
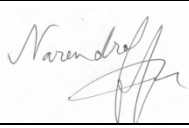



# The Windsor Essex Parkway Project Geotechnical Investigation and Design Report – Bridge B10 (West of Geraedts Drive over Highway 401)



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# 1 Introduction

## 1.1 Preface

The Windsor Essex Parkway (the Parkway, or the WEP) 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 km 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 km of recreational trails, extensive landscaping throughout the corridor, as well as noise and environmental mitigation measures. The environmental mitigation measures were based on 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 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 the 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. This team includes Dragados Canada, Inc., Acciona Infrastructure Canada Inc., and Fluor Canada Ltd.

## 1.2 Report Introduction

This report presents the geotechnical design of Bridge B-10 (Highway 3 Underpass, west of Geraedts Drive), located in Windsor sector of the proposed Windsor-Essex Parkway (WEP) project. This report includes the results of geotechnical investigations carried out to support the design and other relevant background information and addresses review comments from peer reviews and MTO. This is the final report and is issued for construction (IFC).

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

The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines, and the Parkway Infrastructure Constructors (PIC). The proposed four span concrete B-10 structure (HWY 3 underpass) will cross Highway 401 and will support both east and west bound lanes of Highway 3, between Stations 11+000 to 11+275 LaSalle.

The WEMG proposal design for Bridge B-10 incorporated true abutments and piers founded on deep end bearing piles as shown in Drawing 285380-03-060-MST1-S1001. The geotechnical design presented in this report was generally advanced from the preliminary geotechnical design developed for the WEMG proposal in June 2010 (ref. R-43)<sup>1</sup>. This preliminary design concept has generally been maintained through final design. The high walls originally planned at either end of the bridge will be replaced with retained soil structure (RSS) walls. Similarly RSS walls are now planned between both sets of east and west bound abutments. The approach embankment grades are 2 to about 5 m higher than existing grade levels and approximately 9-10 m higher than the Highway 401 (cut) grades. This design report is issued for construction and includes all relevant, available geotechnical investigation information.

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<sup>1</sup> References are listed in Section 10.

For the purposes of the geotechnical design, Bridge B-10 construction was considered to involve the following earthwork, design elements and loading stages:

- General excavations for Highway 401 will be substantially completed prior to construction of Bridge B-10;
- Temporary excavation to approximately 9 to 10 m depths below finished grade at east and west abutments, respectively;
- Installation of Reinforced Granular Mats (RGM) foundation under the RSS walls between the north and south approaches behind the east and west abutments;
- Excavations for of the bridge piers;
- Installation of piles (HP310×110) for all bridge supports
- Backfilling of piers;
- Construction of the structural abutments and bridge deck;
- Construction of the RSS walls and associated permanent sub-drainage works, and approved backfill behind the RSS structure;
- Completion of approach fills including Ultra Light Weight Fill (LWF), Expanded Polystyrene (EPS) and grading; and
- Completion of pavements over Highway 3 and Highway 401.

This 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 of Bridge B-10 complies with the requirements of execution version of the Project Agreement (PA) Schedule 15-2 Part 2, Article 5.

## 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-16, R-18, and R-25). 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 glacio-lacustrine clay. Hudec (ref. R-25) summarized the overburden geology in Windsor as consisting of the following successive strata: desiccated lacustrine clay, normally consolidated lacustrine clay, silty Tavistock till, glacio-lacustrine 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. The eastern part of Windsor is underlain by firm to stiff, glacio-lacustrine silts and clays with upper deposits of stiff sandy to silty weathered clay and a hard to stiff lacustrine clay-silt crust. The western part of Windsor is characterized by a thin surficial granular deposit underlain by a thin crust layer underlain by soft to firm glacio-lacustrine 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 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 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 consists of limestone, dolostone and shale comprising the Devonian Dundee Formation of the Hamilton Group Formation underlain by the Devonian Lucas Formation of the Detroit River Group Formation.

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 latter 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, the Dundee Formation, and the Detroit River Group Onondaga Formation.

## 2.2 Site Seismic Background

Windsor-Tecumseh area is described in the Canadian Highway Bridge Design Code (CHBDC, ref. R-9) 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.

In accordance with CHBDC the soil profile at the site of the project meet in general the description for Soil Profile Type III (soft clay and silts greater than 12 m in depth). A limited number of cross-hole tests were completed during the background investigation program (ref. R-21) at locations distributed strategically 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.

## 2.3 Existing Site Conditions and Proposed Tunnel Layout

Bridge B-10 is situated near the east end of the Windsor segment of the Parkway, close to the border between the Windsor and LaSalle Municipalities in Ontario. The structures at this site are to be constructed under Phase I of WEP. The proposed structure consists of two bridges each with four spans varying in length from 40 to 65 m. Each bridge will be approximately 13 m wide. The WEMG proposal design for Bridge B-10 comprised abutments and three piers founded on deep end bearing piles. The Cahill submerged culvert S-2 is located below Highway 401 approximately 30 m west of the south east abutment of Bridge B-10.

The topography of the lands immediately adjacent the Highway 3 is essentially flat with elevations ranging from between 182 and 183<sup>2</sup>. Adjacent land use is typically urban parkland, residential to the south and industrial to the north.

## 2.4 Frost Depths

In accordance with MTO –SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-37) and Ontario Provincial Standards Drawing (OPSD) 3080.101 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 snow accumulation.

The insulation effects of riprap, and/or other coarse rockfill covers 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 accordingly.

<sup>2</sup> Elevations are in metres and are referred to geodetic datum.



### 3 Geotechnical Investigations

#### 3.1 Scope and Procedures of Geotechnical Investigations

Geotechnical investigations involving a number of boreholes, cone penetration tests (CPT) and Nilcon vane tests had been carried out in 2007-09 by Golder Associates (ref. R-16 to R-23) as 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. The additional investigation was required to support the detailed design development of the WEP embankment and structures. The additional investigation program at and around the proposed location of Bridge B-10 comprised a total of 8 boreholes (B10-1 through B10-8), 4 cone penetration tests (CPT B10-1, CPT 10-2, CPT 10-3, and CPT 10-40-RW), 1 Nilcon vane profile (adjacent Borehole B10-3) and 1 flat plate dilatometer profile (DMT 5-RW). Table 3.1 lists the test holes advanced at or in close proximity to the bridge site during both the previous (2007-2009) and the current (2011) geotechnical investigations.

**Table 3-1: Test Holes at and around Bridge B-10 Site**

Reference	Boreholes	Nilcon Vane Tests	CPT's	DMT's
Additional Investigation (2011)	B10-1		CPT B10-1	DMT 5-RW
	B10-2		CPT B10-2	
	B10-3	NIL B10-3	CPT B10-3	
	B10-4		CPT 40RW	
	B10-5		CPT 41RW	
	B10-6		CPT 42RW	
	B10-7			
	B10-8			
Pre-Bid Investigation (2007-09)	BH/CPT 317		BH/CPT 317	
	BH/CPT320		BH/CPT 320	

Drawing 285380-04-090-WIP1-G1001 shows the locations of the test holes (pre-bid, and additional investigations) and an interpreted soil stratigraphic profile along the WEP centreline and for the general area at and around Bridge B-10, Highway 401 Sta. 10+900L to 11+500L. The test hole locations and stratigraphic sections at the bridge location are illustrated on Drawings 285380-04-090-WIP1-G1002 and 285380-04-091-WIP1-G1003.

#### 3.2 Additional Investigation Fieldwork

The boreholes were advanced using track-mounted CME 55 auger rigs, owned and operated by Marathon Drilling Co. Ltd., under contract to AMICO and under technical supervision by AMEC engineers and technicians. Boreholes were generally advanced using 200 mm OD hollow stem augers, followed by wash boring with NW casing. The depth at which the drilling methods transition occurred is noted on the borehole logs.

Soil sampling was carried out by means of a 50 mm diameter split spoon sampler and 70 mm diameter thin-walled Shelby tubes. Soil sampling was carried out generally at 0.75 m depth interval in the top 7 to 8 m and at 1.5 m depth intervals thereafter. All samples were field identified, placed in airtight containers



and transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing. Rock coring of the bedrock was carried out using NQ core barrels with a length of 1.5 m.

Standard Penetration Tests (SPT, ASTM D1586) were carried out in conjunction with split spoon sampling using an automatic trip hammer. Field vane tests (using conventional vanes) were carried out in between sampling at selected depths. The Nilcon vane test listed in Table 3.1 was carried out adjacent to the Borehole BH10-3 location. Table 3-2 summarizes the depths of overburden penetration and elevation ranges where rock coring and Nilcon vane tests were carried out.

**Table 3-2: Overburden Thickness and Rock Coring/Nilcon Vane Depths in 2011 Boreholes**

Borehole	Location	Overburden Thickness, m	Test Name & Elevation					
			Rock Coring	Nilcon Vane	S-Piez.	VWP	MHSG	IN
B10-1 (2011)	4679159.2N, 332754.5E	33.2	149.4 – 140.3			173.6, 162.0	174.1, 168.0	To 148.0
B10-2 (2011)	4679069.6N, 332866.8E	33.2	149.1 – 147.2			N/A		
B10-3/Nil10-3 (2011)	4679083.2N, 332836.8E	31.1	149.8 - 148	Yes		172.1, 162.1, 149.9		
B10-4 (2011)	4679068.8N, 332867.5E	32.7	149.7 – 147.0					
B10-5 (2011)	4679023.7N, 332901.0E	32.6	149.7 – 147.9		149.4			
B10-5-P32 (2011)	4679021.0N, 332901.4E	32.3	150.0 – 139.5			150.3		
B10-6 (2011)	4678990.9N, 332930.4E	32.3	149.7 – 147.3					
B10-7 (2011)	4678977.5N, 332967.8E	32.6	149.6 – 146.5			173.8, 162.4	173.8 – 162.4	Yes
B10-8 (2011)	4679118.4N, 332824.1	33.1	149.3 – 147.0					

Legend: S-Piez. Standpipe Piezometer  
VWP Vibrating Wire Piezometer  
MHSG Spider Magnet Heave/Settlement Gauge  
IN Inclinator Casing

Rock cores were examined in the field and transported to AMEC's Tecumseh (Windsor) laboratories for further examination. For each core run, rock core recovery and rock quality designation (RQD) were determined. The core recovery and RQD values are given on the borehole logs.

The boreholes were decommissioned using a bentonite-cement grout following completion of sampling, testing and instrument installation.

The Nilcon vane tests, DMT and CPT were carried out in cohesive soil strata after augering through the stiff/dense surficial materials. The Nilcon tests were carried out at 0.5 to 1.0 m depth intervals at an appropriate rate of rotational strain (ASTM D2573).

The CPT cone was pushed at a constant rate into the ground using hydraulic ram system of the drill rig (ASTM D5778). Pore pressure dissipation tests were carried out at selected depths.

The DMT probe was pushed in the ground in increments of 200 mm using the hydraulic ram of the drill rig. The tests were conducted following the provisions of ASTM D 6635. All CPT and DMT were advanced to refusal.

The locations of boreholes, Nilcon tests and CPTs executed during the pre-bid and the most recent 2011 investigation, and the inferred soil profile along the WEP alignment, are shown on Drawings 285380-04-090-WIP1-G1002 and 285380-04-091-WIP1-G1003. Borehole and CPT logs from the additional 2011 investigation are included in Appendix A. Relevant borehole logs from earlier investigations are included in Appendix B.

### 3.3 Geotechnical and Analytical Laboratory Testing

All recovered soil samples and rock cores were examined in the field and the AMEC geotechnical laboratory. Natural moisture content tests were carried out on most of the recovered samples. Grain size distribution and Atterberg limit tests were carried out on selected representative samples.

Selected samples obtained from Boreholes B10-1, B10-2, and B10-7 at depths of 25.9 m, 9.1 m and 30.5 m, respectively were sent to the ALS Environmental Analytical Laboratory in London, Ontario to determine the pH, redox potential, resistivity, sulphide and sulphate content of the soil to assess corrosion potential.

The results of laboratory and analytical tests are indicated on borehole logs and are presented in Appendices C and D, respectively.

### 3.4 Instrumentation

Geotechnical instruments were installed at designated locations on completion of boreholes to monitor pore water pressure and deformation behaviour of the soil strata during and after construction. An instrumentation location plan is provided as Figure I1 in Appendix I (Instrumentation Location Plan). A brief description of these instruments is as follows:

**Standpipe Piezometers:** These piezometers comprise 1.5 m long 10 mil slotted intake screen located at the designated depth and extended to the ground surface using 52 mm diameter, flush-joint, threaded, schedule 40 PVC riser pipe. A silica sand filter pack was placed between the intake screen and the wall of the borehole and extended approximately 0.3 m above the top of the well screen. Bentonite-cement grout was used to restore grade to the ground surface. Screen elevations and details of installations are provided in Table 3-2 and applicable borehole logs.

**Vibrating Wire Piezometers (VWP):** The VWP transducers (RST Model VW2100, 0.35 MPa for shallow to mid-depth and 0.7 MPa for deep installations) were installed at designated depths and electrical wires extended to the monitoring station at the ground surface (outside the parkway footprint area). The borehole was filled with a bentonite-cement mixture designed to match, as near as practical, the permeability and strength-deformation characteristics of the native soils in accordance with standard practice. Sensor elevations and details of installations are provided in Table 3-2 and applicable borehole logs.

**Magnetic Settlement/Heave Gauges (MHSG):** Spider magnets (RST, Model SSMM100 mechanical release spider target for 25 mm pipe) were installed in boreholes at select locations and depths to permit future measurement of heave and settlement. Each magnetic torus was placed around a 25 mm diameter pipe, which was extended to above the ground surface. The spider legs grip into the surrounding soil, which enables the magnetic torus to move up or down on the pipe as the soil settles or heaves. The locations of the magnetic torus are determined by lowering a magnetic probe inside the pipe.

