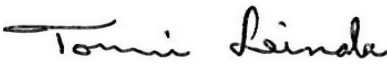




The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report - Culvert CV-4

(Realigned Cahill Drain, 9+954.81 Geraedts Drive, LaSalle)

Revision History					
Revision	Date	Status	Prepared By	Checked By	Reviewed By
0	29/02/2012	Issued for Construction - Final	TL	DD	NSV

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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 kilometers 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 kilometers 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 include 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-4, located at Sta. 9+954.8 below Geraedts Drive, near Sta. 11+650L 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 sequentially 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 the Geraedts Drive and will be used to carry the realigned Cahill Drain as shown on Drawing 285380-03-060-WIP1-5401.

The design presented in this report was generally developed 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.

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-11, R-13 and R-20). 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. P.P. Hudec (ref. R-20) 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 firm to hard surficial crust layer has formed due to weathering 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

In accordance with the Canadian Highway Bridge Design Code (CHBDC) and based on a series of cross-hole tests (ref. R-12), the soil profile at the site of the project in general 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 carried out during the background investigation program at locations distributed along the project alignment between Howard Road (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.

Windsor-Tecumseh area is described in CHBDC by a seismic hazard associated to a Velocity Zone $Z_v = 0$ and Acceleration seismic zone $Z_a = 0$. Zonal Velocity ratio, V , and Zonal Acceleration ratio, A , are both 0.

2.3 Site Conditions

Culvert CV-4 site is situated in the middle of the LaSalle segment of the Parkway, just north of Tunnel T-8. The structures at this site will be constructed under Phase I of WEP mostly within the realigned Cahill Drain. As shown on the Drawing 285380-03-060-WIP1-5401, the banks of the realigned Cahill Drain and against the wing walls will be sloped at 2.4H:1V.

The topography of the lands immediately adjacent the Culvert CV-4 is essentially flat with ground surface elevations about elevation 183 to 184 m. 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 to 1.0 m. This estimate is considered applicable for natural soils and/or conventional pavement materials where the ground surface is usually cleaned from the snow cover.

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 (CV4-1) was advanced within the footprint area of the proposed culvert. However, additional boreholes, CPT and Flat Blade Dilatometer (DMT) were carried out for the nearby structure (Tunnel T-8) and other structures in close proximity (such as Tunnels T-9) and highway design components (slopes, retaining structures). One of the main objectives of Borehole CV4-1 was to examine the site specific subsurface conditions and confirm the background information from the nearby tests and investigations. Table 3.1 lists the test holes located 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-4 Site

Reference	Boreholes	Nilcon Vane Tests	CPT's	DMT's
This Investigation (2011)	CV4-1	Nil T8-1	CPT T8-1	DMT T8-1
	T8-1			
	HG-MW-3			
	TB6-1			
Previous Studies (2007-09)	BH-7		CPT-7	
	BH-118			
	BH-118A			

The locations of boreholes, Nilcon tests, CPTs and DMTs 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-WIP1-5401. 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-WIP1-5401 and 285380-04-091-WIP1-5402 show the location of the test holes and an interpreted soil stratigraphic profile at Culvert CV-4 site.

3.2 Additional Investigation at the Culvert Site

This section presents the exploration procedure and the results of the investigation. The interpreted soil and groundwater data from all the boreholes within the vicinity have been considered in the design of Culvert CV-4.

3.2.1 Fieldwork at culvert site

Borehole CV4-1 was drilled on August 27, 2011 for this study. The borehole was advanced using a track-mounted CME 650 auger rig owned and operated by Marathon Drilling Co. Ltd. under contract to

AMICO and under full-time technical direction by AMEC engineers and technicians. The borehole was advanced to a maximum depth of 10.4 m below grade using 200 mm diameter hollow stem augers.

Soil sampling was advanced using a 50 mm diameter split spoon sampler. Soil sampling was performed at 0.75 m to 1.5 m depth intervals to the depths explored. All samples were 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. Field vane tests (using conventional vanes) were conducted between split spoon sampling at selected depths. 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 performed on most of the recovered samples from Borehole CV4-1. The results are presented on the borehole log (Appendix A).

Analytical testing consisting of pH, redox potential, resistivity, sulphide and sulphate contents were carried out on one sample collected from Borehole CV4-1. 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 (1972) and updated subsequently by Ladd et al (1977) 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 (ref. R-3 and R-18 to R-20). However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), 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:

$$S_{u\text{CPT}} = \frac{Q_t - \sigma_{vo}}{N_{kt}}$$

Where:

- $S_{u\text{CPT}}$ 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.

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

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 (Ladd and Foott, 1974). The following relationship was used to compute the pre-consolidation pressures:

$$OCR = \frac{\sigma'_p}{\sigma'_{vo}} = \left[\frac{S_u / \sigma'_{vo}}{S} \right]^{1/m}$$

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:

$$\sigma'_p = \sigma'_{vo} \times \left[\frac{S_{u,CPT}}{\sigma'_{vo}} \right]^{1.05}$$

Flat Blade Dilatometer (DMT) Test Data:

DMT tests along WEP were conducted following ASTM D6635-01 (2007). The soil properties from the results of these tests were developed in general accordance with the guidelines in 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 represented 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 $0.18 \times \sigma'_{vo}$ 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 of the silty clay to clayey silt stratum and published relationships (refs. R-6 and R-18).

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 and upper granular deposit below the existing ground surface at about elevation 183³, an extensive clayey silt to silty clay deposit below about elevation 181 to 183, and a possible discontinuous lower granular deposit below about elevation 153.7 (BH-118), overlying limestone and dolostone bedrock below about elevation 150 m. The thickness of the clayey silt to sandy/silty clay deposit based on the available nearby boreholes is about 30 to 32 m. The bedrock was encountered at depths approximately 32 to 33 m below the ground surface.

4.1 Topsoil, and Surficial Fills

A layer of topsoil was encountered at the ground surface in Boreholes TB6-1, HG-MW-3, BH T8-1, BH-118, BH-118A, and BH-7. The thickness of the topsoil was about 0.1 to 0.4 m at these locations.

Asphalt was encountered in Borehole CV4-1. The asphalt was about 0.9 m thick at this location. Fill layer consisting of silty clay to clayey silt with trace sand and gravel was encountered beneath this asphalt layer. The fill layer was about 1.20 m thick.

Fill layers were also encountered in Boreholes BH-118 and BH-118A below topsoil. The fills were variable and consisted of silty clay to sand to silty sand and gravel. The fill thickness was about 1.5m at the borehole locations.

4.2 Silty Clay to Clayey Silt Stratum

An extensive cohesive silty clay to clayey silt stratum, was encountered directly underlying the topsoil and/or fill deposit. The encountered depth below existing ground surface varied from 0.1 to 2.1 m corresponding to elevation 181.2 to 182.9 m. 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 clayey silt). The natural water content, Atterberg limits, and total unit weights of the clay sub-strata are summarized in Table 4-1 and illustrated in Figure 3.3.

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

Table 4-1: Summary of Index Properties (Based on CV4-1 and Nearby Boreholes)

Property ¹	Clay Crust	Transition	Upper Silty Clay	Clayey Silt
Elevation Range (m)	183 ² to 178	178 to 175	175 to 163	163 to 151 ²
Natural Water Content, ^w N, %	10 to 23	10 to 18	11 to 38	7 to 35
Liquid Limit, ^w L, %	23 to 27	23 to 25	15 to 43	15 to 31
Plastic Limit, ^w P, %	13 to 15	13 to 14	12 to 22	11 to 18
Plasticity Index, PI	10 to 14	9 to 12	8 to 21	5 to 15
Liquidity Index, LI	-0.34 to -0.07	0.1 to 0.2	-0.35 to 1.05	-0.31 to 0.93
Unit Weight, γ , kN/m ³	N/A ³	N/A	21.4	21

1 – Index Properties are based on laboratory results from Boreholes: CV4-1, BH-7, BH-115, BH-116, BH-118, BH-314, BH 14-RW, BH 15-RW, T8-1, T9-1, TB7-1, TB7-2, TB7-3, CV3-1, CPT 43-RW.

2 – Ground surface elevation reported in reviewed studies varies

3 – 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 kPa
- Transition layer: 80±20 kPa to 70±10 kPa
- Upper silty clay: 70±10 kPa to 60±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 sand, clayey silt, and silty sand and gravel were encountered in the nearest deep borehole, BH-118. 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 153.7 in Borehole BH-118. The thickness of the lower granular deposit was approximately 3.4 m at the borehole location. The lower granular deposit had 'N' values ranging from 19 to 100 indicating a medium dense to very dense state of compactness.

4.4 Bedrock

Boreholes CV4-1, TB6-1, and HGMW-3 were terminated within the overburden deposits. Boreholes T8-1, BH-118, and BH-7 refused on material considered to be bedrock beneath the lower granular deposit or below the silty clay to clayey silt stratum at about elevation 150.3 to 150.0. The bedrock was light grey, fairly porous, and fine grained limestone bedrock. The Rock Quality Designation (RQD) of the recovered rock cores ranged from 0 to 100 percent, indicating a poor to excellent quality.

4.5 Groundwater Conditions

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

The piezometric levels measured in the clayey silt overburden and the limestone bedrock varied from 180.1 to 181.2 and around 177.4 and 177.6, respectively. The highest piezometric levels within the overburden and the bedrock were recorded at about elevations 181.2 and 177.6, 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, occurrence of artesian condition in bedrock cannot be entirely ruled out.

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
T8-1	182.8	VWP	172.1	Clayey Silt	Aug. 29, 2011	181.2
			162.2	Clayey Silt	Aug. 29, 2011	179.9
HG-MW-3	182.91	S-Piez	179.9 – 181.7	Clayey Silt & Sand	Oct. 13, 2011	180.63
BH-7	183.17	S-Piez	165 – 169	Clayey Silt	Nov. 14, 2006	180.1
			145.3 – 149.2	Limestone	Nov. 14, 2006	177.6
BH-118	182.66	S-Piez	146.6 – 149.0	Limestone	Jan. 28, 2009	177.40
BH-118A	182.66	S-Piez	172.6 – 174.0	Clayey Silt	Jan. 28, 2009	180.9

Legend: S-Piez Standpipe Piezometer
VWP Vibrating Wire Piezometer

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.

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 on 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) and the recent investigation, in the boreholes near Culvert CV-4 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 ₂ S	CH ₄
				mg/L	µg/L
BH-118	182.66	151.7	Bedrock	2.55	65

Although the H₂S and CH₄ gases were not detected during the 2011 geotechnical investigation at CV-4 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 4-4.

Table 4-4: Pumping Tests Data

Test #	Approximate Location	H ₂ S 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 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 2.6 m inside height. The invert elevation (i.e., top of the base slab) varies from 180.202 at the inlet end to 180.158 at the outlet med. The general arrangement is shown on Drawing 285380-03-060-WIP1-5401.

Retaining walls flared at 45 degrees will be constructed at both ends of the box culvert structure. The walls will retain the backfill behind the structure at the end of permanent 3H:1V cut slope from the edge of Geraedts Drive pavement near elevation 185 to the bottom of the realigned drain at approximate elevation 179.

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-walls was checked for all potential surfaces of sliding.

5.2 Design Soil Properties

The design undrained shear strength 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 carried out between Sta. 11+500L and Sta. 12+300L (Figures 3.3)

Based on CPT results, the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into 4 layers as follows: brown desiccated stiff to hard clay crust, transition zone, upper grey silty clay to clayey silt deposit (referred to hereafter as upper silty clay), and then a generally coarser lower grey clayey silt deposit (referred to as lower clayey silt).

The S_u profiles inferred from the CPT advanced around Culvert CV-4 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	183 to 178	75 (**)	$\bar{C} = 0$ kPa, $\phi = 30^\circ$	600	>7
Transition	178 to 175	75 to 60	$\bar{C} = 0$ kPa, $\phi = 30^\circ$	600 to 400	7 to 2
Silty Clay (I)	175 to 166	60 to 50	$\bar{C} = 0$ kPa, $\phi = 30^\circ$	400 to 280	7 to 2
Silty Clay (II)	166 to 163	50 to 57	$\bar{C} = 0$ kPa, $\phi = 30^\circ$	280 to 310	2 to 1.2
Clayey Silt (I)	163 to 161	57 to 80	$\bar{C} = 0$ kPa, $\phi = 30^\circ$	310 to 450	2 to 1.2
Clayey Silt (II)	161 to 151	80	$\bar{C} = 0$ kPa, $\phi = 30^\circ$	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

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-22 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-4 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 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 values

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 σ' relationships (ref. R-21 and R-26).

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 foundation. The shapes and slopes of the temporary excavations shown in this report 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 anticipated invert of the excavation for box culvert is near elevation 179.5, i.e., about 3.5 to 4.0 m maximum excavation depth.

Basal hydrostatic uplift stability was calculated based on the highest measured water level, 181.2 measured in the silty sand deposit encountered in BH-T8-1 (Table 4-2) and the anticipated deepest excavation depth (elevation 178.5m). With highest granular deposit encountered at 153.7m (BH 118), the minimum thickness of the silt-clay layer above the silty sand deposit would be 24.5 m. The calculated factor of safety against hydrostatic uplift was greater than 2.0.

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 comprising at least 75 mm of lean concrete over the areas of cast-in-place

foundation, or bedding material over the areas of precast components should be placed the 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 backfilling is approved for subgrade corrections.

The box culvert structure should be founded over granular bedding prepared in accordance to the OPSS 422 requirements unless otherwise specified in the Contract Documents or manufacturer's recommendations. The bedding should be placed on undisturbed stiff grey silty clay at/near elevation 180.

5.4.2 ULS Bearing Resistance

A net factored geotechnical resistance of 175 kPa at Ultimate Limit States (ULS) was determined for the native undisturbed silty clay subgrade soils supporting the box culvert structure near elevation 179.5 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 SLS resistance (soil stress increase) of 125 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 4.6 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 should be verified and refined with respect to the actual performance monitoring in the field.

The settlements discussed above do not include deformations caused by seasonal temperature and moisture variations, or deformation related to compression of the backfill materials, which for well compacted fill should be small. In this regard, stringent compaction control should be exercised to minimize the magnitude of backfill compression (see Section 5.6).

Given the relatively shallow soil cover over the culvert roof and the perceived larger longitudinal flexibility of the precast structure incorporating numerous joints, consideration should be given to the potential risk that the movement at the joints between the precast elements may propagate to the surface causing cracking of the pavement. Therefore, consideration by the structural design should be given to the incorporation of a distribution slab above the culvert.

5.5 Retaining/Head Walls

5.5.1 General

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 179.
- 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

Based on the arrangement of the retaining walls shown on Drawing 285380-03-060-WIP1-5401, the wall heights at both ends of the culvert are similar. It is anticipated that the wall height will be approximately up to 4.1 m with sloping backfill behind the wall.

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	2.57 (19.94*)
Culvert Wall – Long-term Steady State	Drained	D.2	1.80 (1.6)

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

5.5.3 ULS Bearing Resistance

A net factored bearing resistance of 175 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 179 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):

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

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)

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

$\delta = 0^\circ$ (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 178.5 on the basis of a 25 mm maximum post-construction settlement. The footing width considered is not wider than 5 m.

Assuming that the maximum unfactored bearing pressure at the edge of the footing is limited to 1.4 times the SLS resistance, the estimated minimum and maximum footing settlements would vary between 18 mm and 35 mm. Hence, the anticipated maximum wall rotation would be less than 0.34%.

The settlements discussed above do not include deformations caused by seasonal temperature and moisture variations, or deformation related to compression of the backfill materials, 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 wall, and associated retaining walls, bedding and backfill materials should meet the requirements of OPSS 422, 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 SP 105S10. Longitudinal drains should be installed to provide positive drainage of the backfill. Other aspects of the

abutment 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 free draining sand and gravel fill (Granular B Type I, or approved equivalent) should be used. Please refer to OPSD 3101.150 for minimum granular requirements.

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 5-5.

In the case of sloping backfill surface, the coefficients in this table should be modified based on the following equations:

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

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

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

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, $\phi(^{\circ})$	33-35	29-32	22-30
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 undrained and drained analysis for the proposed 2.4:1V drain slope is provided in Figure D-4. The protected slope is presented in Figures D-4. The drained analysis indicates that a suitable slope protection (rip-rap blankets over filter fabric, or equivalent) should be considered to prevent slope surface sloughing.

The analysis model 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	4.29 (3.70*)
Trench – Long-term Drained Slope	Drained	D.4	1.33 (1.29)

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

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, groundwater seepage is anticipated to be minor, which should be controllable by conventional temporary dewatering methods.

Runoff and seepage into the excavations from perched groundwater from the fill, existing and abandoned utility trenches, and upper granular layers should also be anticipated. In adverse conditions, the runoff and seepage from perched groundwater can be significant. Provision should be made to deal with the seepage by pumping from properly filtered sumps located within the excavation.

It is anticipated that piping of fine granular materials from embedded seams and at the granular/clay interface will occur. In this area, blanketing of the excavation slopes with a geotextile and free draining granular material may be required to prevent the loss of ground.

Accordingly, provision should be made to prevent runoff and piping erosion of the slope surface by cut-off 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. Contractors 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.

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.
- 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. The subgrade should be covered the 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.

6.3 Corrosion Potential

A series of pH, Redox Potential, Resistivity, Sulphide, and Sulphate tests were carried out on a sample from Borehole CV4-1. 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 CV4-1 (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 results 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 at strategic locations and surveyed as required. 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 assessed and modifications to the design and construction may be required.

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

- Ground movement in excess of anticipated maxima (> 25 mm)
- Unsterilized movement trend without loading changes
- 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).

A comprehensive instrumentation-monitoring plan for all major components of the WEP project was prepared and should be implemented at this particular structure as appropriate.

