from all open excavations.

6.2 General Construction Requirements

The anticipated construction conditions in this report are discussed only to the extent of their potential influence on the design decisions. References to construction methods are not intended to be the suggestions or directions on the construction methodologies. The Contractor should be aware that the data presented in this report and their interpretations may not be sufficient to assess all factors that may affect the construction.

Reference to the temporary slopes in this report relates to analytical assessment of the slopes as they interact with the stability and design of the structure foundation. The shapes and slopes of the temporary excavations shown do not constitute the actual design of the temporary slopes. The Contractor is fully responsible for the design, construction methods and performance (stability, deformability and deterioration) of the temporary slopes. The Contractor also must ensure that the temporary slopes meet the Project Agreement criteria and the needs to accommodate the construction of the structure as per design.

The following recommendations and comments are considered applicable:

- All excavation works should be carried out in accordance with the guidelines outlined in Occupational Health and Safety Act (OHSA) and Ontario Provincial Standard Specification OPSS 902. The native undisturbed soils may be classified as Type 3 soils. The excavations below the original ground levels may intersect water bearing backfill within trenches of active and/or abandoned utilities. In these cases, Type 4 soil conditions may occur and should be addressed accordingly.

- The silty clay soils at the project site are highly susceptible to disturbance and rapid deterioration when exposed to elements, groundwater inflow, weathering and/ or subjected to direct construction traffic.
- Due to the layering of the clay, the clay is prone to separation along the thin sand-silt stringers/interbeds. The Contractor should therefore monitor the excavation and adjust the rate and thickness of excavation layers when close to design grades to prevent over excavation occurring.
- Temporary slopes, permanent slopes, and subgrade areas must be appropriately protected at all times against surface erosion due to runoff, desiccation, freeze-thaw effects, etc.
- To protect the integrity of subgrade for foundations and pavements, the final excavation lift above the design elevation should not be less than 500 mm and should be carried out only when the contractor is ready to prepare and cover the subgrade with the materials specified in the design same day the final excavation is exposed and approved. No construction traffic should be permitted over subgrade without approved protective covers.
- The excavation of the final soil layer above the design subgrade is to be carried out using buckets equipped with smooth lips. Once exposed, the subgrade must be immediately inspected. Upon approval, the subgrade should be immediately protected; depending on the type of construction, geofabrics, granular mats, a skim coat of lean concrete protection (mud mat), etc. should be used.
- Regular inspection of the condition of the temporary slopes should be carried out by qualified personnel for signs of distress or instability and appropriate mitigation measures should be implemented.
- In the case of rip/rap, or otherwise coarse rockfill cover, the insulation effects of such materials are considered to be one half of the insulation offered by soil deposits /cover, and the depth of frost penetration will have to be increased proportionally.

6.3 Corrosion Potential

A series of pH, Redox Potential, Resistivity, Sulphide, and Sulphate tests were carried out on a sample from Borehole CV3-1. Table 6-1 provides the results of these analyses that could be used to assess the potential for corrosion on concrete:

Table 6-1: Results of Analytical Testing on Soils

Location of Soil Samples	Depth of Soil Sample	pH	Redox Potential, mV	Resistivity, ohm.cm	Sulphide, mg/kg	Sulphate, mg/kg
Borehole CV3-1 (SA#7, L1032540)	5.33 m	7.79	120	3330	<0.2	76

The reported results of laboratory testing indicate that based on CSA A23.1, concrete in contact with the tested soil material would have a negligible degree of exposure to sulphate attack.

Based on the measured electrical resistivity, pH, redox potential, sulphide contents etc., the soil would be considered to have a potential for corrosion to buried metallic elements.

A corrosion specialist should review the test result and provide recommendations to address corrosion concerns.

6.4 Construction Quality Control

To ensure that construction is carried out in a manner consistent with the intent of the recommendations set forth in this report, a construction quality control program, including geotechnical inspection, testing and instrument monitoring, should be developed and implemented throughout the construction phase. In addition, related laboratory testing should be carried out in conjunction with the fieldwork to monitor compliance with the various materials and project specifications.

6.5 Instrumentation and Monitoring

As indicated earlier and in consideration of the relative shallow depth of excavation and small heights of backfill, large heave, uplifts or settlements are not expected to occur during construction of the culvert. Nevertheless, it is important that the ground deformations be visually inspected on a regular basis and the surface pins be installed and surveyed if required based on field experience. The scope of instrumentation and monitoring should be reviewed and adjusted during construction based on performance evaluation.

Alert Levels and Contingencies

The monitoring is expected to provide confirmation of the anticipated ground response to loading. In the event of unexpected response of ground movement, the results of the survey should be adressed and modifications to the design and construction may be required.

Some of the indications of unexpected response could be one of the following:

- Ground movement in excess of anticipated maxima (> 25 mm);
- Unstabilised movement trend without loading changes; and
- Non-responsive porewater pressure to unloading during excavation.

Contingencies associated with the instrumentation monitoring can vary from changes to excavation stages and rates, to rescheduling of the different activities (piling, backfilling, etc), to adjustments in the design (slopes, subdrainage, abutment arrangement, etc).

7 Limitations of Report

The work performed in this report was carried out in accordance with the Standard Terms and Conditions made part of our contract. The conclusions and recommendations presented herein are based solely upon the scope of services and time and budgetary limitations described in our contract.

This report presents the subsurface soil and groundwater conditions inferred from geotechnical investigation and geotechnical design of the structures mentioned in the report. The report was prepared with the condition that the structural and other designs of the WEP will be in accordance with applicable standards and codes, regulations of authorities having jurisdiction, and good engineering practices. Further, the recommendations and opinions expressed in this report are only applicable to the proposed project as described within AMEC’s report.

There should also be an ongoing liaison with AMEC during both the design and construction phases of the project to ensure that the recommendations in this report have been interpreted and implemented correctly. Also, if any further clarification and/or elaboration are needed concerning the geotechnical aspects of this project, AMEC should be contacted immediately.

The conclusions and recommendations given in this report are based on data presented in the pre-bid geotechnical investigation reports and information determined at the test hole locations during the additional investigation carried out for the geotechnical design work. The data obtained from the pre-bid investigations (carried out by others) was assumed to be valid and applicable.

The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated.

The soil boundaries indicated have been inferred from non-continuous sampling, observations of drilling resistance, Nilcon vane, CPT and DMT probing. The boundaries typically represent a transition from one soil type to another and are not intended to define exact planes of geological change. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. Thus, unsuitable foundation soils may be encountered at the foundation grade requiring extra sub-excavations, subgrade improvement, and/or changes to the design. It is important that the AMEC geotechnical design engineer be involved during construction throughout the WEP project site to confirm that the subsurface conditions do not deviate materially from those encountered in test holes, and that any material deviations, if encountered, do not adversely affect the geotechnical design.

The stability analyses assumed a certain sequence of the construction; if different construction approaches are considered the geotechnical design will have to be reviewed. The calculated factors of safety assume strict adherence to good construction practices with respect to the protection of the exposed slopes.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, it is recommended that AMEC be engaged during the final design and construction stages to verify that the design and construction are consistent with AMEC's recommendations.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the structural and other designers and constructor. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of the surficial topsoil and the clay crust layer, the presence of artesian conditions and exsolved natural gases, and the strength of the silty clay stratum may vary markedly and unpredictably. The constructor should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. The work presented in this report has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

The benchmark and elevations mentioned in this report were surveyed and provided by AMICO. They should not be used by any other party for any other purpose.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. AMEC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

8 Closure

The geotechnical design for Culvert CV-3 was developed by Mr. Tommi Leinala, P.Eng. under directions from Dr. Dan Dimitriu, P.Eng. (Technical Lead). Dr. Narendra S. Verma, P.Eng. (Technical Director) provided the senior review of the report. Mr. Matt Oldewening, P.Eng. managed the geotechnical investigation and Mr. Brian Lapos, P.Eng. is the project manager.

Mr. Zuhtu Ozden, P.Eng. and Dr. Andrew Smith of Coffey Geotechnics provided the peer review.

The cooperation received from Ms. Biljana Rajlic, P.Eng. and Mr. Philip Murray, P.Eng. of Hatch Mott McDonald and Mr. Daniel Muñoz, P.Eng. of PIC during the design study is gratefully acknowledged.

Yours truly,
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R-24. Quigley, Robert M., 1980, Geology, mineralogy, and geochemistry of Canadian soft soils: a geotechnical perspective, National Research Council of Canada, Canadian Geotechnical Journal, Vol. 17, pp. 261-285.

R-25. Terzaghi, K., Peck, R.B., and Mesri, G. (1990), Soil Mechanics in Engineering Practice, John Wiley and Sons, NY.

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Drawings

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

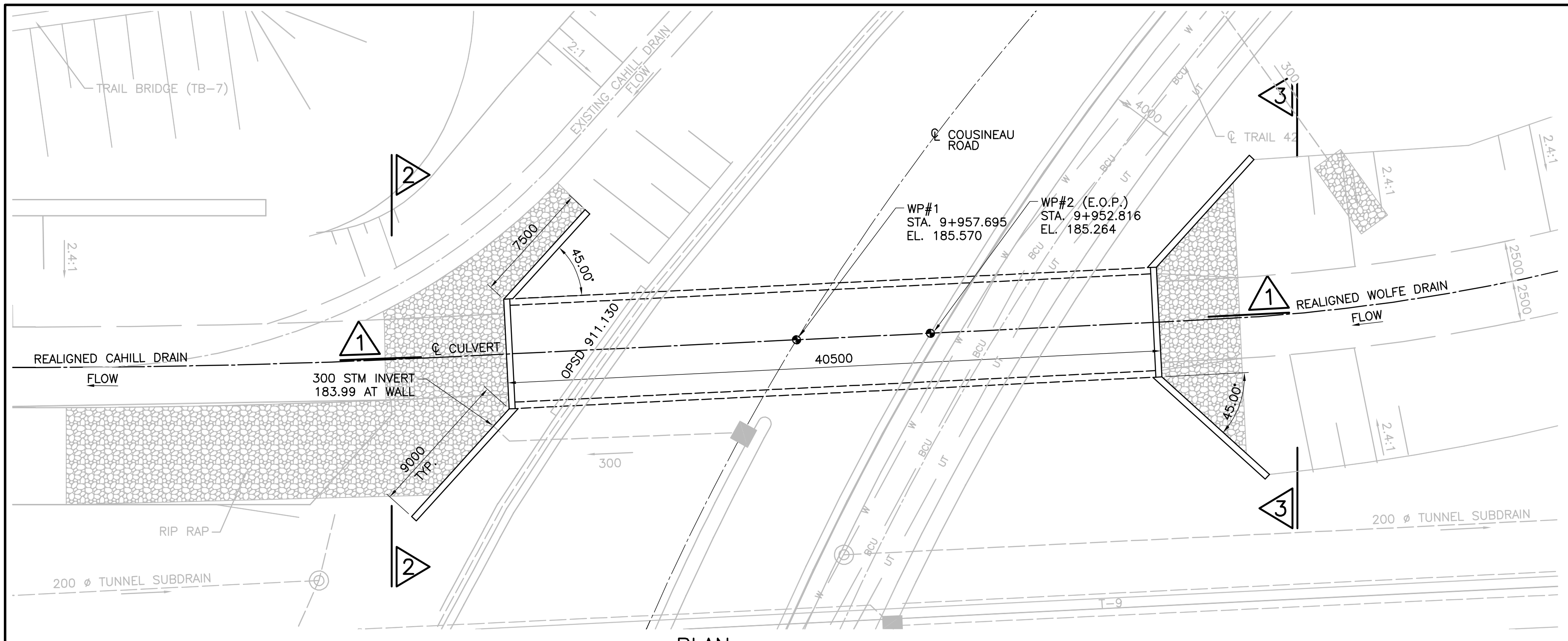
Parkway Infrastructure Engineers
Hatch Mott MacDonald

Windsor–Essex
Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION

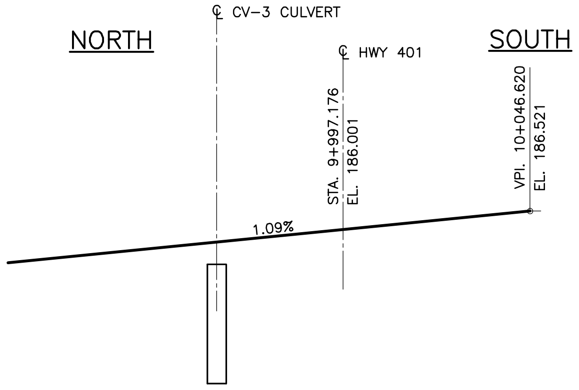
COUSINEAU ROAD CULVERT CV–3
GENERAL ARRANGEMENT

SHEET
S5301
Phase 1
IFC



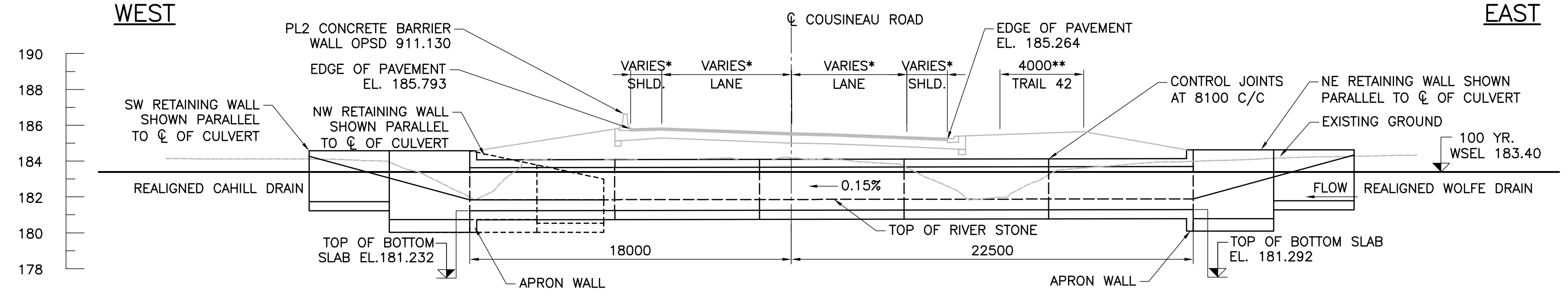
PLAN

SCALE 1:200



PROFILE OF COUSINEAU ROAD

N.T.S



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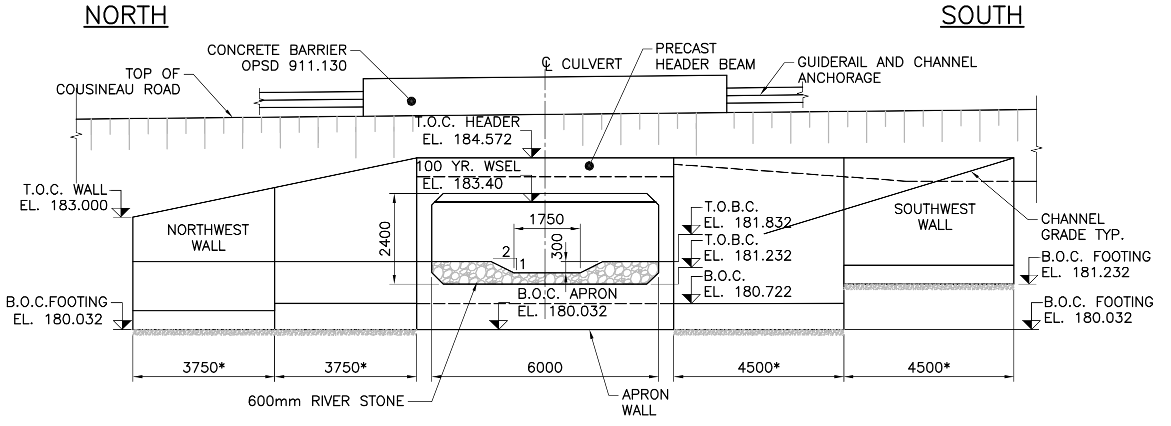
(* DENOTES DIMENSIONS PERPENDICULAR TO ℄ OF TRAFFIC LANE)
(** DENOTES DIMENSIONS PERPENDICULAR TO ℄ OF TRAIL)



2
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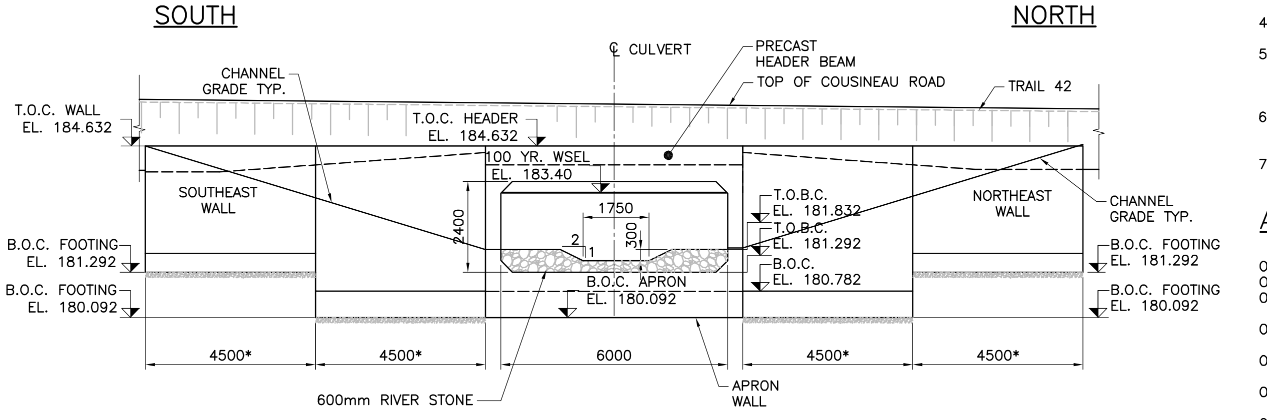
(* DENOTES DIMENSIONS SQUARE TO RETAINING WALL)
RETAINING WALL SHOWN PERPENDICULAR TO ℄ OF CULVERT



3
SCALE 1:100
(* DENOTES DIMENSIONS SQUARE TO RETAINING WALL)
RETAINING WALL SHOWN PERPENDICULAR TO ℄ OF CULVERT

GENERAL NOTES:

1. CLASS OF CONCRETE 30MPa, EXCEPT AS NOTED.
2. CLEAR COVER TO REINFORCING STEEL
   BOTTOM OF TOP SLAB 50±10;
   BOTTOM OF BOTTOM SLAB 100±25;
   REMAINDER 60 ± 20 UNLESS OTHERWISE NOTED.
3. REINFORCING STEEL TO BE GRADE 400W, UNLESS OTHERWISE SPECIFIED. BARS MARKED WITH PREFIX 'C' DENOTE COATED BAR.
4. LEGEND

ALT DENOTES ALTERNATE	OF DENOTES OUTSIDE FACE
IF DENOTES INSIDE FACE	EF DENOTES EACH FACE
TOC TOP OF CONCRETE.	EOP EDGE OF PAVEMENT

5. PEDESTRIAN BARRICADES TO OPSD 980.101. SUPPLY ALONG CULVERT HEADERS AND RETAINING WALLS
6. MAXIMUM FILL HEIGHT OVER CULVERT 2.5 m.
8. SOIL BEARING RESISTANCES
   CULVERT: NET ULS 160kPa; MAX SLS REACTION 135kPa (ESTIMATED MAX SETTLEMENT 25mm).
   RETAINING WALLS: NET ULS 200kPa; MAX SLS REACTION 145kPa (ESTIMATED MAX SETTLEMENT 25mm).
9. FISH COMPENSATION PLAN TO GOVERN FINAL CONFIGURATION OF LOW FLOW CHANNEL AND RIVER STONE WITHIN CULVERT.
10. FOR CULVERT SECTIONS WITH LESS THAN 1000mm COVER, WATERPROOF CULVERT TOP SURFACE AND TOP 300mm OF SIDE WALLS TO OPSD 3370.100 AND OPSS 914. EXTEND PROTECTION BOARD 1000mm ONTO APPROACH SLABS WHERE PRESENT. FOR REMAINDER, APPLY BITUMINOUS DAMP PROOFING TO OPSS 1213 TO TOP SLAB AND TOP 300mm OF SIDE WALLS.
11. FOR ALL HIGHWAY WORKS REFER TO HIGHWAY NEW CONSTRUCTION DRAWINGS.
12. FOR ALL ELECTRICAL AND ATMS WORKS REFER TO ELECTRICAL AND ATMS NEW CONSTRUCTION DRAWINGS.
13. FOR ALL UTILITY WORKS REFER TO UTILITY NEW CONSTRUCTION DRAWINGS.
14. FOR INFORMATION ON EXISTING PAVEMENT AND INFRASTRUCTURE REFER TO HIGHWAYS REMOVAL DRAWINGS AND GENERAL NOTES PROVIDED WITHIN HIGHWAYS REMOVAL DRAWING PACKAGE.

CONSTRUCTION NOTES:

1. SUPPORTS FOR REINFORCING STEEL SHALL BE AS PER OPSD–3329.100 AND OPSD–3329.101 ON FORMED SURFACES. ON NON–FORMED SURFACES, CONCRETE BLOCKS (MIN. 20MPa) SHALL BE USED.
2. 20MPa CONCRETE SUBGRADE PROTECTION SLAB TO BE PLACED AFTER APPROVAL OF SUBGRADE BY GEOTECHNICAL ENGINEER.
3. CORRECT DEFICIENT SUBGRADE AS DIRECTED BY GEOTECHNICAL ENGINEER WITH APPROVED GRANULAR MATERIAL AND COMPACT TO MIN. 95% PROCTOR DENSITY PRIOR TO PLACEMENT OF CONCRETE SUBGRADE PROTECTION SLAB.
4. WET CURE FOR MINIMUM 7 DAYS.
5. JOINT POSITIONS ARE SUGGESTED LOCATIONS TO CONTROL UNWANTED CRACKING. CONSULT DESIGNER IF ALTERNATE LOCATIONS ARE TO BE CONSIDERED.
6. RIVER STONE WITHIN CULVERT TO BE A MINIMUM OF TWO LAYERS THICK WITH A GRADED OR COMPACTED LOW FLOW CHANNEL.
7. GRADATION OF THE RIVER STONE SHALL BE TO NSSP 9999–0229 TO ENSURE VOIDS ARE EFFECTIVELY FILLED DURING PLACEMENT.