**Inclinometers (IN):** An inclinometer casing was installed in Boreholes B10-1 and B10-7. The purpose of this device is to measure the lateral ground movement at the installed location. The bottom ends of the casings were anchored approximately 0.6 m into bedrock, and the annular space around the casing was filled with bentonite-cement grout. The inclinometers comprised 70 mm diameter RST “Snap Seal Inclinometer Casing”, and the probe is IC32005 MEMS digital inclinometer system (0.5 m long).

### 3.5 Data Interpretation – General Discussion

**Field Vane Test Data Correction:** The chart (Figure 3.1<sup>3</sup>) developed initially by Bjerrum (1972) and updated subsequently by Ladd et al (1977) based on circular arc failure analyses of embankment failures suggest correcting the field vane data by multiplying by 1.05 to 1.10 for soils with plasticity index of approximately 15 (ref. R-6 and R-32), the typical value for the silty clay to clayey silt deposit at the WEP. However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), the Canadian Foundation Engineering Manual suggests that the vane test data for clays with  $PI < 20$  should not be corrected (R-1 and R-8, Figure 3-2). The field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI. Interestingly, 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:

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

<sup>3</sup> All figures are included at the end of the report text.

Where:

$S_{u\text{CPT}}$  is the undrained shear strength estimated from the CPT test;

$Q_t$  is the corrected total cone tip resistance;

$\sigma_{vo}$  is the total vertical stress at the corresponding depth of measurement of the  $Q_t$  value;  
and

$N_{kt}$  is an empirical factor that varies, depending on soil type and test arrangement, typically between 8 and 20.

The CPT based  $S_u$  profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles. In this regard, the  $N_{kt}$  factor values used to calibrate the CPT strength profiles varied slightly for different segments of the WEP and the soil strata. For the B-10 bridge site, an  $N_{kt}$  factor of 16 was used for all layers.

*Pre-Consolidation Pressures from Cone Penetration Tests:* The approach used for estimating the pre-consolidation pressures from the estimated  $S_u$  profiles follows the Stress History and Normalized Soil Engineering Properties (SHANSEP) method developed at MIT (ref. R-31). 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 actual undrained shear strength,

$\sigma'_{vo}$  is the vertical effective stress,

$\sigma'_p$  is the pre-consolidation pressure (also referred as maximum past pressure),

$S$  is the normalized strength ratio,  $S_u / \sigma'_{vo}$ , 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,  $\sigma'_p$  was then 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 were conducted following the ASTM D6635-01 (2007) method. The soil properties from the results of these tests were developed in general using the guidelines layout in ISSMGE, 2001 (ref. R-26), except that the undrained shear strength values for the clay deposits were estimated using the relationship  $S_u = S \sigma'_{vo} (0.5 K_d)^{1.25}$ , where  $S = 0.18$ .  $K_d$  is the horizontal stress index obtained from DMT reading and is defined by:

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

Where:

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

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

The undrained shear strength ( $S_u$ ), pre-consolidation pressure ( $\sigma'_p$ ), natural water content ( $w_N$ ) and compression index ( $C_c$ ) profiles based on field and laboratory testing from boreholes, CPT and DMT carried out in the general area of Bridge B-10 (WEP segment between Sta. 10+900L and Sta. 11+500L are presented in Figure 3-3. Also included on the figures are the  $0.18 \times \sigma'_{vo}$  curve (representing undrained strength profile for OCR=1 condition) and the simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units. The constant 0.18 for  $S_u/\sigma'_{vo}$  for OCR=1 curve is based on average plasticity index of the silty clay to clayey silt stratum and Chandler 1988 relationship (Figure 3-1) (ref. R-11).

## 4 Subsurface Conditions

The ground surface elevation at the boreholes was between 182 and 183 m. The general soil stratigraphy at the borehole locations consists of the following successive strata: topsoil and a thin discontinuous upper sand deposit (up to 0.9 m thick), an extensive clayey silt to silty clay deposit to approximately elevation 151, and discontinuous 0 to 2 m thick lower granular deposit overlying limestone and dolostone bedrock below elevation 149.3 to 150.0. The bedrock was encountered at depths ranging from approximately 31.1 and 33.2 m below the existing ground surface.

### 4.1 Topsoil and Upper Granular Deposit

A layer of topsoil was encountered at the ground surface in all B-10 Boreholes varying in thickness from 0.2 to 0.8 m at the borehole locations.

A 0.9 m thick layer of upper granular deposit was encountered beneath the topsoil in Boreholes BH B10-6 and BH B10-8. The upper granular deposit consisted of fine sand with silt to silty fine sand.

### 4.2 Silty Clay to Clayey Silt Stratum

A silty clay to clayey silt stratum was encountered directly underlying the surficial topsoil and/or upper granular deposit. This stratum at borehole locations was between 30.1 and 33 m thick and extended to an approximate elevation of between 151.7 and 149.4. Based on 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 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 properties of the clay layers are summarized in Table 4-1.

**Table 4-1: Summary of Ranges of Index Properties**

Property	Clay Crust	Transition Zone	Silty Clay	Clayey Silt
Elevation Range	183 to 177	177 to 175	175 to 161	161 to 151
Natural Water Content, $w_N$ , %	15 to 23	19 to 21	16 to 30	14 to 34
Liquid Limit, $w_L$ , %	31 to 44	30 to 34	23 to 39	23 to 38
Plastic Limit, $w_P$ , %	14 to 22	15 to 17	13 to 20	13 to 17
Plasticity Index, PI	17 to 22	15 to 17	10 to 19	9 to 18
Liquidity Index, LI,	0.1	0.2 to 0.3	0.1 to 0.9	0 to 1.4
Unit Weight, $\gamma$ , kN/m <sup>3</sup>	20.1 to 22.7	20.1 to 21.4	19.2 to 22.7	18.3 to 21.7

The measured undrained shear strength,  $S_u$ , versus depth profiles are shown in Figure 3.3.

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: 100 kPa to 65±15 kPa
- Upper silty clay: 65±15 kPa to 50±10 kPa
- Lower clayey silt: 60 ±10 kPa to >75 kPa

The stress-strain properties and the effective shear strength properties of the silty clay to clayey silt soils were based on published correlations (Kulhawy and Mayne, 1990, ref. R-29, Leroueil et al, ref. R-33 and Terzaghi et al. Ref. R-42) and confirmed by tests reported in Golder's Subsurface Condition Interpretation Report (ref. R-19) 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 indices were correlated to natural water content ( $w_N$ , expressed as percent) and are illustrated in Figures 4.1 and 4.2. The relationships are summarized as follows:

$$C_c = 0.0086w_N - 0.0086$$

$$C_r = 0.11C_c$$

$$C_s = 0.25C_c$$

$$C_\alpha = 0.028C_c$$

The interpreted average values used for the clay crust, transition zone, silty clay and clayey silt substrata for the Bridge B-10 site are summarized as follows:

**Table 4-2: Summary of Compressibility Properties**

Property	Clay Crust	Transition Zone	Silty Clay	Clayey Silt
Natural Water Content, $w_N$ , %	20	20	20-25	22-39
Virgin Compression Index, $C_c$	0.16	0.16	0.16-0.21	0.18-0.32
Recompression Index, $C_r$	0.018	0.018	0.018-0.023	0.02-0.036
Swelling Index, $C_s$	0.04	0.04	0.04-0.05	0.045-0.08

The modulus of elasticity has been correlated with the undrained shear strength of the material, published information (ref.R-42) and local experience (ref.R-19). For the unweathered portion of the silty clay stratum the empirical relationship were used based on average shear strength profiles for the material, as follows:

$$E_u = 300 S_u$$

$$E' = 0.9E_u$$



**Table 4-3: Summary of Interpreted Soil Deformation Properties**

Soils Stratigraphy	Elastic Modulus-Undrained, MPa	Poisson's Ratio-Undrained (*)	Elastic Modulus-Drained, MPa	Poisson's Ratio-Drained (*)
Clay Crust	22.5	0.49	20.3	0.35
Transition Zone	16.5-22.5	0.49	14.9-20.3	0.35
Silty Clay	12-19.5	0.49	10.8 – 15.6	0.35
Clayey Silt	19.5-21	0.49	15.6 – 18.9	0.35

(\*) Assumed values

The effective shear strength properties applicable to the silty clay to clayey silt stratum were determined from triaxial compression and drained direct tests performed during the pre-bid and additional geotechnical investigations (Figure 4-3) and supported also by published PI versus  $\phi'$  relationships (ref. R-28, R-34 and R-42, Figure 4-4), and are summarized as follows:

**Table 4-4: Effective shear strength Properties**

Apparent cohesion, $c'$	0 kPa
Angle of internal friction, $\phi'$	30°
Residual angle of internal friction, $\phi_r'$	27°
Friction angle at critical state, $\Phi_c^*$	25° - 26°

(\*) Based on triaxial tests (ref. R-17)

Effective cohesion (which may be potentially present) in the upper zones of over-consolidated clayey silt has been neglected for design in consideration of long-term weathering, swelling resulting from unloading, and fissuring effects.

The hydraulic conductivity of the silty clay to clayey silt stratum was interpreted from pore pressure dissipation tests carried out in the CPT probes as well as the laboratory oedometer tests. The hydraulic conductivity values obtained from previous (2007-09) and additional (2011) investigations are plotted on Figure 4.5.

### 4.3 Lower Granular Deposit

Beneath, or near the base of the silty clay to clayey silt, a discontinuous lower granular deposit was encountered in Boreholes B10-2, B10-3, B10-4, B10-5, B10-7, and B10-8. The lower granular deposit varied from sandy silt to sand and gravel. This layer was encountered at approximate elevations between 149.9 and 154.1. The thickness of the lower granular deposit varied from 0.6 to 1.3 m at the borehole locations. The lower granular deposit had 'N' values of 18 to 75 blows per 0.3 m.

### 4.4 Bedrock

Where rock coring was undertaken, a white to grey, limestone bedrock was encountered. The bedrock was generally fresh, medium strong, laminated to thinly laminated, fine grained, faintly to highly porous and highly fractured. Bedrock was encountered at elevations ranging from 149.1 to 150.0 in the vicinity

of B-10. The Rock Quality Designation (RQD) of the recovered rock varied between 23 and 100 per cent, indicating a poor to excellent quality.

Based on this core logging the rock mass classification was estimated to range from 2.8 to 5 for the Q-System (ref. R-3) and 53 to 58 for the Rock Mass Rating (RMR) (ref. R-5) and indicates that the rock mass can be considered as a Fair quality rock mass based on the later system.

It was found during the preliminary pre-bid investigation report (ref. R-16) that little variation in the strength of the rock mass conditions was identified from site to site. For this reason in order to obtain a reasonable statistical sample, the density, unit weight, and uniaxial compressive strength of the samples from all of the key sites have been grouped and are summarised in Table 4-5. The average strength of the limestone is determined to be 85.5 MPa and is 'strong rock' based on the ISRM (ref. R-27). Additionally, based on the coefficient of variation, enough tests have been performed to characterise the compressive strength.

**Table 4-5: Summary of Intact Properties**

	<b>Density (kg/m<sup>3</sup>)</b>	<b>Unit Weight (kN/m<sup>3</sup>)</b>	<b>UCS (MPa)</b>
Number of Samples, N	12	12	16
Average	2502	24.54	85.5
Standard Deviation	96	0.94	25.4
Minimum Value	2340	22.95	35.5
Maximum Value	2660	26.09	135.3

Based on the rock mass classification and the strength properties assuming an  $m_i = 12$  for a crystalline limestone, a disturbance factor of 0.7, and a factor of safety of 3.0, an allowable bearing capacity of the rock has been calculated to range from 5.3 MPa to 13.5 MPa. The mean allowable bearing capacity is determined to be 9.2 MPa using the Hoek and Brown strength criterion for determining the bearing capacity of a fractured rock mass (ref. R-44).

Photographs of rock cores recovered from Boreholes B10-5 and B10-6 are provided in Appendix H.

## 4.5 Groundwater Conditions

Shallow and deep standpipe and vibrating wire piezometers were installed in selected boreholes to measure the water levels within overburden and bedrock, respectively Table 3-2

The piezometric water levels within the overburden and the bedrock were measured to be at approximate elevations 178 to 181 and 177, respectively. These measurements suggest a slight downward gradient between the overburden and the bedrock. Nevertheless, given the general prevalence in the Windsor area, local occurrence of artesian condition in bedrock cannot be ruled out.

Perched groundwater is known to accumulate seasonally within the upper deposits of topsoil and granular layers, and within the fissures in the clay crust. In adverse conditions, the perched groundwater levels can rise to near the ground surface. Groundwater was measured in a shallow test pit near Borehole B10-3 at 0.45 m below grade.

**Table 4-6: Summary of Measured Water Levels**

Borehole	Surface Elevation	Piezometer Type	Screen / Sensor Elevation	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	Elevation
BH T10-1	182.6	VWP	173.6	Clayey Silt	June 4, 2011	181.4
					July 23, 2011	181.0
		VWP	162.0	Clayey Silt	June 4, 2011	180.6
					July 23, 2011	180.4
BH T10-3	182.2	VWP	172.1	Clayey Silt	June 25, 2011	179.8
					July 23, 2011	179.1
		VWP	162.1	Clayey Silt	June 25, 2011	179.9
					July 23, 2011	179.4
		VWP	149.9	Silty Sand	June 25, 2011	177.1
					July 23, 2011	176.6
BH-10-5-32	182.3	VWP	150.3	Limestone	June 4, 2011	178.3
					July 23, 2011	177.0
BH-10-7	182.2	VWP	173.8	Clayey Silt	June 25, 2001	181.0
					July 23, 2011	180.7
		VWP	162.4	Clayey Silt	June 25, 2001	178.3
					July 23, 2011	178.1

Legend: VWP Vibrating Wire Piezometer

#### 4.6 Subsurface Gases

The groundwater in the project area, especially within the lower granular deposit and bedrock, is known to contain dissolved hydrogen sulphide ( $H_2S$ ) and methane ( $CH_4$ ) gases that are liberated from the water on exposure to atmospheric pressure. The  $H_2S$  gas can frequently be detected by odour at concentrations on the order of 0.5 ppm and can be corrosive at concentrations of about 2 ppm to 3 ppm in the groundwater. The gas odour was not detected during the drilling at the bridge site.