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 structure 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-4 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. was 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 Rajilic, 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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Narendra S. Verma, Ph.D., P.Eng, F.ASCE, D.GE.
Principal Geotechnical Engineer
(Designated MTO RAQS Contact)

9 References

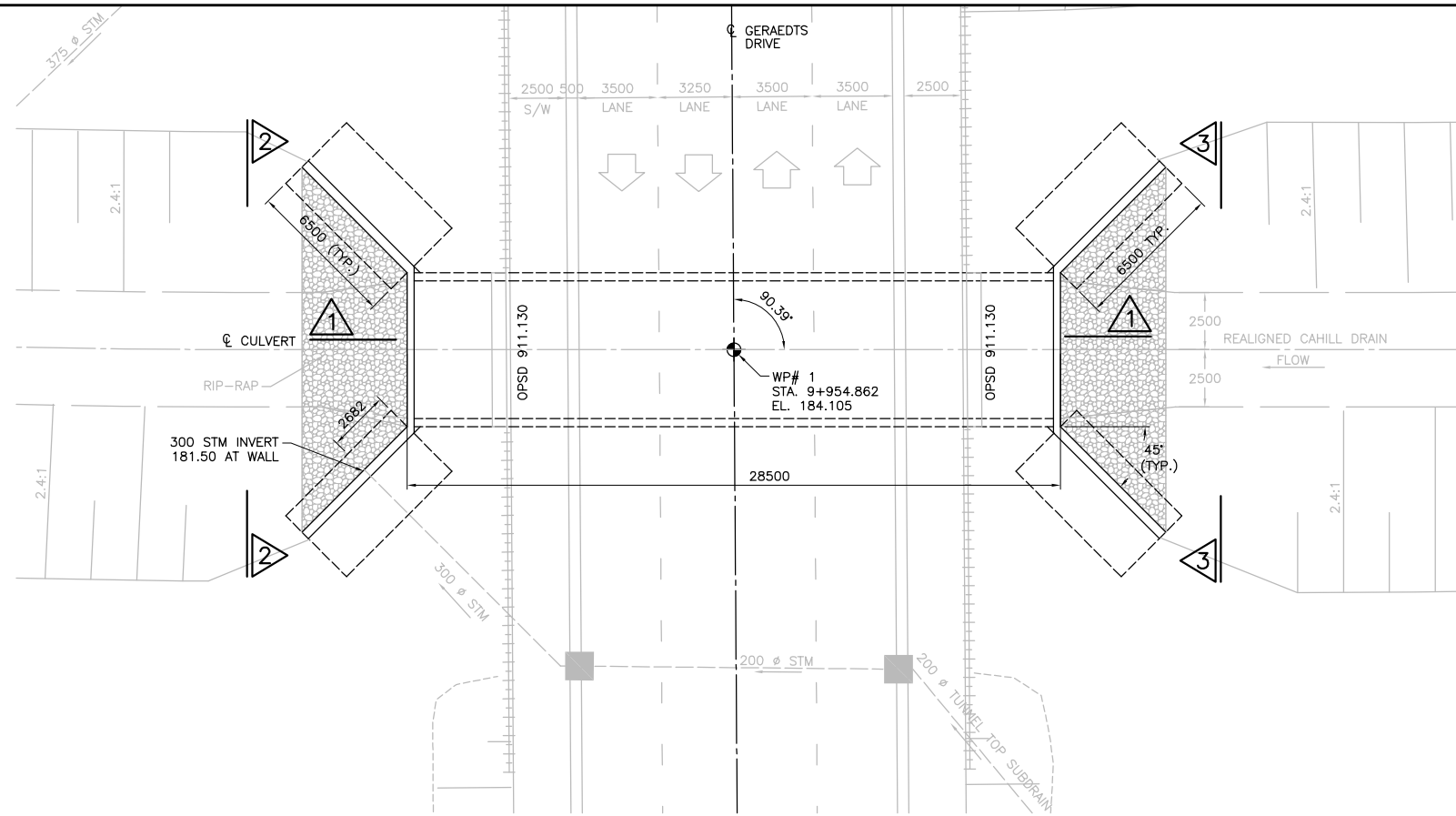
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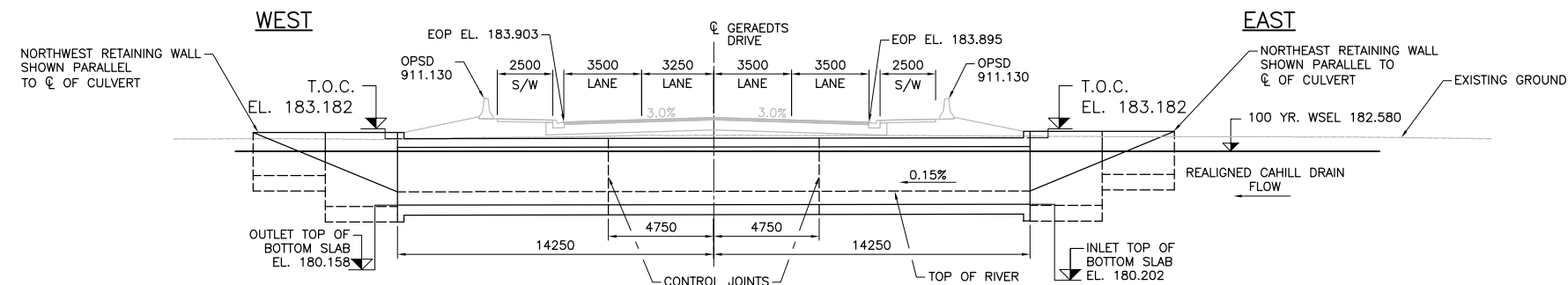
Drawings

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report – Culvert CV-4
(Realigned Cahill Drain, 9+954.81 Geraedts Drive, LaSalle)
Doc No.: 285380-04-119-0022(Geocres No. 40J3-11)

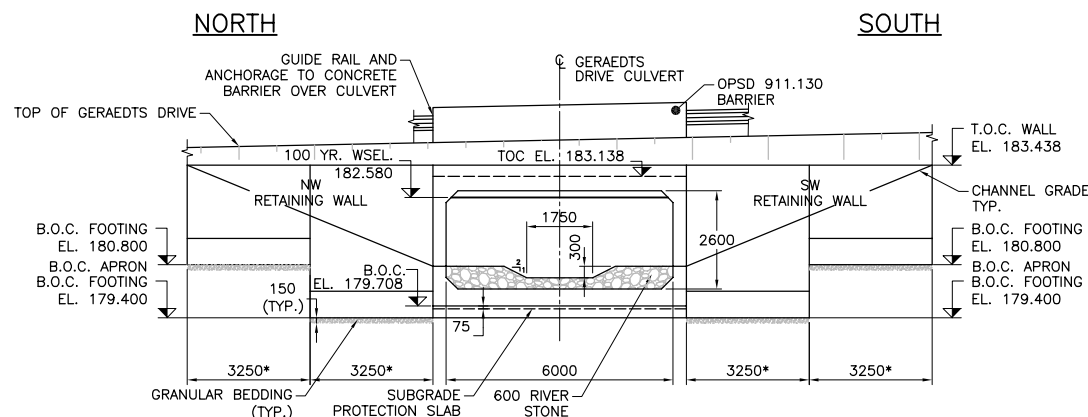
Date: February/2012
Rev: 0
Page No.: Drawings



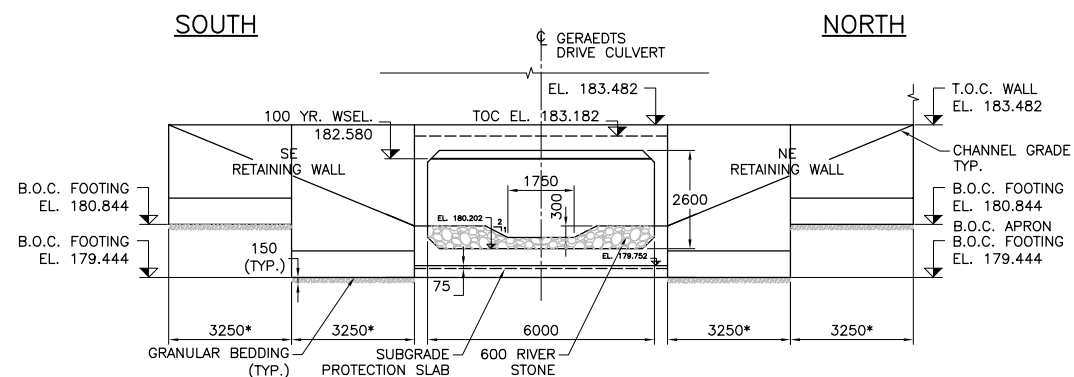
PLAN
SCALE 1:150



1
SCALE 1:150



2
SCALE 1:100

(* DENOTES DIMENSIONS
SQUARE TO RETAINING WALL)


 SCALE 1:100

(* DENOTES DIMENSIONS
SQUARE TO RETAINING WALL)

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

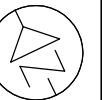
NOT FOR
CONSTRUCTION

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



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



SHEET
S5401

Phase 1
IFC

GENERAL NOTES:

1. CLASS OF CONCRETE TO BE 30MPa, UNLESS OTHERWISE 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, BLACK, UNLESS OTHERWISE NOTED.
4. LEGEND
ALT DENOTES ALTERNATE
IF DENOTES INSIDE FACE
TOC TOP OF CONCRETE
OF DENOTES OUTSIDE FACE
EF DENOTES EACH FACE
EOP EDGE OF PAVEMENT
5. PEDESTRIAN BARRICADES TO OPSD 980.101. SUPPLY ALONG CULVERT HEADERS AND RETAINING WALLS.
6. MAXIMUM FILL HEIGHT OVER CULVERT 2.0 m.
7. SOIL BEARING RESISTANCES
CULVERT: NET ULS 160kPa; MAX SLS REACTION 150kPa
(ESTIMATED MAX SETTLEMENT 25mm).
RETAINING WALLS: NET ULS 210kPa; MAX SLS REACTION 145kPa
(ESTIMATED MAX SETTLEMENT 25mm).
8. FISH COMPENSATION PLAN TO COVER FINAL CONFIGURATION OF LOW FLOW CHANNEL AND RIVER STONE WITHIN CULVERT.
9. 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.
10. FOR ALL HIGHWAY WORKS REFER TO HIGHWAY NEW CONSTRUCTION DRAWINGS.
11. FOR ALL ELECTRICAL AND ATMS WORKS REFER TO ELECTRICAL AND ATMS NEW CONSTRUCTION DRAWINGS.
12. FOR ALL UTILITY WORKS REFER TO UTILITY NEW CONSTRUCTION DRAWINGS.
13. FOR INFORMATION ON EXISTING PAVEMENT AND INFRASTRUCTURE REFER TO HIGHWAYS REMOVAL DRAWINGS AND GENERAL NOTES PROVIDED WITHIN HIGHWAYS REMOVAL DRAWING PACKAGE.

CONSTRUCTION NOTES:

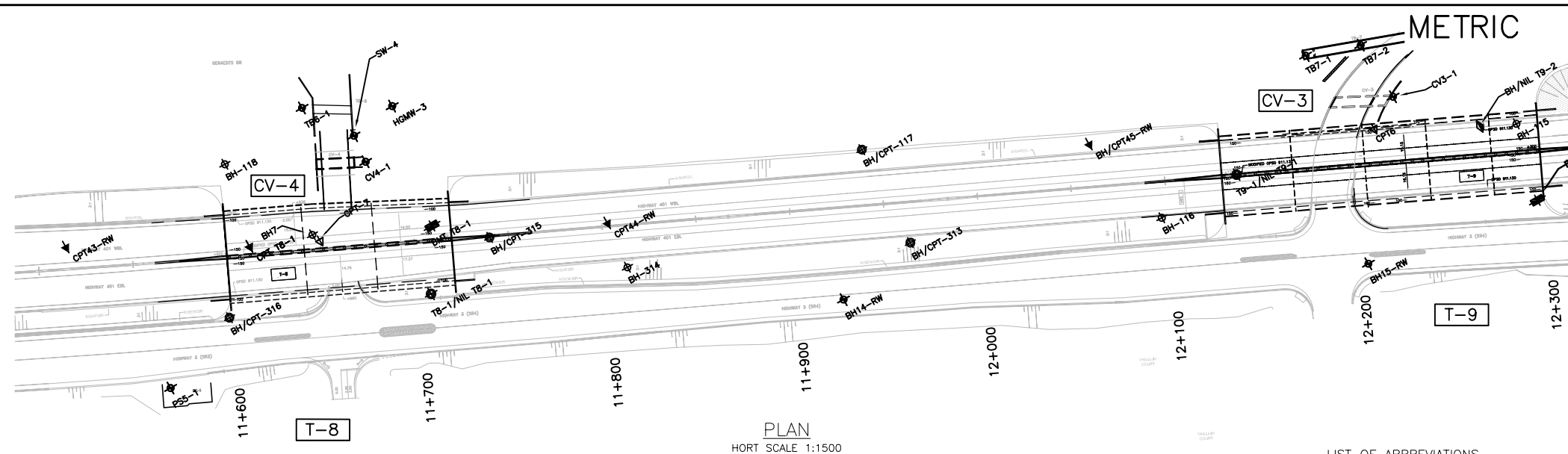
1. SUPPORTS FOR REINFORCING STEEL SHALL BE AS PER OPSD-3329.101 AND OPSD-3329.100 ON FORMED SURFACES. ON NON-FORMED SURFACES, CONCRETE BLOCKS (MIN. 20MPa) SHALL BE USED.
2. 20MPa CONCRETE SUBGRADE PROTECTION SLAB TO BE PLACED AFTER APPROVAL OF SUBGRADE BY GEOTECHNICAL ENGINEER.
3. CORRECT DEFICIENT SUBGRADE 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. CONTROL JOINT POSITIONS ARE SUGGESTED LOCATIONS TO CONTROL UNWANTED CRACKING CAUSED BY STRUCTURE CONTRACTIONS.
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

[illegible]

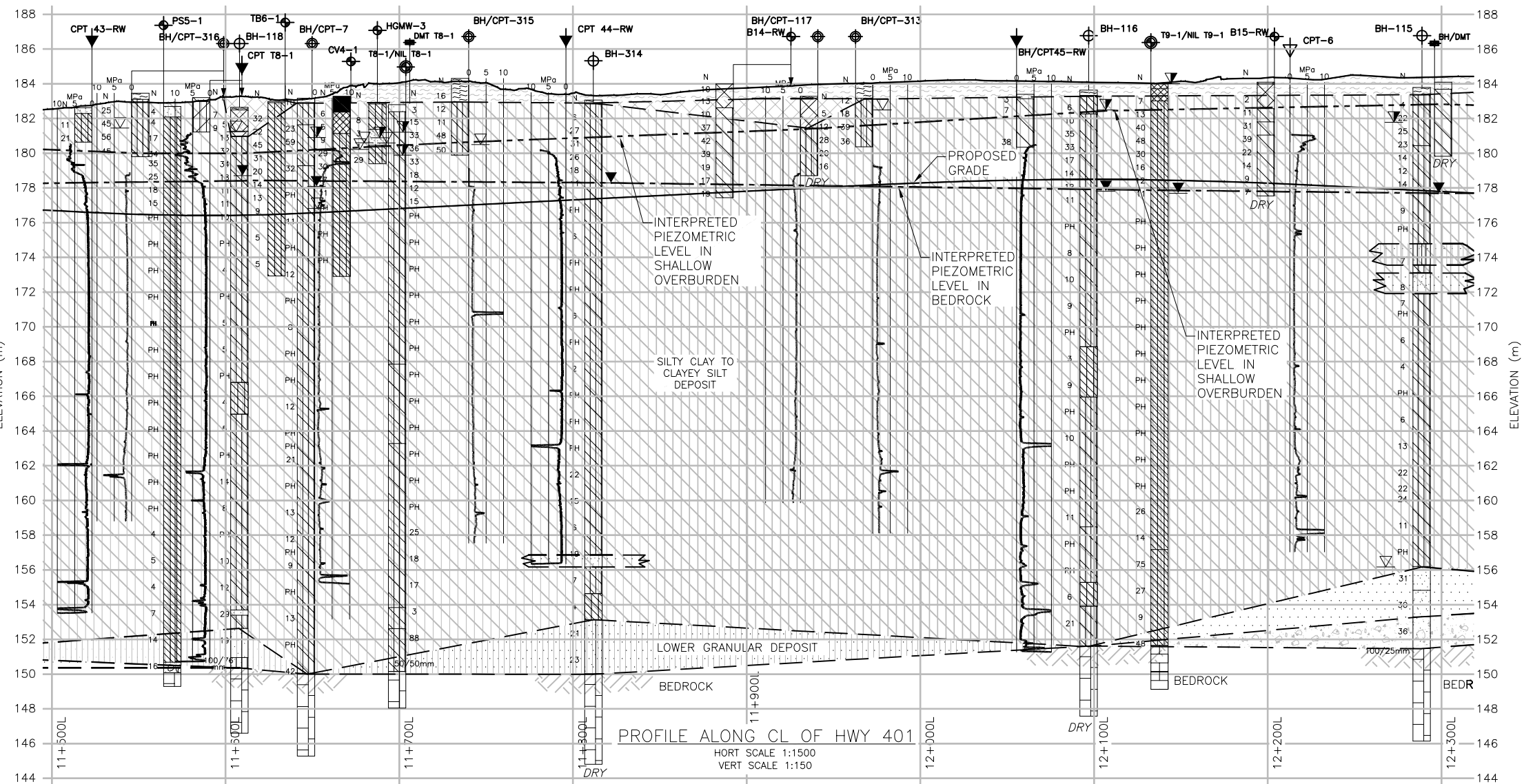
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PLAN
HORIZONTAL SCALE 1:1500

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



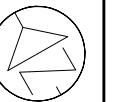
PROFILE ALONG CL OF HWY 401

HORIZONTAL SCALE 1:1500
VERTICAL SCALE 1:150

LOCATION PLAN & INTERPRETED STRATIGRAPHIC PROFILE

STA 11+500L TO STA 12+300L

REVISIONS	DATE	BY	REV.	DESCRIPTION
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DESIGN	GN	APR	NSV	DATE MAY 30/11

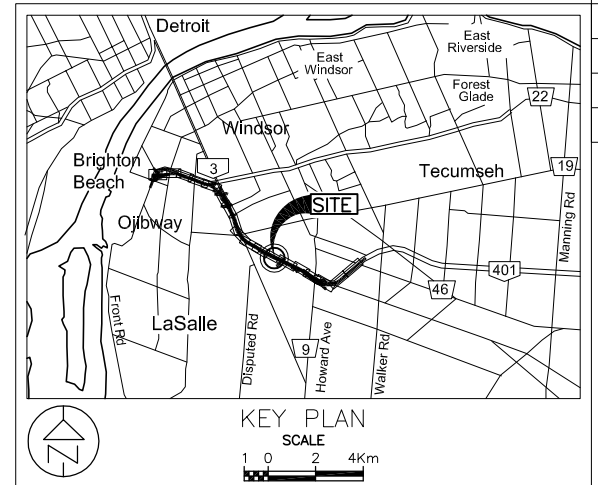


SHEET

G5401

Phase 1

IFC



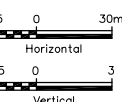
LEGEND

- 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 /BEDROCK
DOLOSTONE

NOTES

- THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
- 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.
- 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