APPLICABLE STANDARD DRAWINGS:

OPSD–803.010	BACKFILL AND COVER FOR CONCRETE CULVERTS
OPSD–980.101	PEDESTRIAN BARRICADE INSTALLATION
OPSD–3121.150	WALLS, RETAINING, BACKFILL. MINIMUM GRANULAR REQUIREMENT
OPSD–3329.100	DECK, REINFORCEMENT SUPPORTS FOR REINFORCING STEEL FOR SLAB DEPTHS 300mm OR LESS
OPSD–3329.101	DECK, REINFORCEMENT SUPPORTS FOR REINFORCING STEEL FOR SLAB DEPTHS 300mm OR MORE
OPSD–3370.100	DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD–3941.200	FIGURES IN CONCRETE SITE NUMBER AND DATE LAYOUT

NORTH FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

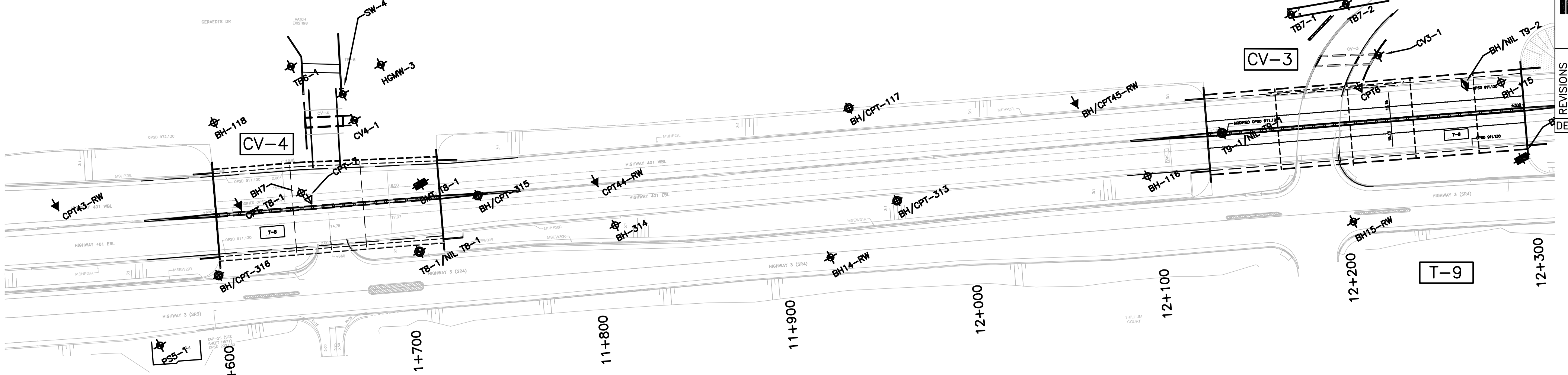
NOT FOR
CONSTRUCTION

REVISIONS

DATE	REV.	BY	DESCRIPTION
TBD	0	CRD	ISSUED FOR CONSTRUCTION

DESIGN CRD	CHK JF	CODE CAN/CSA S6–06	LOAD CL 625–ONT
DRAWN BW	CHK BR	SITE 6–629	DATE 16–AUG–11

METRIC



PLAN
HORT SCALE 1:1500

LIST OF ABBREVIATIONS

PH – SAMPLER ADVANCED BY HYDRAULIC PRESSURE
PM – SAMPLER ADVANCED BY MANUAL PRESSURE
WH – SAMPLER ADVANCED BY STATIC WEIGHT OF HAMMER
WR – SAMPLER ADVANCED BY WEIGHT OF SAMPLER RODS

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amec
Hatch Mott MacDonald

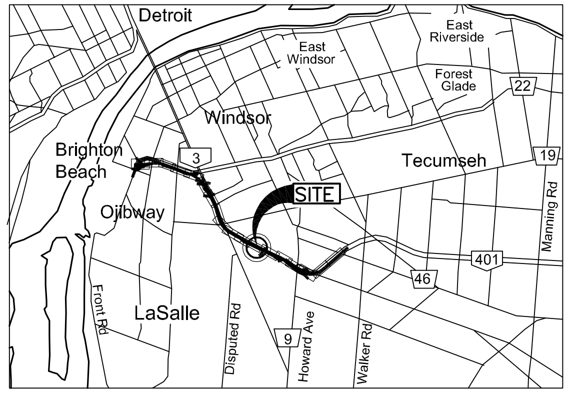
Windsor–Essex
Parkway Project
RFP No. 09–54–1007

REVISIONS			
02–MAR–12	0	TL	ISSUED FOR CONSTRUCTION
DATE	BY	REV.	DESCRIPTION

DESIGN GN APR NSV DATE MAY 30/11

LOCATION PLAN & INTERPRETED STRATIGRAPHIC PROFILE
STA 11+500L TO STA 12+300L

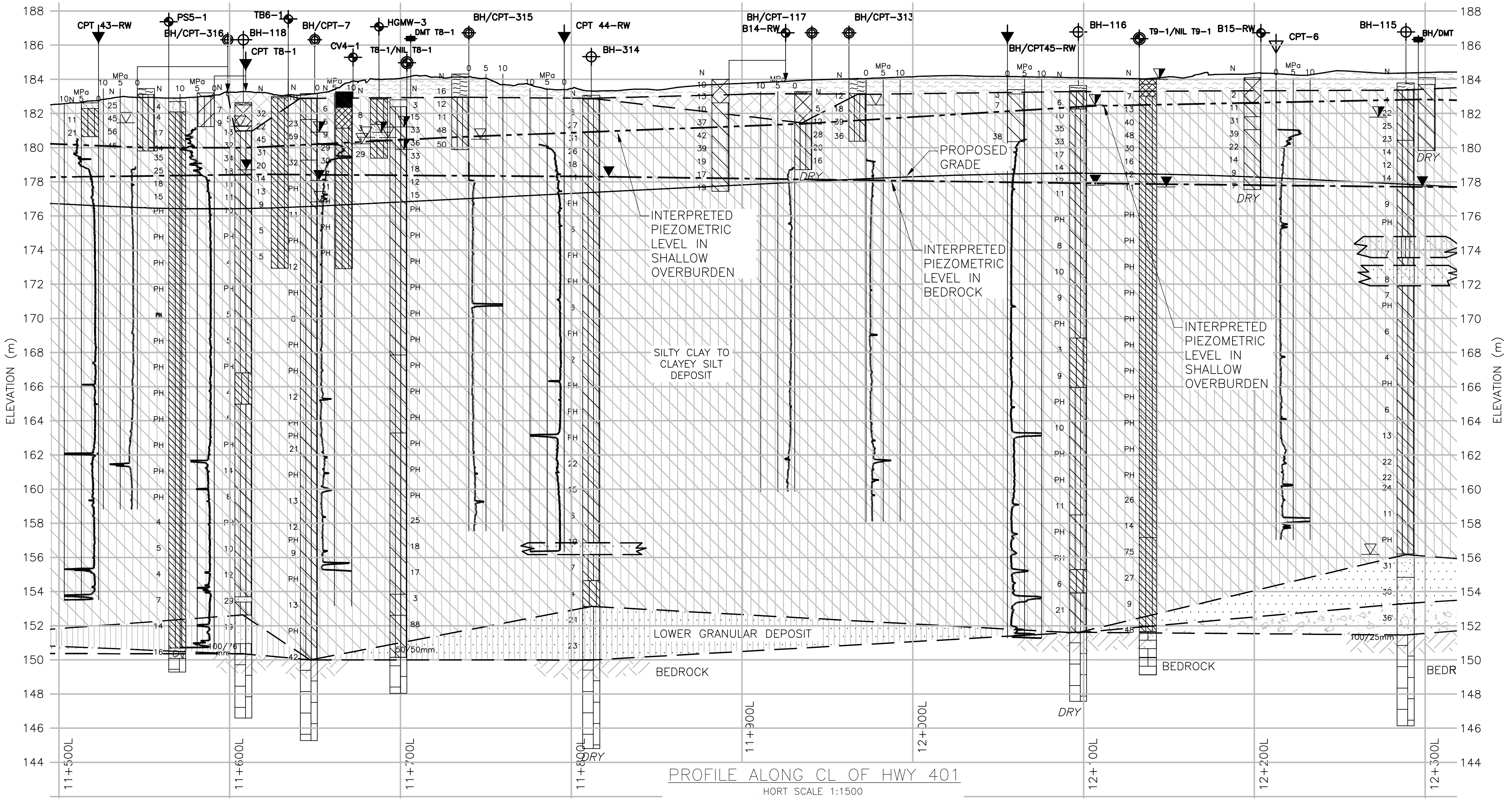
SHEET
G5301
Phase 1
IFC



KEY PLAN
SCALE
1 0 2 4Km

LEGEND

- ⊕ BOREHOLE – CURRENT INVESTIGATION
- ◉ BOREHOLE & NILCON VANE – CURRENT INVESTIGATION
- ◆ NILCON VANE – CURRENT INVESTIGATION
- ▼ CPT–CURRENT INVESTIGATION
- ▬ DMT–CURRENT INVESTIGATION
- ⊕ SW/SP HOLE (HYDROGEOLOGY)
- ⊕ BOREHOLE–PREVIOUS INVESTIGATIONS
- ◎ BOREHOLE, CPT & NILCON VANE – PREVIOUS INVESTIGATIONS
- ▽ CPT – PREVIOUS INVESTIGATIONS
- N SPT N–VALUE
- ▽ WATER LEVEL DURING DRILLING
- *DRY* BOREHOLE DRY DURING DRILLING
- ▼ WATER LEVEL (SHALLOW PIEZO)
- ▼ WATER LEVEL (DEEP PIEZO)
- PH – SAMPLE OBTAINED UNDER HYDRAULIC PRESSURE
- MPa 10 5 0 CPT, qc
- TOPSOIL/ ORGANICS
- FILL
- SAND
- SILTY CLAY
- SILTY SAND
- SILT
- SANDY SILT
- CLAYEY SILT
- SAND AND GRAVEL
- SILTY SAND AND GRAVEL
- LIMESTONE DOLOSTONE /BEDROCK



PROFILE ALONG CL OF HWY 401
HORT SCALE 1:1500
VERT SCALE 1:150

NOTES

1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
2. THE INTERPRETED STRATIGRAPHY REPRESENTS SIMPLIFIED SUBSURFACE CONDITIONS. SEE BORING LOGS FOR DETAILED STRATIGRAPHY. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN DEFINED AT BOREHOLE LOCATIONS ONLY. CONDITIONS BETWEEN BOREHOLE LOCATIONS COULD DIFFER FROM ILLUSTRATED CONDITIONS.
3. ELEVATIONS ARE REFERENCED TO GEODETIC DATUM. GROUND SURFACE ELEVATIONS WILL BE UPDATED AFTER SURVEYING. LOCATIONS ALONG THE PROPOSED WEP ARE REFERRING TO STATIONS IN LASALLE (L) SECTOR.

SCALES
15 0 30m
Horizontal
1.5 0 3
Vertical

DATE PLOTTED: 3/2/2012 3:24:26 PM
FILE LOCATION: C:\pwworking\hmmg_285380\stephen.labute@amec.com\dms09817\285380-04-090-WIP1-5301.dwg

METRIC

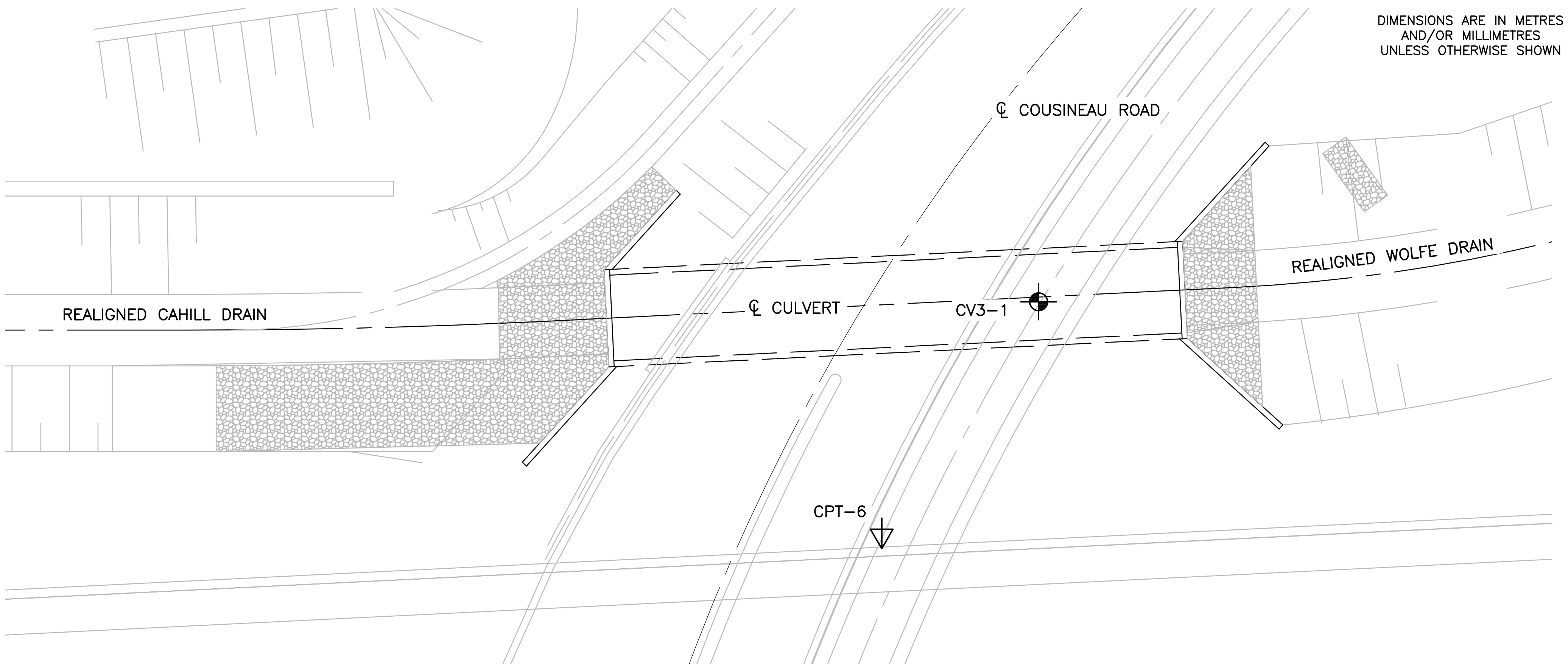
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

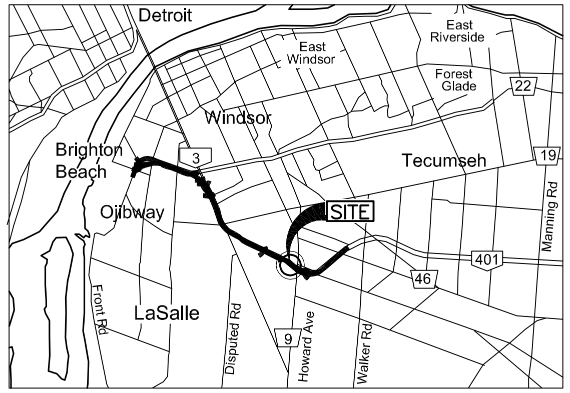
Windsor–Essex
Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
COUSINEAU ROAD CULVERT CV–3
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
G5302
Phase 1
IFC



PLAN
SCALE 1:200



KEY PLAN
SCALE
1 0 2 4Km

LEGEND

- ⊕ BOREHOLE CURRENT INVESTIGATION
- ⊕ BOREHOLE AND NILCON VANE CURRENT INVESTIGATION
- ◊ NILCON VANE CURRENT INVESTIGATION
- ▼ CPT – CURRENT INVESTIGATION
- ■ DMT – CURRENT INVESTIGATION
- ⊕ BOREHOLE PREVIOUS INVESTIGATION
- ◎ BOREHOLE, CPT AND NILCON VANE PREVIOUS INVESTIGATIONS
- ▽ CPT –PREVIOUS INVESTIGATION
- N SPT N–VALUE
- ⊕ SW/SP HOLE (HYDROGEOLOGY)
- 16 BLOWS/0.3m UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
- ☐ MHSG – MAGNETIC HEAVE/SETTLEMENT GAUGE
- ▯ P – VIBRATING WIRE PIEZOMETER
- *DRY* BOREHOLE DRY DURING DRILLING
- ▽ WATER LEVEL DURING DRILLING
- ▼ WATER LEVEL (SHALLOW PIEZO)
- ▼ WATER LEVEL (DEEP PIEZO)
- MPa 0 5 10 CPT–qc

NOTES

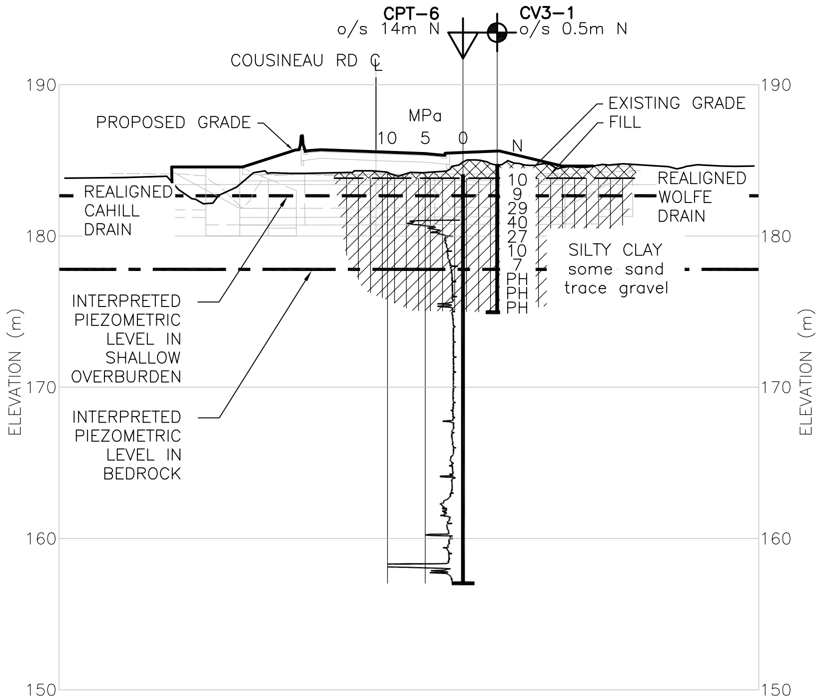
1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
2. THE INTERPRETED STRATIGRAPHY REPRESENTS SIMPLIFIED SUBSURFACE CONDITIONS. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN DEFINED AT BOREHOLE LOCATIONS ONLY. CONDITIONS BETWEEN BOREHOLE LOCATIONS COULD DIFFER FROM ILLUSTRATED CONDITIONS.
3. ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

LIST OF ABBREVIATIONS

PH – SAMPLER ADVANCED BY HYDRAULIC PRESSURE
PM – SAMPLER ADVANCED BY MANUAL PRESSURE
WH – SAMPLER ADVANCED BY STATIC WEIGHT OF HAMMER
WR – SAMPLER ADVANCED BY WEIGHT OF SAMPLER RODS

MATERIAL LEGEND

- ▨ TOPSOIL/ ORGANICS
- ▨ FILL
- ▨ SAND
- ▨ SILTY CLAY
- ▨ SILTY SAND
- ▨ SILT
- ▨ SANDY SILT
- ▨ CLAYEY SILT
- ▨ SAND AND GRAVEL
- ▨ SILTY SAND AND GRAVEL
- ▨ LIMESTONE DOLOSTONE /BEDROCK



PROFILE ALONG CL OF CULVERT
HORT SCALE 1:500
VERT SCALE 1:250

No.	ELEVATION	CO–ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC TESTHOLES			
CV3–1	184.5	4678630.0	333861.1
PREVIOUS TESTHOLES			
CPT–6	184.08	4678621.0	338844.0

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS			
02–MAR–12	0	TL	ISSUED FOR CONSTRUCTION
DATE	REV.	BY	DESCRIPTION

DESIGN TF	CHK NSV	CODE CAN/CSA S6–06	LOAD CL–625–ON
DRAWN MM	CHK TF	SITE 6–629	DATE 27–JUL–11

Figures

Figure 3.1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures (Figure 5.1, Ladd & DeGroot, 2004, ref. R-18)

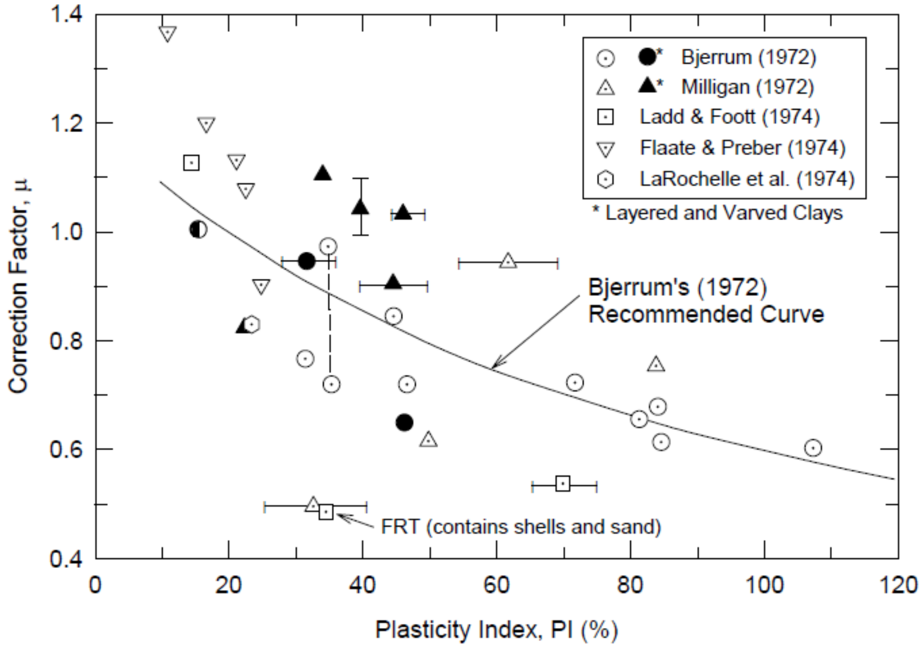
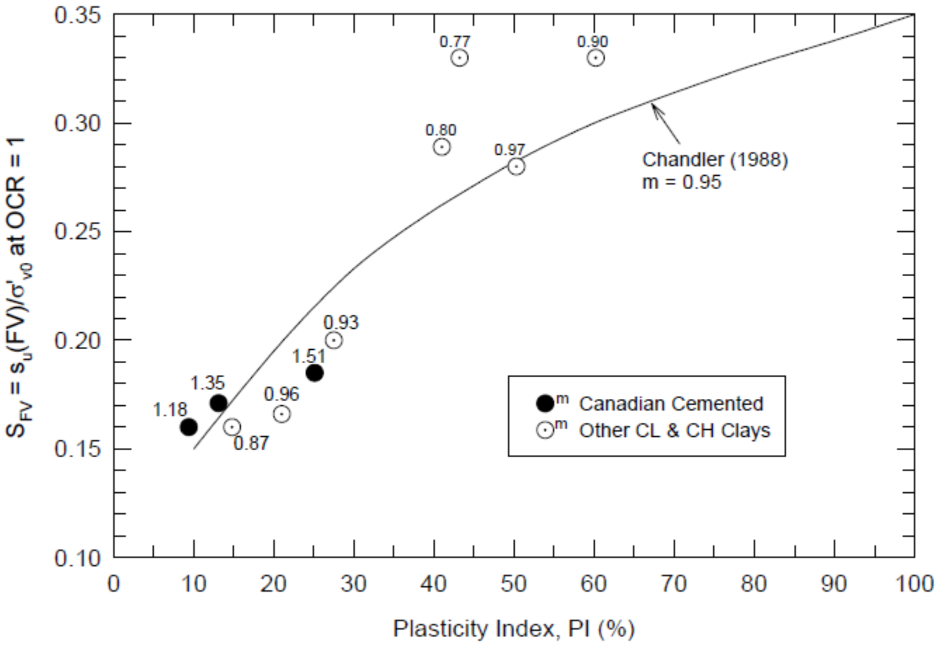
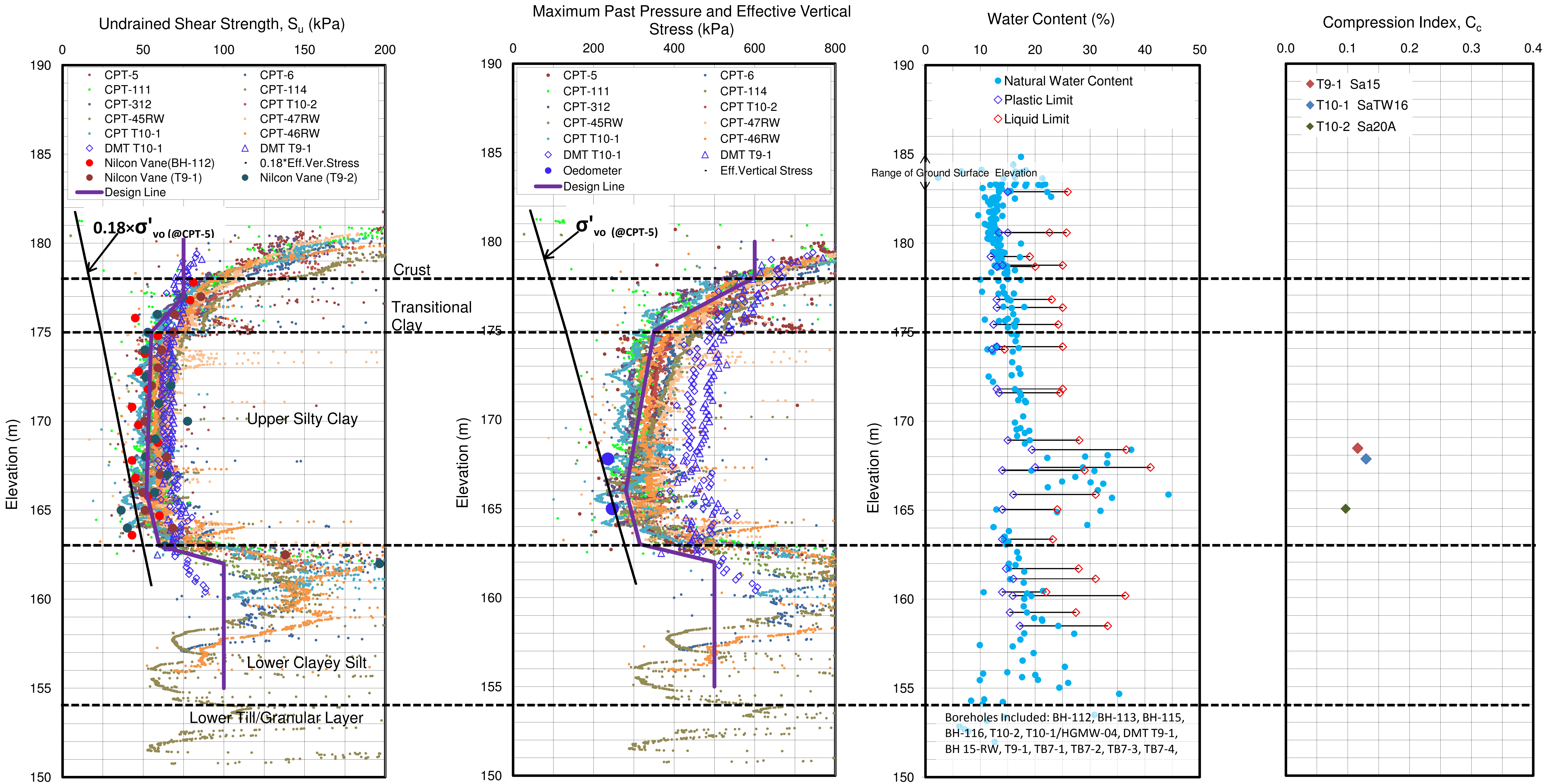