However, although the  $H_2S$  and  $CH_4$  gases were not detected during the 2011 geotechnical investigation at Bridge B-10 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 which suggest very low concentration in the area.

**Table 4-7: Pumping Tests Data**

Test #	Approximate Location	H <sub>2</sub> 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

The understanding of the engineering behaviour (related to the impact on design and construction) of the gassy soils is rather limited. In the case of low permeability cohesive soils it is known that these soils may experience rapid drop in undrained shear strength during unloading. Due to the relatively high compressibility of the pore water fluid in gassy soils, the immediate pore water pressure response ( $\Delta U$ ) to total stress changes can be very low. This phenomena leads to reduction in effective stress and hence shear strength (ref. R-24 and R-41). It is, therefore, recommended that the design and construction methodologies should be developed in consideration of the potential presence of these gases (ref. R-14). Air quality and subgrade pore pressure monitoring should be carried out during construction. The equipment operating in confined spaces should be selected to safely operate in a potentially gaseous environment. Excavation layers should be decided in consideration of the pore pressure monitoring data and the potential ground softening.

## 5 Development of Geotechnical Design

The proposed four span concrete Bridge B-10 structure (HWY 3 underpass) will cross Highway 401 and will support both east and west bound lanes of Highway 3. Bridge B-10 will be constructed over the below-grade section of WEP between 11+000 to 11+275 LaSalle. Bridge B-10 is 229.5 m long with individual spans ranging from 40 to 65 m.

Bridge B-10 incorporates two bridges side-by-side each comprising a four span deck-on-girder structure incorporating true abutments and three centre piers founded on deep end-bearing HP 310×110 steel piles. Batter piles are used to carry horizontal loads.

Approach embankments adjacent to Highway 401 are supported by high RSS wall. Also, the approach fills between the east and west bound lanes of B-10 are designed using RSS walls.

The geotechnical analyses discussed in the subsequent sections has been used to determine suitable configurations for the B-10 bridge site. The proposed geotechnical configurations have been illustrated on Figures G1. These drawings illustrate the configurations at the west side of the bridge which was chosen for the design case as the design heights are slightly greater. The configurations at the east side are similar (reversed).

### 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' designs were as per the principles of 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 wing-walls. The stability of the soil mass containing the bridge end abutments and wing walls was checked for all potential surfaces of sliding and has a minimum factor of safety of 1.3 according to the Project Agreement.

WS method was also used for the external stability analysis of the RSS structures.

### 5.2 Design Soil Properties

The design soil properties for the silty clay to clayey silt stratum were interpreted from the CPT and Nilcon vane test profiles, DMTs and the laboratory test results. The undrained shear strength,  $S_u$  profiles were estimated from the CPT's based on the calibration described in Section 3.2. The  $S_u$  profiles inferred from the CPTs advanced around Bridge B-10 are shown in Figure 3.3. Selected typical design values obtained from the profiles are summarized in Table 5-1.

**Table 5-1: Summary of Interpreted Design Clay Strength**

Clay Substratum	Undrained Shear Strength ( $S_u$ ), kPa	Effective Stress Parameters	Preconsolidation Pressure ( $\sigma_p'$ ), kPa	Over-Consolidation Ratio
Clay Crust	75	Cohesion, $c' = 0$ Friction Angle, $\phi = 30^\circ$	450 to 600	>5.6
Transition Zone	55 to 75		300 to 450	5.6 to 2.7
Silty Clay	43 to 65		220 to 380	2.7 to 1.1
Clayey Silt	65 to 70		380	1.5 to 1.1

Note: The ranges of  $S_u$  and  $\sigma_p'$  values indicate variation top to bottom with depth.

Legend:  $\phi'_{\max}$  = peak friction angle

The design values of the coefficient of horizontal permeability ( $k_h$ ) and the hydraulic conductivity anisotropy ratio ( $A=k_h/k_v$ ) used for the analysis of stress and deformation response of the soils are provided in Table 5-3. These values are typically 2 to 5 times higher than the values interpreted from the field test results (Figure 4.5) and are considered to be within range of precision of the measurements.

**Table 5-2: Design Hydraulic Conductivity Parameters for Silty Clay Stratum**

Clay Substratum	Horizontal Permeability, cm/sec	Anisotropy ratio, $k_h/k_v$	Initial Void Ratio, $e_0$
Clay Crust	$3.4 \times 10^{-7}$	1	0.54
Transition Zone	$3.4 \times 10^{-7}$	2	0.54
Silty Clay	$1.1 \times 10^{-7}$	1	0.54-0.68
Clayey Silt	$1.1 \times 10^{-7}$	1	0.59

### 5.3 Stability of Excavations 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. The shapes and slopes of the anticipated temporary excavations shown do not constitute the actual design of the temporary slopes. The Contractors are fully responsible for the design, construction methods, performance (stability, deformability and deterioration) and maintenance 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.

For the present design purposes, the bulk of the general excavation is assumed to be conducted prior to construction of the walls and abutments.

Basal hydrostatic uplift was calculated based on the highest measured water level in the bedrock (179.0), anticipated deepest excavation depth (Hwy 401 subgrade at elevation 176), and a silty clay to clayey silt stratum thickness of 22 m (Borehole BH B10-5) below the deepest general excavation. The calculated factor of safety against hydrostatic uplift considering the weight only of the clay cap was 1.8 (1.4 at pier excavations).



As described in Section 4.6 presence of gassy soils near bedrock surface could potentially be encountered during construction, which could impact the pore pressure and undrained shear strength condition of the lower part of the silty clay deposit. Given the significant soil stress relief due to depth of excavations it is recommended that in the case of excavations deeper than 5 m, careful monitoring of basal heave and pore water pressures below of the bottom of the excavations be carried out during construction. Adequate number of heave gauges and low-displacement type piezometers (e.g., vibrating wire piezometers) should be installed prior to initiation of the major excavations. If warranted by the monitoring of the excavation progress performance, the excavation rates will have to be adjusted to allow sufficient time to dissipate the pore pressures to safe levels. The excavation guidelines can be revised based on on-site experience.

## **5.4 Stability of Abutments and RSS Walls**

Bridge B-10 is comprised of two separate bridges, one for eastbound traffic (south bridge), and one for the westbound traffic (north bridge). The abutments have been identified on Drawing 285380-03-060-MST1-S1001, as Northeast and Northwest for the westbound bridge, and Southwest and Southeast for the eastbound bridge. Four configurations were modelled as presented in Appendix E. Two sections were analyzed in a transverse direction (North-South), through the Northwest and Southwest abutments. A third section was taken through the northwest abutment in a longitudinal direction (East-West). The fourth configuration was taken as a transverse section through the proposed West RSS wall. These sections are illustrated on Figure E1. The west side of the bridge was used for the design sections as the grades are slightly higher. The designs are applicable to the east side (mirrored configuration).

Limit equilibrium stability analyses for the abutments and approaches at the end of construction were based on undrained clay strength; the long term (effective stress) conditions were modeled using the drained soil properties as summarized below: For analytical purposes the upper silty clay was subdivided into 3 substrata, the lower silty clay was divided into 2 substrata.

The groundwater table has been assumed near 181m outside of the excavations, and stabilizing at the Highway 401 subgrade level beyond the abutments. The final design should incorporate permanent ground water control to ensure the slopes and abutments are suitably drained.



**Table 5-3: Summary of Soil Design Values For Global Stability Purposes**

Soil Layer	Unit Weight kN/m <sup>3</sup>	Undrained Analyses Cohesion, kPa	Drained Analyses	
			$\phi$ °	Effective Cohesion (kPa)
Clay Crust	22	75	30	0
Transition Zone	22	75 to 55	30	0
Silty Clay 1	19.5	55 to 43	30	0
Silty Clay 2	19.5	43 to 49	30	0
Silty Clay 3	19.5	49 to 65	30	0
Clayey Silt 1	19.5	65	30	0
Clayey Silt 2	19.5	65 to 70	30	0
Lower Till /Granular Layer	19	-	30	0
Light Weight Fill	12	-	35	0
RSS Regular Reinforced fill	21	-	35	50
RSS Reinforced LWF	12	-	35	50
RGM	22	-	35	50
Regular Granular Fill	21	-	35	0
LWF	21	-	35	0
Expanded Polystyrene	0.5	15	-	15
Clay Fill	21	50	30	0

The analysis using undrained soil properties was carried out considering that the pavement structure is not applied over the subgrade of Highway 401 footprint. The drained analyses assumed that all the components of the pavement structure are present. The results of analyses, summarized in Table 5-4, indicate calculated factors of safety of 1.3 or greater which satisfies the design criteria. The stability analyses reports are provided in Appendix E.

**Table 5-4: Summary of B-10 Global Abutment Stability**

Location	Loading Condition	Figure No.	Strength Parameters	Factor of Safety <sup>(1)</sup>
West RSS Wall, Section A	Short Term <sup>(2)(3)</sup>	E2	Undrained	135 (1.28)
	End of Construction	E3	Undrained	1.39 (1.28)
	Long Term	E4	Drained	1.51 (1.37)
West RSS Wall, Section B	Short Term <sup>(2)</sup>	E5	Undrained	1.39 (1.26)
	End of Construction	E6	Undrained	1.44 (1.33)
	Long Term	E7	Drained	1.57 (1.47)
East RSS Wall, Section A	Short Term	E8	Undrained	1.45 (1.31)
	End of Construction	E9	Undrained	1.64 (1.49)
	Long Term	E10	Drained	1.67 (1.57)
Northwest Abutment, Longitudinal Section	End of Construction	E11	Undrained	1.65 <sup>(4)</sup>
	Long Term	E12	Drained	1.53 <sup>(4)</sup>

Notes: 1) Values in brackets represent “optimized” non-circular failure surfaces.

2) During construction - assumes no WBR traffic surcharge loading until 401 granulars installed.

3) Roadway granulars required in short-term at Highway 401 level.

4) Unrealistic “optimized” failure surface

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The various configurations have been optimised to achieve the minimum required FS ( $FS > 1.3$ ) against global instability of the abutments. This has been accomplished through the use of light weight fills and reinforced granular mat. The configurations required are as shown on the stability sections Appendix E and on the configuration sketches, Appendix G.

#### 5.4.1 RSS Wall External Stability

The external stability factors of safety against base sliding, overturning about the toe and bearing capacity failures were checked by means of the Working Stress method in accordance with the CFEM guidelines in conjunction with the undrained and drained soils shear strength properties described in Section 5.3.

##### *Bearing Capacity:*

Table 5-5 presents the net ultimate bearing capacity values ( $q_u$ ) that were determined for the native subgrade soils at the abutments for short-term (undrained) and long-term (drained) loading conditions based on the wall configurations provided in Table 5-5.

**Table 5-5: Summary of Bearing Capacity Below RGM**

RSS Wall	Assumed Wall Height (m)	Assumed Sub-grade Elevation (m)	Short-Term (Undrained), kPa (*)	Long-Term (Drained), kPa (*)
West Side – Section A	6	180	260	270
West Side – Section B	7	179	260	370
East Side – Section A	8.5	177.3	260	495
East Side – Section B	7.2	178.6	260	495

(\*) Net bearing capacity (no embedment)

##### *Base Sliding:*

The ultimate geotechnical horizontal resistance ( $H_{ri}$ ) can be determined in accordance to the following expression:

$$H_{ri} = A'c' + V \tan \delta > 1.5 H_f$$

Where:

- $A'$  ( $m^2$ ) = effective contact area of the base;
- $c'$  (kPa) = cohesion/adhesion at sliding interface;
- $\delta$  ( $^\circ$ ) = friction angle at sliding interface;
- $V$  (kN) = vertical force (kN); and
- $H_f$  (kN) = design horizontal load.

Allowance for buoyancy should be made, where applicable.

The following soil properties (Table 5-6) can be used in the design at the interfaces between the RSS, RGM and silty clay subgrade:

**Table 5-6: Soil Properties for use at Sliding**

Interface	Undrained (Short-Term)		Drained (Long-Term)	
	$\delta$ , degrees	c, kPa	$\delta'$ , degrees	c', kPa
RSS to RGM	30	0	30	0
RGM to Silty Clay	0	75		

### 5.4.2 Abutment Configurations

Based on geotechnical analyses discussed in Sections 5.1 to 5.4, abutment configurations and dimensions were developed, which are summarized in Table 5-7. The abutment configurations and dimensions indicated in these analyses are preliminary (e.g., the indicated width of the RSS is the minimum width) and are to be finalized by proprietary suppliers. The final design of the abutments is to be developed in consultation with the proprietary component suppliers.

**Table 5-7: Summary of RSS Wall Dimensions**

RSS Wall	RGM Size, Thickness x Width (m)	RSS Structure Size, Width x Height (m) <sup>(1)</sup>	LWF (m <sup>2</sup> )
West Side – Section A	1.5 x 8	5.0 x 6.0	-
West Side – Section B	1.5 x 9	6.0 x 7.0	9 to 15
East Side – Section A	1.5 x 11	8 x 8.5	44 to 60
East Side – Section B	1.5 x 10	7.0 x 7.2	21

(1) The RSS supplier may require wider structures to meet the internal design requirement.

### 5.4.3 RGM Foundation Loads

A 1.5 m thick RGM foundation was considered under the RSS false abutment walls to improve the bearing soils and satisfy the WS bearing capacity requirements for undrained conditions at the North and South abutments. For preliminary estimates, a simplified approach was used considering that the RGM foundation distributes the vertical pressures at the base of the RSS walls to the subgrade below the RGM within a 45° angle. The following loads in (Table 5-8) were estimated to act on top of the RGM on the basis of conventional calculation of the bearing pressures under gravity retaining walls:

**Table 5-8: Estimated load on RGM at the underside of RSS**

Abutment Location	Maximum Edge Bearing Pressure, kPa	Average Unfactored Bearing Pressure, kPa
West Side – Section A	225	180
West Side – Section B	235	180
East Side – Section A	245	190
East Side – Section B	220	175

Based on the above load on RGM, an estimated unfactored horizontal tensile load of 65 kN per meter of RGM for both walls was estimated across the entire height of 1.5 m. For cost estimates, this tensile load can be accommodated by 4 layers of UX1000HS, or equivalent.