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



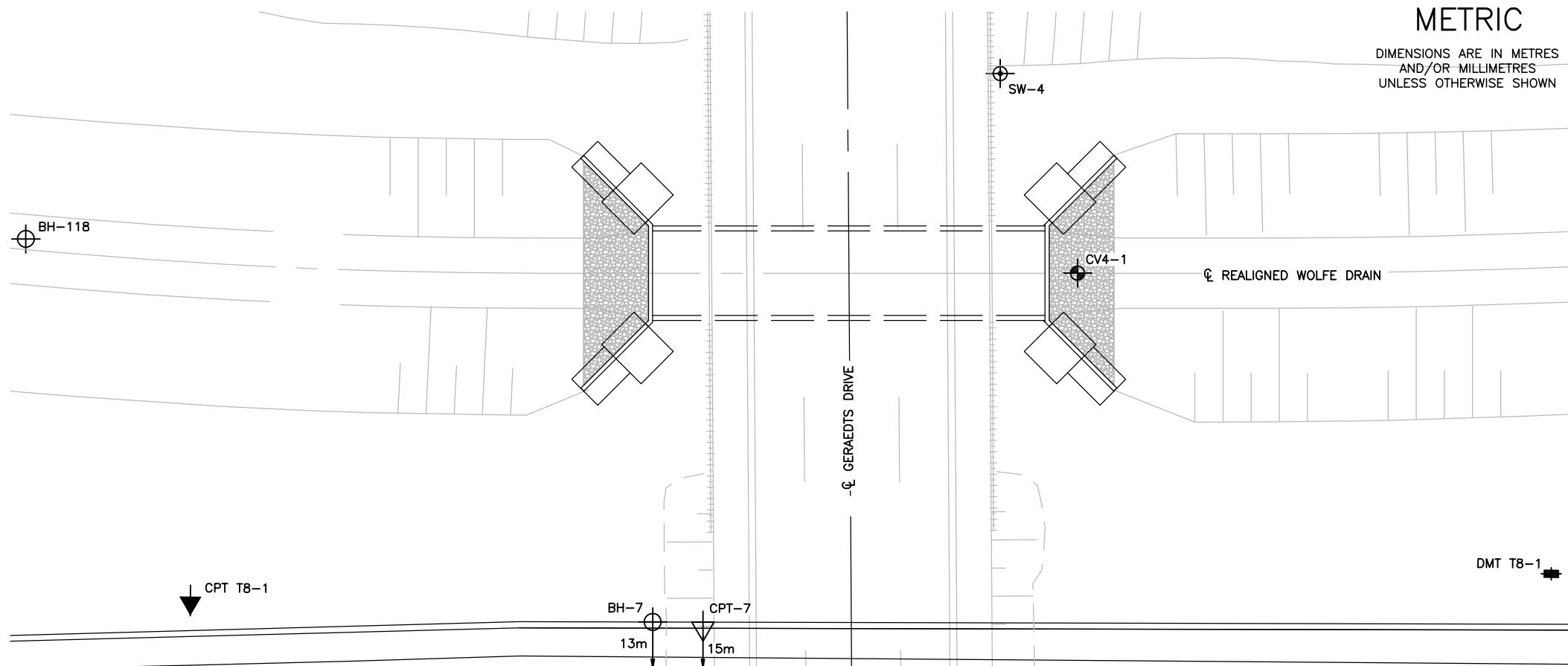
Windsor-Essex
Parkway Project
RFP No. 09-54-1007



NEW CONSTRUCTION
HWY 401
GERAEDTS DRIVE CULVERT CV-4
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
G5402

Phase 1
IFC



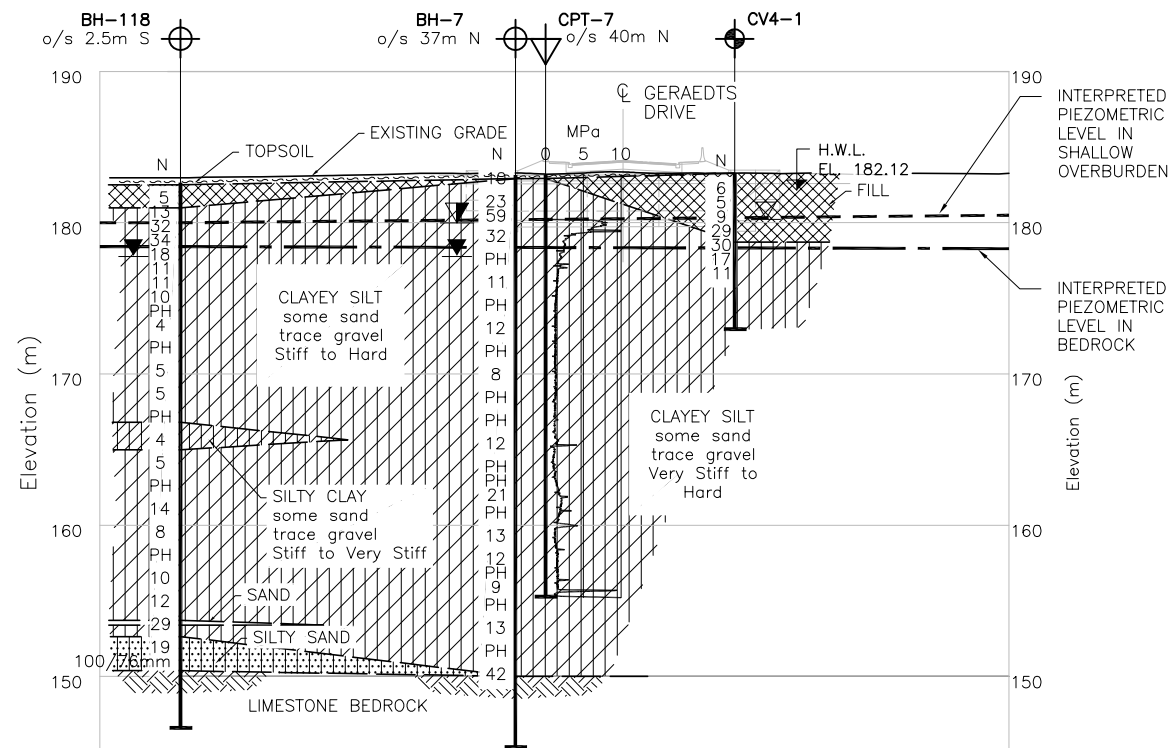
PLAN
HORIZONTAL SCALE 1:200

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

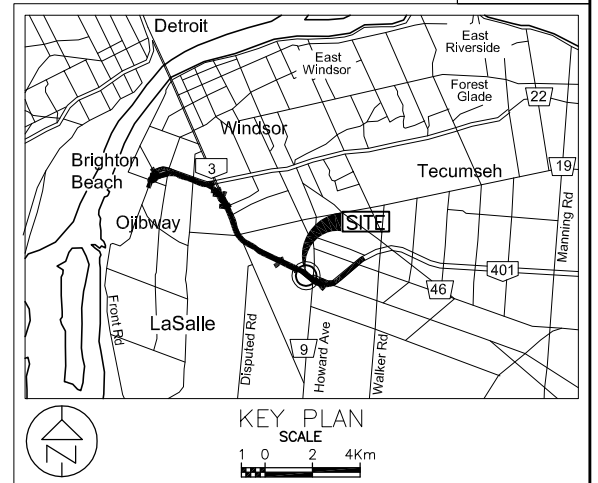
MATERIAL LEGEND

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



PROFILE ALONG CL OF CULVERT
HORIZONTAL SCALE 1:500
VERTICAL SCALE 1:250

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING



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)
- BLOWS/0.3m UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
- MHSG - MAGNETIC HEAVE/SETTLEMENT GAUGE
- P - VIBRATING WIRE PIEZOMETER
- BOREHOLE DRY DURING DRILLING
- WATER LEVEL DURING DRILLING
- WATER LEVEL (SHALLOW PIEZO)
- WATER LEVEL (DEEP PIEZO)

NOTES

- THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
- 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.
- ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

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

CONSTRUCTION NOTES – BACKFILL AT STRUCTURES

1.0 GENERAL REQUIREMENTS

- 1.1.

THESE CONSTRUCTION NOTES RELATE TO THE SUPPLY AND PLACEMENT OF BACKFILL MATERIALS AT THE STRUCTURES AT THE WINDSOR–ESSEX PARKWAY (WEP) PROJECT AS ILLUSTRATED ON THE ACCOMPANYING DRAWINGS. THE REQUIREMENTS GIVEN HEREFTER ARE THE GENERAL REQUIREMENTS. FOR DETAILED REQUIREMENTS, THE CONTRACTOR SHOULD REFER TO APPROPRIATE ONTARIO PROVINCIAL STANDARD SPECIFICATIONS (OPSS) LISTED IN SECTION 1.5.
- 1.2.

THESE CONSTRUCTION NOTES ARE TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN DRAWINGS AND REPORT.
- 1.3.

FOR LIGHTWEIGHT FILL (LWF), REFER TO CONSTRUCTION NOTES FOR LIGHTWEIGHT FILL MATERIAL.
- 1.4.

FOR EXPANDED POLYSTYRENE (GEOFOAM, EPS) FILL, REFER TO CONSTRUCTION NOTES FOR EXPANDED POLYSTYRENE FILL.
- 1.5.

THESE REQUIREMENTS DO NOT APPLY TO THE HIGHWAY PAVEMENT CONSTRUCTION.
- 1.6.

THE CONSTRUCTION WORKS SHALL BE EXECUTED IN ACCORDANCE WITH THE GEOTECHNICAL DESIGN ILLUSTRATED ON THE ACCOMPANYING DRAWINGS, THE SUPPLIER SPECIFICATIONS AND THE REQUIREMENTS SPECIFIED IN THE FOLLOWING STANDARDS, SPECIFICATIONS AND PUBLICATIONS:
- ASTM D422

•

ASTM D2216

•

ASTM D2850

PARTICLE–SIZE ANALYSIS OF SOILS
MOISTURE CONTENT OF SOILS
UNCONSOLIDATED–UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS

•

ASTM D2922

•

ASTM D3017

•

ASTM D5856

DENSITY OF SOIL AND SOIL–AGGREGATE IN PLACE BY NUCLEAR METHODS
WATER CONTENT OF SOIL AND ROCK IN PLACE BY NUCLEAR METHODS
HYDRAULIC CONDUCTIVITY OF POROUS MATERIALS USING A RIGID WALL PERMEAMETER

•

OPSS 201

•

OPSS 206

•

OPSS 212

•

OPSS 401

•

OPSS 501

•

OPSS 517

CLEARING, CLOSE CUT CLEARING, GRUBBING, REMOVAL OF SURFACE AND PILES BOULDERS
GRADING
BORROW
TRENCHING, BACKFILLING AND COMPACTING
COMPACTING
DEWATERING AT PIPELINE, UTILITY AND ASSOCIATED STRUCTURE EXCAVATION

•

OPSS 518

•

OPSS 805

•

OPSS 902

CONTROL OF WATER FROM DEWATERING OPERATIONS
TEMPORARY EROSION AND SEDIMENT CONTROL MEASURES
CONSTRUCTION SPECIFICATIONS FOR EXCAVATING AND BACKFILLING – STRUCTURES

•

OPSS 1001

•

OPSS 1004

•

OPSS 1010

AGGREGATES – GENERAL
AGGREGATES – MISCELLANEOUS
AGGREGATES – BASE, SUBBASE, SELECT SUBGRADE AND BACKFILL MATERIAL

•

OPSS 1860

•

OPSD 208.010

GEOTEXTILE
BENCHING OF EARTH SLOPES

1.7.

IF THERE IS ANY CONFLICT BETWEEN THE REQUIREMENTS GIVEN ON THIS DRAWING AND THE STANDARDS AND SPECIFICATIONS DOCUMENTS LISTED IN SECTION 1.3, THE DESIGNER SHOULD BE CONSULTED FOR CLARIFICATION AND RECOMMENDATIONS.

1.8.

IN THE FOLLOWING CONSTRUCTION NOTES, THE CONTRACTOR MEANS PIC AND ITS SUB–CONTRACTORS, THE SUPPLIER MEANS THE MANUFACTURER AND PROPRIETARY SUPPLIER, THE ENGINEER MEANS THE GEOTECHNICAL SITE ENGINEER, AND THE DESIGNER MEANS THE GEOTECHNICAL DESIGNER OF THE PROJECT.
- 2.0 SITE PREPARATION AND EXCAVATION
- 2.1

CLEARING AND GRUBBING AREA SHALL EXTEND MINIMUM 3 M BEYOND THE FOOTPRINT AREA OF THE STRUCTURE, OR AS REQUIRED BY THE ENGINEER. THE TREES AND SHRUBS REMOVED FROM THE GROUND SHALL BE TRANSPORTED TO DESIGNATED AREAS.

2.2

THE STRIPPING AREA SHALL EXTEND MINIMUM 1 M BEYOND THE FOOTPRINT AREA OF THE STRUCTURE, OR AS REQUIRED BY THE ENGINEER. ALL PEAT/MUSKEG, WETLAND VEGETATION AND OTHER UNSUITABLE MATERIAL SHOULD BE STRIPPED AND TRANSPORTED TO DESIGNATED AREAS.

2.3

CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS.

2.4

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. NATIVE DEWATERED SOILS AT THE SITE AND COMPACTED FILLS MAY BE CLASSIFIED IN GENERAL AS TYPE 3 SOILS. UNDEWATERED FILLS, NATIVE SAND AND SILTS, AND WATER BEARING BACKFILL WITHIN TRENCHES OF ACTIVE AND/OR ABANDONED UTILITIES MAY DEVELOP TYPE 4 SOIL CONDITIONS AND SHALL BE ADDRESSED ACCORDINGLY.
- 2.5

THE SOILS AT THE PROJECT SITE ARE HIGHLY SUSCEPTIBLE TO RAPID DETERIORATION WHEN EXPOSED TO ELEMENTS, WEATHERING, WATER INFLOW AND PONDING, DISTURBANCE FROM CONSTRUCTION TRAFFIC, AND THE LIKE. SUBGRADE SOILS AND BACKFILL IN PROGRESS SHALL BE APPROPRIATELY PROTECTED AT ALL TIMES AGAINST SURFACE EROSION, DESICCATION, AND FREEZE–THAW EFFECTS, REGULARLY INSPECTED AND MONITORED, AND TREATED AS REQUIRED.

2.6

TO PROTECT THE SUBGRADE INTEGRITY, THE FINAL EXCAVATION LAYER ABOVE THE DESIGN ELEVATION IN GENERAL SHOULD NOT BE LESS THAN 0.5 M AND SHOULD BE CARRIED OUT ONLY WHEN THE CONTRACTOR IS READY TO PREPARE AND COVER/PROTECT THE SUBGRADE SAME DAY THE FINAL EXCAVATION IS EXPOSED AND APPROVED.

2.7

NO CONSTRUCTION TRAFFIC SHOULD BE PERMITTED OVER THE SUBGRADE WITHOUT APPROVED PROTECTIVE COVERS.

2.8

THE SUBGRADE EXCAVATION SHALL BE CUT TO NEAT LINES AND GRADES USING BUCKETS EQUIPPED WITH SMOOTH LIPS. ONCE EXPOSED, THE SUBGRADE MUST BE IMMEDIATELY INSPECTED. UPON APPROVAL, THE SUBGRADE SURFACE SHOULD BE COVERED WITH SKIM COAT OF LEAN CONCRETE MUD MAT, GRANULAR OVER GEO–FABRIC, GRANULAR OVER SUBGRADE, ETC., AS APPROVED BY THE ENGINEER, FOR PROTECTION AGAINST DISTURBANCE AND TO PROVIDE A WORKING SURFACE.

2.9

THE TEMPORARY EXCAVATION SURFACES SHALL BE BENCHED ACCORDING TO OPSD 208.010. UNLESS THE GRANULAR BACKFILL IS FILTER GRADED WITH RESPECT TO THE NATIVE SUBGRADE MATERIAL, A GEOTEXTILE LAYER (TERRAFIX 360R OR EQUIVALENT) SHALL BE PLACED AT THE BENCHED INTERFACE BETWEEN THE EXCAVATED SURFACE AND THE GRANULAR BACKFILL TO FUNCTION AS A SEPARATOR AND PREVENT MIGRATION OF FINES.

2.10

IF PRESENCE OF GASSY SOILS IS EVIDENCED (FOR EXAMPLE, DISSOLVED GAS BUBBLES COMING OUT OF SOLUTION AND/OR SOFTENING OF THE EXCAVATION FACE), THE EXCAVATION PROGRESS SHALL BE REVIEWED WITH THE ENGINEER IN TERMS OF TIMING, STAGING AND OTHER MITIGATION MEASURES.

2.11

THE CONTRACTOR SHOULD EMPLOY APPROPRIATE GROUND IMPROVEMENT APPROACH (E.G., SUITABLE FILL LAYER, GEOGRID SHEET, ETC.) TO FACILITATE CONSTRUCTABILITY, WHERE REQUIRED, AS APPROVED BY THE ENGINEER.

2.12

THE SUBGRADE SHOULD BE SLOPED APPROPRIATELY TO ACHIEVE POSITIVE DRAINAGE OF SEEPAGE AND SURFACE WATER TO SUBDRAINS, DITCHES OR SUMPS TO AVOID PONDING BENEATH ANY FILL PLACED. NO PONDING OR FLOODING SHALL BE ALLOWED TO OCCUR IN AREAS OF FINAL EARTHWORKS (SEE SECTION 6 ON DRAINAGE – REQUIREMENTS).
- 3.0 REINFORCED GRANULAR MAT (RGM)
- 3.1

THE RGM ARE REINFORCED SOIL MATS COMPRISING SELECT COMPACTED GRANULAR FILL AND REINFORCEMENT (GEOSYNTHETICS OR METALLIC)

3.2

THE LOCATION SEGMENTS AND WIDTHS OF THE RGM ARE INDICATED ON APPROPRIATE DRAWINGS.

3.3

GRANULAR FILL FOR RGM: THE FILL MATERIAL SHALL BE GRANULAR A OR GRANULAR B TYPE II (OPSS 1010).