Figure 3.2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays (Figure 5.2, Ladd & DeGroot, 2004, ref. R-18)





Notes:
1. Shear strength profiles were estimated from CPT data using the equation $S_u = (q_t - \sigma_{VO}) / N_{KT}$. The cone factor N_{KT} was estimated by comparing the CPT profiles with a nearby Nilcon Vane profile.
2. Maximum past pressure profiles estimated using SHANSEP method. $OCR = [(S_u/\sigma'_V)/S]^{1/m}$
3. Data on the graph is dervied from the current investigation by AMEC and historic data from Golder Associates.

amec Earth & Environmental

CLIENT :

PROJECT:	**WINDSOR ESSEX PARKWAY**
TITLE:	SOIL PROPERTIES PROFILES STA.12+000L TO 12+800L

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.:	REV.
Sep 2011	SW8801.1002		3.3	

Figure 5.1: Data Summary of Compression Indices Cc, Cs and Cr
(Figure 6.7, Golder 2009, Ref. 9)

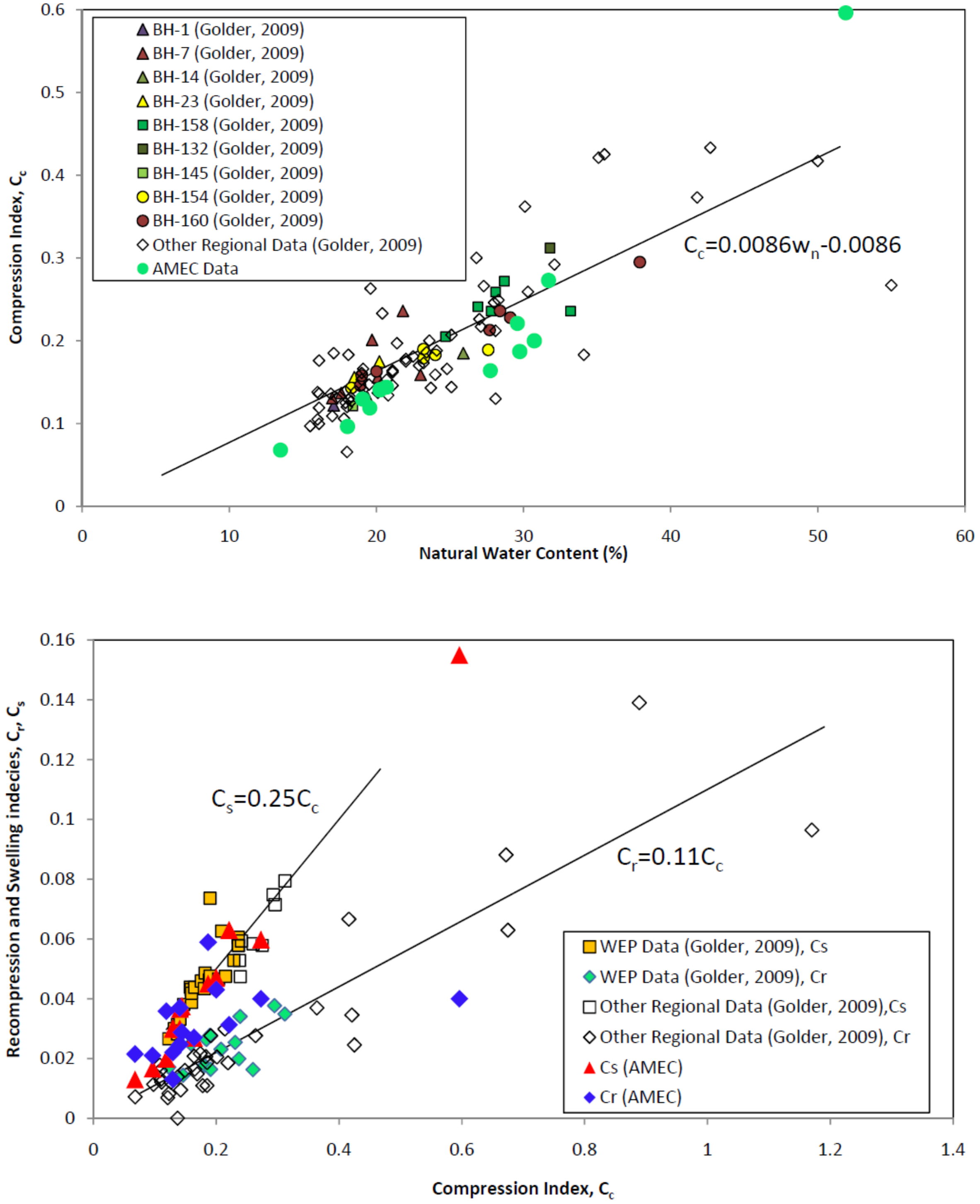
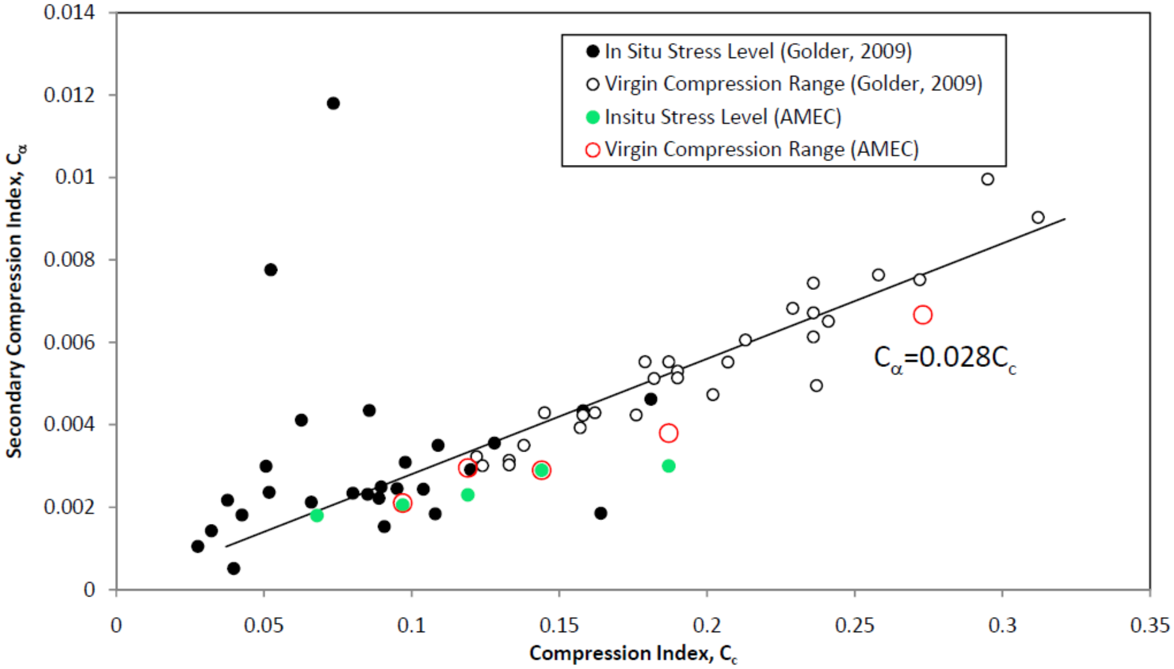
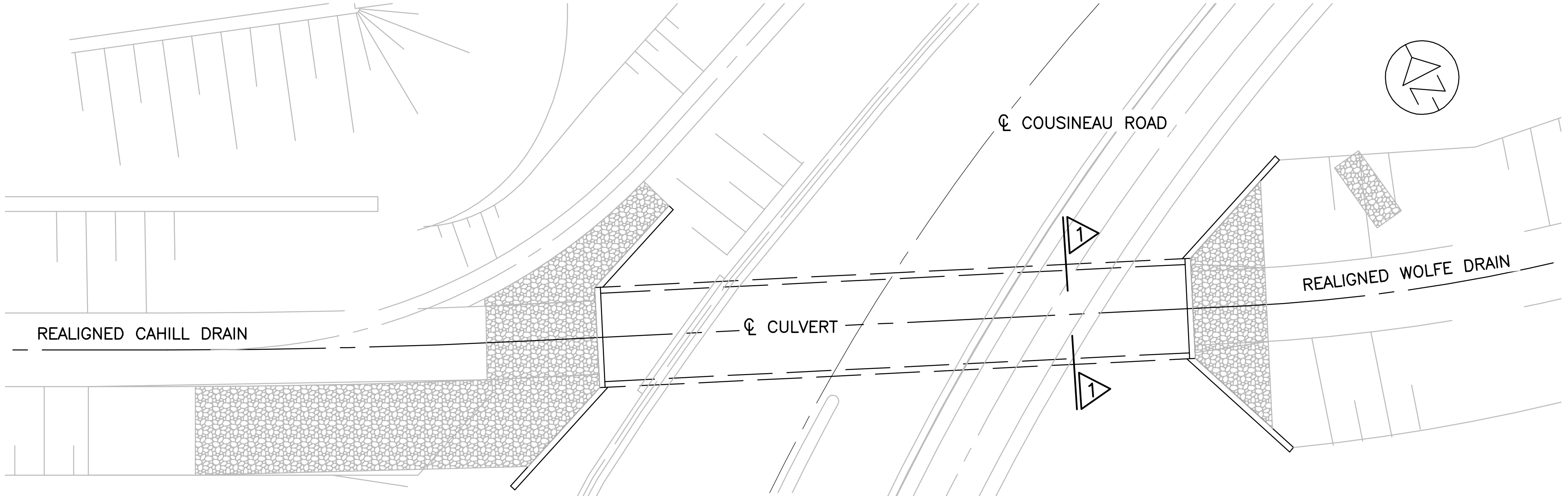
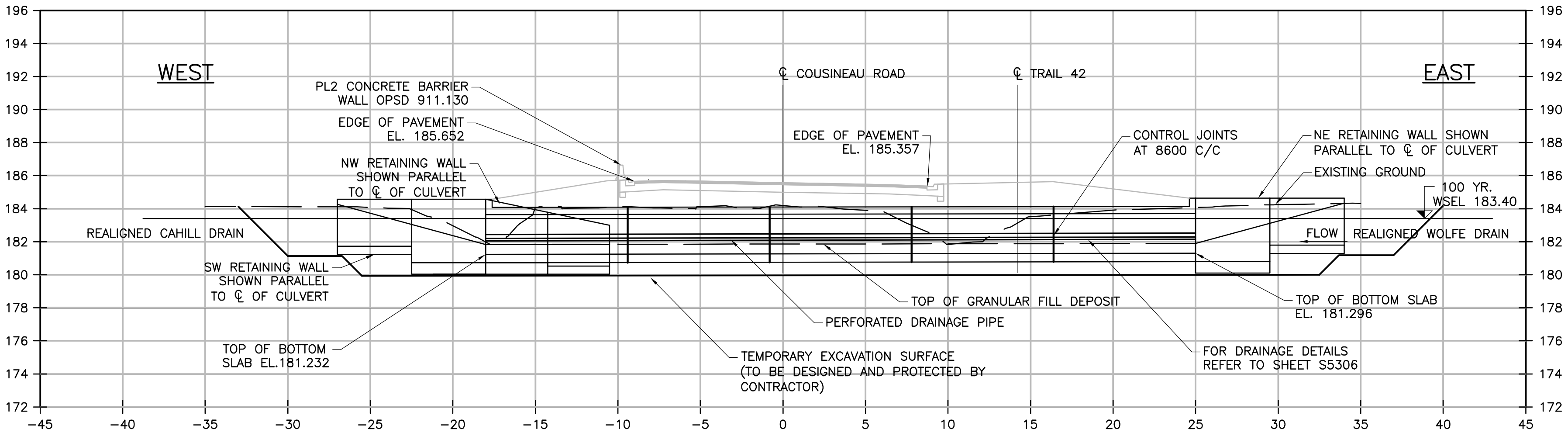


Figure 5.2: Data Summary of Compression Indices Cc and Cα
(Figure 6.7, Golder 2009, Ref. 9)

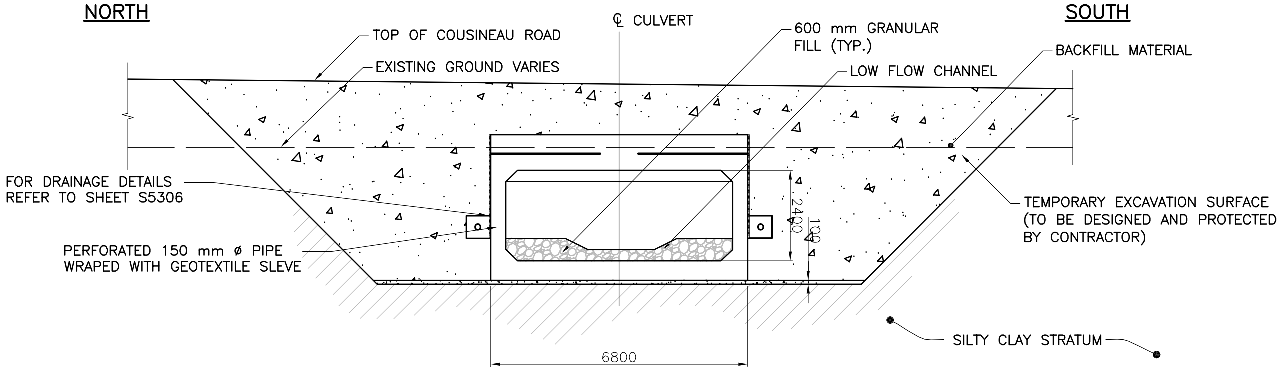




PLAN
HORT SCALE 1:200



PROFILE ALONG CL CULVERT CV–3
HORT SCALE 1:200
VERT SCALE 1:200



1
HORT SCALE 1:100
VERT SCALE 1:100

NOTES:

1. THIS FIGURE ILLUSTRATES THE GEOTECHNICAL DESIGN ARRANGEMENT OF CULVERT CV–3. THE DESIGN OF WOLFE DRAIN BEYOND THE CULVERT IS BY OTHERS.
2. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT AND CONSTRUCTION NOTES.
3. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE SHOWN. ELEVATIONS ARE REFERRED TO GEODETIC DATUM.
4. CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS. EXCAVATED CLAY SURFACES ARE SUSCEPTIBLE TO DETERIORATION AND EXPERIENCE DEFORMATIONS AND INSTABILITY. THEY ARE TO BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED AND TREATED AS REQUIRED.
5. THE EXPOSED SUBGRADE SHOULD BE INSPECTED AND UPON APPROVAL, A SUBGRADE PROTECTION LAYER OF AT LEAST 75 mm OF LEAN CONCRETE SHOULD BE PLACED SAME DAY AS EXCAVATED OVER THE AREAS TO SUPPORT CAST–IN–PLACE STRUCTURES.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

CULVERT CV–3
HWY 401 COUSINEAU ROAD CULVERT
CULVERT EXCAVATION AND BACKFILL DETAILS

DWG. BY: SJL	CHK. BY: TL	FIGURE NO.: 5.3
DATE: Mar–12	SHEET: 1 OF 1	

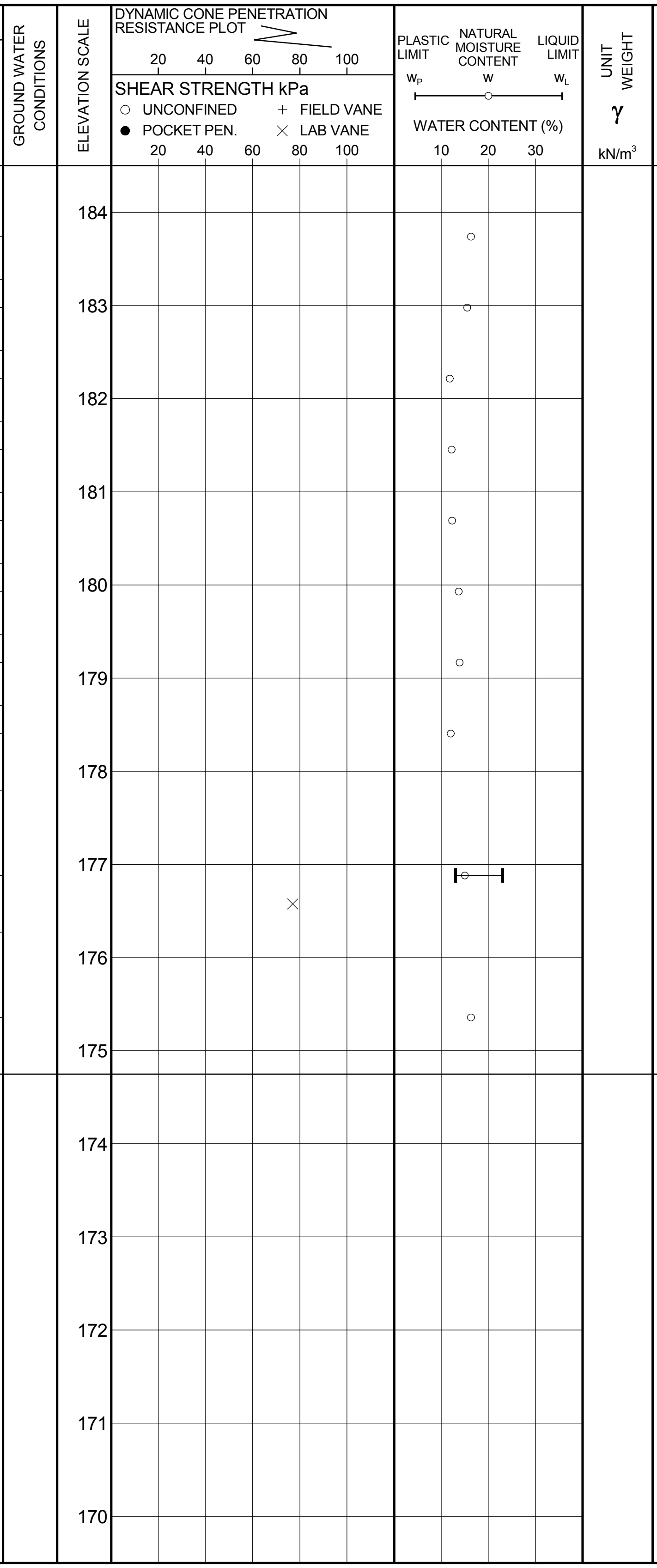
Appendix A Borehole, CPT and DMT logs from Additional Geotechnical Investigation

RECORD OF BOREHOLE No CV3-1

1 OF 1 **METRIC**

W.P. RFP No. 09-54-1007
LOCATION N4678630.0, E333861.1
ORIGINATED BY TA
DIST
HWY WEP
BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE Jul 12, 11 - Jul 12, 11
CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
184.5	Fill Surface							
0.0	Brown **FILL** Silty Clay Some sand, trace gravel Trace to some topsoil							
183.6 0.9	Mottled Brown and Grey **SANDY CLAYEY SILT** Some sand, trace gravel Stiff	1	SS	10				
		2	SS	9				
	Brown -Trace fissures, trace silt seams Very stiff	3	SS	29				
	Hard	4	SS	40				
	Very stiff	5	SS	27				
	Grey Stiff	6	SS	10				
		7	SS	7				
		8	TW	PH				
		9	TW	PH	1	32	42	26
		10	TW	PH				
174.7 9.8	**END OF BOREHOLE** (no refusal) Borehole dry on completion							



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

RECORD OF BOREHOLE No T9-1

1 OF 3 **METRIC**

W.P. RFP No. 09-54-1007
LOCATION N4678634.9, E333766.7
ORIGINATED BY NB
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE Jul 15, 11 - Jul 16, 11
CHECKED BY MSO