## 5.5 Stress Deformation Analysis Models

The finite element stress-deformation analyses (SDA) were carried out using the SIGMA/W software. The purpose of the SDA was to assess the following:

- Settlement and tilting of the RSS wall and associated approach fills;
- Settlements of approaches and fills behind bridge abutments;
- Potential long-term heave along the permanent cut (as this relates to potential uplift at the piers and heave of Hwy 401);
- Soil settlements below the proposed abutment and pier piles (as this relates to potential downdrag); and
- Lateral soil movements along the abutment and pier piles (as this relates to additional bending moments within the piles).

Three configurations were modelled as presented in Appendix F on Figures F2 to F4. The first configuration was taken at a section through the RSS wall located between the staggered abutments at the west side of the bridge (Southwest abutment and Northwest abutment). The second and third configurations were taken as transverse and longitudinal sections through the Northwest abutment. These sections were taken as the design sections for the site. The locations of the configurations are illustrated on Figure F1. The east end data and configuration requirements generally mirror the locations described. In all cases the 0 m offset is located at the exposed face of the wall or abutment. Positive offsets are towards Highway 401 and negative offsets are in the direction of the back fill and / or approaches. The longitudinal deformations behind the south west and north east approaches will be governed by the highwall/RSS walls at those locations. These are addressed under a separate cover.

The SDA was carried out using drained (effective stress) soil properties.

The model is based on the following loading steps:

- Day 0 – Definition of the initial (in-situ) stress condition for level ground conditions;
- Day 1 – Completion of Bulk excavation to the subgrade level under the highway pavement, RSS/RGM, and/or pile caps;
- Day 1 to 90 – Waiting period to simulate excavation duration and associated porewater dissipation;
- Day 90 to 97 – Pile Installations;
- Day 97 – Construction of RSS wall and/or backfill behind abutments up to the underside of pavement structure;
- Day 97 to 110 – Waiting period to simulate construction duration from the new loads placed on day 97 and associated porewater pressure dissipation;
- Day 110 – Completion of pavement structure for approaches and Highway 401; and
- Day 110 to 30110 – Long-term Dissipation of excess pore pressures.

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The stratigraphy and selection of the soil properties was based on the design soil properties discussed in Section 4.

In the case of effective stress, coupled hydro-mechanical Modified Cam-Clay constitutive models were considered for the unweathered silty clay and clayey silt below the transition zone, and the elastic-plastic Mohr-Coulomb model for the remaining soil layers (Crust, Transition, and Backfill). The Modified Cam-Clay model required the input of the critical state friction angle, pre-consolidation pressure, initial void ratio, primary compression and recompression indices. The latter were selected as the recompression index given in Table 4-2. The drained elastic-plastic Mohr-Coulomb model required inputs of the peak friction angle, the drained initial Young's modulus, and Poisson's ratio.

Ground deformations (i.e., heave/settlement, horizontal displacement, etc.) were estimated at the time steps described above.

As the abutments and piers will be constructed on piled foundations (including battered piles), these were modelled as rigid structures with full lateral restraint and negligible weight.

### 5.5.1 SLS Performance

Three models were examined for the SDA, the locations are presented on Figure F1. Figure F2 represents a section of the RSS wall at the West Abutment. Figure F3 represent the NW bridge abutment in the transverse direction. Figure F4 shows the longitudinal section (parallel to Hwy 3) at the NW bridge abutment and approachway.

#### RSS Wall:

Graphs and sections of the SDA modeling are provided on Figures F2-1 to F2-4; the following is a brief description.

Drained SDA analysis results are presented on Figures F2-1 to F2-4. Once construction is complete and the backfill placed to design grades, about 60 mm of cumulative settlement will have occurred behind the abutment (Day 111, Figure F2-1). Most of this will be corrected by construction activities.

In the longer term, (Year 20 / 7300 days, Figure F2-2), cumulative settlements behind the abutment will total about 65 mm (this includes construction settlements). Figure F2-3 provides an estimate of the cumulative settlements anticipated behind the RSS wall. Based on the analysis maximum post construction settlements behind the wall will be in the order of 15 mm occurring at 16 m behind the wall

The ground movements generated by the construction loads are anticipated to stabilize within approximately 15 to 25 years following completion of construction.

Vertical movement of the top of the RSS wall and lateral movements estimated at the face of the RSS wall are provided on Figure F2-4. This figure indicates that the top of the RSS wall will settle less 10 mm before rebounding less than 5 mm. Lateral movement of the RSS wall will be approximately 10 mm inward. The total rotation of the wall inward is anticipated to be < 0.2 %. Most of the movement will take place after construction is completed and as the underlying soils undergo stress redistribution.

### Northwest Bridge Abutment, Transverse Sections:

The deformations at the northwest abutments have been analyzed in the transverse direction (north – south, across the approach alignment). The transverse SDA deformations model of the Bridge B-10, Northwest Abutment have been illustrated Figure F3. Graphs and sections of the SDA modeling are provided on Figures F3-1 to F3-4.

At the end of construction (Day 111, Figure F3-1) approximately 60 mm of cumulative settlement is anticipated behind the excavation (north), with 40 mm of heave occurring below Highway 401.

In the longer term, (Year 25, Figure F3-2), cumulative settlements behind the abutment will total approximately 60 mm (this includes construction settlements) and at Highway 401 will be approximately 95 mm with post construction heave approximately 55 mm (Figure 3-3).

### Northwest Bridge Abutment, Longitudinal Sections:

The deformations at the northwest abutments have also been analyzed in the longitudinal direction (east-west parallel to the approach alignment). The longitudinal SDA deformations model of the Bridge B-10, Northwest Abutment has been illustrated on Figure F4.

SDA analyses results are presented on Figures F4-1 to F4-4. At the end of construction (Day 111, Figure F4-1) approximately 90 mm of cumulative settlement are expected behind the excavation (west). Most of this will occur during construction and it is expected that most of this settlement will be accounted for by construction activities.

In the longer term, (Figure F4-2), cumulative settlements behind the abutment will total approximately 110 mm (this includes construction settlements).

Figure F4-3 provides an estimate of the cumulative settlements anticipated behind the abutment at the end of construction and long-term. Table 5-9 summarizes representative long-term deformation response obtained behind the northwest abutment of Bridge B-10. The ground movements generated by the construction loads are anticipated to stabilize within approximately 15 to 25 years following completion of construction.

Ground movement monitoring should be undertaken during and after construction at the locations indicated.

**Table 5-9: Summary of Calculated Deformations**

<b>Vertical Ground Movement<sup>(2)</sup> at Various Distances behind the Bridge Abutment<sup>(3)</sup> (mm)</b>				
<b>Loading Stage</b>	<b>10 m*</b>	<b>25 m</b>	<b>50 m</b>	<b>&gt;75 m</b>
End of Construction <sup>(1)</sup>	-80	-80	-5	0
Long-Term Settlements	-80	-105	-35	-30

(1) Represents cumulative total settlements which will be corrected during construction activities.

(2) Settlements are indicated by negative numbers

(3) Distances measured perpendicular to the bridge abutment (westerly from edge of wall)

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## 5.6 Pile Foundations

### 5.6.1 Resistance to Axial Loads

It is understood that HP310x110 steel H piles will be used at this project. The pile driving equipment and installation procedure should be established in the field by the Contractor with approval of the Engineer. A number of static load tests should be carried out at key locations along the alignment of WEP in conjunction with Pile Driving Analyzer (PDA) testing to facilitate proper calibration of the PDA, and determine the hammer performance and appropriate driving criteria (set).

The piles are expected to be driven to bedrock and accordingly they would mobilize a Ultimate Limit States (ULS) axial geotechnical resistance in excess of 4000 kN. Hence, a factored ULS geotechnical resistance of at least 2000 kN is anticipated.

The Serviceability Limit States (SLS) resistance of the HP310x110 piles, based on the conventional 25 mm settlement, is estimated to exceed the ULS resistance due to the unyielding nature of the bearing surface. Hence, the SLS resistance does not govern the design.

Based on the available borehole data at this structure, the bedrock surface elevation varies between 152.1 and 153, where the tips of piles are anticipated to be set.

In cases where some of the piles cannot be driven to bedrock due to refusal within dense lower granular deposit lying immediately above the bedrock, and/or a perceived risk of damaging the piles by overdriving is apparent, consideration should be given to supplementing the field testing to prove the actual mobilized resistance. If lower than assumed 4000 kN pile resistances are proven, options based on the most economical approaches may be considered (e.g., changes to the driving method and equipment, or addition of more piles).

The actual mobilized resistance of the production piles should be confirmed by dynamic testing using PDA methods on a minimum of 3% of the piles.

The following general pile installation recommendations should be considered:

- The steel H piles should be installed and monitored in accordance with OPSS 903 requirements. The piles should be reinforced with Type I shoe flanges as shown in OPSD 3000.100, or approved alternatives.
- Survey of all the pile head elevations should be completed at the end of driving and just prior to forming the pile cap. Re-tapping of the piles will be necessary where uplift exceeding 5 mm is noted, or as directed by engineer.
- While unlikely to occur at this site, considering the general geologic conditions in the region, indications of natural gas venting, water, and fines washout should be monitored during driving. Provision to mitigate such occurrences (by heavy mud, grouting of the cavities, etc.) should be in place. It is recommended that the pile splicing be completed by butt-welding (OPSD 3000.150, Section A-A) to minimize the pathways for upward flow of artesian water along the piles to the surface.



- Consideration should be given to potential driving difficulties due to the presence of dense to very dense lower granular soils and potential presence of cobbles and boulders above the bedrock.
- Vibrations generated by piling should be monitored. It is not expected that the vibrations during piling will have a significant impact on the stability of temporary slopes. Nonetheless, if the vibration intensities at the toe and top of the slopes exceed 10 mm/s, appropriate mitigation measures (slope flattening or vibration dampening by dumping sand around the piles) should be considered.
- Noise monitoring should be carried out during pile driving at the site.

### 5.6.2 ULS and SLS Resistance to Lateral Loads

The ULS and SLS geotechnical resistances to lateral loads should be determined on the basis of field load tests. Both the ULS and SLS lateral load resistances are strongly dependent on the soil properties, structural configuration of the pile and pile foundation, load configuration and deformations.

The SLS geotechnical resistance to lateral loads is dependent on the acceptable levels of the lateral pile deflections under the design loads and should be obtained on the basis of field load tests. In the absence of field tests, the preliminary design may be based as per Table 5-10.

The ULS lateral resistance is defined as the lateral force applied to the pile shaft causing unstabilised pile displacements due to soil failure or pile structural failure. In the absence of field tests, the ULS lateral resistance may be assumed as per Table 5-10.

**Table 5-10: Lateral Load Resistances**

Location	Limit State Loading	Pile Head Boundary	H-Pile Strong Axis		H-Pile Weak Axis	
			Lateral Load (kN)	Lateral Deflection (mm)	Lateral Load (kN)	Lateral Deflection (mm)
Vertical Pile	SLS	Fixed	125	5	61	2
	ULS		249	24	121	11
	SLS	Free	81	10	51	8
	ULS		217	99	101	41
Exterior Pile	SLS	Fixed	116	6	56	3
	ULS		231	27	112	12
	SLS	Free	71	10	45	8
	ULS		195	112	90	46

SLS - 10 mm of lateral deflection or 50% of ULS (whichever gives the lesser lateral load)

ULS - maximum bending moment = 384 kN-m (Strong Axis)

ULS - maximum bending moment = 125 kN-m (Weak Axis)

The above estimates were based on a pile model assumed to be embedded within firm to stiff silty clay. The above resistances were estimated using the “p-y” model (LPile 5.0 model Ensoft 2010). The “p-y” curves were generated using the Reese method described in the Technical manual for LPILE, using the Reese “Stiff-Clay without free water” model in conjunction with the following soil parameters:

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- Effective Unit Weight = 10 kN/m<sup>3</sup>;
- Average Undrained Strength = 50 kPa; and
- Strain corresponding to 50% of the ultimate shear stress,  $E_{50} = 0.010$ .

As mentioned earlier, the SLS criterion was set to 10 mm lateral deflection at the assumed ground surface. The ULS criterion for the above modeling was set at the onset of the plastic yielding in the pile section subjected to an induced bending moment.

It should be noted that during driving, significant soil disturbance and damage occur around the pile shaft forming sizeable gaps between the pile and the surrounding soils. These gaps cause significant reduction of the actual SLS and ULS resistances. Where the design relies on the lateral resistance provided by the soils, “repairs” to the disturbed soils must be undertaken (typically, the voids are grouted using non-shrink fills).

The stress-deformation analysis of the piles to lateral loads may be carried out using the horizontal subgrade reaction method. The coefficient of horizontal subgrade reaction,  $k_h$ , may be based on the following equations:

$$k_h = n_h (z/d) \quad \text{for cohesionless soils; and}$$

$$= 67 (S_u/d) \quad \text{for cohesive soils.}$$

Where:

$k_h$  (MPa/m) = Soil modulus of horizontal subgrade reaction;

$n_h$  (MPa/m) = Soil coefficient;

$S_u$  (MPa) = Undrained shear strength;

$z$  (m) = Depth below finished grade; and

$d$  (m) = Pile diameter/width.