3.4

REINFORCEMENT FOR RGM: AS PER DESIGN DRAWINGS
- 4.0 FILL MATERIALS
- 4.1

ALL FILL MATERIALS TO BE USED FOR TUNNEL AND BRIDGE CONSTRUCTION SHALL BE INERT MATERIAL, FREE OF ORGANIC MATERIAL AND DELETERIOUS SUBSTANCES. ALL FILL MATERIALS SHALL BE APPROVED BY THE ENGINEER AT THE BORROW SOURCE AND AT PLACEMENT LOCATION.

4.2

SILTY CLAY FILL: THE UPPER CLAY CRUST ZONE MATERIAL OBTAINED FROM REQUIRED EXCAVATIONS IN THE DEPRESSED SEGMENTS OF THE WEP OR OTHER SOURCES APPROVED BY THE ENGINEER SHALL BE USED AS PER DRAWINGS PROVIDED IT MEETS THE OPSS 902 REQUIREMENTS AND CAN BE COMPACTED TO AT LEAST 95% SPMDD. THE SUITABILITY OF THE CLAY FILL MATERIALS SHALL BE VERIFIED IN TERMS OF ITS GRADATION (E.G., SILTY CLAY TO CLAYEY SILT), PLASTICITY CHARACTERISTICS (LOW TO MEDIUM PLASTICITY INDEX) AND THE IN–SITU MOISTURE CONTENT. ALL SUITABLE METHODS TO ACHIEVE THE SPECIFIED PLACEMENT MOISTURE CONTENT SHALL BE EMPLOYED.

4.3

GRANULAR FILL FOR GENERAL BACKFILL: THE GRANULAR FILL MATERIAL SHALL BE GRANULAR B TYPE I OR II, OR ALTERNATIVE GRANULAR MATERIALS APPROVED BY THE ENGINEER. THE SUITABILITY OF GRANULAR FILL MATERIALS SHALL BE DETERMINED AS PER THE OPSS 1010 STANDARD AND THE REQUIREMENTS OF THE RSS/RGM SUPPLIER.

4.4

RIPRAP: THE RIPRAP MATERIAL FOR EROSION PROTECTION OF PERMANENT SLOPES AND CHANNEL SURFACES SHALL BE R–10 (MINUS 180 MM) FOR LIGHT TO MEDIUM EROSION RISK CONDITIONS AND R–50 (MINUS 305 MM) FOR HIGH RISK CONDITIONS, AS SHOWN ON THE DESIGN DRAWINGS OR AS REQUIRED BY THE ENGINEER (OPSS 1004). GEOTEXTILE SHALL BE USED AT INTERFACE BETWEEN THE SOIL SLOPES AND RIPRAP LAYER TO PREVENT LOSS OF MATERIAL FROM THE SOIL SLOPE.

4.5

LWF AND EPS: SEE RESPECTIVE CONSTRUCTION NOTES.
- METRIC
- DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN
-
- Windsor–Essex
Parkway Project
RFP No. 09–54–1007
-
- NEW CONSTRUCTION
HWY 401
GERAEDTS DRIVE CULVERT CV–4
CONSTRUCTION NOTES – BACKFILL AT STRUCTURES
- SHEET
G5408
- Phase 1
- IFC
- 5.0 FILL PLACEMENT AND COMPACTION
- 5.1 GENERAL:
- THE CONSTRUCTOR SHALL SUBMIT TO THE ENGINEER THEIR QC/QA INSPECTION AND TEST PLAN FOR REVIEW/COMMENT PRIOR TO THE PLACEMENT/COMPACTION OF FILL.

•

FILL SHALL NOT BE PLACED ON SURFACES HAVING STANDING WATER, OR SURFACES WHICH HAVE BEEN RUTTED AND HEAVED BY TRAFFICKING. FILL SHALL NOT BE PLACED ON FROZEN SURFACES. FROZEN FILL IS DEFINED AS MATERIALS WITH SOIL WATER IN FROZEN STATE.

•

ALL EARTHWORKS TO BE ADEQUATELY PROTECTED AGAINST EROSION, FROST AND WATER INGRESS UNTIL THE LANDSCAPING REQUIREMENTS HAVE BEEN INSTALLED (SEE SECTIONS 2.6 TO 2.8).
- 5.2

IF NOT SPECIFIED IN THE CONTRACT DOCUMENTS, TARGET DENSITIES WILL BE ESTABLISHED UTILIZING CONTROL STRIPS AS PRESENTED IN OPSS 501. THE MINIMUM TARGET DENSITIES SHALL BE AS PER NOTES 5.3 AND 5.4.

5.3

THE SILTY CLAY FILL SHALL BE PLACED IN MAXIMUM 200 MM THICK LOOSE LIFTS AND COMPACTED AT WOPT±2% MOISTURE CONTENT TO A MINIMUM OF 95% SPMDD UNLESS OTHERWISE SPECIFIED IN THE CONTRACT DOCUMENTS. THE TERMS WOPT AND SPMDD REFER TO OPTIMUM WATER CONTENT AND MAXIMUM DRY DENSITY, RESPECTIVELY, DETERMINED BY STANDARD PROCTOR TESTS.

5.4

THE GRANULAR FILL MATERIALS SHALL BE PLACED IN MAXIMUM 300 MM THICK LOOSE LIFTS AND COMPACTED AT WOPT±2% MOISTURE CONTENT TO A MINIMUM OF 95% SPMDD UNLESS OTHERWISE SPECIFIED IN THE CONTRACT DOCUMENTS.

5.5

THE COMPACTION EQUIPMENT SHALL BE APPROPRIATE FOR THE MATERIAL TO BE COMPACTED AND THE SITE CONDITIONS, AND SHOULD BE PROPOSED TO THE ENGINEER FOR APPROVAL. ADEQUATE NUMBER OF PASSES SHALL BE EMPLOYED TO ACHIEVE THE SPECIFIED PLACEMENT DENSITIES. HEAVY COMPACTION EQUIPMENT SHOULD NOT BE EMPLOYED NEAR STRUCTURAL WALLS.

5.6

COMPACTION AND PLACEMENT OF GRANULAR MATERIALS FOR RSS WALLS WILL CONFORM TO THE MANUFACTURES RECOMMENDATIONS.

5.7

FILL PLACEMENT SHALL CONFORM TO THE REQUIREMENTS PRESENTED IN OPSS 501. THE CONTRACTOR SHOULD USED APPROPRIATELY SIZED EQUIPMENT TO AVOID DAMAGING ANY STRUCTURES, DEGRADING THE AGGREGATE, OR EPS BLOCKS.
- 6.0 DRAINAGE – DEWATERING
- 6.1

REFER TO OPSS 518 FOR DEWATERING REQUIREMENTS.

6.2

THE CONSTRUCTION SITE WILL BE KEPT CLEAN AND DRY, FREE OF WATER PUDDLES, MUD AND DEBRIS.

6.3

MINOR TO SIGNIFICANT SEEPAGE FROM RUNOFF INFILTRATIONS OR PERCHED WATER WITHIN UPPER GRANULAR DEPOSITS AND/OR FILL IS ANTICIPATED. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE TEMPORARY DEWATERING SYSTEM.
- 7.0 USE
- 7.1

THIS DRAWING PROVIDES CONSTRUCTION REQUIREMENTS FOR GEOTECHNICAL ASPECTS OF BACKFILLING AT CULVERTS.
- DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING
- | | | | | | |
|-----------|-----------|------|-----|-------------------------|----------------|
| REVISIONS | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
| | 29–FEB–12 | 0 | TL | ISSUED FOR CONSTRUCTION | |
| | DATE | REV. | BY | DESCRIPTION | |
| DESIGN | SF | CHK | NSV | CODE CAN/CSA S6-06 | LOAD CL–625–ON |
| DRAWN | MM | CHK | DD | SITE 6–630 | DATE 20–DEC–11 |
- DOC: 285380–04–094–WP1–5408

Figures

Figure 3.1 Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures

(Figure 5.1, Ladd & DeGroot, 2004, ref. R-18)

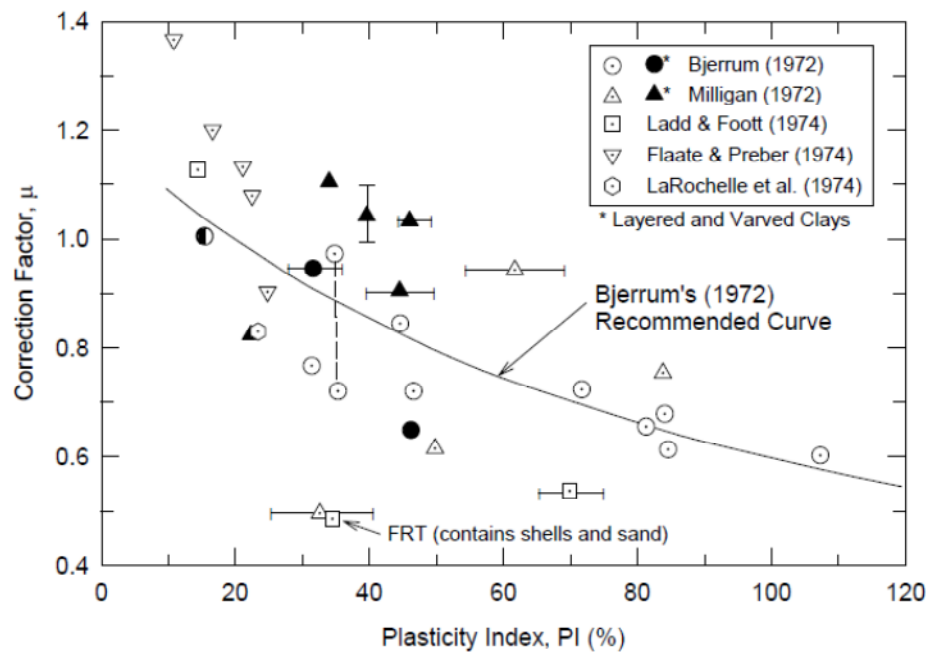
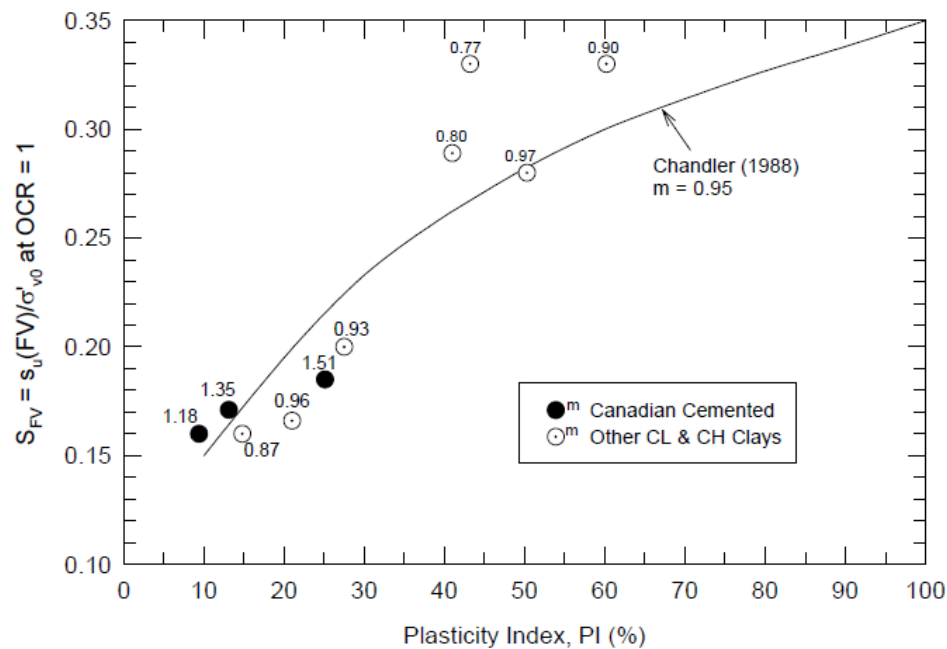
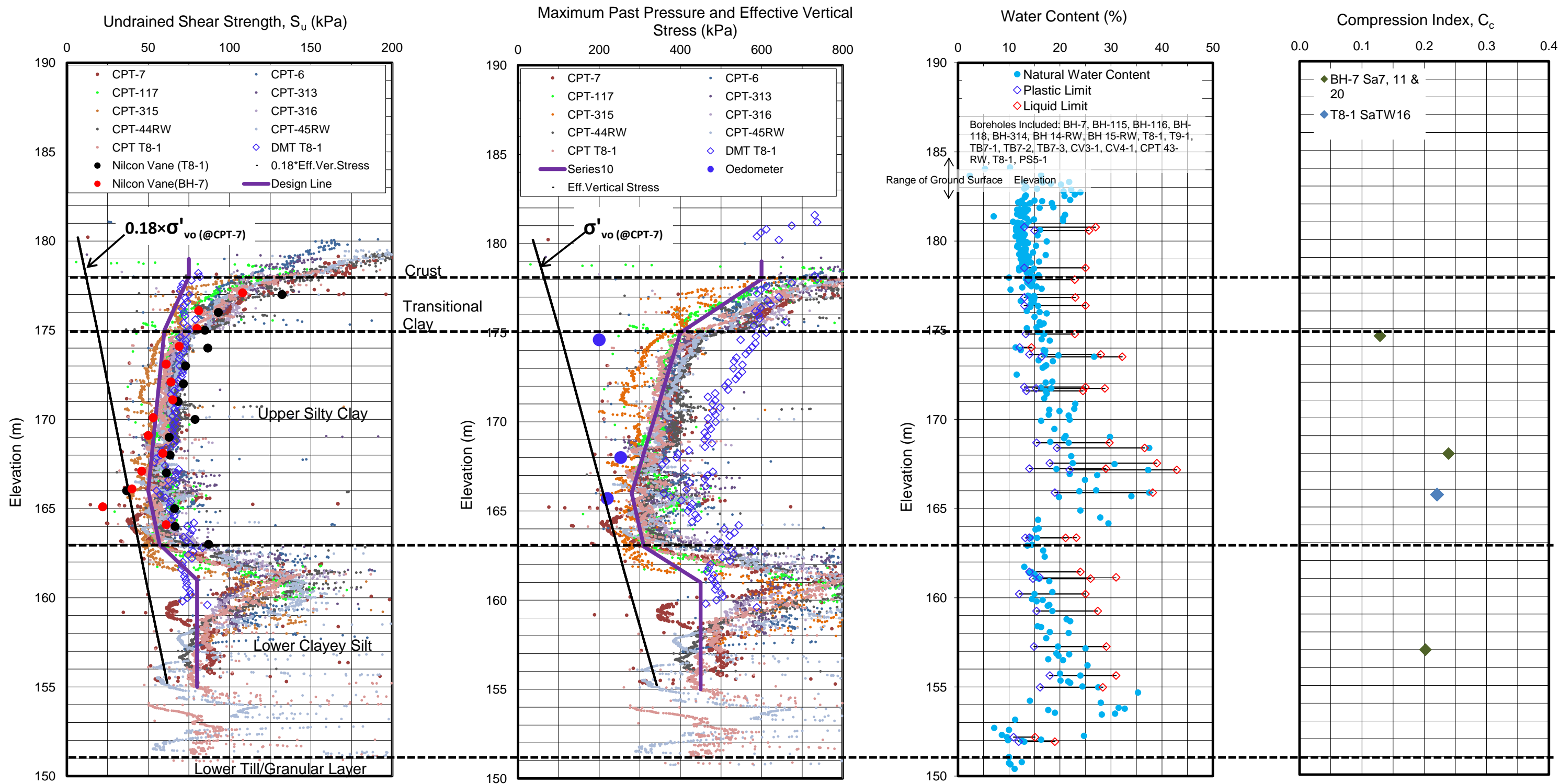


Figure 3.2 Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays (Figure 5.2, Ladd & DeGroot, 2004, ref. R-18)





Notes:

1. Shear strength profiles were estimated from CPT data using the equation $S_u = (q_t - \sigma_{v0}) / N_{KT}$. 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 derived from the current investigation by AMEC and historic data from Golder Associates.



Earth & Environmental

CLIENT :

PROJECT:

WINDSOR ESSEX PARKWAY

TITLE:

**SOIL PROPERTIES PROFILES
STA.11+500L TO 12+300L**

DATE:

Feb 2012

JOB NO.:

SW8801.1002

CAD FILE:

FIGURE NO.:

3.3

REV.