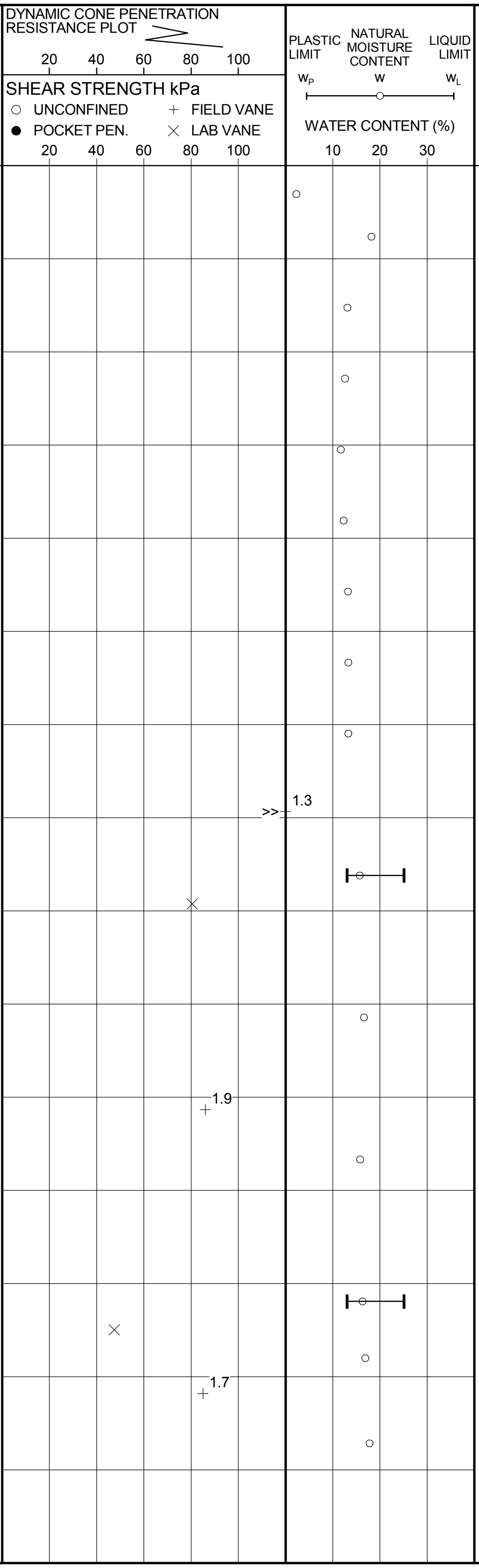
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
184.0	Pavement Surface
0.0 183.7 0.3	280mm **ASPHALT**
	Grey **FILL** Crushed Limestone Silty sand and gravel
183.0 1.0	Brown **SANDY CLAYEY SILT** Trace gravel Hard
	Grey
	Very stiff
	Stiff

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS						
2	SS	7					
3	SS	13					
4	SS	40					
5	SS	48					
6	SS	30					
7	SS	16					
8	SS	12					
9	SS	11					
	VT						
10	TW	PH	21.6	2	31	41	26
11	TW	PH					
	VT						
12	TW	PH					
13	TW	PH	21.5	2	32	40	26
	VT						
14	TW	PH					

GROUND WATER CONDITIONS



REMARKS & GRAIN SIZE DISTRIBUTION (%)

-Slope Inclinometer casing installed in sampled borehole; Vibrating Wire Piezometers (VWP) installed in adjacent boring at N4678635.6, E333769.0
-Spider Magnets (MG) installed in adjacent boring

-end of drilling July 15; continue July 16

-VWP #P9 and MG installed at 9.1m below ground surface

Continued Next Page

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

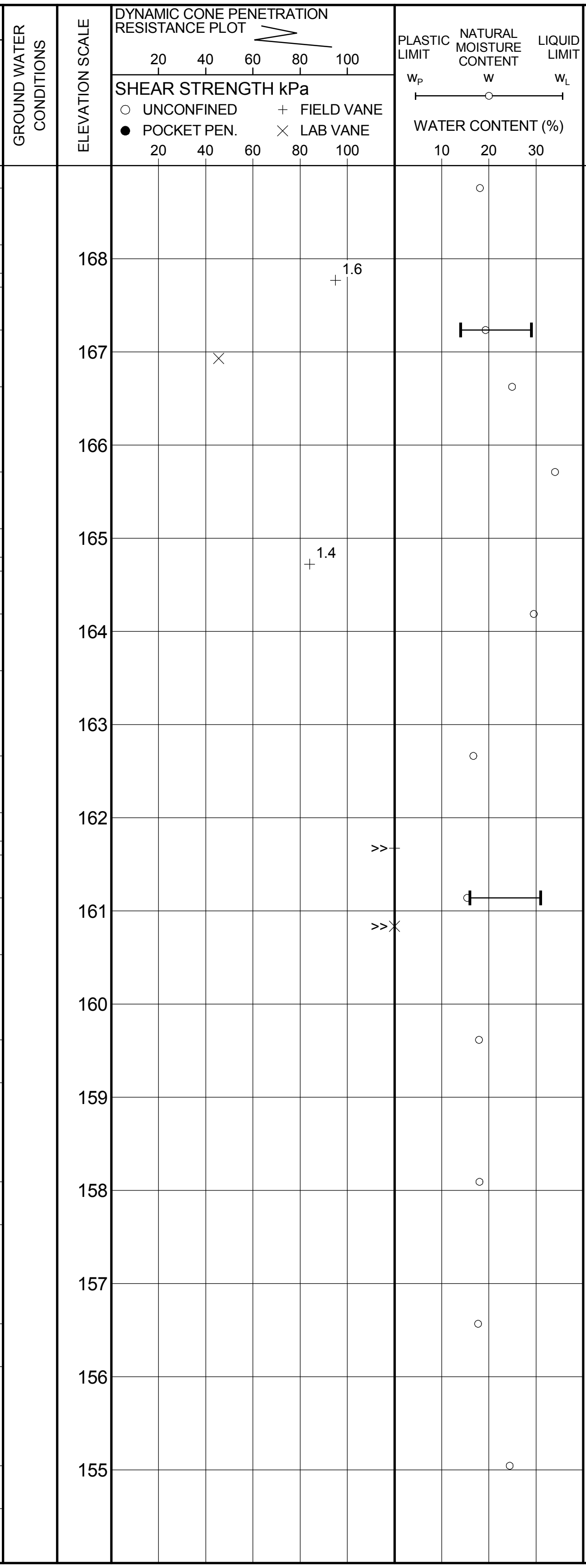
RECORD OF BOREHOLE No T9-1

2 OF 3

METRIC

W.P. RFP No. 09-54-1007
LOCATION N4678634.9, E333766.7
ORIGINATED BY NB
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE Jul 15, 11 - Jul 16, 11
CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
	Grey **SANDY CLAYEY SILT** Trace Gravel	15	TW	PH						
			VT							
		16	TW	PH	21.0	2	28	38	32	
		17	TW	PH						
			VT							
		18	TW	PH						
		19	SS	PH						-no recovery with shelby tube; Sample retrieved by pushing split spoon -Attempt at vane shear test exceeded max torque of apparatus.
			VT							
	Stiff	20	TW	PH	21.8	2	23	44	31	-MG installed at 22.97m below ground surface
		21	SS	26						
		22	SS	14						
157.2 26.8	Grey **SILTY CLAY**									
		23	SS	75						
	Soft to firm, interbedded with layers of compact/dense silt	24	SS	27						



Continued Next Page

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

RECORD OF BOREHOLE No T9-1

3 OF 3 **METRIC**

W.P. RFP No. 09-54-1007	LOCATION N4678634.9, E333766.7		ORIGINATED BY NB
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic		DATE Jul 15, 11 - Jul 16, 11	CHECKED BY MSO

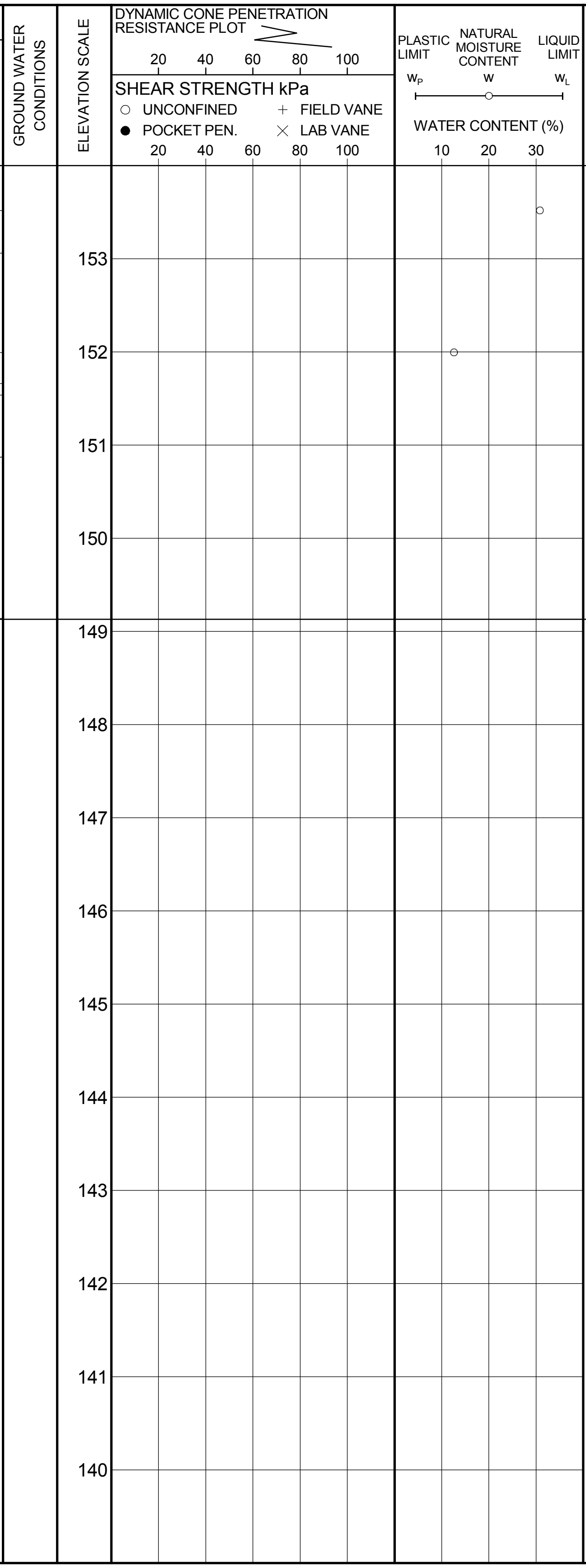
ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Grey **SILTY CLAY** *(continued)* Stiff	25	SS	9		
152.0 32.0	Grey **SANDY CLAYEY SILT** Trace some gravel Hard	26	SS	48		

151.7
32.3
	Grey
	LIMESTONE
	Fine Grained, fossiliferrous
	Pitted with black inclusions, porous and fractured at locations 32.6m-32.7m and rubble between 33.3m-33.8m.
		27	RC			-VWP #P33 installed at 32.6m below ground surface
						RQD = 87%
						TCR = 97%
						SCR = 65%

150.1 33.9	Grey to Brown **LIMESTONE** Laminated, pitted	28	RC			RQD = 70% TCR = 95% SCR = 63%
149.1 34.9	**END OF BOREHOLE**					

Piezometric Levels at VWP #P9:
August 6, 2011: EL. 183.9m
August 29, 2011: EL. 184.0m

Piezometric Levels at VWP #P33:
August 6, 2011: EL. 177.5m
August 29, 2011: EL. 177.7m



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

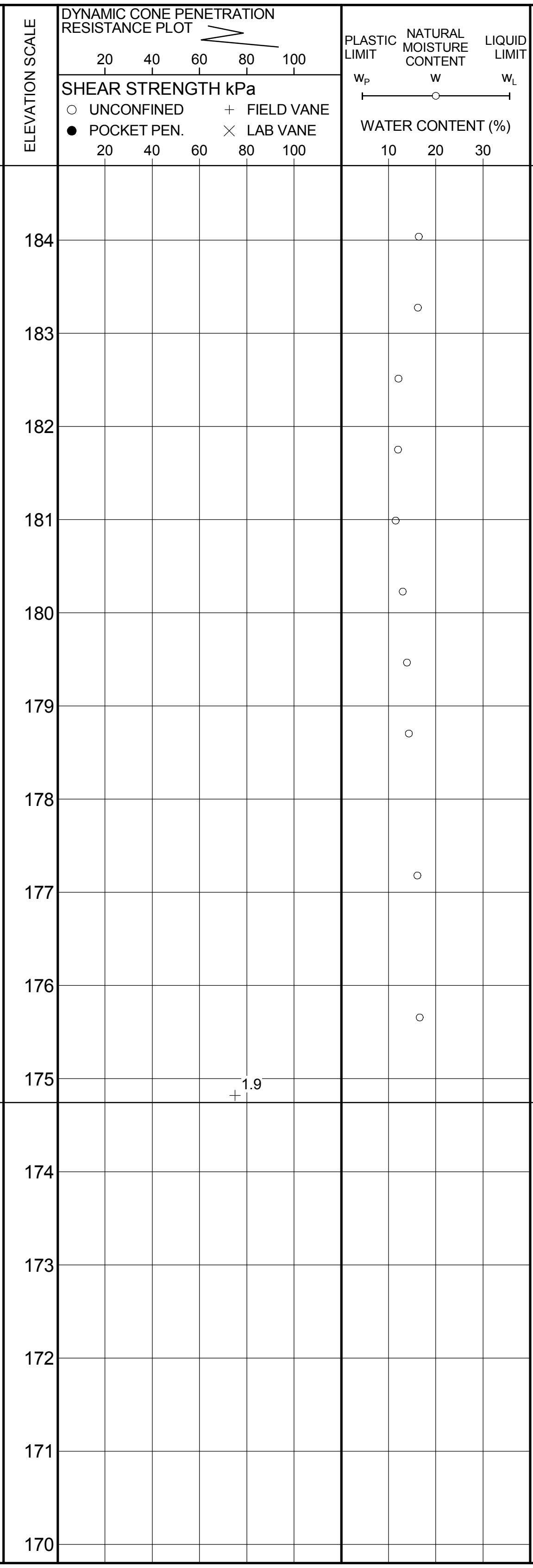
RECORD OF BOREHOLE No TB7A-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007
LOCATION N4678506.6, E334190.2
ORIGINATED BY TA
DIST
HWY WEP
BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE Jul 14, 11 - Jul 14, 11
CHECKED BY MSO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		
184.8	Ground Surface					
0.0	**TOPSOIL**					
184.2 0.6	Mottled Brown-Grey **CLAYEY SILT** Some sand, trace gravel Stiff	1	SS	8		
	Brown - Trace fissures	2	SS	14		
	Very stiff	3	SS	26		
	Hard	4	SS	35		
	Grey, very stiff Stiff	5	SS	18		
		6	SS	12		
		7	SS	9		
		8	SS	7		
			VT			-attempt at vane shear test exceeded max of apparatus (approx. 115. kPa)
	Firm to stiff	9	SS	6		
		10	SS	3		
			VT			
174.7 10.1	**END OF BOREHOLE** (no refusal) Borehole dry on completion					



UNIT WEIGHT γ kN/m^3

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

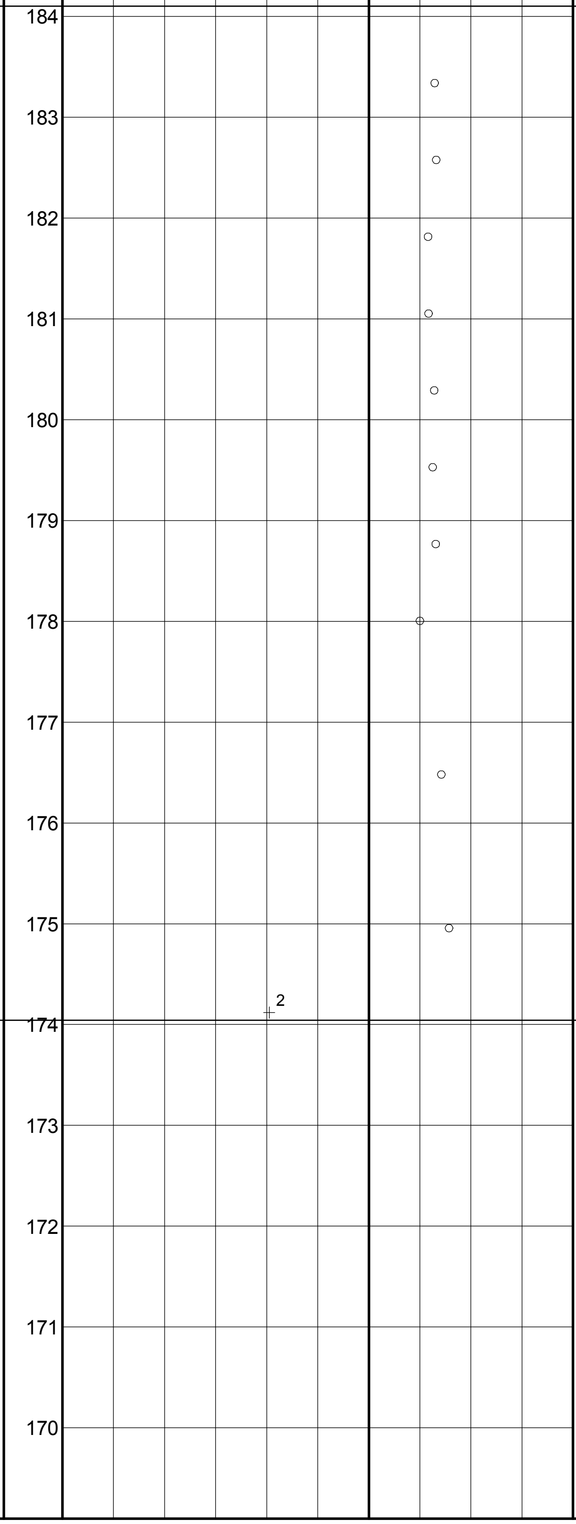
RECORD OF BOREHOLE No TB7-2

1 OF 1 **METRIC**

W.P. RFP No. 09-54-1007	LOCATION N4678662.3, E333859.6		ORIGINATED BY TA
DIST	HWY WEP	BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE Jul 10, 11 - Jul 10, 11		CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
184.1	Ground Surface				
0.0	**TOPSOIL**				
183.8 0.3	Mottled Brown-Grey **CLAYEY SILT** Some sand, trace gravel Stiff	1	SS	12	
	Trace fissures, very stiff	2	SS	25	
	Hard	3	SS	37	
	Trace to some oxidized fissures, very stiff	4	SS	28	
	Grey, stiff	5	SS	15	
		6	SS	10	
		7	SS	9	
		8	SS	9	
		9	SS	5	
			VT		-driller found it difficult to push vane
		10	SS	5	-corrosivity sample
			VT		
174.0 10.1	**END OF BOREHOLE** (no refusal) Borehole dry on completion				

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



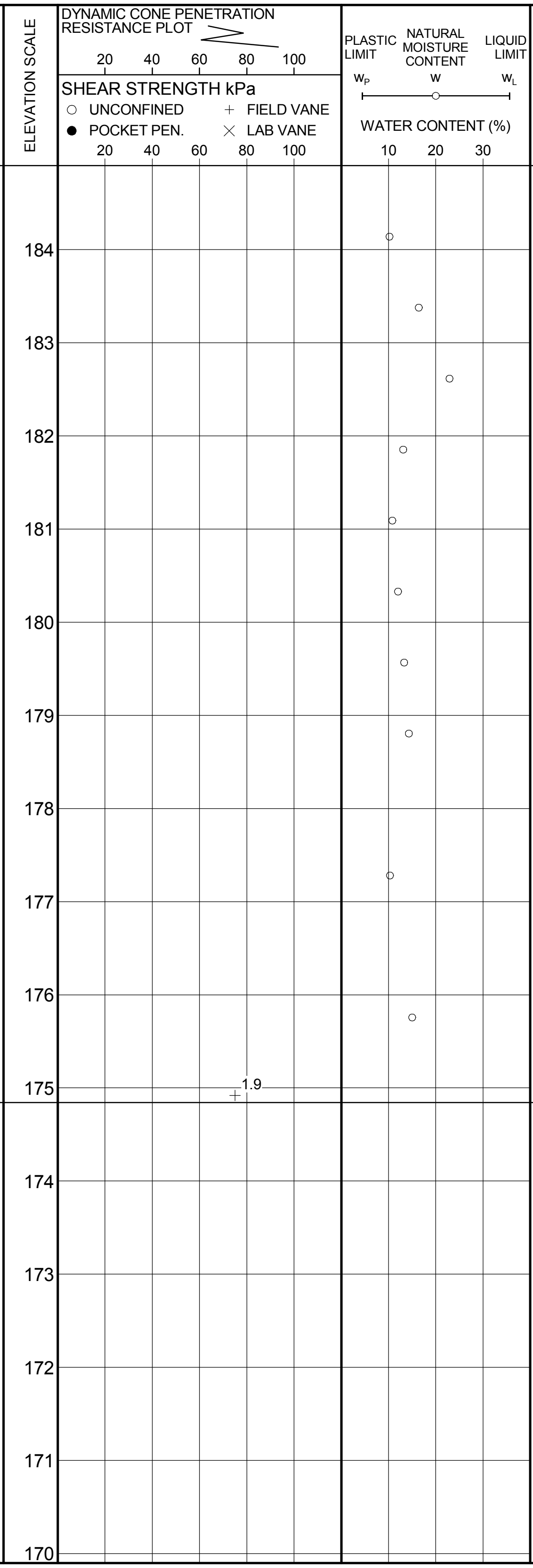
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No TB7-3

1 OF 1 **METRIC**

W.P.	RFP No. 09-54-1007	LOCATION	N4678644.6, E333911.0	ORIGINATED BY	TA
DIST		HWY WEP	BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	Jul 12, 11 - Jul 12, 11	CHECKED BY	MSO

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.9	Fill Surface						
184.7 0.0	Topsoil						
0.2	Brown **FILL** Silty clay and topsoil	1	SS	6			
183.8 1.1	Mottled Brown-Grey **CLAYEY SILT**	2	SS	7			
	Brown, firm, moist to wet Trace fissures, very stiff below approx. 2.5m	3A, B	SS	6			
		4	SS	25			
	Hard	5	SS	37			
	Grey, very stiff	6	SS	20			
	Stiff	7	SS	10			
		8	SS	9			
		9	SS	9			
		10	SS	5			-corrosivity sample taken
			VT				
174.8 10.1	**END OF BOREHOLE** (no refusal) Borheole dry on completion						