The recommended ranges of soil parameters are tabulated as follows:

**Table 5-11: Soil Parameters for Lateral Load Resistance Calculations**

Soils Around the Piles	Elevation Range	Undrained Shear Strength, $S_u$ (kPa)
Clay Crust	183 to 177	75
Transition Zone	177 to 175	65
Silty Clay	175 to 161	45
Clayey Silt	161 to 151	65

Alternative pile design methods can be considered using the nonlinear “p-y” interaction method (discussed in the next subsection) or elastic continuum theory as discussed in the Canadian Foundation Engineering Manual (ref. R-6).

Significant lateral loads in excess of the preliminary previously cited should be resisted fully or partially by the use of battered piles. Batter piles are considered to be more effective in resisting horizontal loads, as a part of lateral load is converted into axial load and consequently the induced bending moments are less. For ease of constructability and to provide hammer energy sufficient for pile driving, batters are usually limited to no steeper than 1H:5V. However, higher batter may be achieved, if required.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, as indicated in Table 5.12. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed this table.

**Table 5-12: Lateral Load Capacity Reduction Factor For Pile Groups using the Horizontal Subgrade Reaction Method**

Pile Spacing in Direction of Loading	Subgrade reaction Reduction Factor
8d	1
6d	0.7
4d	0.4
3d	0.25

d = pile diameter

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2, Department of the Navy, Naval Facilities Engineering Command (1986).

#### *Alternative Nonlinear ‘p-y’ Curve Method:*

The p-y curve represents the total lateral soil reaction pressure ‘p’ (kPa) to the pile lateral deflection ‘y’ (m) relative to the surrounding soil mass at a particular section of the pile shaft in contact with the surrounding soils. Where only pile head loads are applied and there are no lateral movements of the surrounding soil mass, y is the absolute lateral deflection. Where lateral ground movements occur, y is the relative movement between the pile and the soil. The p-y curves reflect the non-linear soil behaviour under moderate to high stress levels where the more traditional elastic modeling of the soil response is considered to be insufficient.

The general procedure for computing p-y curves is summarized in the Canadian Foundation Engineering Manual of 2006. A detailed description for the generation of the p-y curves can be found in the Technical Manual for the commercial software LPILE Plus by Ensoft Inc. For a given foundation configuration, pile size, and soil stratification, the soil properties required for the generation of the p-y curves are provided in the table below. “Stiff clay” p-y curves as given in the LPILE manual should be developed appropriate for either static or cyclic loading conditions in absence of free water. For p-y curves below the water table, effective unit weights in the soil mass shall be used.

**Table 5-13: Soil Parameters for p-y Curve Calculation**

Soils Around the Piles	Elevation Range	Design Bulk Unit Weight (kN/m <sup>3</sup> )	Undrained Shear Strength, $S_u$ (kPa)	$\epsilon_{50}$
Clay Crust	Above 177	22	75	0.005
Transition Zone	177 to 175	22	65	0.007
Silty Clay	175 to 161	19.5	45	0.010
Clayey Silt	161 to 151	19.5	65	0.007

$\epsilon_{50}$  = Soil axial strain at 50% of the maximum deviatoric stress determined from undrained triaxial compression tests or estimated from correlations between  $S_u$  and  $\epsilon_{50}$ .

The obtained p-y curves may require to be scaled by a factor (“modifier”) to account for batter and for group effects.

In the case of batter of 1H:5V, the p-y curve modifier will be  $B_m = 0.75$  and 1.25 for the batter in the direction of the lateral load, and opposite direction of the lateral load, respectively.

In the case of group of piles, the modifier factors for the p-y curves are calculated as follows:

$$F_{mi} = \prod \beta_{ki}$$

where :

$\beta_{ki}$  = the influence factor of pile ‘k’ in the group on pile ‘i’, with  $k \neq i$ , and is calculated with one of the following expressions depending on the relative position of pile ‘k’ in the group with respect to pile ‘i’.

**Table 5-14: Lateral Load Capacity Reduction Factor For Pile Groups for p-y Method**

Relative Pile Position	Pile Spacing Ratio, $s/d$	$\beta_{ki}$
In Row (perpendicular to the load direction)	$< 3.75$	$0.64(s/d)^{0.34} \leq 1$
Leading pile in Line (first pile in line parallel to the load direction)	$\leq 4$	$0.70(s/d)^{0.26} \leq 1$
Trailing piles in line (piles behind the leading pile)	$\leq 7$	$0.48(s/d)^{0.38} \leq 1$

The modifier factor applies to the “p” values.

The spacing between the piles under the abutments is general less than 2200 mm (Sheet S021); appropriate reduction factors shall be based on individual abutment pile configurations. At the piers a closer spacing of approximately 1000 mm is anticipated. Group reduction factors will apply for lateral pile loadings.

LPILE software and other similar products provide automatic generation of the p-y curves along with the stress-deformation calculation of a pile subjected to various lateral loads applied at the pile cap and/or along the pile shaft, and various boundary conditions at the pile head and / or along the pile shaft.

### 5.6.3 Downdrag Loads (Negative Skin Friction – NSF)

Potential for downdrag loads on piles was considered in conjunction with the anticipated ground movements (rebound and settlements) that are assumed to occur during and following excavation of the overburden of up to 9 m to accommodate the future depressed highways, followed by re-placement of fills to construct the abutments.

Soil stress-deformation analyses described in Section 5.5 were conducted using the SIGMA/W software. The net estimated ground vertical movement (settlement/heave) at various times after excavation in the vicinity of the pile shafts is shown on Figures F5-1 to F5-4. Figures F5-1 and Figures F5-2 present the net estimated vertical ground deformations at the high side or “up-hill”, and pile on the “down-hill”, (Highway 401) side of the northwest abutment.

Considering the construction staging, the anticipated settlement-rebound of the soils and the transient nature of the NSF at the site, the recommended dead load and drag load combinations are as follows:

**Table 5-15: Summary of Anticipated Negative Skin Friction**

Pile Location	NSF Loads (kN)	Structural Loads
“Up-hill” Piles – i.e. North side of West Abutment and South side of east abutment*	600	No Dead Load
“Up-hill” Piles – i.e. North side of West Abutment and South side of east abutment	Nominal (<300)	Structural Dead Loads
Other piles at abutment locations	Nominal (< 300)	All cases
Piles at Pier Locations	No down-drag loads anticipated.	

\*Based on calculations for west abutment, east abutment results will be similar due to similarities in configuration.

In accordance with the Canadian Foundation Engineering Manual (ref. R-6), the service loads should not be reduced by any portion of the drag loads unless required by insufficient structural strength of the pile. Downdrag load and live load do not combine and two separate loading cases should be considered:

- Dead load plus downdrag load (but no transient live load); and
- Dead load and live load (but no downdrag load).

### 5.6.4 Shaft Bending

The approach to estimate the pile shaft bending caused by lateral deformations of the soil mass surrounding the piles was as follows:

- The pile was modelled as a HP section. The ground lateral movement (Figures F6-1 to F6-4) along the pile shaft anticipated to occur after the installation of the piles was estimated using the stress-deformation analysis described in Section 5.5.

- The pile head was assumed to be a free head.
- The above soil deformation field was imposed as “loads” along the pile shaft. The calculation was conducted using the “p-y” model (LPile 5.0 model Ensoft 2010). The “p-y” curves were generated using the Reese method described in the Technical manual for LPILE, using the soil parameters indicated in Table 5-16:

**Table 5-16: Assumed Soil Properties for Pile Interaction Assessment**

Material	Elevation (m)	Soil Model in L-Pile	Effective Unit Weight, kN/m <sup>3</sup>	S <sub>u</sub> , kPa
Transitional Clay	177 to 175	Stiff Clay without Free Water (Reese)	11	75
Silty Clay 1	175 to 165	Stiff Clay without Free Water (Reese)	11	55
Silty Clay 2	165 to 163	Stiff Clay without Free Water (Reese)	11	39
Native Silty Clay 3	163 to 161	Stiff Clay without Free Water (Reese)	11	43
Clayey Silt 1	161 to 153	Stiff Clay without Free Water (Reese)	11	65
Clayey Silt 2	153 to 151	Stiff Clay without Free Water (Reese)	11	65
Native Silty Clay	< 151	Stiff Clay without Free Water (Reese)	11	70

- The earth pressures from backfill and surcharge loads against the pile cap were not considered in the analyses.
- The shear force, bending moment and displacement within the pile were calculated from LPile model.

Based on this above approach the estimated maximum unfactored bending moments, shear force and deflection in the pile shaft due to the deformations of the soil mass around the piles are listed in Table 5-17 16 for both longitudinal and transverse directions.

**Table 5-17: Estimated Unfactored Loads on Pile Shaft**

**Free Pile Head - H-Pile Strong Axis**

Location	Maximum Induced Bending Moment, kN-m	Depth of maximum bending section below Pile Cap (m)	Lateral Load transferred by Pile Shaft to Pile Cap, kN	Deflection of the Pile at underside of Pile Cap, mm
Vertical Pile	35	12	30	7
Exterior Pile	35	12	30	7



### Free Pile Head - H-Pile Weak Axis

Location	Maximum Induced Bending Moment, kN-m	Depth of maximum bending section below Pile Cap (m)	Lateral Load transferred by Pile Shaft to Pile Cap, kN	Deflection of the Pile at underside of Pile Cap, mm
Vertical Pile	11	2	11	8
Exterior Pile	11	2	10	9

Based on the above approach and anticipated lateral ground displacement, the estimated induced maximum unfactored bending moment in the shaft was 35 kN-m for the strong axis pile loadings, and a free head condition. The shear force diagram indicated that the maximum shear force transferred by the pile shaft to the pile cap was <30 kN. These results should be considered in the structural design of the piles and in the design of pile cap structural components. These bending moments, shear forces and deflections are in addition to those caused by bridge loads applied to the piles. The maximum computed moment in the pile under assumed pile head load equal to the conventional SLS resistance (80 kN) was 85 kN-m for the strong axis pile loadings. Accordingly, a potential combination of the maximum bending stresses from pile head shear force and ground displacement field would lead to a maximum bending moment of 120 kN-m, which is less than the yield moment of the pile.

As indicated, the stress and deformation discussed above are in addition to the stress and deformation caused by the bridge loads. The structural designer should review the assumptions and analysis approach and satisfy himself with these findings.”

### 5.6.5 Earth Pressures on Abutment Walls

Behind the abutments and wing walls, non-frost susceptible free draining granular fill should be placed in accordance with CHBDC.

The granular backfill should be compacted in maximum 200 mm thick loose lifts in accordance with OPSS 501. Longitudinal drains and weep holes should be installed, as required, to ensure positive drainage of the backfill.

Heavy compaction equipment should not be used adjacent to the walls of the structure, where the backfill should be placed in maximum 100 mm thick loose lifts and compacted with small compactors. 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 the CHBDC.

For retained backfill that is placed and compacted in layers, the lateral force caused by compaction should be considered. In the absence of detailed analysis, the total lateral pressure due to soil weight and compactive effort should not be less than 12 kPa in any section of the wall.

**Table 5-18: Soil Parameters for Earth Pressure Calculations**

Soil Parameter	Group I Soils	Group II Soils	Group III Soils
Fill Unit Weight, $\text{kN/m}^3$	22	21	20.5
Friction angle, (degrees)	33 -35	29-32	22-30
Coefficients of Static Lateral Earth Pressure:			
'Active' or Unrestrained, $K_a^{(*)}$	0.27 to 0.30	0.310 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.

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)

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

## 6 Cahill Submerged Culvert

Cahill Submerged Culvert S-2 site is situated in the LaSalle segment of the Parkway, approximately 1 km east of the border between the Windsor and LaSalle Municipalities in Ontario. The culvert is located beneath Bridge B-10 (Highway 3) and west of bridge's east abutment. The existing open ditch Cahill drain running roughly east-west crosses the existing Highway 3 through the proposed culvert. Culvert S-2 will traverse under the proposed Highway 401, Highway 3 (Bridge B-10), EBR7, EBR8 and WBR6. The culvert will traverse under Bridge B-10 between bridge Pier 3N and Pier 3S located west of bridge's east abutment. The geotechnical aspects of Cahill culvert are being addressed under a separate cover. These include stability and potential soil deformations.

The submerged drain will consist of three 3.0 m diameter pipes which will be laid down in parallel array near Sta. 11+175L to 11+200L. As detailed in the Report Geotechnical Investigation and 90% Design Report, Cahill Submerged Culvert S-2, the post-construction ground deformations (settlements) caused along Hwy 401 would be less than 25 mm. Hence, no tangible impact on the bridge foundations are anticipated.

The bridge is supported on piles in the vicinity of the culvert and no loadings from the bridge structure onto the culvert are anticipated. It is expected that the piles will be driven before the culvert installation so that no concerns about the dynamic effects should arise with respect to the stability of temporary works, or with the performance of the backfill to the culvert and culvert structure itself.

## 7 Other Geotechnical Considerations

### 7.1 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, old farm tiles and/or utility trenches, and upper granular layers are likely to occur. In addition, random water bearing seams or pockets of fine sand and silts may be intersected by the excavations slopes. In adverse conditions, the runoff and seepage from perched groundwater and sand/silt pockets can be significant and accompanied by piping and wash-outs of the fines causing sloughing of the slopes.

Accordingly, provision should be made to prevent runoff and piping erosion of the slope surfaces by blanketing 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.

### 7.2 General Construction Requirements

The anticipated construction conditions in this report are discussed only to the extent of their potential influence on the design decisions to be made by the Contractors. 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.