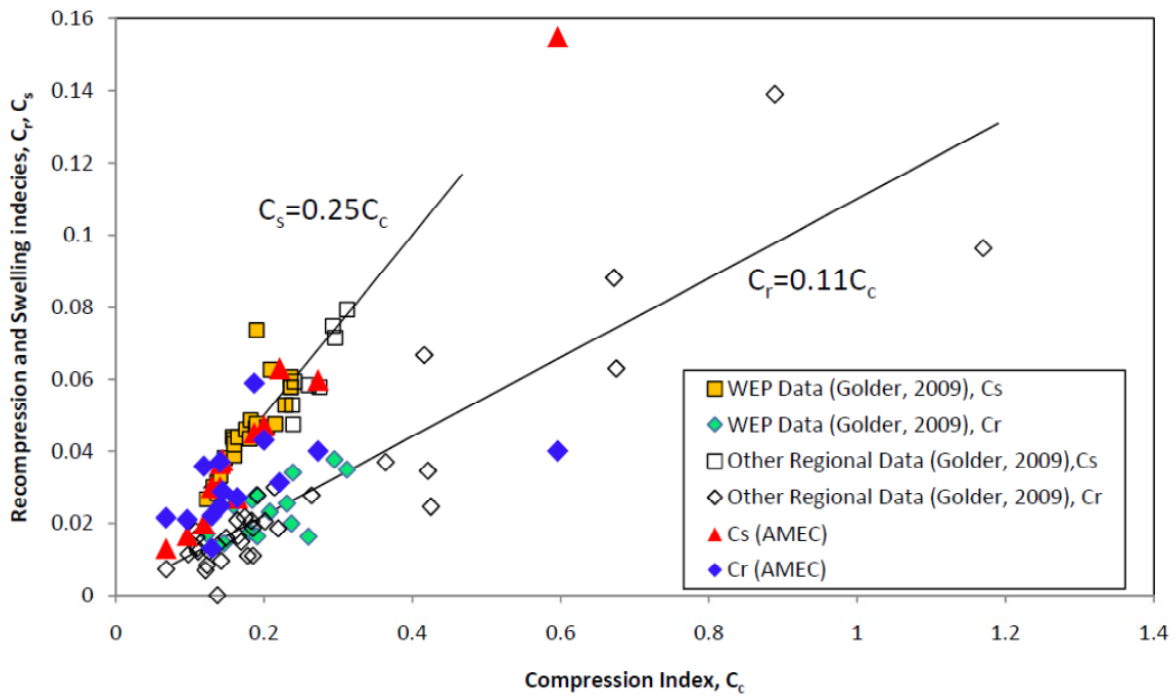
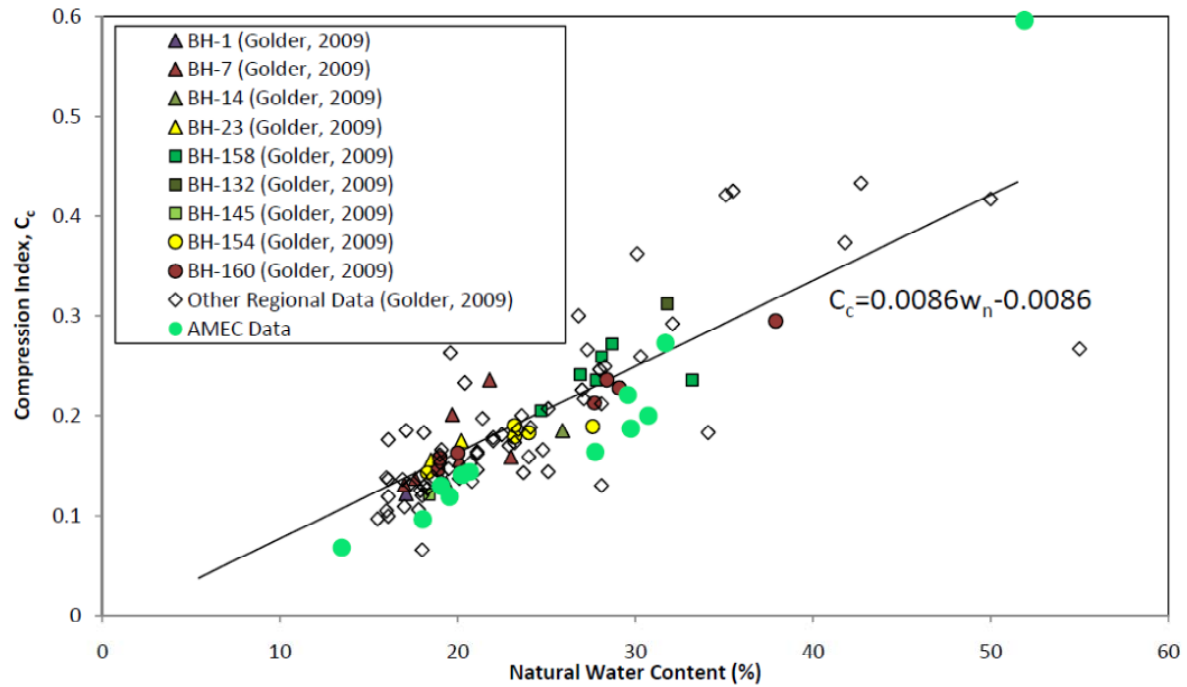


Figure 5.1 Data Summary of Compression Indices C_c , C_s and C_r
(Figure 6.7, Golder 2009, Ref. 9)

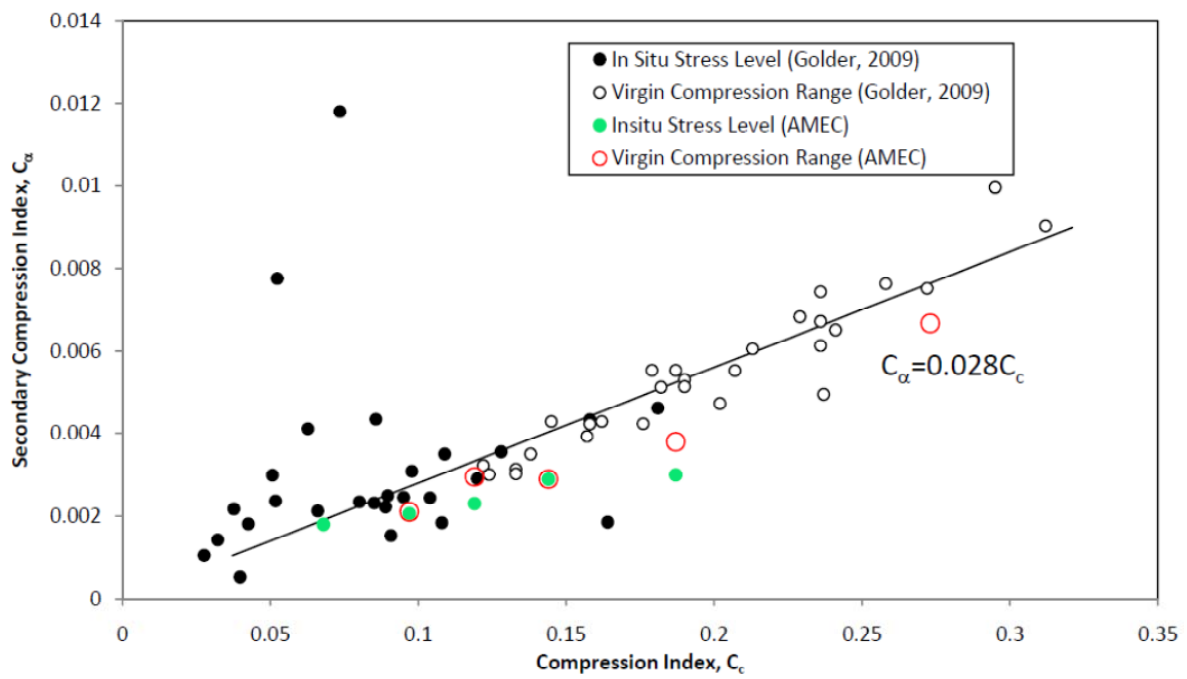
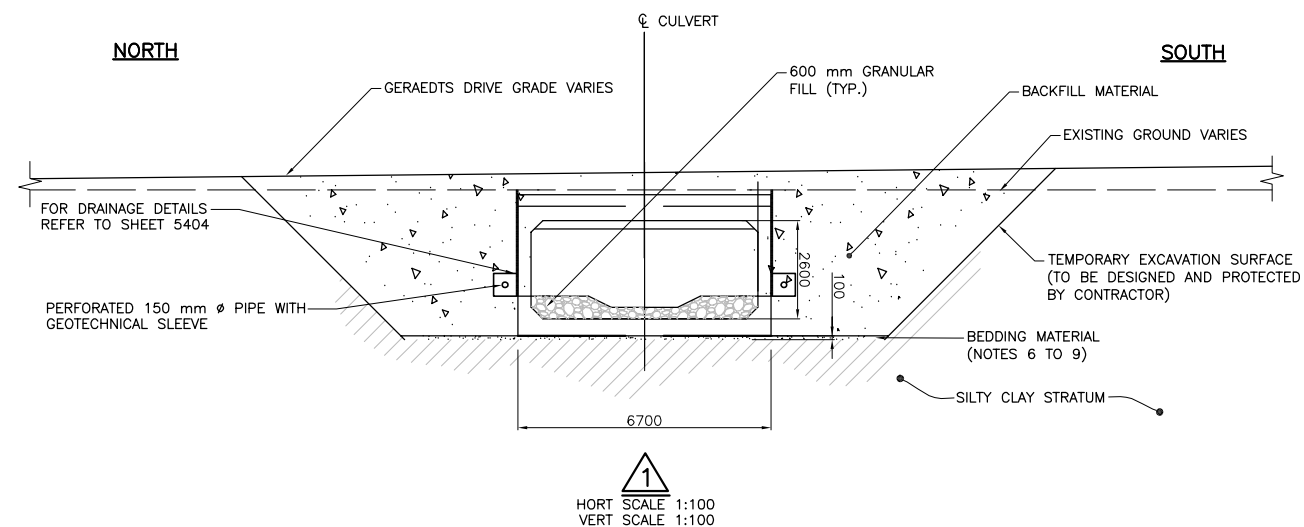
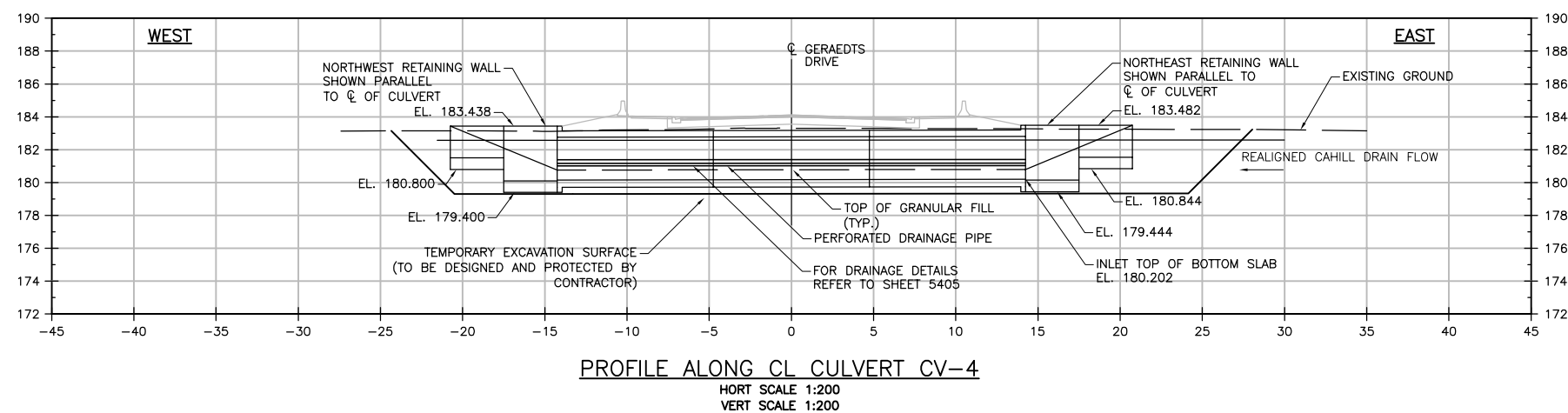
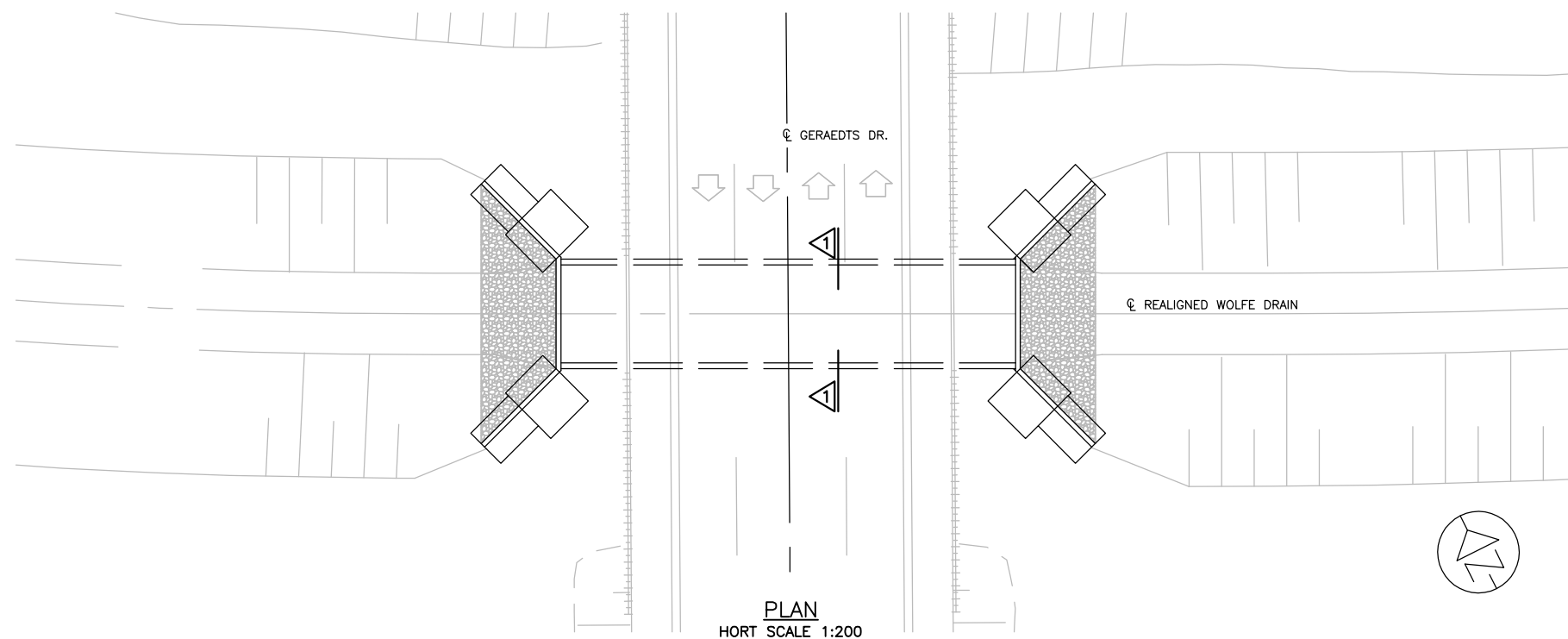


Figure 5.2 Data Summary of Compression Indices C_c and C_α
 (Figure 6.7, Golder 2009, Ref. 9)



NOTES:

1. THIS FIGURE ILLUSTRATES THE GEOTECHNICAL DESIGN ARRANGEMENT OF CULVERT CV-4. 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, AND AT LEAST 100 mm OF CONCRETE WITHIN THE AREAS OF THE PRECAST STRUCTURE.
6. BEDDING SHALL BE AS SPECIFIED IN THE CONTRACT AND CONSTRUCTED TO PROVIDE A UNIFORM SUPPORT FOR THE FULL LENGTH AND WIDTH OF EACH BOX UNIT.
7. BEDDING MATERIAL SHALL NOT BE PLACED ON A DISTURBED OR FROZEN EARTH GRADE.
8. BEDDING REQUIRING COMPACTION SHALL BE PLACED IN LAYERS NOT EXCEEDING 200 mm IN THICKNESS, LOOSE MEASUREMENT, AND EACH LAYER SHALL BE COMPACTED IN CONFORMANCE TO OPSS 501.
9. THE SURFACE TO SUPPORT THE PRECAST BOX UNITS SHALL BE PREPARED IN ACCORDANCE WITH THE CULVERT DESIGN AND SUPPLIER REQUIREMENTS.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR
CONSTRUCTION

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

RECORD OF BOREHOLE No CV4-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678867.9, E333368.7 ORIGINATED BY DG
 DIST HWY WEP BOREHOLE TYPE CME 850 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Aug 27, 11 - Aug 27, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100	20 40 60 80 100	10 20 30								
183.3 0.0	Fill Surface					▽	183											
182.4 0.9	75mm ASPHALT Over FILL, sand and gravel						182							○				
	FILL Silty Clay/Clayey Silt Some topsoil, trace fine gravel, trace sand, brown		1	SS	6										○			
			2	SS	5											○		
181.2 2.1	CLAYEY SILT Some sand, trace fine-coarse gravel Stiff to hard Mottled brown-grey		3	SS	9			181							○			
			4	SS	29			180								○		
			5	SS	30			179								○		
	Grey		6	SS	17			178								○		
			7	SS	11			177								○		
			8	TW	PH			176								○		
			VT					175										
			9	TW	PH			174								○		
			10	TW	PH			173										
172.9 10.4	END OF BOREHOLE (no refusal) Groundwater observed at 3.0 m (El. 180.3 m) during drilling on Aug. 27, 2011		VT				172											
							171											
							170											
							169											

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 22/11/11

METRIC

Continued Next Page

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

RECORD OF BOREHOLE No T8-1

2 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678789.7, E333364.5 ORIGINATED BY NB
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Jul 19, 11 - Jul 20, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED ● POCKET PEN.	+ FIELD VANE × LAB VANE								
14.9	SILTY CLAY Some silt nodules Firm to stiff Grey, some pink nodules <i>(continued)</i>		15	TW	PH		167						20.4	1 19 35 45			
				VT													
					16		TW	PH	166								
									165								
					17		TW	PH	164								
					VT												
									163								
					18		TW	PH	162								
163.3 19.5	CLAYEY SILT Some sand, trace gravel Very stiff Grey					161							22.02	-end of drilling July 19; continue July 20 -VWP T8-1-P21 installed at 20.6m below ground surface (El. 162.2 m) -MG T8-1-SM20 installed at 19.7 m below ground surface (El. 163.1 m) -attempt at vane shear test exceeded max torque of apparatus			
					19	TW	PH	160									
								159									
								158									
					21	SS	25	157									
								156									
								155									
					23	SS	17	154									
								153									
153.8 29.0	SILTY CLAY Some silt seams Soft Grey Wet		24	SS	3												

Continued Next Page

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

ONTARIO MOT. SW8801.1004.101.GPJ ONTARIO MOT.GDT 22/11/11

METRIC

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

RECORD OF BOREHOLE No TB6-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678909.5, E333353.3 ORIGINATED BY TA
DIST HWY WEP BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE Jul 9, 11 - Jul 9, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE								
								20 40 60 80 100									10 20 30		
183.0	Ground Surface																		
182.9	TOPSOIL																		
0.1	Mottled Brown-Grey CLAYEY SILT Some sand, trace gravel Sandy, dry		1	SS	32														
	Brown -Trace fissures Very Stiff		2	SS	22														
	-Trace inferred cobbles, trace fissures Hard		3	SS	45														
	Grey Very stiff		4	SS	31														
	Stiff		5	SS	20														
			6	SS	14														
			7	SS	13														
			8	SS	9														
			9	SS	5														
				VT															
			10	SS	5														
				VT															
172.9	END OF BOREHOLE (no refusal)																		
10.1	Borehole dry on completion																		

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 22/09/11

RECORD OF BOREHOLE No HG-MW-3

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678886.8, E333395.5 ORIGINATED BY TA
 DIST HWY WEP BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Jul 9, 11 - Jul 9, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● POCKET PEN. × LAB VANE									
				WATER CONTENT (%)													
182.9	Ground Surface						20	40	60	80	100						GR SA SI CL
180.9	TOPSOIL Brown CLAYEY SILT Some sand, trace gravel, trace topsoil		1	SS	8												-Observation Well installed in borehole
181.4																	
181.5	Brown Poorly-Graded SAND Trace gravel, trace silt		2	SS	3												9 68 13 11
180.5																	
180.5	Brown CLAYEY SILT Some sand, trace gravel Trace fissures		3A, B	SS	1												
179.4			4	SS	29												
179.4	END OF BOREHOLE (no refusal) Piezometric levels in observation well: July 29, 2011: EL. 180.9m October 13, 2011: EL. 180.6m																
179.5																	
						</											