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

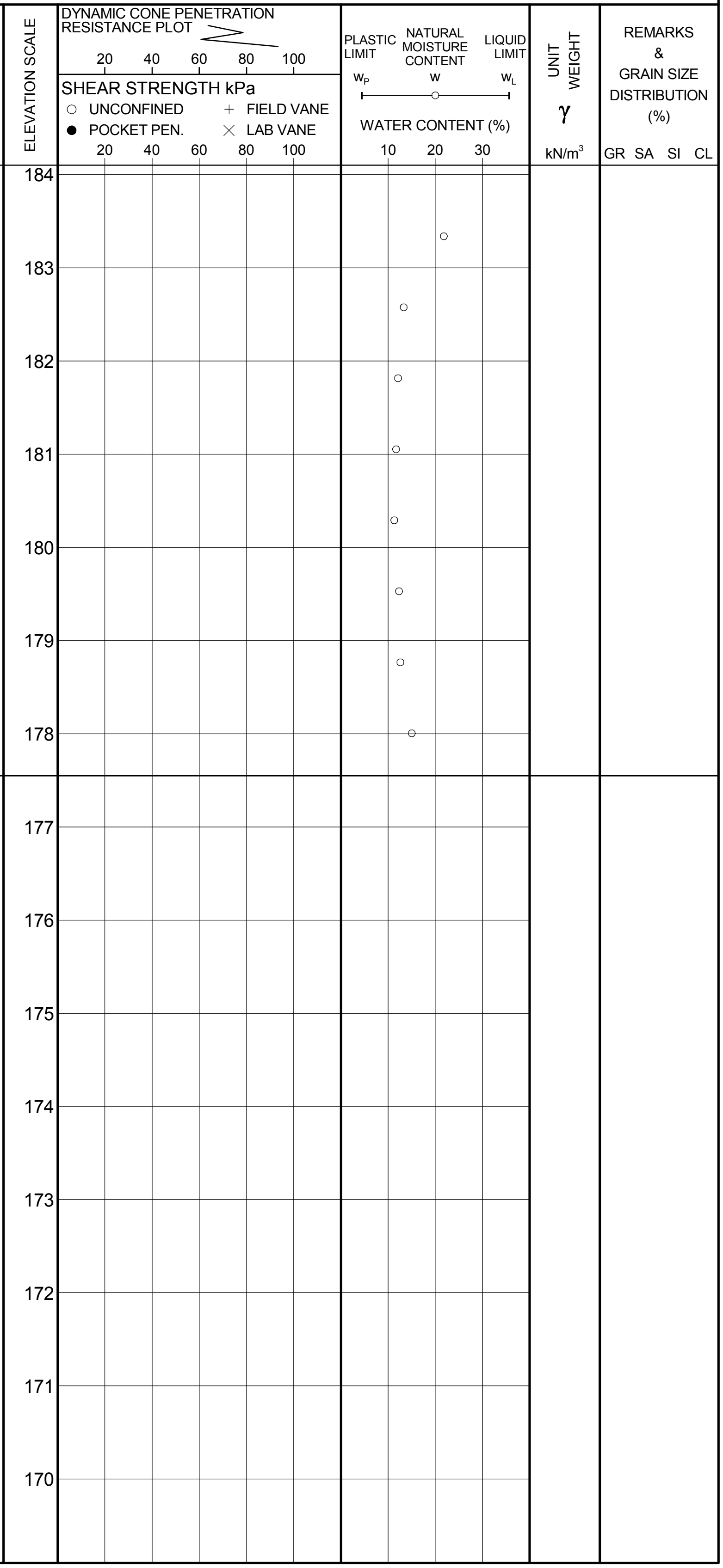
RECORD OF BOREHOLE No BH15-RW

1 OF 1

METRIC

W.P. RFP No. 09-54-1007
LOCATION N4678559.2, E333806.1
ORIGINATED BY LC
DIST
HWY WEP
BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE Jul 15, 11 - Jul 15, 11
CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
184.1	Fill Surface					
184.0 0.1	**FILL** Silty Topsoil to Mixed Clay/Silt/Sand/Roots/concrete					
	FILL Soft Clay/Topsoil		1	SS	2	
182.6 1.5	Brown **SILTY CLAY** Weathered, fissures Some sand, trace gravel		2	SS	11	
181.8 2.3	Brown-Grey **SANDY CLAYEY SILT** Fissured, hard, dry		3	SS	31	
181.1 3.0	Brown **CLAYEY SILT** Some sand, trace gravel hard, dry		4	SS	39	
	Grey Very stiff		5	SS	22	
	Stiff		6	SS	14	
			7	SS	9	
	Firm		8	SS	7	
177.5 6.6	**END OF BOREHOLE** (no refusal) Borehole dry on completion					



+ 3, × 3: Numbers refer to Sensitivity

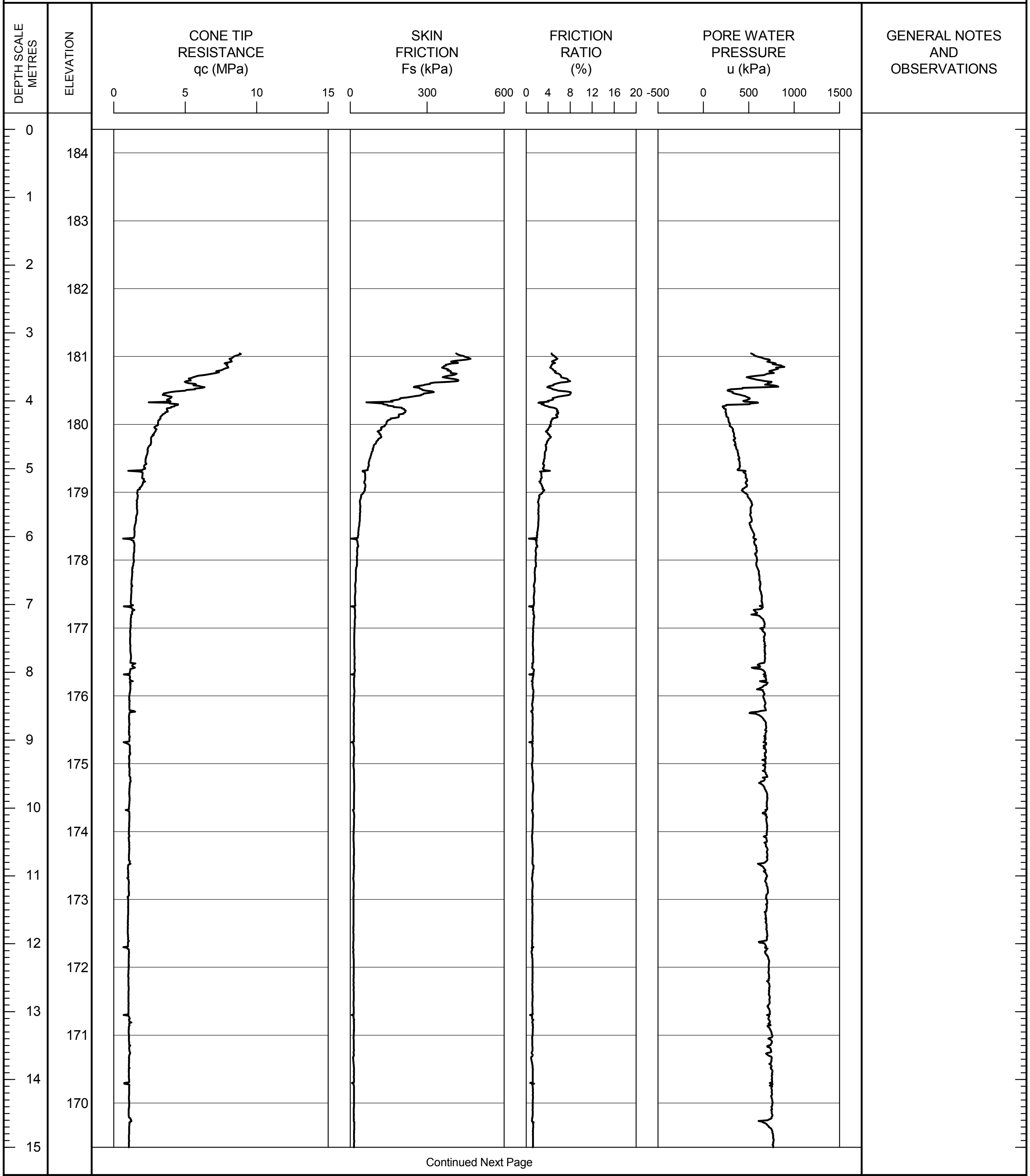
○ 3% STRAIN AT FAILURE

RECORD OF CONE PENETRATION TEST CPT 46-RW

METRIC

PROJECT	Windsor-Essex Parkway	TEST DATE	8/5/2011 - 8/5/2011	SHEET	1 OF 2
LOCATION	N4678505.0; E333977.6			DATUM	Geodetic

GROUND SURFACE ELEVATION: 184.3 PREDRILL DEPTH: 3 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

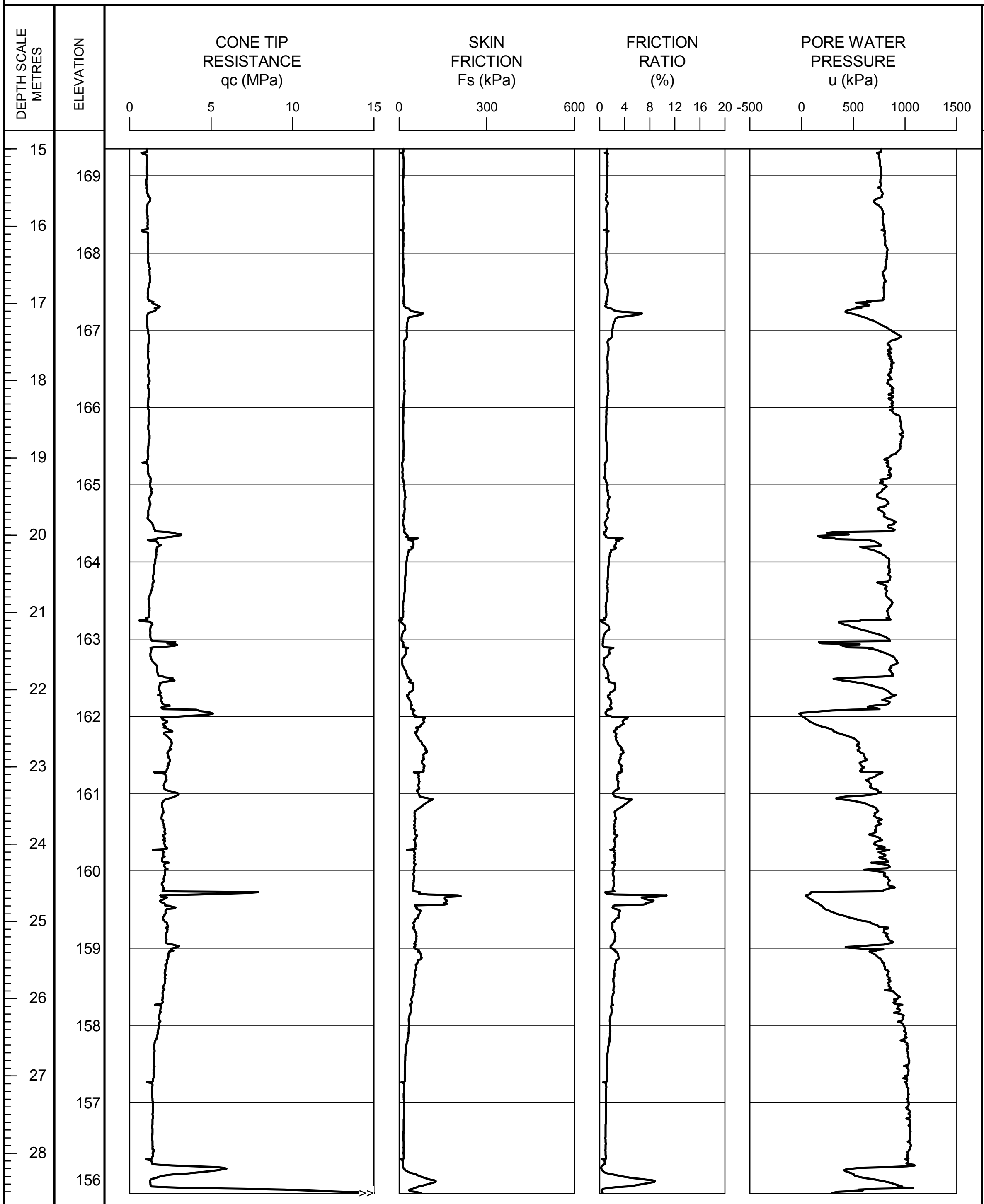
CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 46-RW

METRIC

PROJECT	Windsor-Essex Parkway	TEST DATE	8/5/2011 - 8/5/2011	SHEET	2 OF 2
LOCATION	N4678505.0; E333977.6			DATUM	Geodetic

GROUND SURFACE ELEVATION: 184.3 PREDRILL DEPTH: 3 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



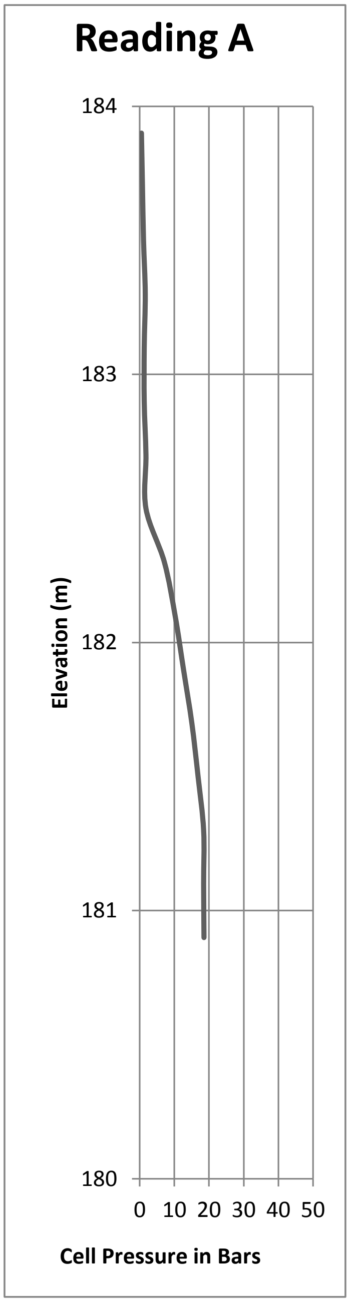
GENERAL NOTES AND OBSERVATIONS

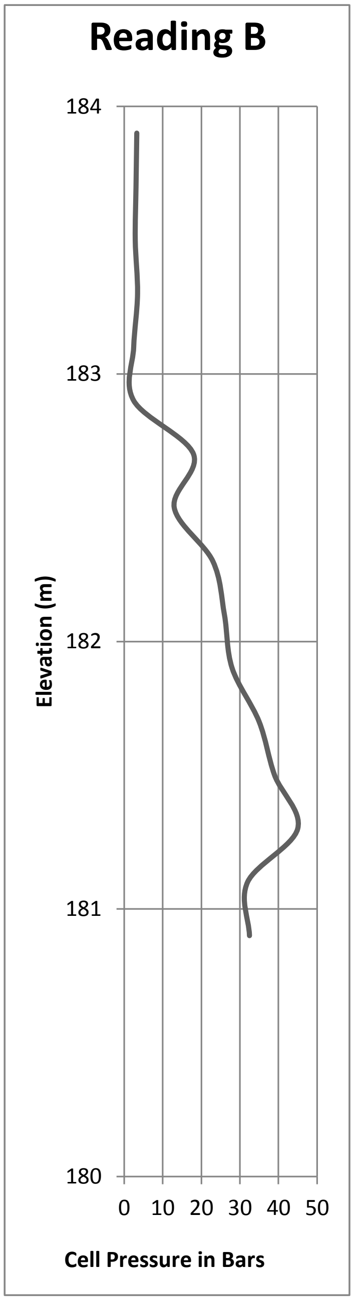
OPERATOR: TA

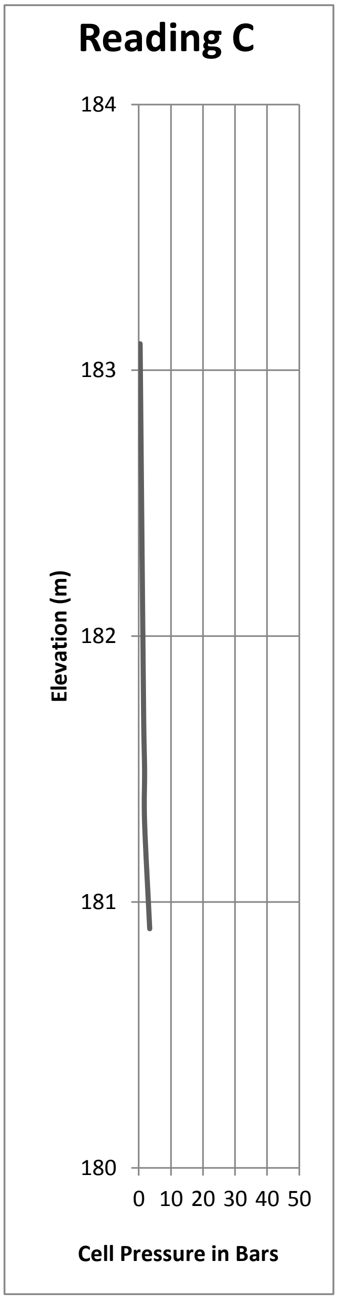
CHECKED: DD

RECORD OF DILATOMETER TEST DMT T9-1-SHALLOW

Project : Windsor-Essex Parkway	Test Date: 7/19/2011	Sheet 1 of 1
Location: N 4678544.5; E 333900.9	Predrill Depth : 0.2 m	Datum Geodetic
Ground Surface Elevation : 184.1	Delta A: 0.14 Bar	Delta B: 0.22 Bar


Reading A


Reading B


Reading C

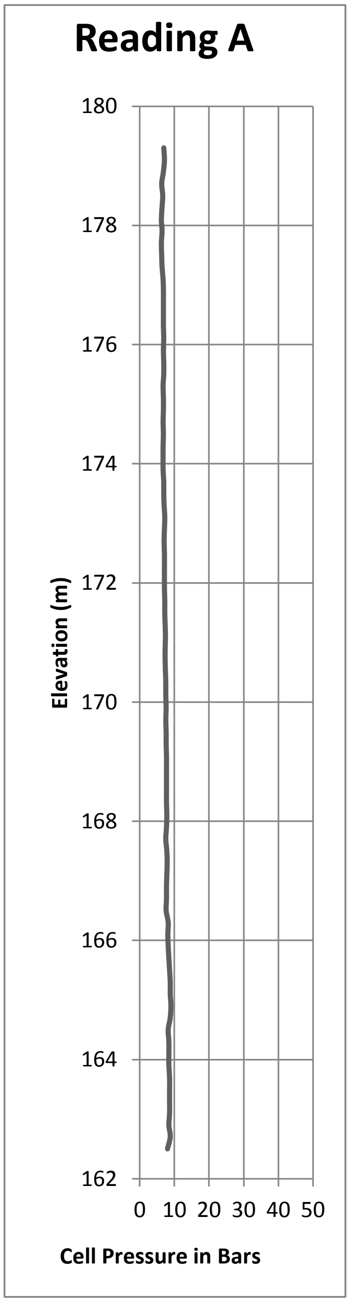
Note: DMT refusal at elevation 180.9m .Redrill to elevation 179.5m
Resumed DMT to elevation 162.5m

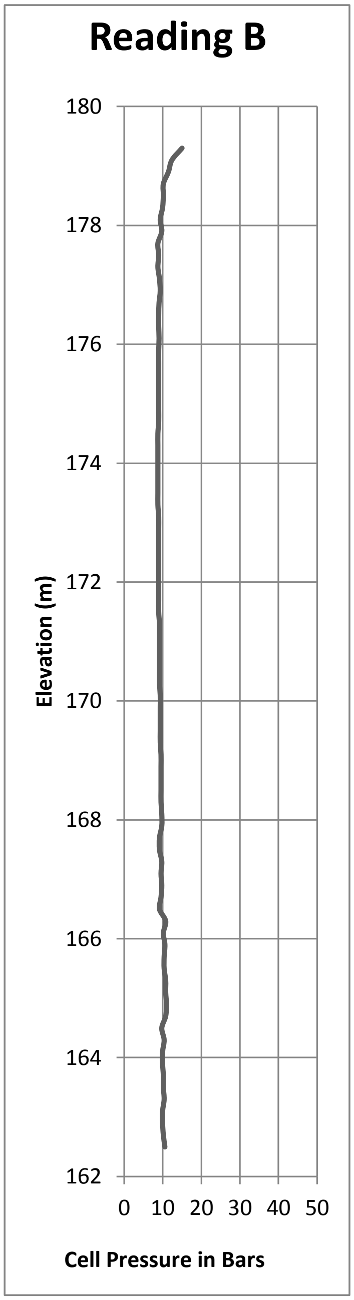
Operator: LC

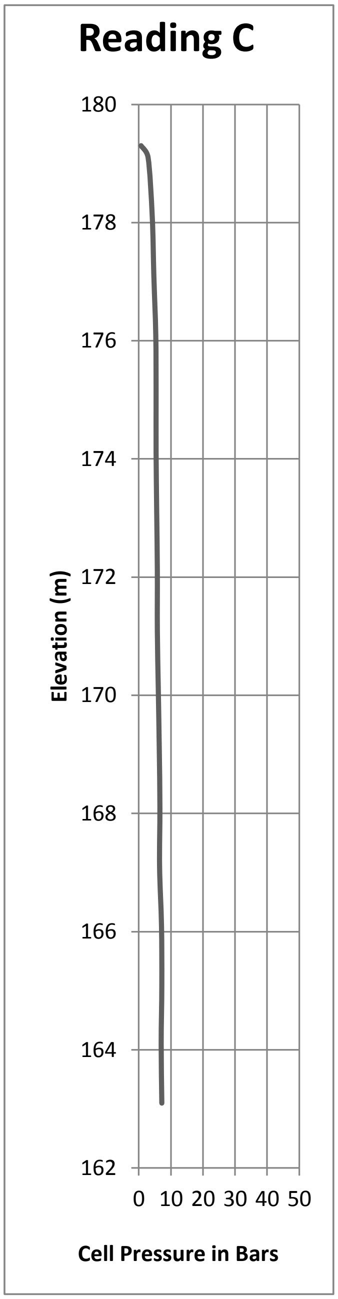
Checked: DD

RECORD OF DILATOMETER TEST DMT T9-1-DEEP

Project : Windsor-Essex Parkway	Test Date: 7/19/2011	Sheet 1 of 1
Location: N 4678544.5; E 333900.9	Predrill Depth : 4.6 m	Datum Geodetic
Ground Surface Elevation : 184.1	Delta A: 0.10 Bar	Delta B: 0.37 Bar