As mentioned earlier, the Contractors are fully responsible for the design, construction methods, performance (stability, deformability and deterioration) and maintenance of the temporary slopes and temporary works. 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 rapid deterioration when exposed to elements, 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 prevent damage during excavation to the subgrade for foundations and pavements, the final excavation lift above the design elevation should not be less than 500 mm and should be carried out only when the Contractor is ready to prepare and cover the subgrade with the materials specified in the design same day the final excavation is exposed and approved. No construction traffic should be permitted over subgrade without approved protective covers.
- The final lift above the design subgrade should 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, geo-fabrics, granular mats, a skim coat (minimum 75 mm thick) of lean concrete protection (mud mat), etc. should be used.
- Regular monitoring and inspections of the condition of the temporary slopes for signs of instability, deterioration, sloughing, etc should be carried out by qualified personnel. Appropriate mitigation measures should be implemented.
- Excavations in this area should be limited in size in the area and appropriate monitoring of the existing nearby structures should take place. Monitoring should consist of a precondition survey along with regular surveying conducted of the nearby utilities, residences, etc.
- Riprap, and other coarse rockfill cover are considered to have half the insulation effect as offered by the fine grained soil deposits/cover, and therefore, the depth of frost penetration will have to be increased proportionally.
- The complete excavation for Highway 401 does not need to be advanced to the roadway subgrade within the same excavation operation as for the abutment walls. For the present design purposes, the bulk of the general excavation is assumed to be conducted close to the profiles shown on Figures G1 to G4.
- Staging limits of backfill will be required at some locations. These are indicated on the Conceptual Cross section Drawings, Appendix G.
- The contractor shall monitor for the potential emissions of natural gas (primarily hydrogen sulphide (H<sub>2</sub>S) and methane (CH<sub>4</sub>) gases) during construction.

### 7.3 Corrosion Potential

Analytical testing was carried out on samples of the clay obtained from Boreholes B10-1, B10-2 and B10-3 at Bridge B-10. A copy of the analytical report is provided in Appendix D. Table 7-1 provides the results of various analyses carried out on the soil samples to assess the potential for corrosion on concrete:

**Table 7-1: Results of Analytical Testing on Soils.**

	Elevation of Soil Sample	pH	Redox Potential, mV	Resistivity, ohm.cm	Sulphide, mg/kg	Sulphate, mg/kg
Borehole B10-1, Sample 25	150.6 m	7.98	163	2550	<0.2	338
Borehole B10-2, Sample 10	173.2 m	8.19	44	6450	<0.2	<20
Borehole B10-3, Sample 25	150.3 m	7.95	130	3470	<0.2	267
Borehole B10-4, Sample 24	151.9 m	7.0	123	1910	<0.2	567
Borehole B10-5, Sample 11	171.6 m	7.89	188	3420	<0.2	186
Borehole B10-6, Sample 23	153.0 m	7.84	218	1680	<0.2	657
Borehole B10-7, Sample 26	151.7 m	7.94	102	3220	<0.2	172
Borehole B10-8, Sample 10	173.3 m	8.00	87	5380	<0.2	52

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 be satisfied with their adequacy.

## 7.4 Construction Quality Control

To ensure that construction is carried out in a manner consistent with the intent of the PA, the design and 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.

As indicated in Section 5.4, the excavations below 5 m should be carefully monitored for basal heave and pore water response below the bottom of the excavation. If required, depth should be carried out in stages and in limited lifts (maximum 1 m thick) and sufficient time should be allowed for piezometric levels in the foundation substratum to subside following each stage of excavation.

## 7.5 Instrumentation and Monitoring during Construction

A program of site instrumentation and monitoring of the temporary works during construction should be implemented by the Contractor in addition to the limited instrumentation already installed during the geotechnical investigation.

The Contractor is responsible for planning, installation and maintenance of instrumentation as well as the completion of monitoring of the response of the excavations (ground movement) during construction. Detailed plans and procedures should be submitted to HMQ for approval prior to commencement of the monitoring of the works.



Monitoring is required to check the safety of the work, assess the effects of construction on surrounding ground and existing facilities, evaluate design assumptions, and refine estimates of future performance.

*Instrumentation:*

A limited number of geotechnical instruments have been installed during the recent geotechnical investigation at the locations of boreholes. These are described in Section 3.1.3 and are located as illustrated on Figure I-1, Appendix I.

Additional instruments may be appropriate at strategic locations to adequately cover the footprint of the construction area and the adjacent zone of influence. Figure I-1 also indicates a recommended preliminary layout of geotechnical instrumentation. The detail design of the monitoring instrumentation is the responsibility of the contractor. The type, number and locations of the instrumentations should be developed and revised in consideration of the observations during construction. Particular attention must be given to the nature and condition of the nearby facilities (residences, utilities, etc.) that may be affected by construction and may require additional and / or different type of instruments.

The instruments should be installed, and baseline monitoring should be completed before significant excavation has been occurred.

The instrument monitoring should be completed on a regular basis. As a general guideline, the following schedule should be considered after the completion of the baseline survey:

**Table 7-2: Monitoring Schedule of the Instruments**

<b>Instruments</b>	<b>Active Excavation</b>	<b>Active Construction inside the Excavation</b>	<b>Backfilling</b>	<b>Post-Construction</b>
Piezometers	EOD	D	W	M
Heave Gauge	EOD	EOD	W	M
Inclinometer	TPW	EOD	BW	M
Survey Pins	TPW	EOD	BW	M

D = Daily, EOD = Every Other Day, TPW = Twice per Week, W = Weekly, BW = Biweekly, M= Monthly

The frequency of monitoring can be modified depending on the ground response.

*Monitoring 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 the ground movements, the results of the survey should be assessed and modifications to the design and construction methodologies revisions may be required.

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

- Ground movement in excess of anticipated maxima (> 60 mm);
- Unstabilized movement trend without loading changes; and
- Non-responsive pore water 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.).

## 8 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 provides recommendations current to the issue date only. The geotechnical analyses and optimisations of the project are ongoing as design configurations and material requirements are updated and provided to us. These analyses will be updated and reported at a later date.

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, and CPT 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 the 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.

## 9 Closure

The design for Bridge B-10 was developed by Mr. Wayne Hurley, P.Eng. of TBT Engineering under design direction of Dr. Dan Dimitriu, P.Eng. and the report was reviewed by Dr. Don Dotson, P.Eng. The final design report was prepared by Mr. Tommi Leinala, P.Eng. and checked by Dr. Dan Dimitriu, P.Eng. (Lead Designer). The project was executed under the technical direction of Dr. Narendra S. Verma, P.Eng., who also reviewed the report. Mr. Matt Oldewening, P.Eng. managed the geotechnical investigation and Mr. Brian Lapos, P.Eng. is the project manager.

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

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

Yours truly,

**AMEC Environment & Infrastructure,**  
**a Division of AMEC Americas Limited**



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Principal Geotechnical Engineer  
Designated MTO RAQS Contact

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**Date:** March/2012

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## Drawings

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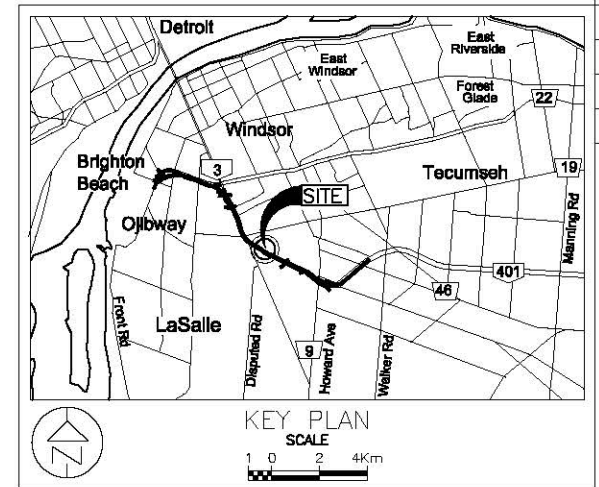
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









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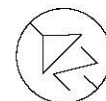
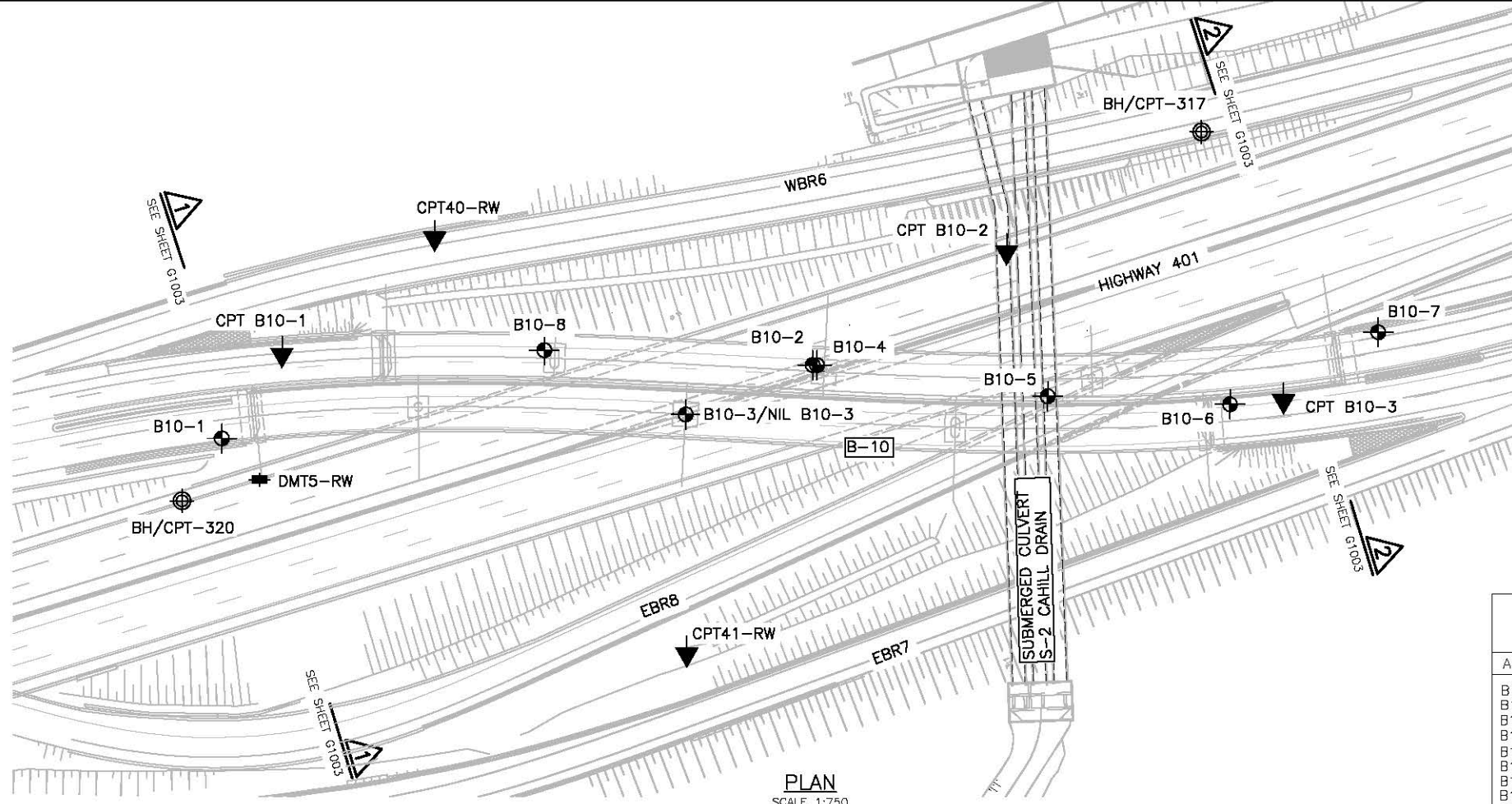
	BOREHOLE - CURRENT INVESTIGATION	N	SPT N-VALUE
	BOREHOLE & NILCON VANE - CURRENT INVESTIGATION		WATER LEVEL DURING DRILLING
	NILCON VANE - CURRENT INVESTIGATION	<i>DRY</i>	BOREHOLE DRY DURING DRILLING
	CPT-CURRENT INVESTIGATION		WATER LEVEL (SHALLOW PIEZO)
	DMT-CURRENT INVESTIGATION		WATER LEVEL (DEEP PIEZO)
	SW/SP HOLE (HYDROGEOLOGY)	PH -	SAMPLE OBTAINED UNDER HYDRAULIC PRESSURE
	BOREHOLE-PREVIOUS INVESTIGATIONS		
	BOREHOLE, CPT & NILCON VANE - PREVIOUS INVESTIGATIONS		
	CPT - PREVIOUS INVESTIGATIONS		
	TOPSOIL/ ORGANICS		SILT
	FILL		SANDY SILT
	SAND		CLAYEY SILT
	SILTY CLAY		SAND AND GRAVEL
	SILTY SAND		SILTY SAND AND GRAVEL
			LIMESTONE DOLOSTONE /BEDROCK

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

DOC: 285380-04-090-WP1-1001



## METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWNParkway  
Infrastructure  
Engineersamec  
Hatch Mott  
MacDonaldWindsor-Essex  
Parkway Project  
RFP No. 09-54-1007NEW CONSTRUCTION  
BRIDGE B-10  
HWY 3 UNDERPASS WEST OF GERAEDTS DRIVE  
BOREHOLE LOCATIONS & SOIL STRATASHEET  
G1002Phase 1  
IFCPLAN  
SCALE 1:750

No.	ELEVATION	CO-ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC BOREHOLES			
B10-1	B10-3	4679159.2	332754.5
B10-2		4679069.6	332866.8
B10-3/NIL		4679083.2	332836.8
B10-4		4679068.8	332867.5
B10-5		4679023.7	332901.0
B10-6		4678990.9	332930.4
B10-7		4678977.5	332967.8
B10-8		4679118.4	332824.1
CPT 10-1		4679147.4	332774.5
CPT 10-2		4679056.3	332920.1
CPT 10-3	182.1	4678983.4	332941.1
CPT40-RW	182.7	4679157.3	332826.1
DMT5-RW	183.8	4679146.3	332752.1
PREVIOUS BOREHOLES			
BH/CPT-317	182.6	4679041.7	332972.4
BH/CPT-320	183.5	4679155.5	332737.0