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

RECORD OF BOREHOLE No CPT T8-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678860.0, E333292.9 ORIGINATED BY TA
DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE Aug 4, 11 - Aug 4, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20	40	60	80	100						20	40	60
183.2	Fill Surface																			
180.0	FILL Crushed Limestone, Grey																			
0.2	FILL Clayey silt, some gravel, brown																			
182.4	SANDY SILT Some clay, trace gravel Mottled brown-grey to brown		1	SS	7															
0.8																				
181.2			2	SS	9															
2.0	END OF SAMPLED BOREHOLE Continue with CPT from 2 m to refusal at 32.4 m (El. 181.2 m to El. 150.8 m) No groundwater observed on Aug. 4, 2011																			
							181													
							180													
							179													
							178													
							177													
							176													
							175													
							174													
							173													
							172													
							171													
							170													
							169													

RECORD OF CONE PENETRATION TEST CPT T8-1

METRIC

PROJECT Windsor-Essex Parkway

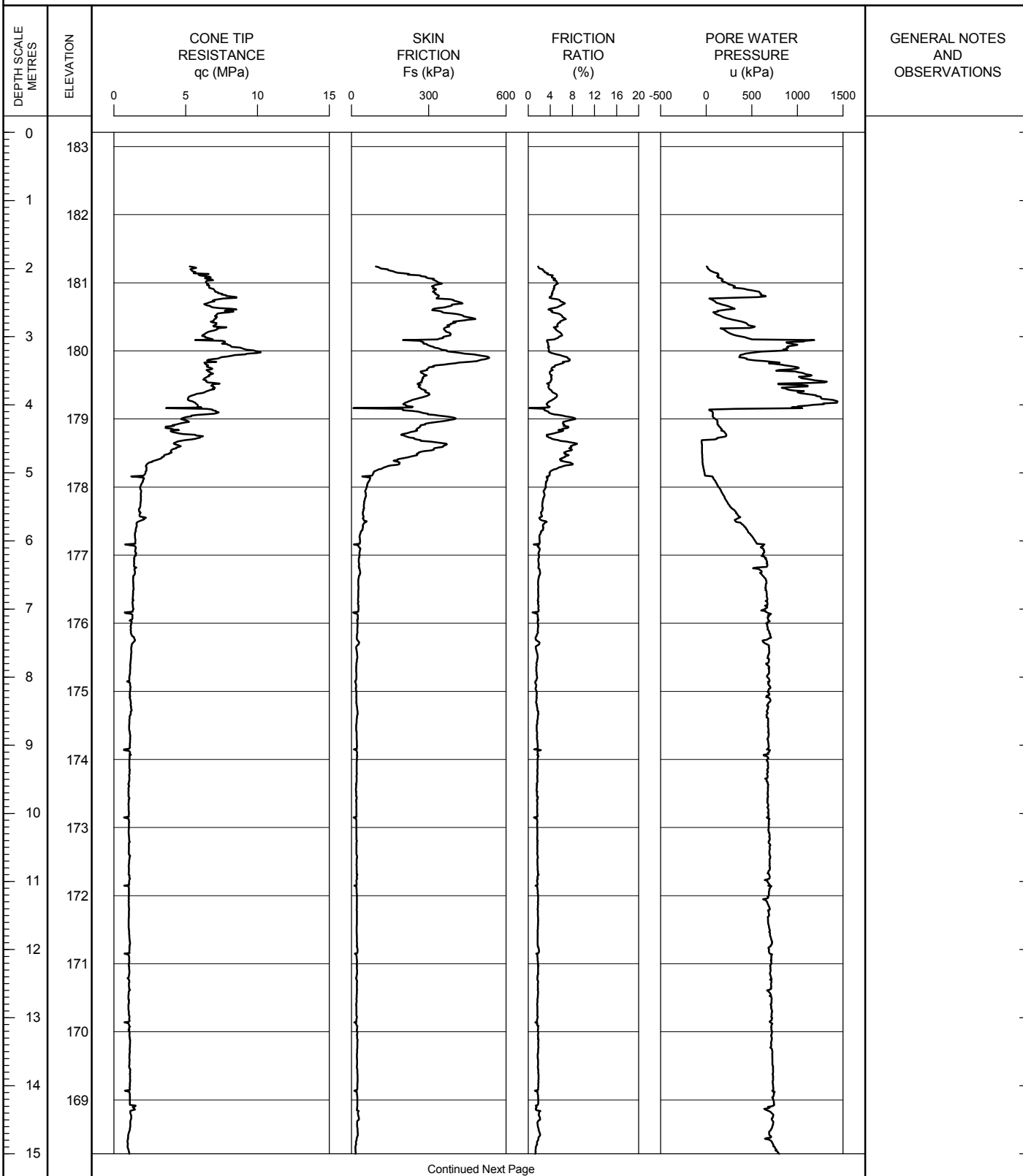
TEST DATE 8/4/2011 - 8/4/2011

SHEET 1 OF 3

LOCATION N4678860.0; E333292.9

DATUM Geodetic

GROUND SURFACE ELEVATION: 183.2 PREDRILL DEPTH: 1.82 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEP CPT LOG CPT T8-1.GPJ ONTARIO.MOT.GDT 22/12/11

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T8-1

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 8/4/2011 - 8/4/2011

SHEET 2 OF 3

LOCATION N4678860.0; E333292.9

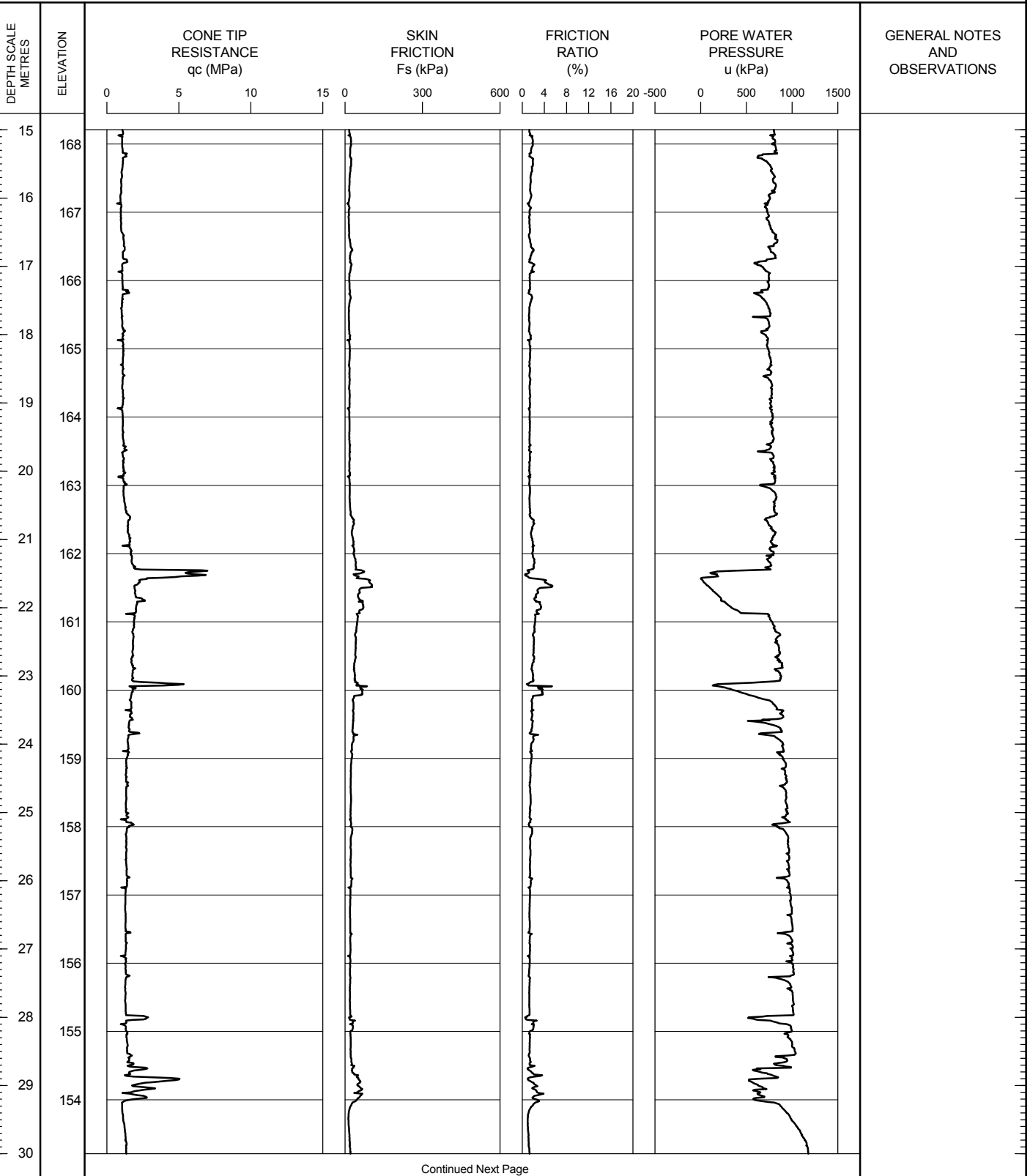
DATUM Geodetic

GROUND SURFACE ELEVATION: 183.2

PREDRILL DEPTH: 1.82

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T8-1

METRIC

PROJECT Windsor-Essex Parkway

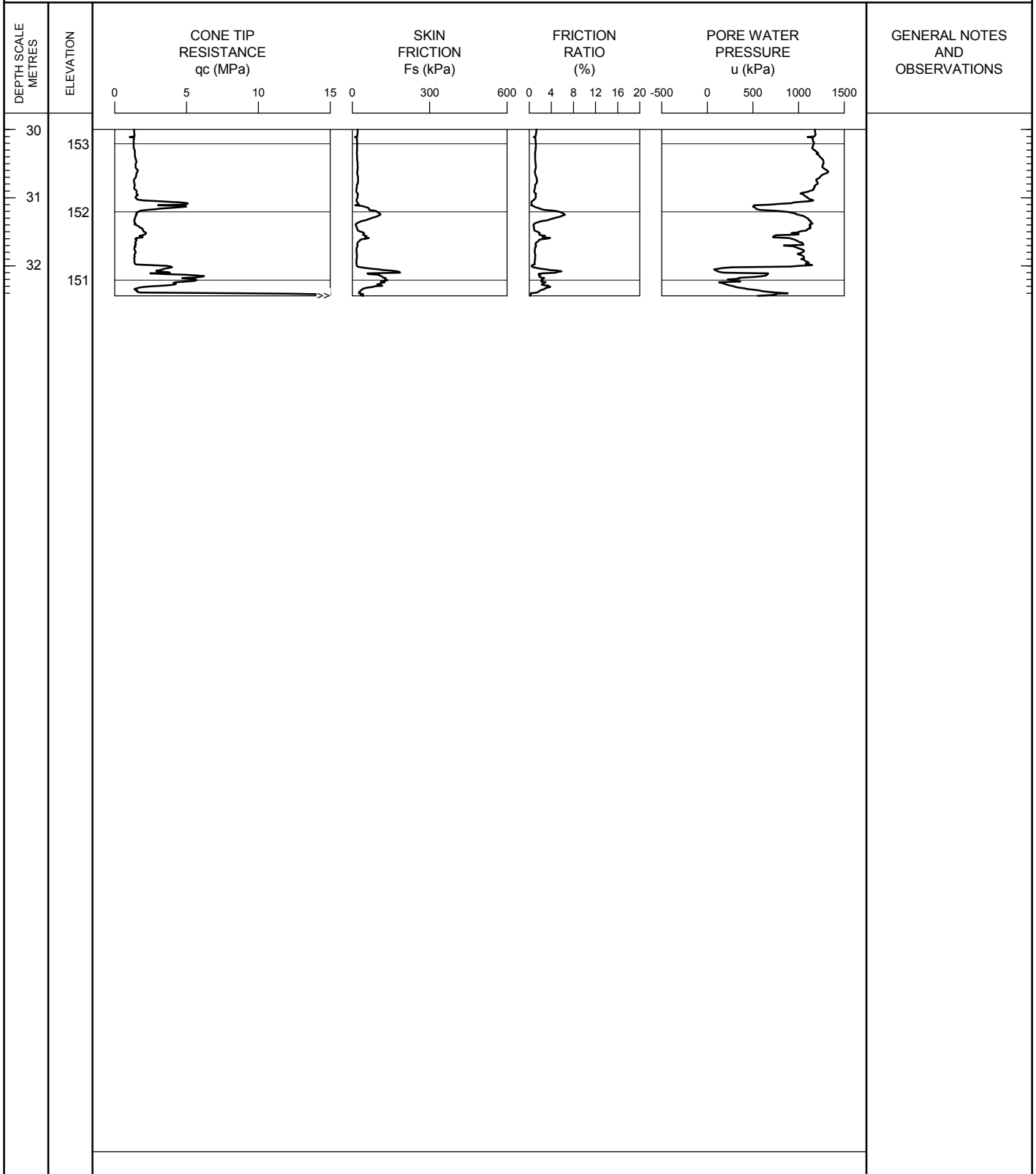
TEST DATE 8/4/2011 - 8/4/2011

SHEET 3 OF 3

LOCATION N4678860.0; E333292.9

DATUM Geodetic

GROUND SURFACE ELEVATION: 183.2 PREDRILL DEPTH: 1.82 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

Appendix B Borehole Logs from Previous Investigations

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report – Culvert CV-4
(Realigned Cahill Drain, 9+954.81 Geraedts Drive, LaSalle)
Doc No.: 285380-04-119-0022(Geocres No. 40J3-11)

Date: February/2012
Rev: 0
Page No.: Appendix B

RECORD OF BOREHOLE No 7

1 OF 4

METRIC

PROJECT 04-1111-060

W.P.

LOCATION

N 4678848.0 :E 333325.0

ORIGINATED BY C.C.

DIST WEST HWY 401 / 3

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

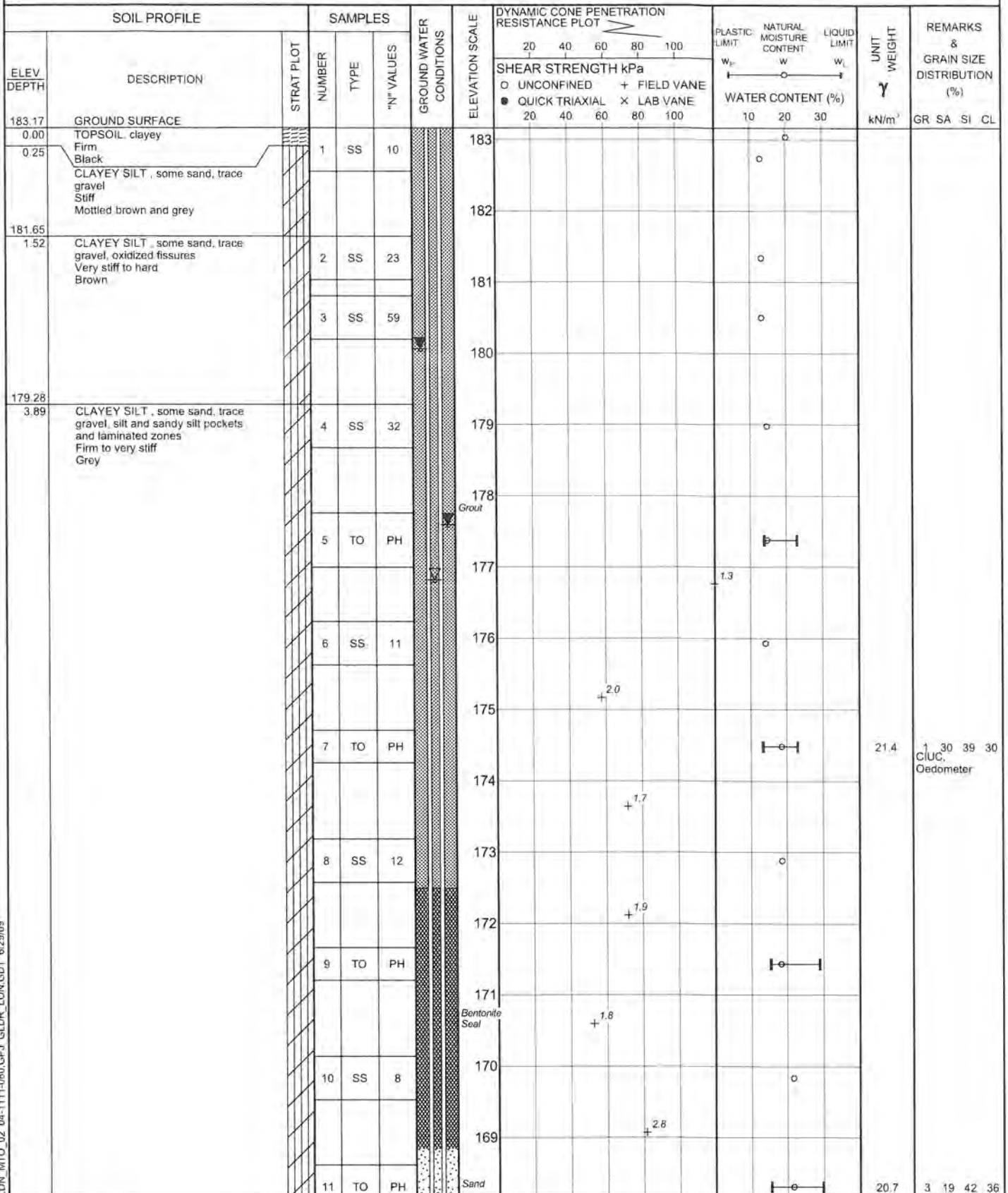
COMPILED BY T.M.