Reading A


Reading B


Reading C

Operator: LC

Checked: DD

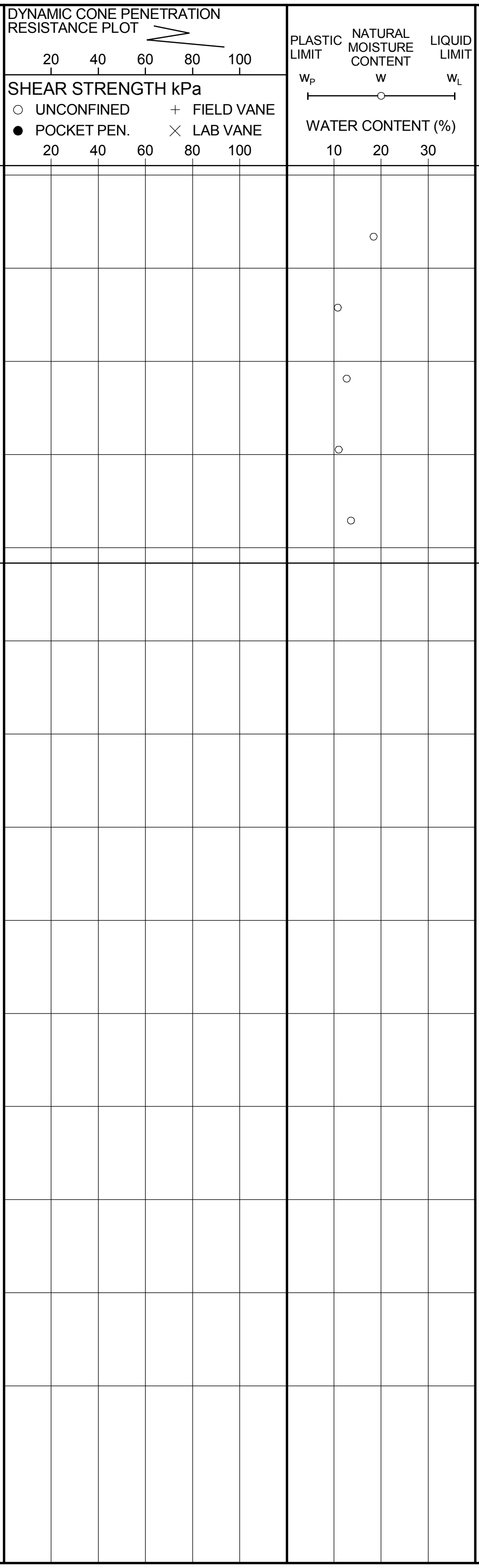
RECORD OF BOREHOLE No DMT T9-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007
LOCATION N4678544.5, E333900.9
ORIGINATED BY LC
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE Jul 19, 11 - Jul 19, 11
CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.1	Ground Surface								
0.0 183.7	Clayey **TOPSOIL**						184		
0.4	Brown-Grey **SILTY CLAY/CLAYEY SILT**								
			1	AS			183		
	Brown-Grey -Weathered fissures -Some sand, trace gravel with topsoil/organics in fissures		2	SS					-hydrocarbon odour
	-Brown fissures Stiff		3	SS			182		-spoon driven into hole after DMT removed. Sampled/Blow Counts disturbed/affected
	-Oxidized Hard		4	SS			181		
179.8 4.3	Grey -Silty fissures Oxidized clay Very stiff		5	SS			180		
	END OF SAMPLED BOREHOLE (Continue with DMT to refusal)						179 178 177 176 175 174 173 172 171 170		



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

Appendix B Borehole Logs from Previous Investigations

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 115

1 OF 4 **METRIC**

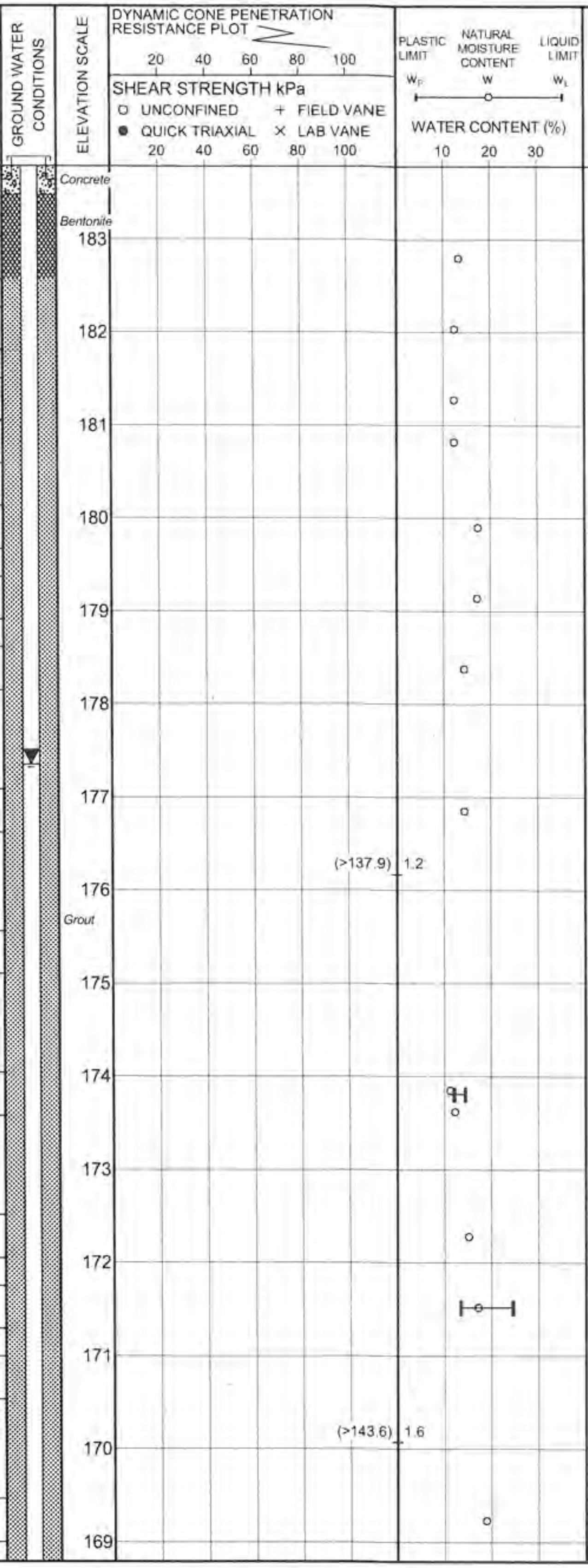
PROJECT 07-1130-207-0			
W.P.	LOCATION N 4678585.3 ;E 333911.1	ORIGINATED BY MA
DIST WEST	HWY 401/3	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC	COMPILED BY BRS
DATUM GEODETIC		DATE February 15, 2008 - February 21, 2008	CHECKED BY SJB

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
183.79	GROUND SURFACE
0.00	TOPSOIL, silty Brown
183.36 / 0.43	CLAYEY SILT, some sand, trace gravel Soft to very stiff Brown
180.44 / 3.35	CLAYEY SILT, some sand, trace gravel Stiff Grey
174.80 / 8.99	SANDY SILT, some clay, trace gravel Loose Grey
173.58 / 10.21	CLAYEY SILT, some sand, trace gravel Firm Grey
173.12 / 10.67	SAND, trace gravel, trace silt Loose Grey
171.90 / 11.89	CLAYEY SILT, some sand, trace gravel Soft to very stiff Grey

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	22	
3	SS	25	
4	SS	23	
5	SS	14	
6	SS	12	
7	SS	14	
8	SS	9	
9	TO	PH	
10	SS	7	3 43 39 14
11	SS	8	(9)
12	SS	7	2 30 40 28
13	TO	PH	
14	SS	6	



UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$○^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 115

2 OF 4 **METRIC**

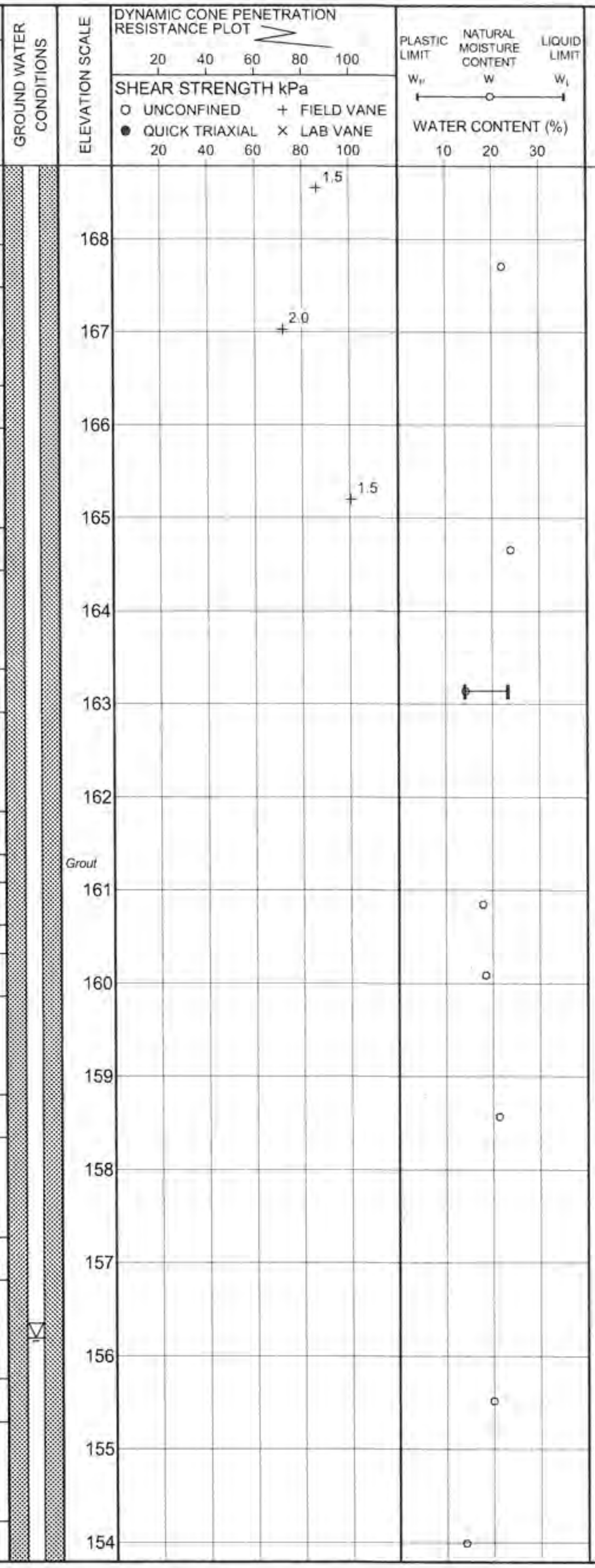
PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4678585.3 ;E 333911.1
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE February 15, 2008 - February 21, 2008
ORIGINATED BY MA
COMPILED BY BRS
CHECKED BY SJB

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
	CLAYEY SILT, some sand, trace gravel Soft to very stiff Grey
156.21 27.58	SAND, trace sand, trace gravel, trace clay Dense Grey
154.83 28.96	SAND, trace gravel Compact to dense Grey

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
15	SS	4	
16	TO	PH	
17	SS	6	
18	SS	13	4 25 47 24
19	SS	22	
20	SS	22	
21	SS	24	
22	SS	11	
23	TO	PH	
24	SS	31	1 86 8 5
25	SS	30	



UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 115

3 OF 4

METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4678585.3 ;E 333911.1	ORIGINATED BY MA	
DIST WEST HWY 401/3	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC	COMPILED BY BRS	
DATUM GEODETIC	DATE February 15, 2008 - February 21, 2008	CHECKED BY SB	

SOIL PROFILE · SAMPLES · GROUND WATER CONDITIONS · ELEVATION SCALE · DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 · SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 · PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L · WATER CONTENT (%) 10 20 30 · UNIT WEIGHT γ kN/m³ · REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	GR SA SI CL
153.31 30.48	SAND AND GRAVEL, trace silt Dense Grey				153				
		26	SS	36	Grout				25 66 6 3
					152				
151.48 32.31	LIMESTONE, fresh, medium strong, laminated, fine grained Light grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	27	SS	100/ 25mm	151 Bentonite				
		28	NQ RC		150	96	90	86	
		29	NQ RC		149 Sand 148	100	100	100	
		30	NQ RC		Screen 147	100	97	86	UC
146.15 37.64	END OF BOREHOLE								

Water level in borehole at about elev. 156.19m during drilling on February 21, 2008.

Water level measured in deep piezometer at elev. 178.00m on February 21, 2008.

Water level measured in deep piezometer at elev. 178.10m on March 20, 2008.

Water level measured in deep piezometer at elev. 177.69m on July 24, 2008.

Water level measured in deep piezometer at elev. 175.99m on September 19, 2008.

Water level measured in deep piezometer at elev. 177.25m on November 14, 2008.

Water level measured in deep piezometer at elev. 177.35m on January 28, 2009.

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

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 115

SHEET 4 OF 4

LOCATION: N 4678585.3 ;E 333911.1

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: February 15, 2008 - February 21, 2008

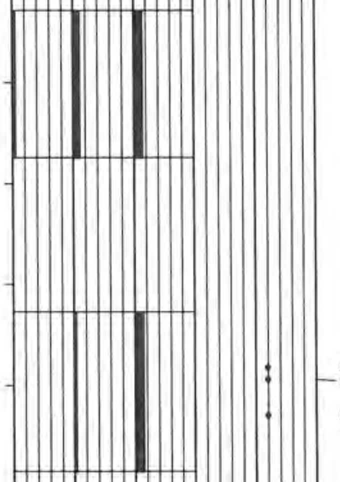
DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DATUM: GEODETIC

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	Br - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	NOTE: For additional abbreviations refer to list of abbreviations & symbols
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular		

DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION
	ROCK SURFACE		151.48 32.31		
MUD ROTARY NO ROCK CORE	LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous to porous, light brown to grey, fossiliferous		149.99 33.80	1	
	LIMESTONE, fresh, medium strong, thinly laminated, medium grain, faintly porous to porous, light grey		149.53 34.26		
	LIMESTONE, fresh, medium strong, laminated, fine grained, porous to vuggy, light grey		148.80 34.99		
	LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, brown - grey		148.19 35.60	2	
	LIMESTONE, fresh, medium strong, laminated, porous, light brown and grey		147.06 36.73		BD,PL,Ro Cl BD,ST,Ro Si
	LIMESTONE, fresh, medium strong, laminated, porous, light grey to white		146.15 37.64	3	BD,PL,Ro Cl
	END OF DRILLHOLE				



DEPTH SCALE
1 : 75

Golder Associates

LOGGED: SG
CHECKED: SJB

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 115A

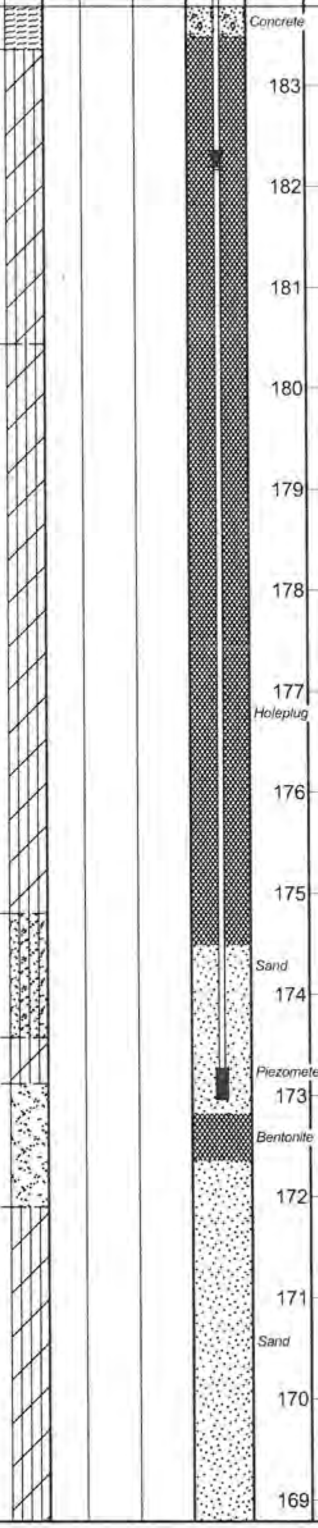
1 OF 2

METRIC

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4678585.3 ;E 333911.1
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE February 20, 2008 - February 21, 2008
ORIGINATED BY MA
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	DESCRIPTION SOIL CONDITIONS INFERRED FROM BOREHOLE No. 115
183.79	GROUND SURFACE
0.00 183.36	TOPSOIL, silty Brown
0.43	CLAYEY SILT, some sand, trace gravel Soft to very stiff Brown
180.44 3.35	CLAYEY SILT, some sand, trace gravel Stiff Grey
174.80 8.99	SANDY SILT, some clay, trace gravel Loose Grey
173.58 10.21	CLAYEY SILT, some sand, trace gravel Firm Grey
173.12 10.67	SAND, trace gravel, trace silt Loose Grey
171.90 11.89	CLAYEY SILT, some sand, trace gravel Soft to very stiff Grey

SOIL PROFILE | STRAT PLOT | SAMPLES: NUMBER, TYPE, "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 115A

2 OF 2 **METRIC**

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4678585.3 ;E 333911.1
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE February 20, 2008 - February 21, 2008
ORIGINATED BY MA
COMPILED BY BRS
CHECKED BY SJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	CLAYEY SILT, some sand, trace gravel Soft to very stiff Grey						168 167 Sand 166 165 164				
163.98 19.81	END OF BOREHOLE Water level measured in shallow piezometer at elev. 182.36m on March 20, 2008. Water level measured in shallow piezometer at elev. 182.34m on July 24, 2008. Water level measured in shallow piezometer at elev. 182.26m on September 19, 2008. Water level measured in shallow piezometer at elev. 182.20m on January 28, 2009.										

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 116

1 OF 4 **METRIC**

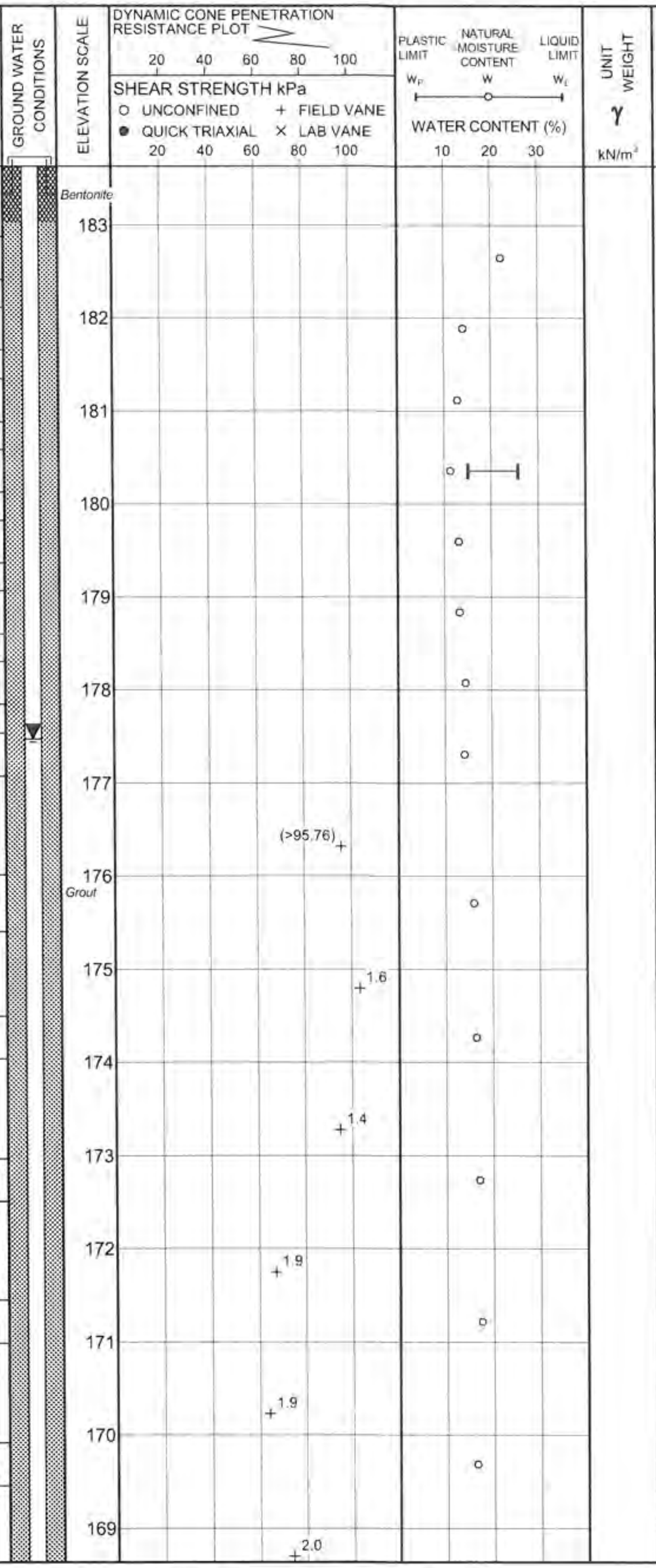
PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4678634.3 ;E 333722.5
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE February 20, 2008 - February 25, 2008
ORIGINATED BY SM
COMPILED BY BRS
CHECKED BY SJB

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
183.64	GROUND SURFACE
0.00	TOPSOIL, clayey Black
0.30	SILTY CLAY, some sand, trace gravel Firm Mottled brown and grey
182.27 / 1.37	CLAYEY SILT, some sand, trace gravel Stiff to hard Brown
179.98 / 3.66	CLAYEY SILT, some sand, trace gravel Firm to very stiff Grey
168.86 / 14.78	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	6	
2	SS	10	
3	SS	35	
4	SS	33	6 30 39 25
5	SS	17	
6	SS	14	
7	SS	12	
8	SS	11	
9	TO	PH	
10	SS	8	
11	SS	10	
12	SS	9	
13	TO	PH	



Continued Next Page

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

RECORD OF BOREHOLE No 116

2 OF 4 **METRIC**

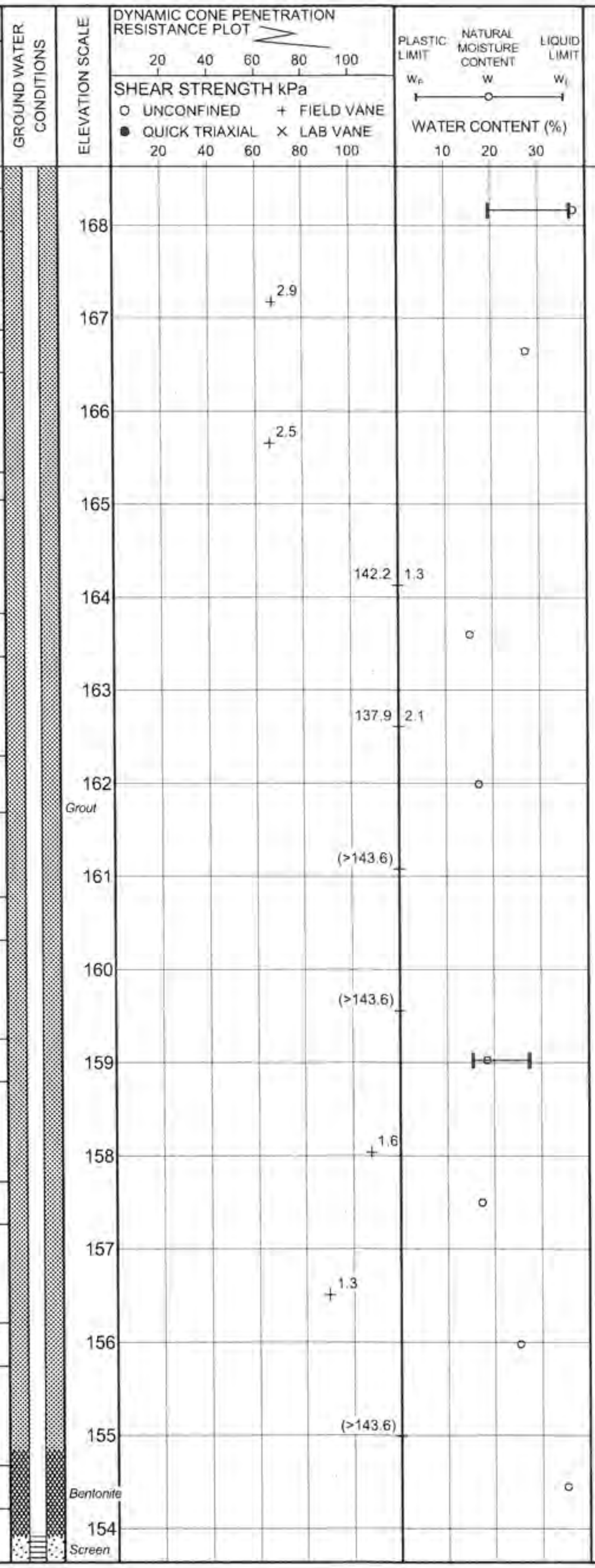
PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4678634.3 ;E 333722.5
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE February 20, 2008 - February 25, 2008
ORIGINATED BY SM
COMPILED BY BRS
CHECKED BY SJB

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
	SILTY CLAY, trace sand, trace gravel Soft to stiff Grey
165.89 17.75	CLAYEY SILT, some sand, trace gravel Stiff to very stiff Grey
158.49 25.15	CLAYEY SILT, some sand, trace gravel with sand seams Firm to very stiff Grey
155.22 28.42	SILTY CLAY Firm Grey
153.92 29.72	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14	SS	3	1 4 36 59
15	SS	9	
16	TO	PH	
17	SS	10	
18	TO	PH	
19	TO	PH	
20	SS	11	3 21 44 32
21	TO	PH	
22	TO	PH	
23	SS	6	



Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

PROJECT 07-1130-207-0

RECORD OF BOREHOLE No 116

3 OF 4

METRIC

W.P.