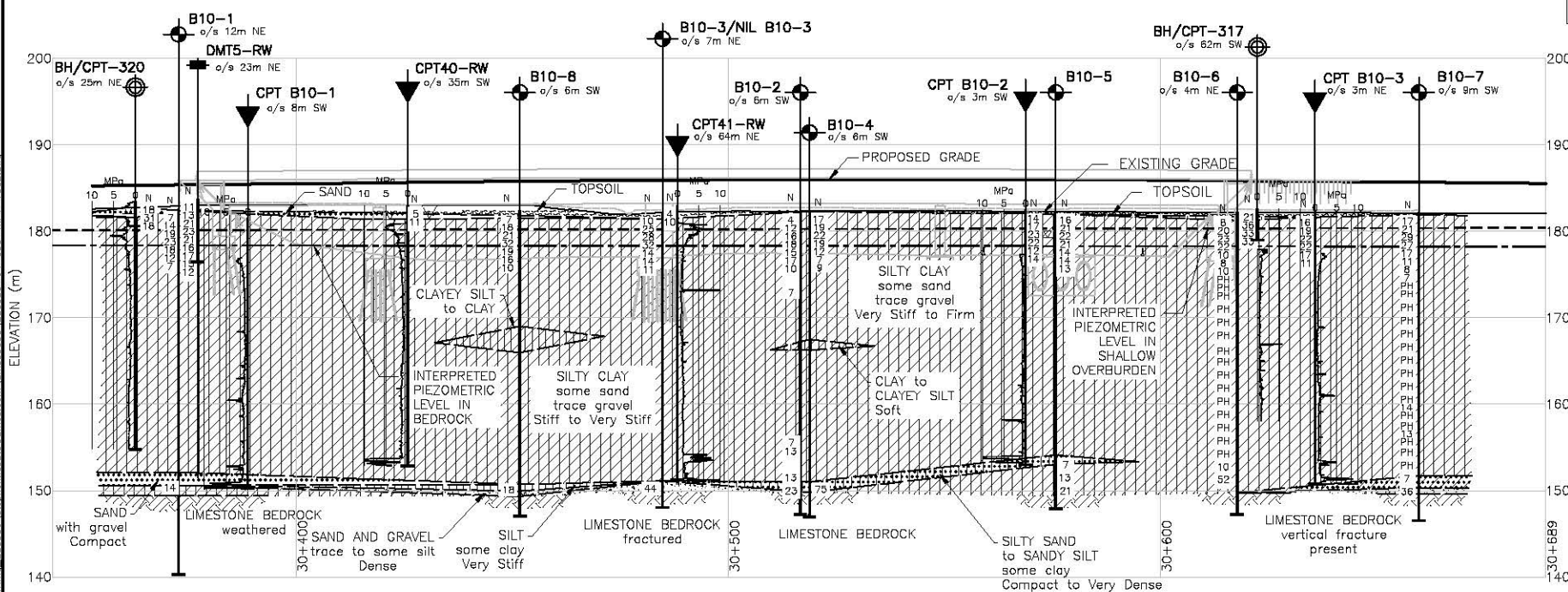
## LIST OF ABBREVIATIONS

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

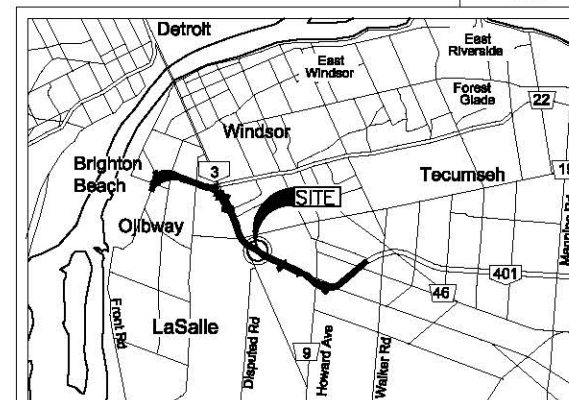
## MATERIAL LEGEND

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

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



PROFILE ALONG CL OF HIGHWAY 3 (SR3)

HORIZ SCALE 1:750  
VERT SCALE 1:375DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

KEY PLAN

SCALE  
1:0 2:0 4:0 km

## 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
- 16 BLOWS/0.3m UNLESS  
OTHERWISE STATED  
(STD. PEN. TEST, 475 J/BLOW)  
SEAL
- P-VIBRATING WIRE PIEZOMETER
- DRY BOREHOLE DRY DURING DRILLING
- WATER LEVEL DURING DRILLING
- WATER LEVEL (SHALLOW PIEZO)
- WATER LEVEL (DEEP PIEZO)
- MPa 0 5 10
- CPT, qc
- MHS - MAGNETIC  
HEAVE/SETTLEMENT  
GAUGE

## 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.

REVISIONS	19-MAR-12				ISSUED FOR CONSTRUCTION			
	DATE	REV.	BY	DESCRIPTION	DATE	REV.	BY	DESCRIPTION
DESIGN	WH	CHK	NSV	CODE CAN/CSA S6-06	LOAD	CL-625-ON		
DRAWN	MM	CHK	DD	SITE	6-610	DATE	18-JUL-11	

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



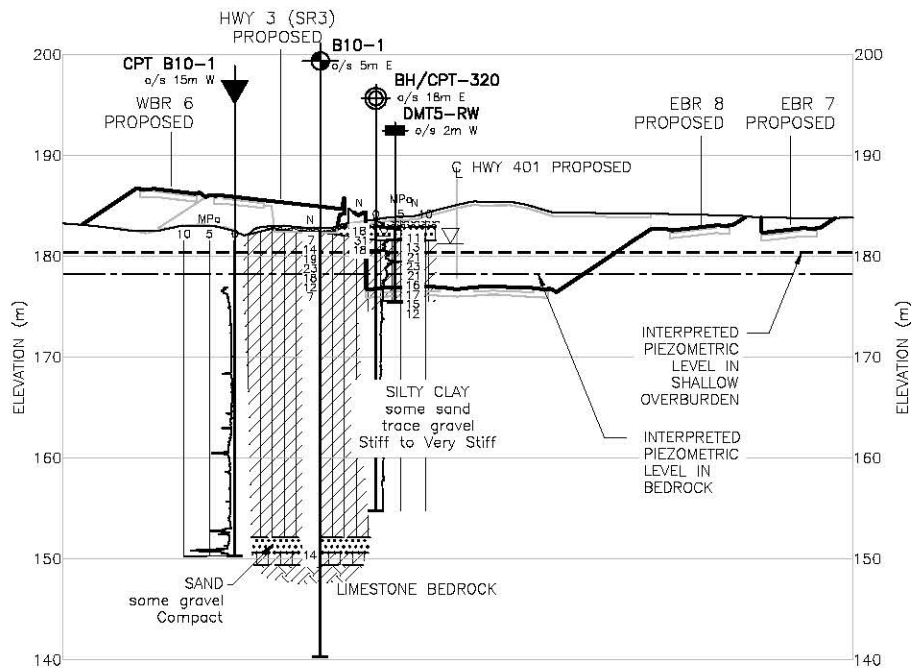
Windsor-Essex  
Parkway Project  
RFP No. 09-54-1007

NEW CONSTRUCTION  
BRIDGE B-10  
HWY 3 UNDERPASS WEST OF GERAEDTS DRIVE  
SOIL STRATIGRAPHY

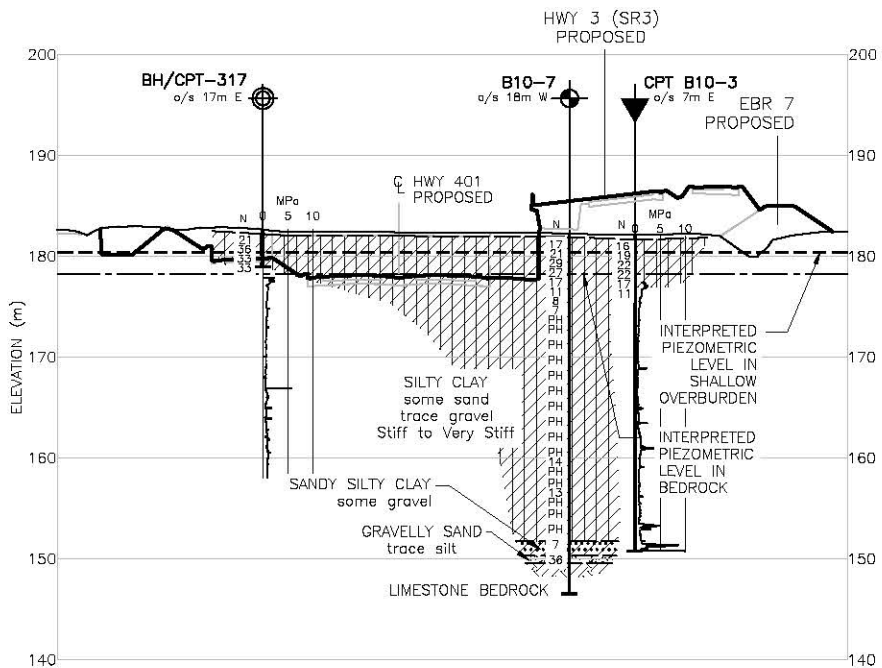


SHEET  
G1003

Phase 1  
IFC



HORT SCALE 1:750  
VERT SCALE 1:375



HORT SCALE 1:750  
VERT SCALE 1:375

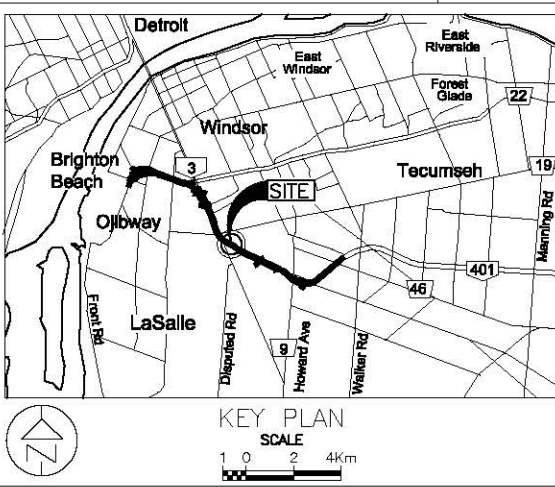
LIST OF ABBREVIATIONS

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PM - SAMPLER ADVANCED BY MANUAL PRESSURE  
WH - SAMPLER ADVANCED BY STATIC WEIGHT OF HAMMER  
WR - SAMPLER ADVANCED BY WEIGHT OF SAMPLER RODS

MATERIAL LEGEND

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

No.	ELEVATION	CO-ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
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BH/CPT-320	183.5	4679155.5	332737.0



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
- 16 BLOWS/0.3m UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
- SEAL
- P-VIBRATING WIRE PIEZOMETER
- BOREHOLE DRY DURING DRILLING
- WATER LEVEL DURING DRILLING
- WATER LEVEL (SHALLOW PIEZO)
- WATER LEVEL (DEEP PIEZO)
- MHSG - MAGNETIC HEAVE/SETTLEMENT GAUGE
- MPa 0 5 10
- CPT, qc

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.

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

REVISIONS	DATE	REV.	BY	DESCRIPTION
19-MAR-12	0	TL		ISSUED FOR CONSTRUCTION
DESIGN	WH	CHK	NSV	CODE CAN/CSA S6-06 LOAD CL-625-ON
DRAWN	MM	CHK	DD	SITE 6-610 DATE 18-JUL-11

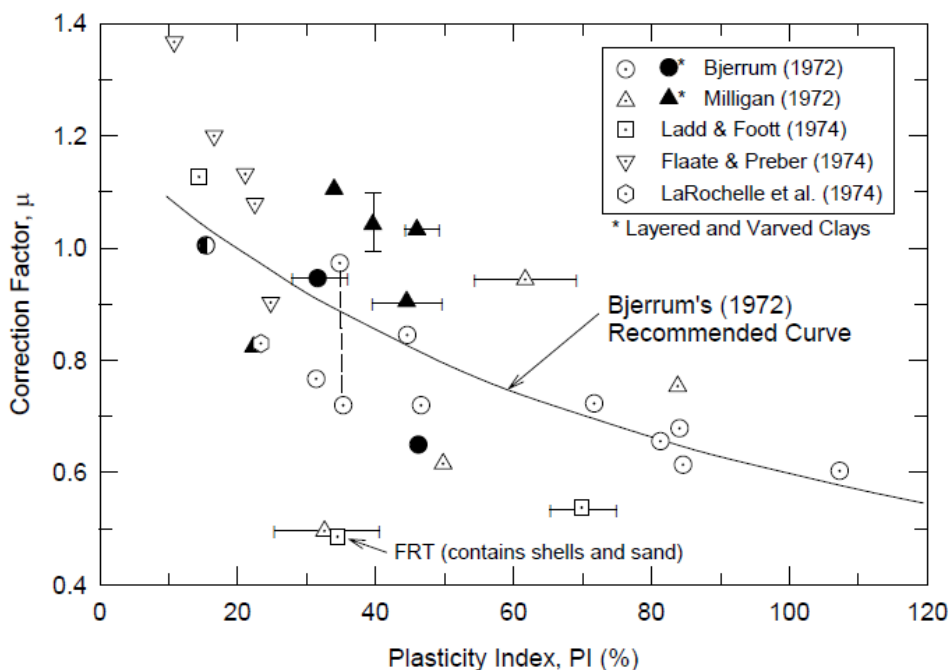


## Figures

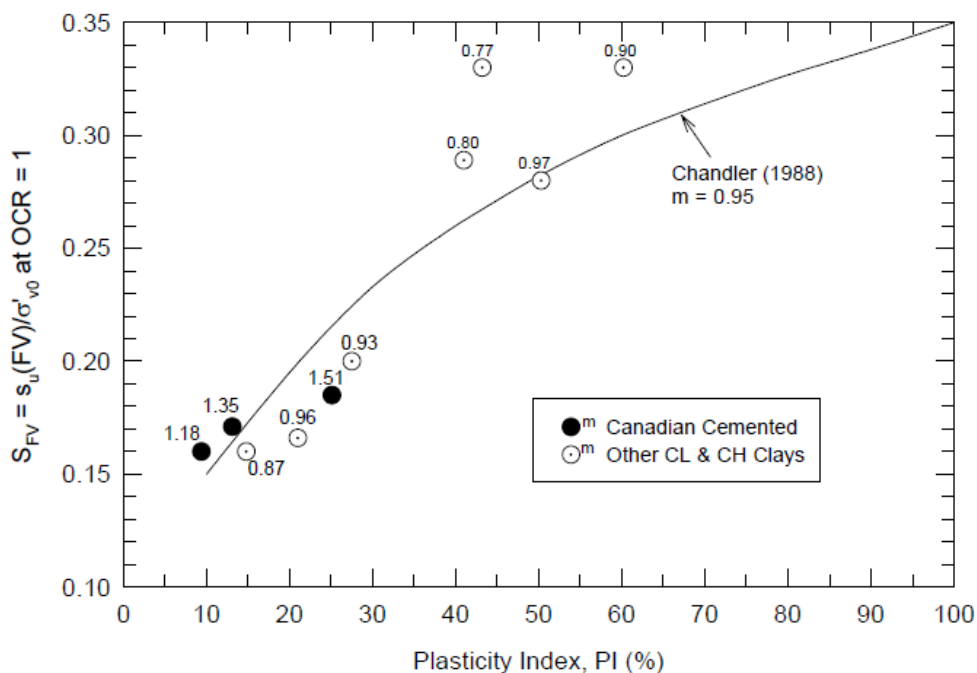
**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018