DATUM Geodetic

DATE

November 10, 2006 - November 16, 2006

CHECKED BY *SB*



Continued Next Page

+ 3 x 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

LON_MTO_02 04-1111-060.GPJ GLDR_LON.GDT 6/29/09

PROJECT 04-1111-060		RECORD OF BOREHOLE No 7		2 OF 4	METRIC
W.P.	LOCATION	N 4678848.0 : E 333325.0		ORIGINATED BY C.C.	
DIST WEST HWY 401/3	BOREHOLE TYPE	POWER AUGER/HOLLOW STEM		COMPILED BY T.M.	
DATUM Geodetic	DATE	November 10, 2006 - November 16, 2006		CHECKED BY SJS	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED 20 40 60 80 100	+ FIELD VANE 20 40 60 80 100						
	CLAYEY SILT , some sand, trace gravel, silt and sandy silt pockets and laminated zones Firm to very stiff Grey														
							168							CIUC, Oedometer	
									3.7						
			12	TO	PH		167						43		
							166		3.3						
			13	SS	12										
							165								
									1.8						
			14	TO	PH		164								
			15	TO	PH		163								
			16	SS	21		162								
									>95.7						
			17	SS	PH		161								
							160		>143.6						
			18	SS	13		159								
									2.0						
							158								
			19	SS	12										
			20	TO	PH		157						21.0	2 19 42 37 CIUC, Oedometer	
			21	SS	9		156								
									>95.7						
							155								
			22	SS	PH										
							154								

LDN_MTO_02 04-1111-060.GPJ GLDR LON.GDT 6/29/03

Continued Next Page

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

RECORD OF BOREHOLE No 7

3 OF 4

METRIC

PROJECT 04-1111-060

W.P.

LOCATION

N 4678848.0 ; E 333325.0

ORIGINATED BY C.C.

DIST WEST HWY 401 / 3

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY T.M.

DATUM Geodetic

DATE

November 10, 2006 - November 16, 2006

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
								● QUICK TRIAXIAL	× LAB VANE						
						20 40 60 80 100	20 40 60 80 100	10 20 30							
150.02	CLAYEY SILT , some sand, trace gravel, silt and sandy silt pockets and laminated zones Firm to very stiff Grey		23	SS	13		153								
							152								
			24	SS	PH										
							151								
			25	SS	42										
33.15	LIMESTONE, fresh, medium strong, laminated, very fine grained, moderately porous, light grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		26	NQ RC			150								
			27	NQ RC			149								
							148								
			28	NQ RC			147								
							146								
145.28	END OF BOREHOLE		29	NQ RC										UC	
37.89	Water level in borehole at about elevation 176.82m on October 16, 2006 Lower piezometer 32mm PVC screen and riser pipe. Second (Upper) piezometer 13mm porous tip and CPVC riser pipe. Water level in Upper Piezometer at about elevation 180.06m on November 14, 2006. Water level in Lower Piezometer at about elevation 177.59m on November 14, 2006.														

SHEET 4 OF 4

DATUM: Geodetic

DRILLING CONTRACTOR:

[illegible]

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-7

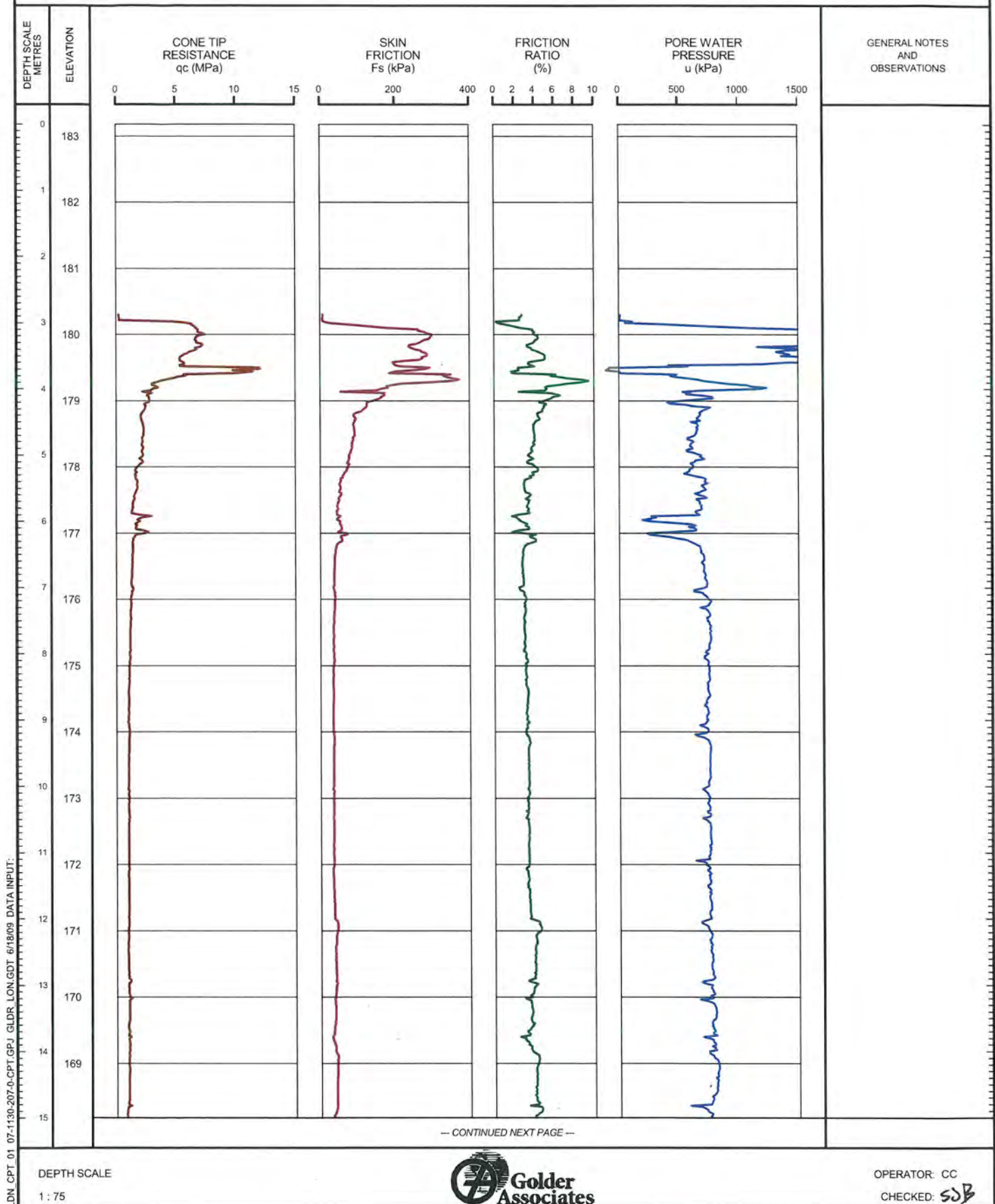
SHEET 1 OF 2

LOCATION: N 4678844.0 ;E 333327.0

TEST DATE: November 12, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 2.90m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-7

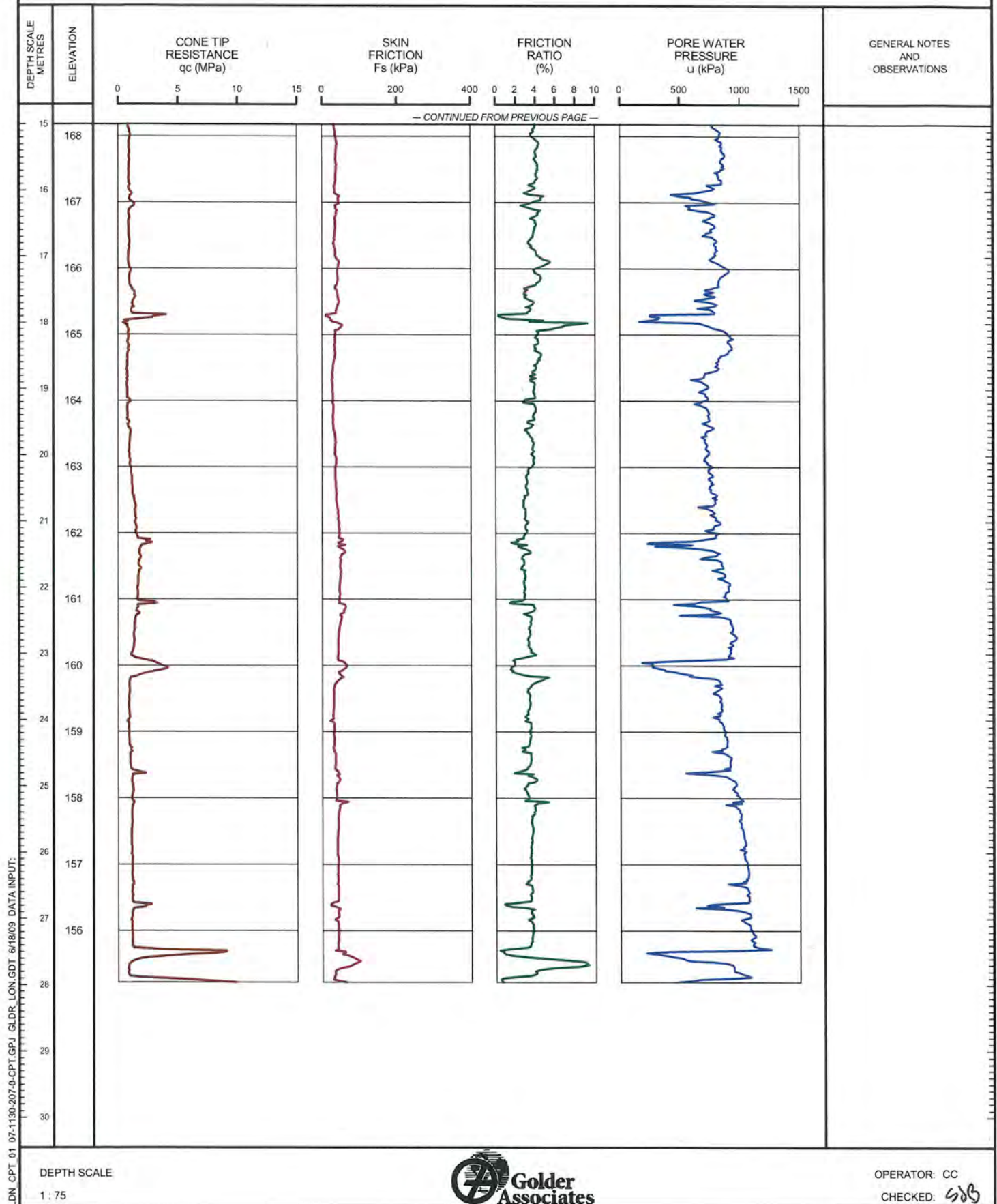
SHEET 2 OF 2

LOCATION: N 4678844.0 :E 333327.0

TEST DATE: November 12, 2006

DATUM: GEODETIC

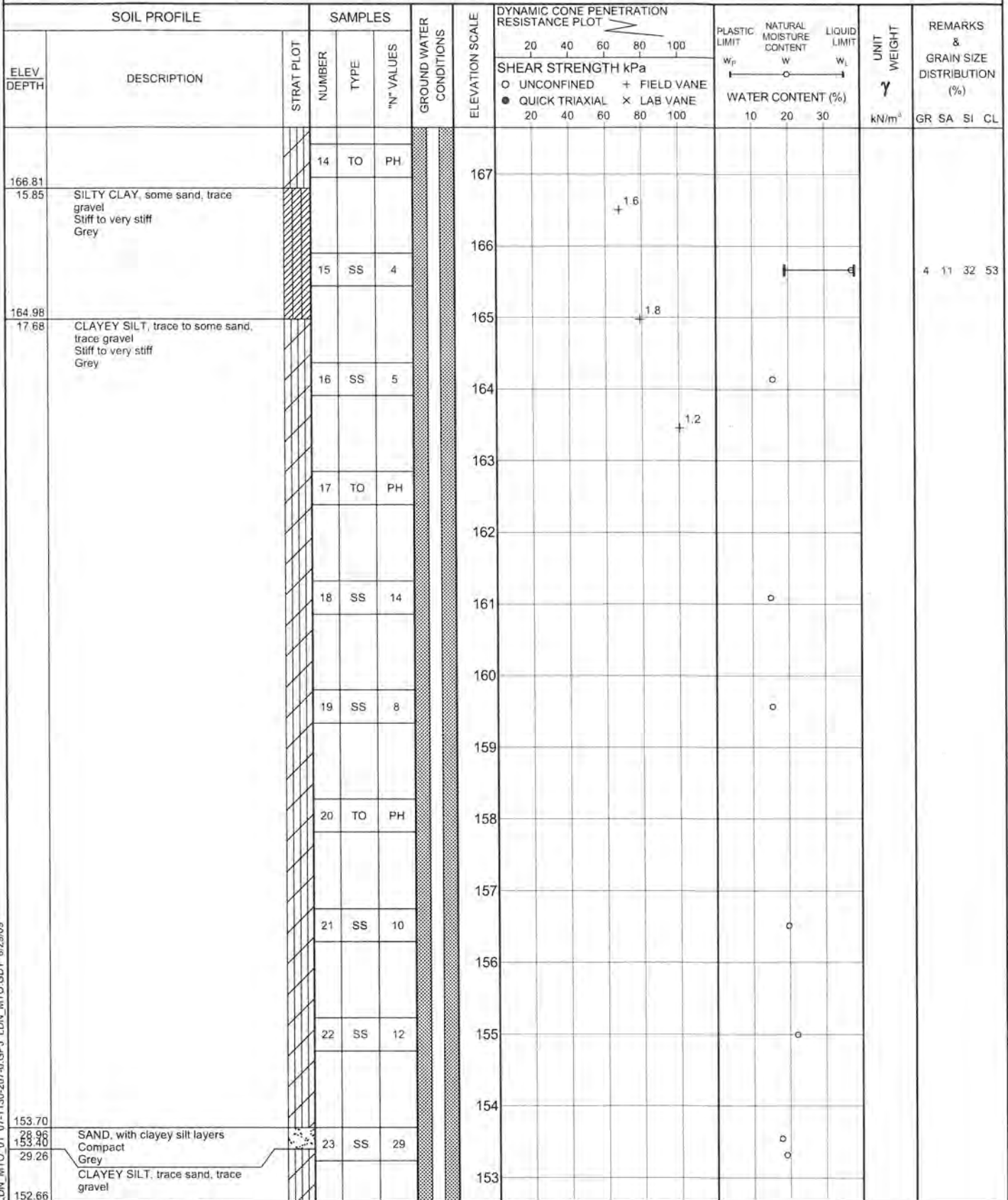
GROUND SURFACE ELEVATION: PREDRILL DEPTH: 2.90m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LON CPT 01 07-1130-207-0-CPT.GPJ GLDR LON.GDT 6/18/09 DATA INPUT:

DN MTO 01 07-1130-207-0.GPJ LDN MTO.GDT 6/29/09

PROJECT <u>07-1130-207-0</u>		RECORD OF BOREHOLE No 118		2 OF 4	METRIC
W.P. _____		LOCATION <u>N 4678903.5 :E 333302.9</u>		ORIGINATED BY <u>MA</u>	
DIST <u>WEST</u> HWY <u>401/3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>BRS</u>	
DATUM <u>GEODETIC</u>		DATE <u>February 28, 2008 - March 4, 2008</u>		CHECKED BY <u>SJB</u>	



Continued Next Page

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

LDN MTO_01 07-1130-207-0.GPJ LDN_MTO_GDT 6/29/09

RECORD OF BOREHOLE No 118

3 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4678903.5 :E 333302.9

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 28, 2008 - March 4, 2008

CHECKED BY SJS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
								● QUICK TRIAXIAL	× LAB VANE						
						20 40 60 80 100			10 20 30						
30.02	Very stiff Grey SILTY SAND, trace clay, trace gravel Compact Grey		24	SS	19		152							4 48 39 9	
150.96															
31.70	SILTY SAND AND GRAVEL, trace clay Dense Grey		25	SS	100/ 76mm		151								
150.32															
32.34	LIMESTONE, fresh, medium strong, thinly laminated, fine grained, moderately porous Whitish grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		26	NQ RC			150	100 60 25						UC	
			27	NQ RC			149	99 98 82							
			28	NQ RC			148	99 96 90							
146.61							147								
36.05	END OF BOREHOLE														
	Water levels in borehole at about elev. 181.29m, 153.70m and 150.96m during drilling between February 28 and March 4, 2008.														
	Water level measured in deep piezometer at elev. 176.77m on March 4, 2008.														
	Water level measured in deep piezometer at elev. 177.30m on March 20, 2008.														
	Water level measured in deep piezometer at elev. 177.78m on July 24, 2008.														
	Water level measured in deep piezometer at elev. 177.32m on September 19, 2008.														
	Water level measured in deep piezometer at elev. 177.28m on November 14, 2008.														
	Water level measured in deep piezometer at elev. 177.40m on January 28, 2009.														

LDN_MTO_01_07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 118

SHEET 4 OF 4

LOCATION: N 4678903.5 ;E 333302.9

DRILLING DATE: February 28, 2008 - March 4, 2008

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR FLUSH % RETURN	ELEVATION											DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION	
									RECOVERY		FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec							
									TOTAL CORE %	SOLID CORE %		TYPE AND SURFACE DESCRIPTION									
									80 60 40 20	80 60 40 20		DIP w.r.t. CORE AXIS		10 ⁻⁸ 10 ⁻⁶ 10 ⁻⁴ 10 ⁻²							

		ROCK SURFACE		150.32																
		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous, whitish grey		32.34	1			150												
33				149.56																
		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous, grey		33.10																
				149.22																
				33.44																
34		LIMESTONE, fresh, medium strong, thinly laminated, fine to very fine grained, pitted, whitish grey			2			149												
				147.97																
		LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, moderately porous, light grey		34.69																
				147.61																
				35.05																
35		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, pitted to vuggy, light brown to grey			3			147												
				146.60																
36		END OF DRILLHOLE		36.06																
37																				
38																				
39																				
40																				
41																				
42																				
43																				
44																				
45																				
46																				
47																				

3 07-1130-207-0-ROCK GPJ GLDR LDN GDT 6/29/09 DATA INPUT WDF

DEPTH SCALE

1:75



LOGGED: SG

CHECKED: SSB

Appendix C Analytical Laboratory Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report – Culvert CV-4
(Realigned Cahill Drain, 9+954.81 Geraedts Drive, LaSalle)
Doc No.: 285380-04-119-0022(Geocres No. 40J3-11)

Date: February/2012
Rev: 0
Page No.: Appendix C



AMEC EARTH & ENVIRONMENTAL-
WINDSOR

ATTN: SHANE MACLEOD

11865 County Road 42

TECUMSEH ON N8N 2M1

Date Received: 16-SEP-11

Report Date: 23-SEP-11 06:20 (MT)

Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1059690

Project P.O. #: NOT SUBMITTED

Job Reference: SW8801.1004.101

C of C Numbers: 112773

Legal Site Desc:

Gayle Braun
Senior Account Manager

[This report shall not be reproduced except in full without the written authority of the Laboratory.]