LOCATION N 4678634.3 ;E 333722.5

ORIGINATED BY SM

DIST WEST HWY 401/3

BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY BRS

DATUM GEODETIC

DATE February 20, 2008 - February 25, 2008

CHECKED BY SSB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, some sand, some gravel, with cobbles and boulders Very stiff Brown									(>143.6)			
		24	SS	21	Screen	153					○		(49)
						152							
151.66 31.98	LIMESTONE AND DOLOSTONE, fresh, medium strong, laminated, fine grained, faintly porous Light brown to grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)				Bentonite								
		25	NQ RC			151	53	33	33				
													UC
		26	NQ RC		Sand	150	80	72	69				
						149							
		27	NQ RC			148	57	56	89				
147.58 36.06	END OF BOREHOLE												

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

Borehole dry during drilling between February 20 and 25, 2008.

Water level measured in deep piezometer at elev. 180.79m on March 20, 2008.

Water level measured in deep piezometer at elev. 177.95m on July 22, 2008.

Water level measured in deep piezometer at elev. 176.69m on August 11, 2008.

Water level measured in deep piezometer at elev. 176.09m on September 19, 2008.

Water level measured in deep piezometer at elev. 177.26m on November 11, 2008.

Water level measured in deep piezometer at elev. 177.48m on January 28, 2009.

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 116

SHEET 4 OF 4

LOCATION: N 4678634.3 ;E 333722.5

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: February 20, 2008 - February 25, 2008

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

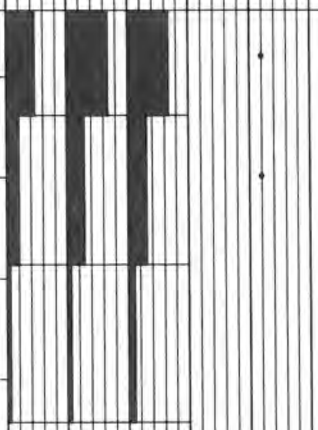
DRILLING CONTRACTOR: AARDVARK DRILLING INC

DATUM: GEODETIC

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
Br - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	ELEVATION	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
		ROCK SURFACE	151.66			
32	MUD ROTARY NQ ROCK CORE	LIMESTONE, fresh, medium strong, thinly laminated, medium grained, porous, light brown to grey	31.98	1	151	Broken Rock
33		LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, light grey	150.63 33.01	2		
		DOLOSTONE, fresh, medium strong, thinly laminated, very fine to fine grained, porous, light grey	150.36 33.28		150	BD,PL,SM Sl
34		LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, brown	150.02 33.62			
35		LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, brown to grey	149.23 34.41	3	149	
		LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, light grey	148.56 35.08		148	
36		END OF DRILLHOLE	147.58 36.06			
37						
38						
39						
40						
41						
42						
43						
44						
45						
46						



HYDRAULIC CONDUCTIVITY k, cm/sec

DIAMETRAL POINT LOAD INDEX (MPa)

NOTES WATER LEVELS INSTRUMENTATION

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: SG
CHECKED: SSB

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 116A

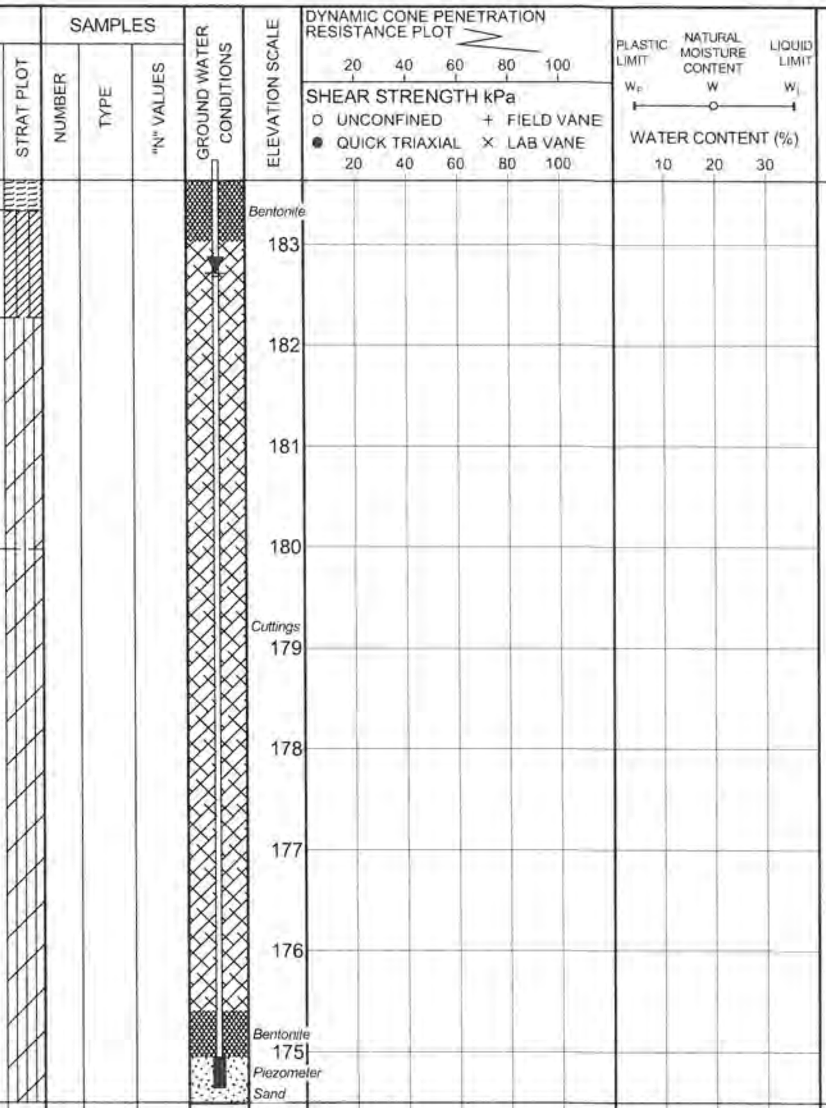
1 OF 1 **METRIC**

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC

LOCATION N 4678634.3 ;E 333722.5
BOREHOLE TYPE POWER AUGER, SOLID STEM
DATE February 25, 2008

ORIGINATED BY SM
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION SOIL CONDITIONS INFERRED FROM BOREHOLE No. 116	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.64	GROUND SURFACE					
0.00	TOPSOIL, clayey Black					
0.30	SILTY CLAY, some sand, trace gravel Firm Mottled brown and grey					
182.27 1.37	CLAYEY SILT, some sand, trace gravel Stiff to hard Brown					
179.98 3.66	CLAYEY SILT, some sand, trace gravel Firm to very stiff Grey					
174.50 9.14	END OF BOREHOLE					



Water level measured in shallow piezometer at elev. 182.55m on March 20, 2008.

Water level measured in shallow piezometer at elev. 182.80m on July 22, 2008.

Water level measured in shallow piezometer at elev. 182.59m on August 11, 2008.

Water level measured in shallow piezometer at elev. 182.57m on September 19, 2008.

Water level measured in shallow piezometer at elev. 182.72m on January 28, 2009.

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

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-6

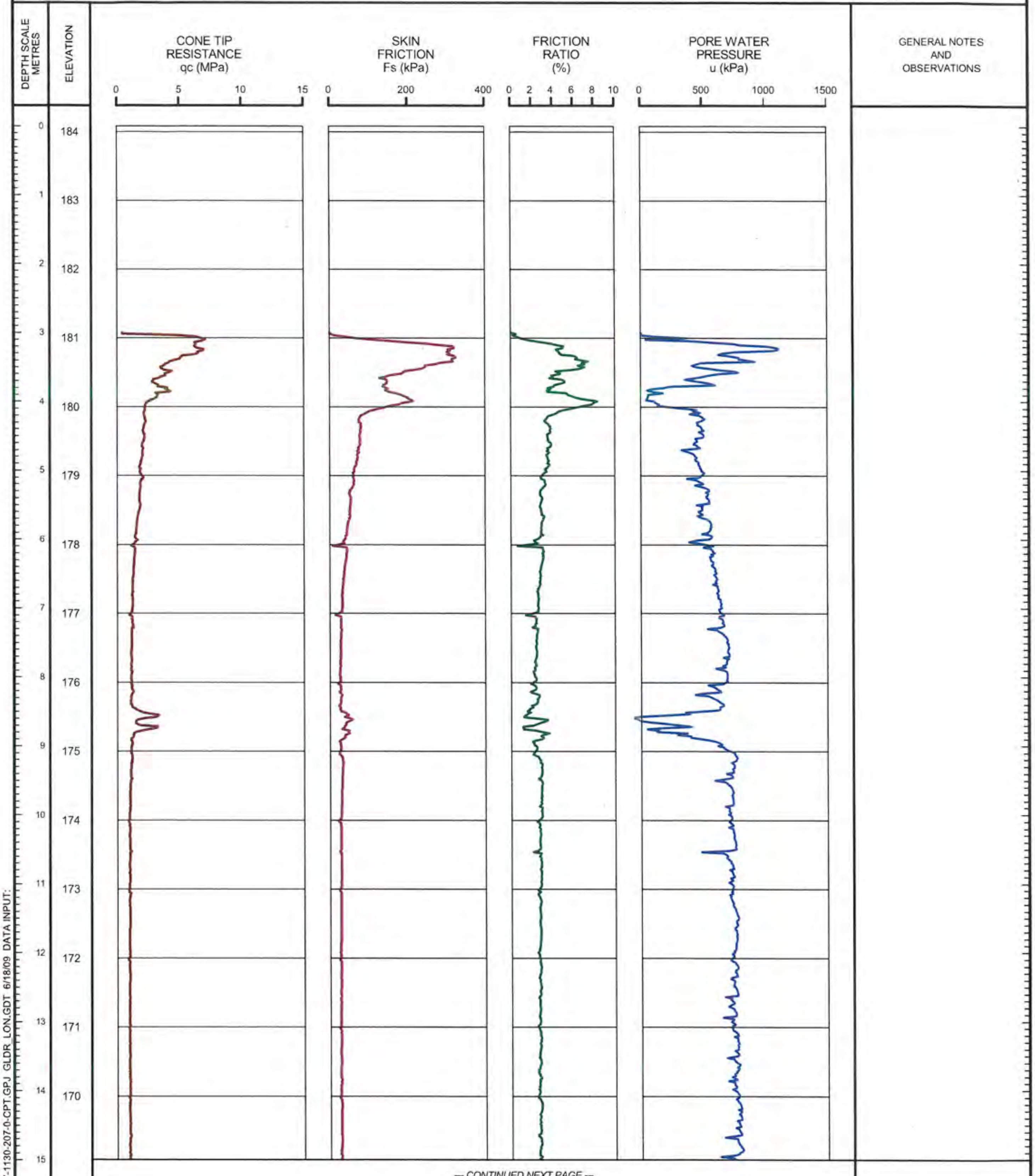
SHEET 1 OF 2

LOCATION: N 4678621.0 ;E 333844.0

TEST DATE: November 13, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 3.00m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



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DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJO

LDN_CPT_01 07-1130-207-0-CPT.GPJ GLDR_LON.GDT 6/18/09 DATA INPUT:

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-6

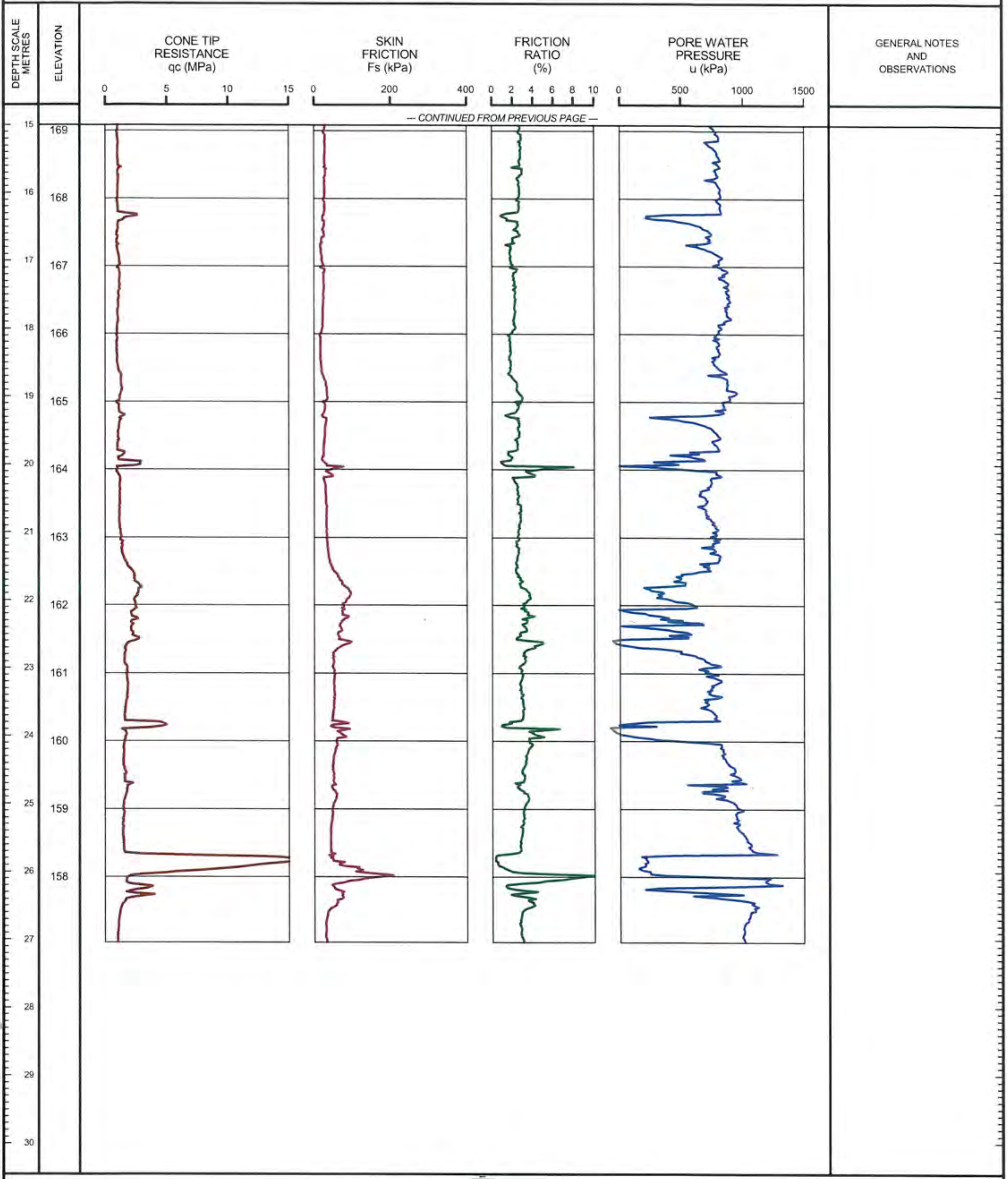
SHEET 2 OF 2

LOCATION: N 4678621.0 ;E 333844.0

TEST DATE: November 13, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 3.00m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-114

SHEET 1 OF 2

LOCATION: N 4678526.7 ;E 334018.6

TEST DATE: September 10, 2008

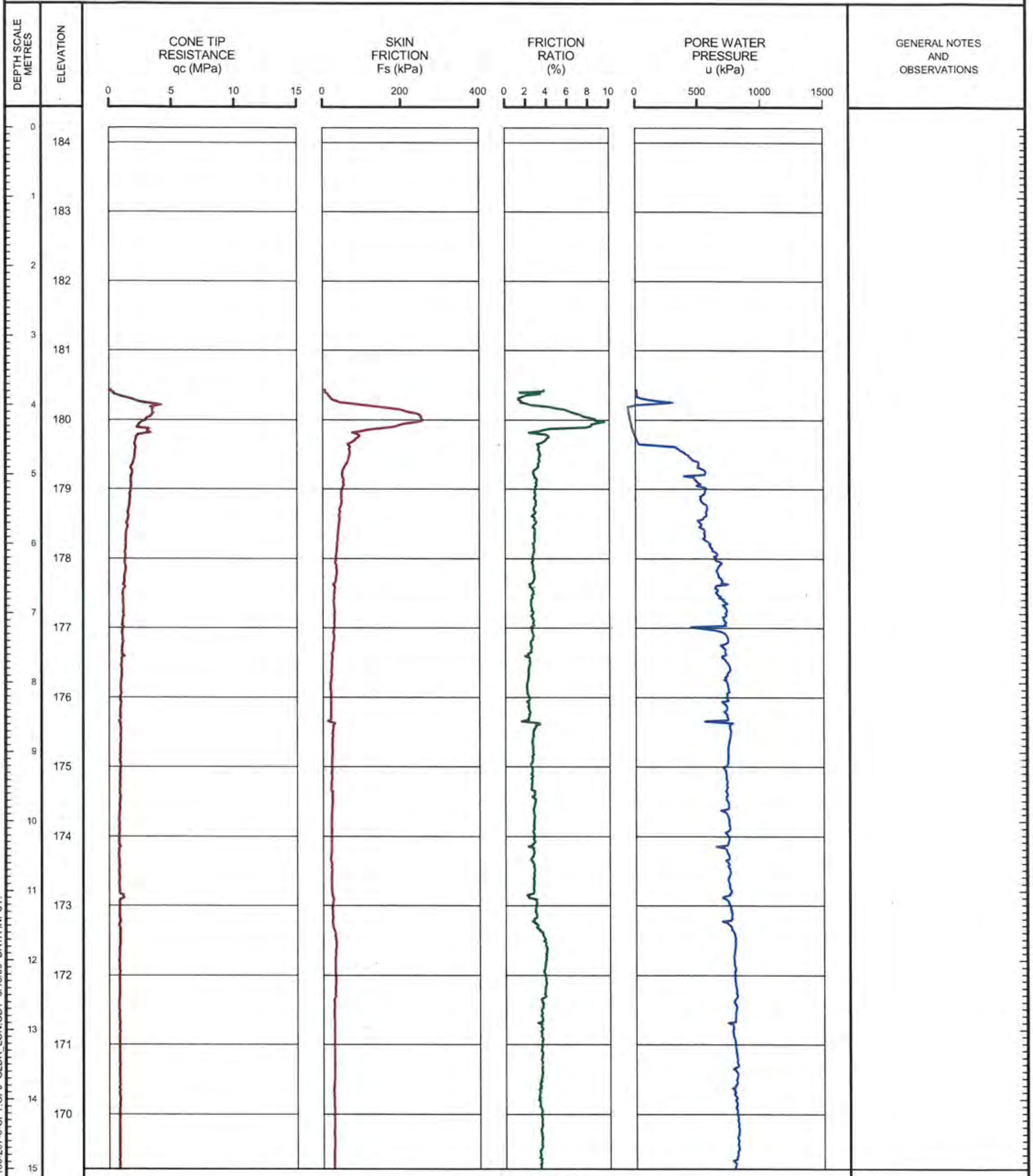
DATUM: GEODETIC

GROUND SURFACE ELEVATION:

PREDRILL DEPTH: 3.80m

CORRECTION FACTOR A: 0.584

CORRECTION FACTOR B: 0.012



— CONTINUED NEXT PAGE —

DEPTH SCALE

1 : 75

OPERATOR: CC

CHECKED: SJB

LDN_CPT_01 07-1130-207-0-CPT.GPJ GLDR_LON.GDT 6/18/09 DATA INPUT:

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-114

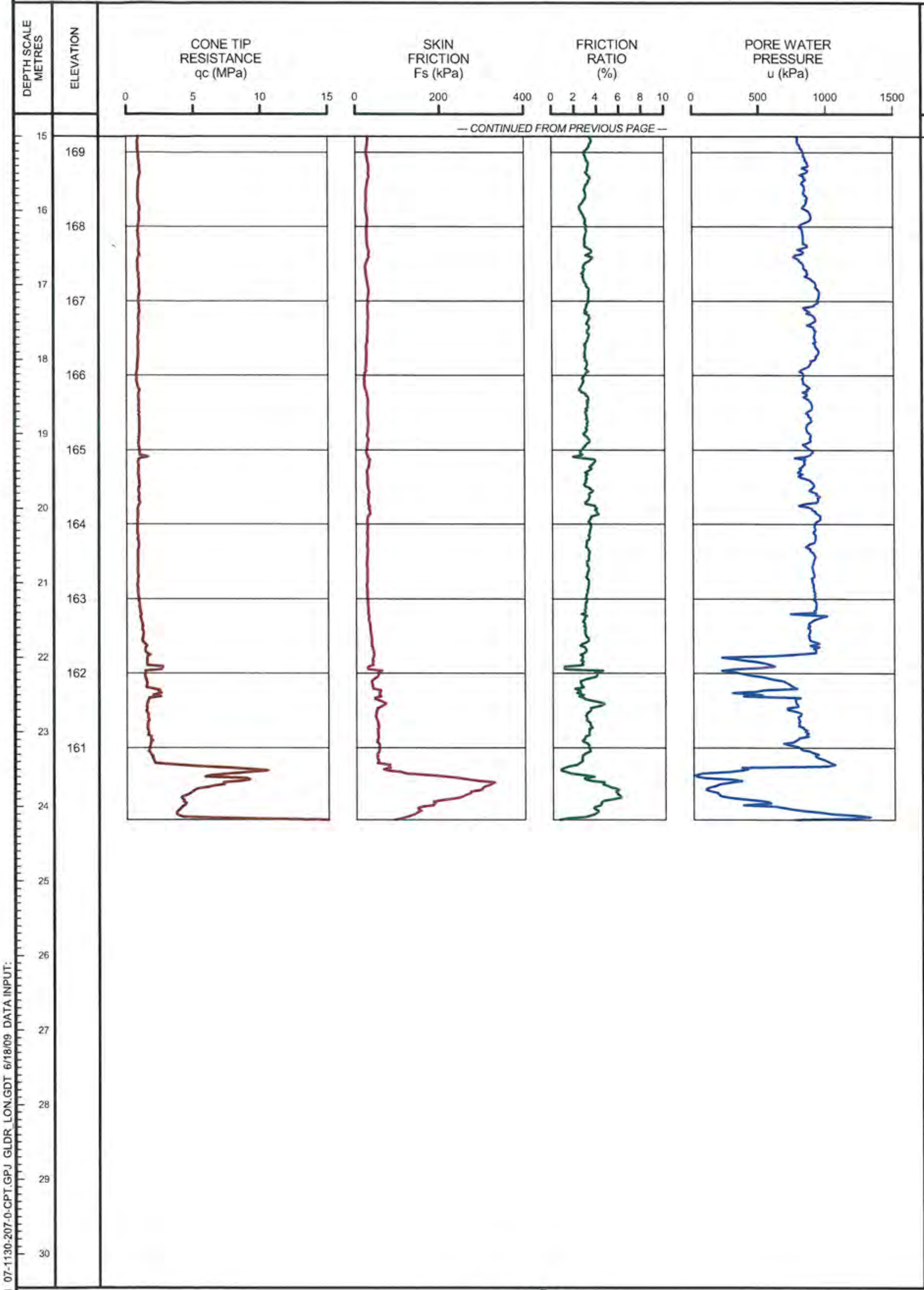
SHEET 2 OF 2

LOCATION: N 4678526.7 ;E 334018.6

TEST DATE: September 10, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 3.80m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



GENERAL NOTES AND OBSERVATIONS

LDN_CPT_01 07-1130-207-0-CPT.GPJ GLDR_LON.GDT 6/18/09 DATA INPUT:

DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SDB

Appendix C Analytical Laboratory Results

AMEC EARTH & ENVIRONMENTAL
ATTN: SHANE MACLEOD
11865 County Road 42
TECUMSEH ON N8N 2M1

Date Received:	18-JUL-11
Report Date:	26-JUL-11 07:25 (MT)
Version:	FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #:	**L1032540**
Project P.O. #:	NOT SUBMITTED
Job Reference:	SW8801.1004.101
Legal Site Desc:	
C of C Numbers:	092959-G

Gayle Braun

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L1032540-1 BHCV3-1,SS7@17.5', GREY SILTY CLAY Sampled By: CLIENT on 15-JUL-11 Matrix: SOIL							
Physical Tests							
% Moisture	12.1		0.10	%	18-JUL-11	18-JUL-11	R2220531
pH	7.79		0.10	pH units	22-JUL-11	22-JUL-11	R2223567
Redox Potential	120		-1000	mV	22-JUL-11	22-JUL-11	R2223536
Resistivity	3330		100	ohm cm	22-JUL-11	22-JUL-11	R2223537
Leachable Anions & Nutrients							
Sulphide	<0.20		0.20	mg/kg	21-JUL-11	21-JUL-11	R2222299
Anions and Nutrients							
Sulphate	76		20	mg/kg	20-JUL-11	20-JUL-11	R2222247

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
MOISTURE-WT	Soil	% Moisture	Gravimetric: Oven Dried
PH-WT	Soil	pH	MOEE E3137A
Soil samples are mixed in the deionized water and the supernatant is analyzed directly by the pH meter.			
REDOX-POTENTIAL-WT	Soil	Redox Potential	APHA 2580
RESISTIVITY-WT	Soil	Resistivity	MOEE E3137A
SO4-WT	Soil	Sulphate	EPA 300.0
SULPHIDE-WT	Soil	Sulphide	APHA 4500S2D

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
WT	ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA

Chain of Custody Numbers:

092959-G

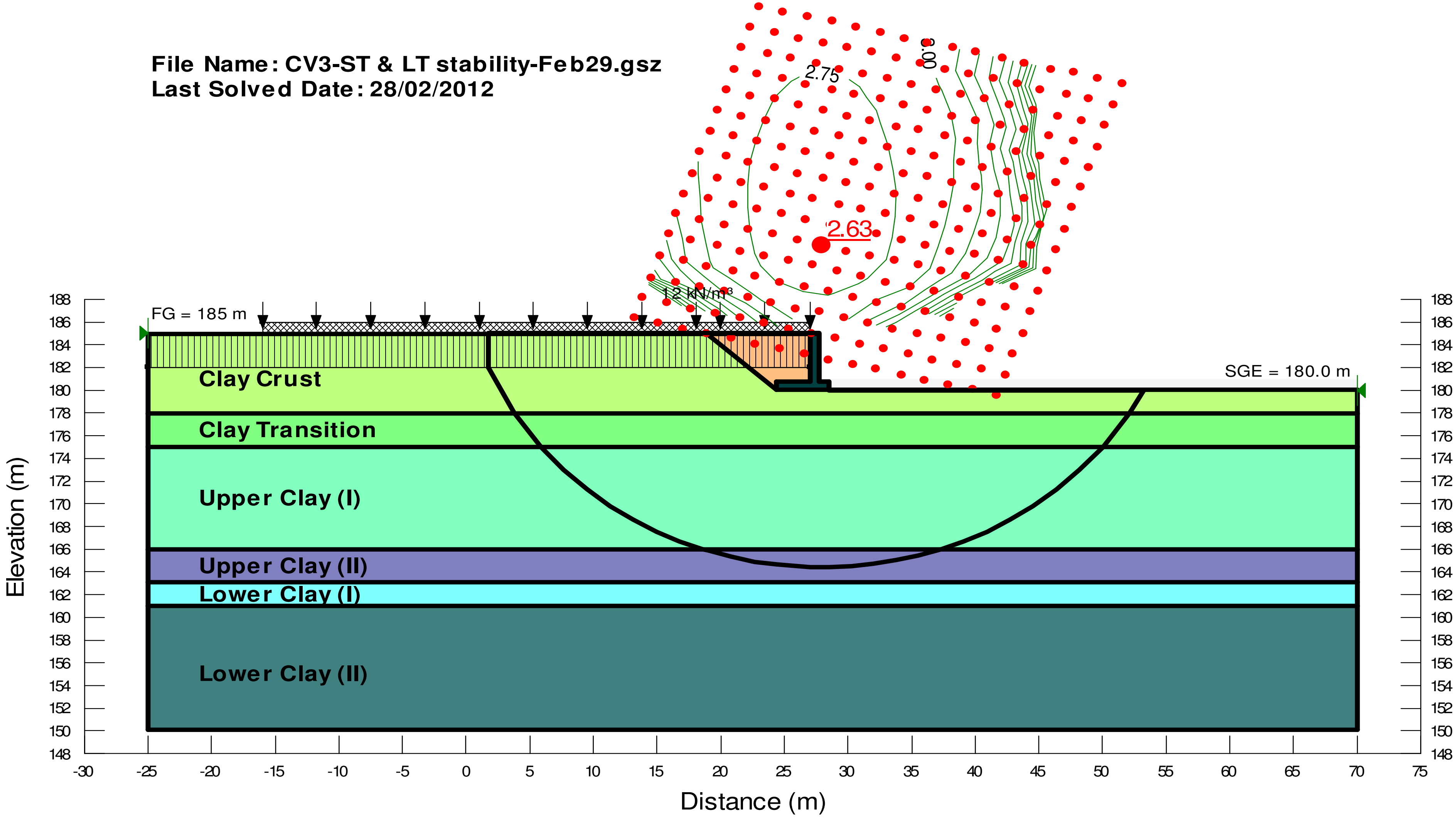
GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid weight of sample
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Appendix D Slope Stability Analyses

File Name: CV3-ST & LT stability-Feb29.gsz
Last Solved Date: 28/02/2012



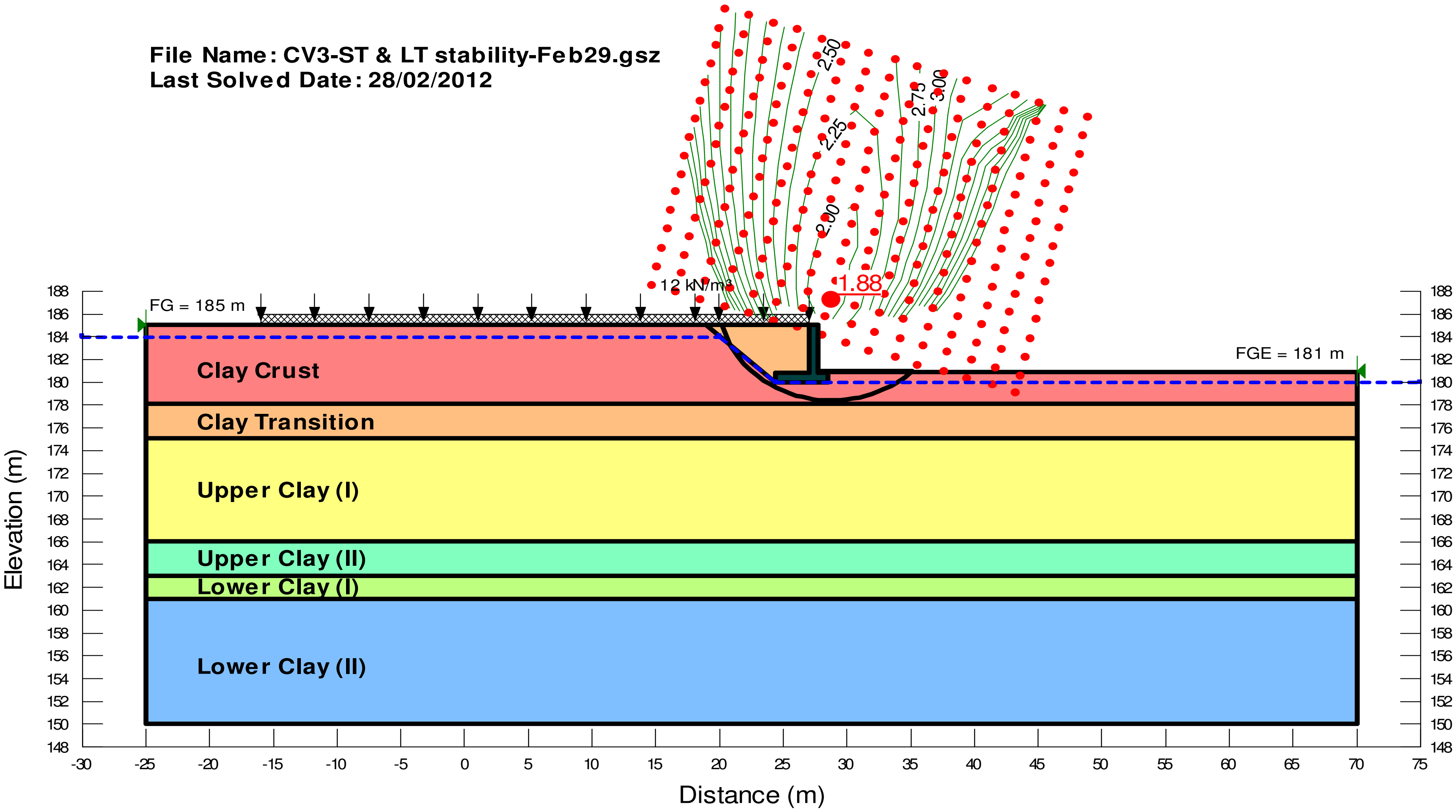
Name: Backfill (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clay Crust (U) Unit Weight: 21 kN/m³ Cohesion: 75 kPa
Name: Clay Transition (U) Unit Weight: 21 kN/m³ C-Top of Layer: 75 kPa C-Rate of Change: -5 kPa/m Limiting C: 60 kPa
Name: Upper Clay-1 (U) Unit Weight: 20 kN/m³ C-Top of Layer: 60 kPa C-Rate of Change: -1.1 kPa/m Limiting C: 50 kPa
Name: Lower Clay-1 (U) Unit Weight: 21 kN/m³ C-Top of Layer: 57 kPa C-Rate of Change: 11.5 kPa/m Limiting C: 80 kPa
Name: Retaining Wall Unit Weight: 24 kN/m³ Cohesion: 500 kPa Phi: 40 °
Name: Upper Clay-2 (U) Unit Weight: 20 kN/m³ C-Top of Layer: 50 kPa C-Rate of Change: 2.3 kPa/m Limiting C: 57 kPa
Name: Lower Clay-2 (U) Unit Weight: 21 kN/m³ Cohesion: 80 kPa

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

PARKWAY
INFRASTRUCTURE CONSTRUCTORS

PROJECT: **WINDSOR ESSEX PARKWAY**
TITLE: SHORT-TERM (UNDRAINED) STABILITY ANALYSES
CULVERT CV-3 RETAINING WALL
DATE: Feb 2012
JOB NO.:
CAD FILE:
FIGURE NO.: D.1
REV.

File Name: CV3-ST & LT stability-Feb29.gsz
Last Solved Date: 28/02/2012



Name: Backfill (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Clay Crust (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Clay Transition (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Upper Clay-1 (D) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Lower Clay-1 (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Retaining Wall Unit Weight: 24 kN/m³ Cohesion: 500 kPa Phi: 40 ° Piezometric Line: 1
Name: Upper Clay-2 (D) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Lower Clay-2 (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1

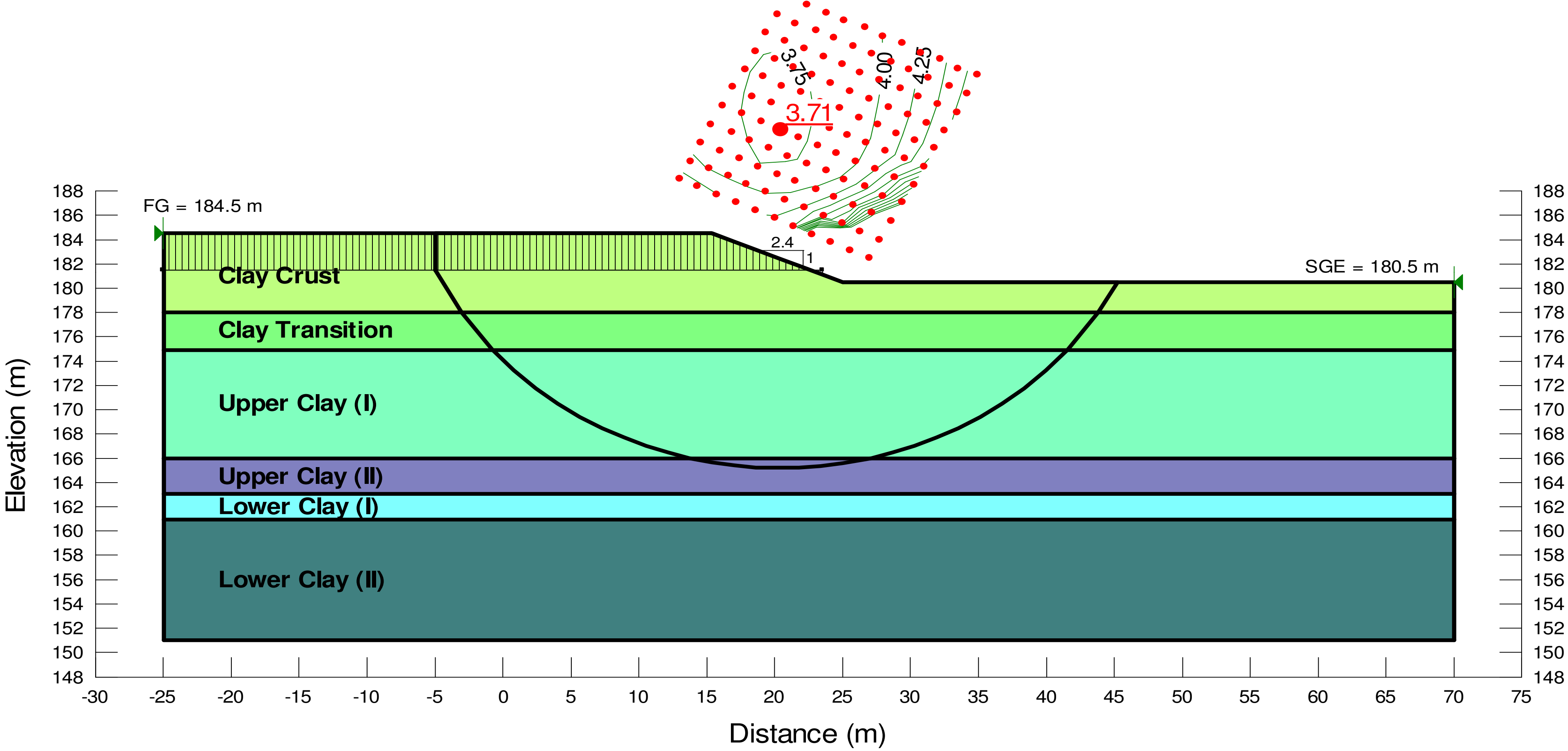
Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

PARKWAY
INFRASTRUCTURE CONSTRUCTORS

PROJECT: WINDSOR ESSEX PARKWAY
TITLE: LONG-TERM (DRAINED) STABILITY ANALYSES
CULVERT CV-3 RETAINING WALL
DATE: Feb 2012
JOB NO.:
CAD FILE:
FIGURE NO.: D.2
REV.

File Name: CV3-ST & LT Trench Stability-SEEP Feb29.gsz
Last Solved Date: 28/02/2012

Name: Clay Crust (U) Unit Weight: 21 kN/m³ Cohesion: 75 kPa
Name: Clay Transition (U) Unit Weight: 21 kN/m³ C-Top of Layer: 75 kPa C-Rate of Change: -5 kPa/m Limiting C: 60 kPa
Name: Upper Clay-1 (U) Unit Weight: 20 kN/m³ C-Top of Layer: 60 kPa C-Rate of Change: -1.1 kPa/m Limiting C: 50 kPa
Name: Lower Clay-1 (U) Unit Weight: 21 kN/m³ C-Top of Layer: 57 kPa C-Rate of Change: 11.5 kPa/m Limiting C: 80 kPa
Name: Upper Clay-2 (U) Unit Weight: 20 kN/m³ C-Top of Layer: 50 kPa C-Rate of Change: 2.3 kPa/m Limiting C: 57 kPa
Name: Lower Clay-2 (U) Unit Weight: 21 kN/m³ Cohesion: 80 kPa



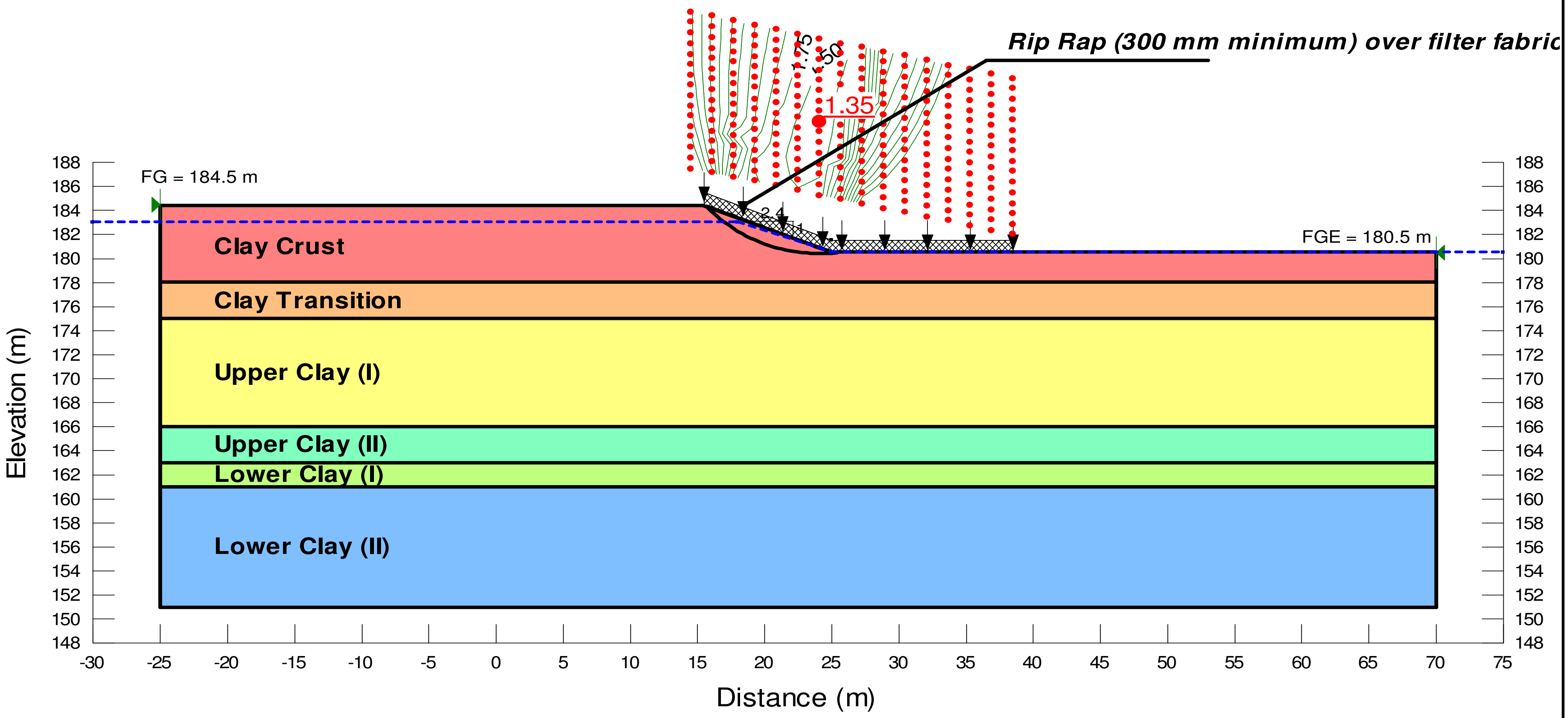
Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

PARKWAY
INFRASTRUCTURE CONSTRUCTORS

PROJECT: **WINDSOR ESSEX PARKWAY**
TITLE: SHORT-TERM (UNDRAINED) STABILITY ANALYSES
CULVERT CV-3 TRENCH
DATE: Feb 2012
JOB NO.:
CAD FILE:
FIGURE NO.: D.3
REV.

File Name: CV3-ST & LT Trench Stability-SEEP Feb29.gsz
Last Solved Date: 01/03/2012

Name: Clay Crust (D) Unit Weight: 21 kN/m³ Cohesion: 1 kPa Phi: 30 ° Piezometric Line: 1
Name: Clay Transition (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Upper Clay-1 (D) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Lower Clay-1 (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Upper Clay-2 (D) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
Name: Lower Clay-2 (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1



Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

PARKWAY
INFRASTRUCTURE CONSTRUCTORS

PROJECT:
WINDSOR ESSEX PARKWAY
TITLE:
LONG-TERM (DRAINED) STABILITY ANALYSES
CULVERT CV-3 TRENCH,
DATE: Feb 2012
JOB NO.:
CAD FILE:
FIGURE NO.: D.4
REV.