ADDRESS: 309 Exeter Road Unit #29, London, ON N6L 1C1 Canada | Phone: +1 519 652 6044 | Fax: +1 519 652 0671

ALS CANADA LTD Part of the ALS Group A Campbell Brothers Limited Company

		Sample ID Description Sampled Date Sampled Time Client ID	L1059690-1 SOIL 27-AUG-11 CV4- 1,SS7@175',GREY SILTY CLAY				
Grouping	Analyte						
SOIL							
Physical Tests	% Moisture (%)	11.4					
	pH (pH units)	7.87					
	Redox Potential (mV)	209					
	Resistivity (ohm cm)	2890					
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20					
Anions and Nutrients	Sulphate (mg/kg)	244					

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:

112773

GLOSSARY OF REPORT TERMS

Surrogate - A compound that is similar in behaviour to target analyte(s), but that does not occur naturally in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery.

mg/kg - milligrams per kilogram based on dry weight of sample.

mg/kg ww - milligrams per kilogram based on wet weight of sample.

mg/kg lwt - milligrams per kilogram based on lipid-adjusted weight of sample.

mg/L - milligrams per litre.

< - Less than.

D.L. - The reported Detection Limit, also known as the Limit of Reporting (LOR).

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.

Quality Control Report

Workorder: L1059690

Report Date: 23-SEP-11

Page 1 of 3

Client: AMEC EARTH & ENVIRONMENTAL-WINDSOR

11865 County Road 42

TECUMSEH ON N8N 2M1

Contact: SHANE MACLEOD

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT		Soil						
Batch	R2254382							
WG1351428-2	LCS							
% Moisture			94		%		70-130	19-SEP-11
WG1351428-1	MB							
% Moisture			<0.10		%		0.1	19-SEP-11
PH-WT		Soil						
Batch	R2254003							
WG1351581-1	CVS							
pH			101		%		80-120	19-SEP-11
WG1351581-2	DUP	L1059690-1						
pH		7.87	7.85		pH units	0.25	20	19-SEP-11
RESISTIVITY-WT		Soil						
Batch	R2255410							
WG1353108-1	CVS							
Resistivity			102		%		70-130	21-SEP-11
SO4-WT		Soil						
Batch	R2255430							
WG1352527-3	LCS							
Sulphate			101		%		60-140	20-SEP-11
WG1352527-1	MB							
Sulphate			<20		mg/kg		20	20-SEP-11
SULPHIDE-WT		Soil						
Batch	R2254650							
WG1352442-1	CVS							
Sulphide			107		%		50-120	20-SEP-11
WG1352431-2	DUP	L1059690-1						
Sulphide		<0.20	<0.20	RPD-NA	mg/kg	N/A	20	20-SEP-11
WG1352431-1	MB							
Sulphide			<0.20		mg/kg		0.2	20-SEP-11

Quality Control Report

Workorder: L1059690

Report Date: 23-SEP-11

Page 2 of 3

Legend:

Limit	ALS Control Limit (Data Quality Objectives)
DUP	Duplicate
RPD	Relative Percent Difference
N/A	Not Available
LCS	Laboratory Control Sample
SRM	Standard Reference Material
MS	Matrix Spike
MSD	Matrix Spike Duplicate
ADE	Average Desorption Efficiency
MB	Method Blank
IRM	Internal Reference Material
CRM	Certified Reference Material
CCV	Continuing Calibration Verification
CVS	Calibration Verification Standard
LCSD	Laboratory Control Sample Duplicate

Sample Parameter Qualifier Definitions:

Qualifier	Description
RPD-NA	Relative Percent Difference Not Available due to result(s) being less than detection limit.

Quality Control Report

Workorder: L1059690

Report Date: 23-SEP-11

Page 3 of 3

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
% Moisture	1	27-AUG-11	19-SEP-11 10:35	14	23	days	EHTR
Redox Potential	1	27-AUG-11	21-SEP-11	24	603	hours	EHTR
Resistivity	1	27-AUG-11	21-SEP-11	7	25	days	EHTR
Leachable Anions & Nutrients							
Sulphide	1	27-AUG-11	20-SEP-11 13:08	7	24	days	EHTR

Legend & Qualifier Definitions:

EHTR-FM: Exceeded ALS recommended hold time prior to sample receipt. Field Measurement recommended.
EHTR: Exceeded ALS recommended hold time prior to sample receipt.
EHTL: Exceeded ALS recommended hold time prior to analysis. Sample was received less than 24 hours prior to expiry.
EHT: Exceeded ALS recommended hold time prior to analysis.
Rec. HT: ALS recommended hold time (see units).

Notes*:

Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1059690 were received on 16-SEP-11 09:00.

ALS recommended hold times may vary by province. They are assigned to meet known provincial and/or federal government requirements. In the absence of regulatory hold times, ALS establishes recommendations based on guidelines published by the US EPA, APHA Standard Methods, or Environment Canada (where available). For more information, please contact ALS.

The ALS Quality Control Report is provided to ALS clients upon request. ALS includes comprehensive QC checks with every analysis to ensure our high standards of quality are met. Each QC result has a known or expected target value, which is compared against pre-determined data quality objectives to provide confidence in the accuracy of associated test results.

Please note that this report may contain QC results from anonymous Sample Duplicates and Matrix Spikes that do not originate from this Work Order.



112773
C of C # 00000

Notes

1. Quote number must be provided to ensure proper pricing. 2. TAT may vary dependent on complexity of analysis and lab workload at time of submission. 3. Any known or suspected hazards relating to a sample must be noted on the chain of custody in comments section.

Appendix D Slope Stability Analyses

File Name: CV4-ST & LT stability - Feb27.gsz

Last Solved Date: 27/02/2012

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

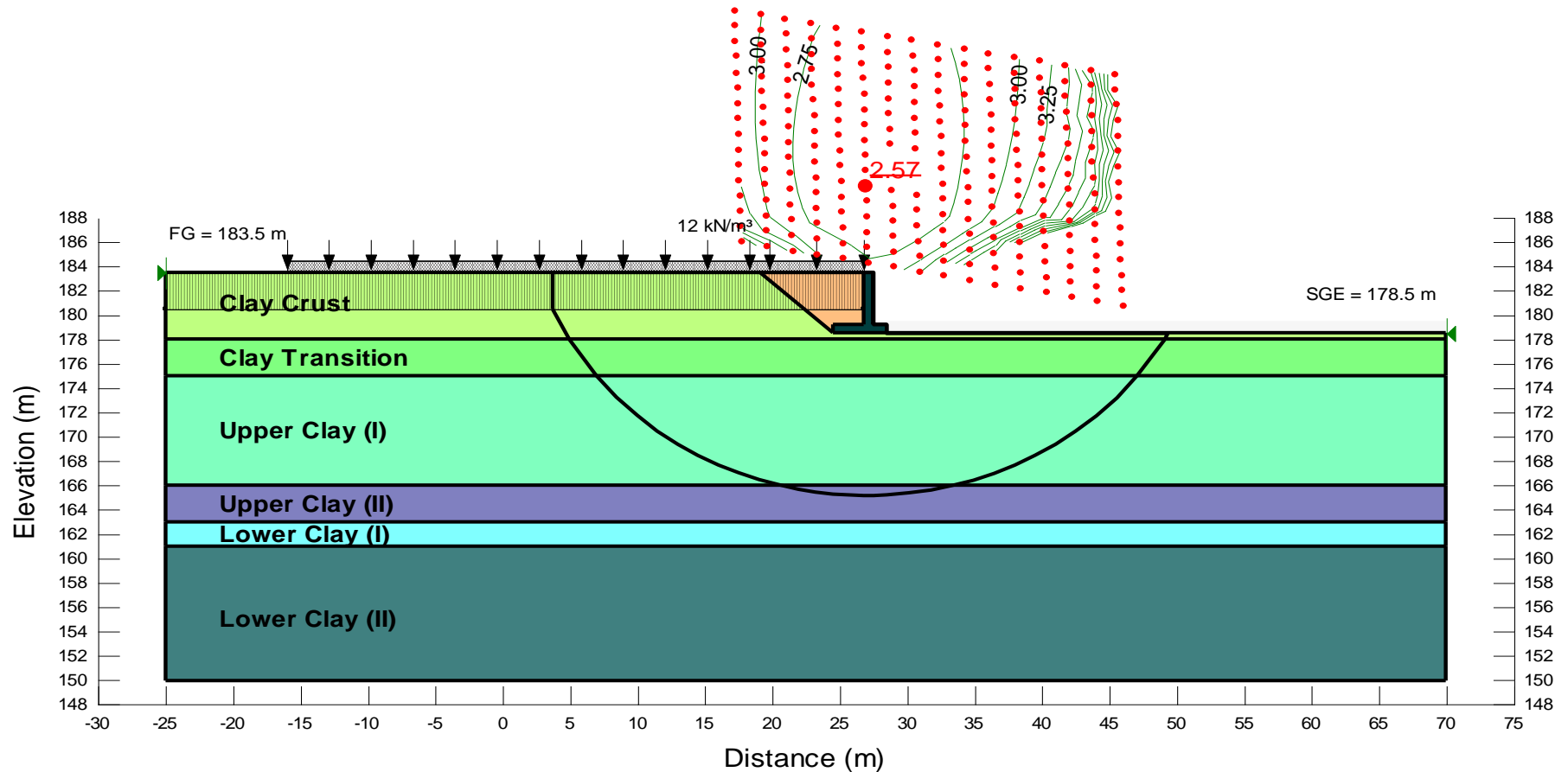
Name: Upper Clay-1 (U) Unit Weight: 20 kN/m³ C-Top of Layer: 60 kPa C-Rate of Change: -1.1 kPa/m Limiting C: 50 kPa

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

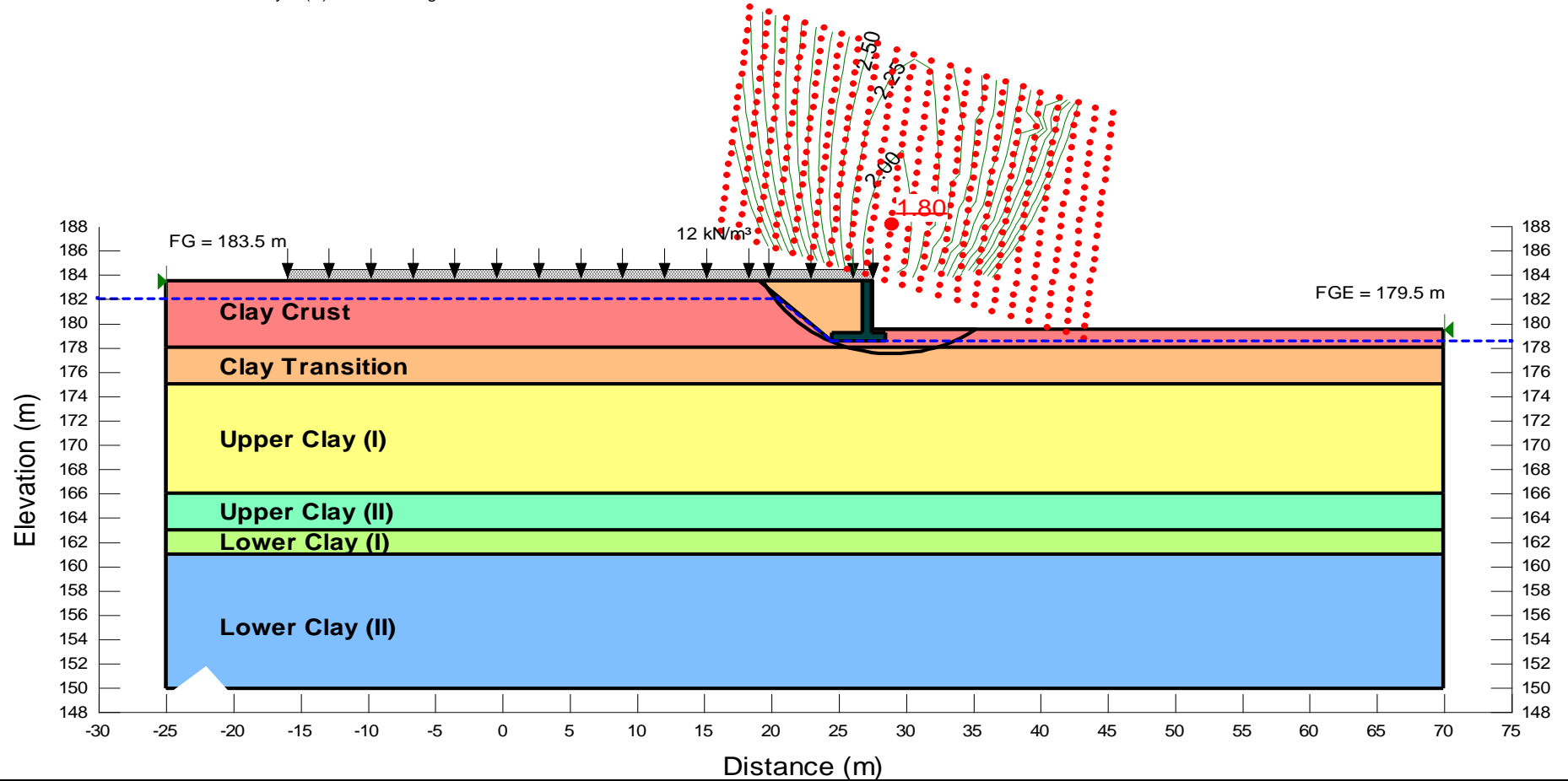
Name: Lower Clay-2 (U) Unit Weight: 21 kN/m³ Cohesion: 80 kPa



File Name: CV4-ST & LT stability - Feb27.gsz

Last Solved Date: 27/02/2012

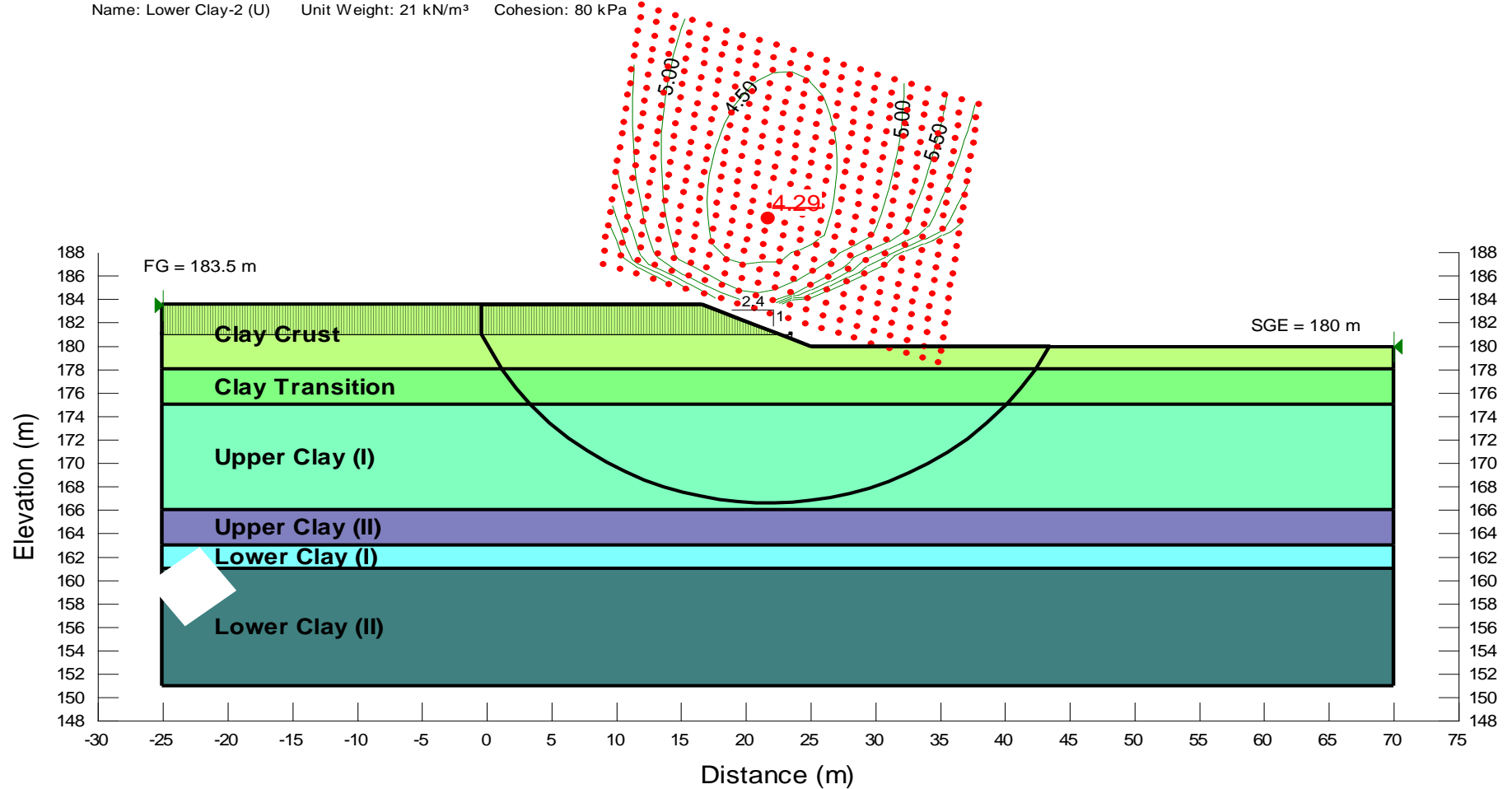
Name: Backfill (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Clay Crust (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Clay Transition (D) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Piezometric Line: 1
 Name: Upper Clay-1 (D) Unit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



File Name: CV4-ST & LT Trench Stability-SEEP - Feb27.gsz

Last Solved Date: 27/02/2012

Name: Clay Crust (U)	Unit Weight: 21 kN/m ³	Cohesion: 75 kPa		
Name: Clay Transition (U)	Unit Weight: 21 kN/m ³	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/m ³	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/m ³	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/m ³	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/m ³	Cohesion: 80 kPa		



Name: Clay Crust (D)	Unit Weight: 21 kN/m ³	Cohesion: 1 kPa	Phi: 30 °	Piezometric Line: 1
Name: Clay Transition (D)	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	Piezometric Line: 1
Name: Upper Clay-1 (D)	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	Piezometric Line: 1
Name: Lower Clay-1 (D)	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	Piezometric Line: 1
Name: Upper Clay-2 (D)	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	Piezometric Line: 1
Name: Lower Clay-2 (D)	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	Piezometric Line: 1



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