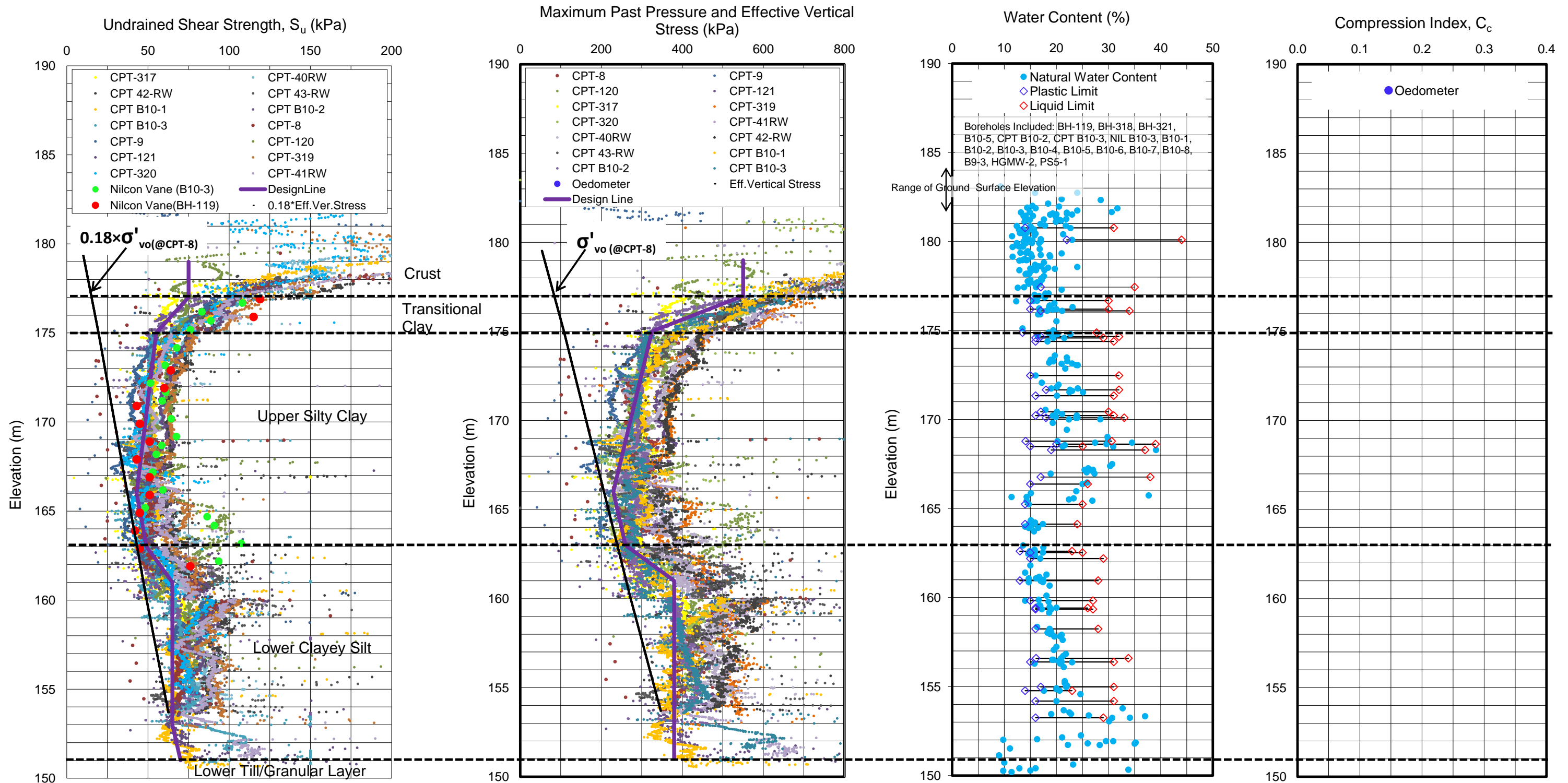
**Date:** March/2012  
**Rev:** 0  
**Page No.:** Figures

**Figure 3-1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures (Figure 5.1, Ladd & DeGroot, 2004, ref. R-30)**



**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-30)**



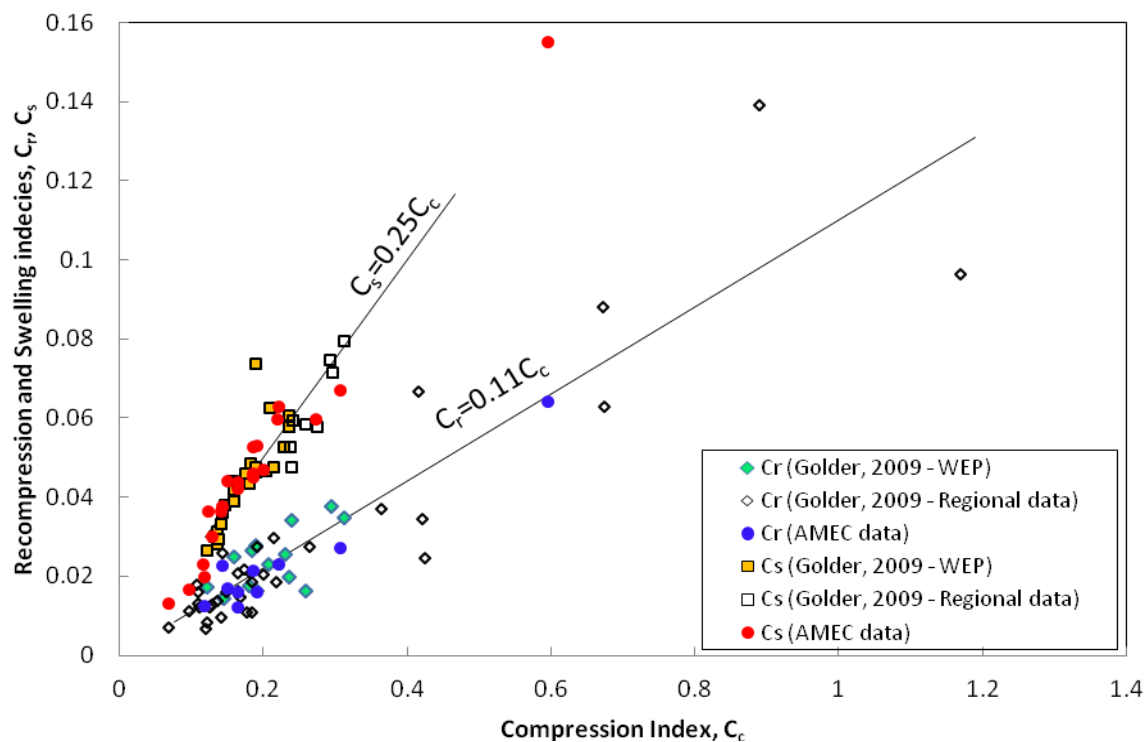
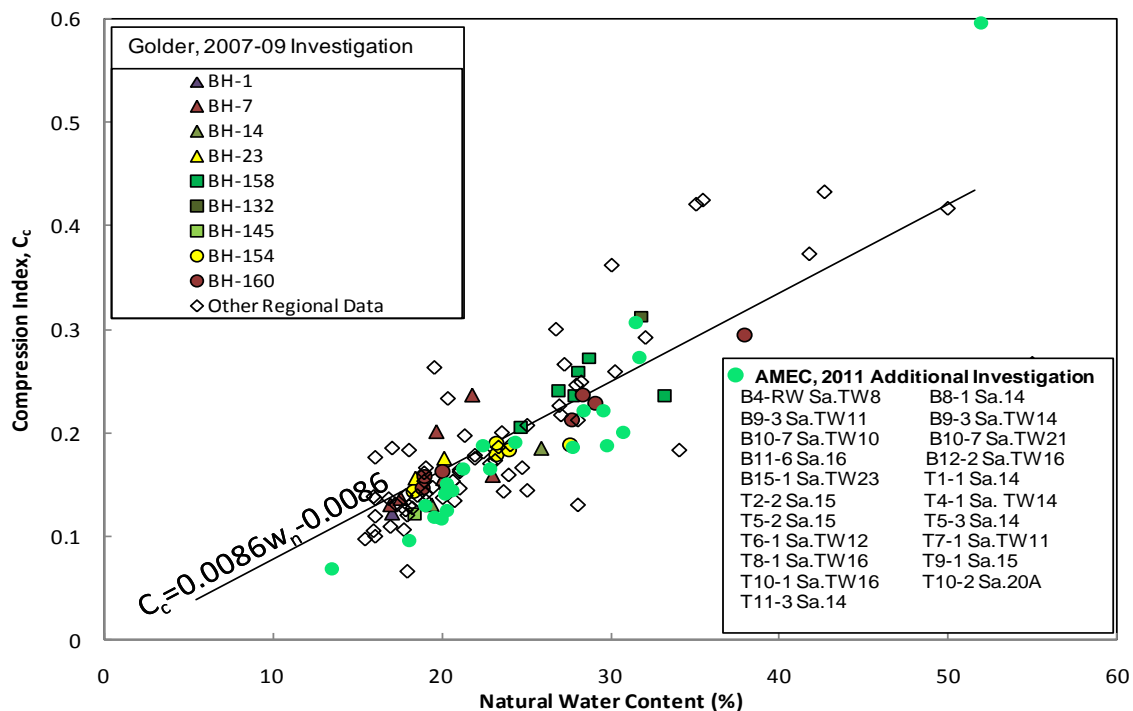


Notes:

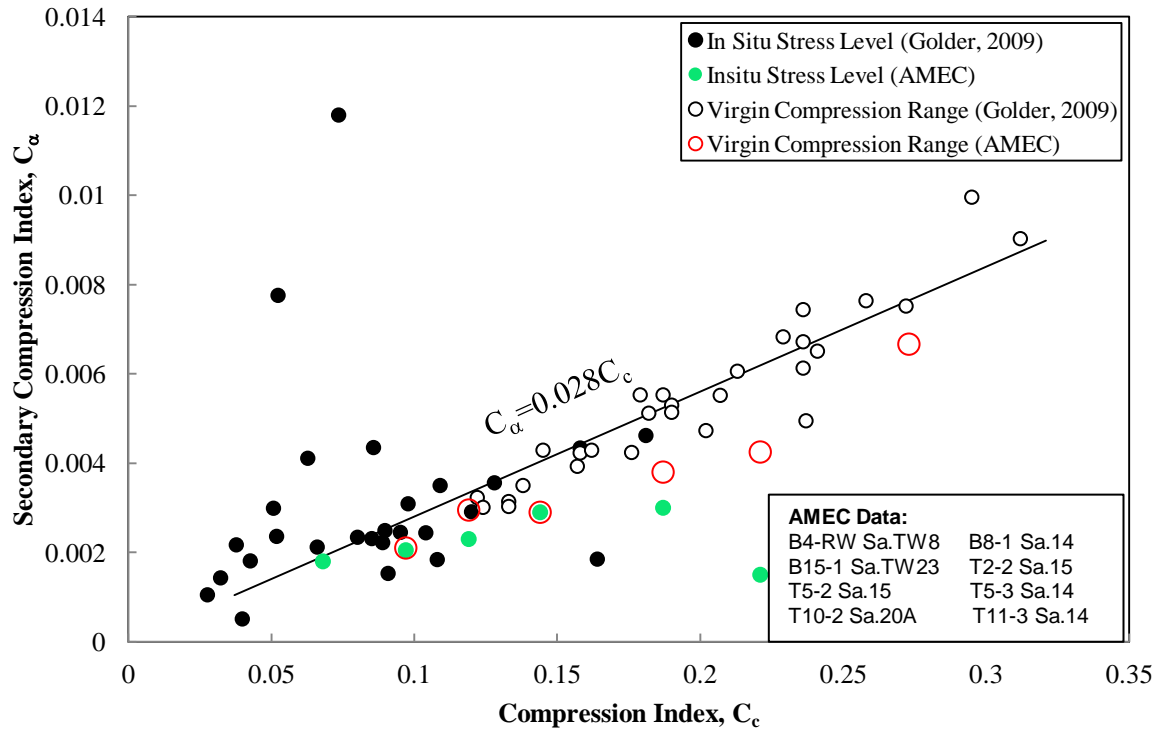
1. Shear strength profiles were estimated from CPT data using the equation  $S_u = (q_t - \sigma_{vo}) / N_{KT}$ . The cone factor  $N_{KT}$  was estimated by comparing the CPT profiles with a nearby Nilcon Vane profile.
2. Maximum past pressure profiles estimated using SHANSEP method.  $OCR = [(S_u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

Earth & Environmental	PROJECT: <b>WINDSOR ESSEX PARKWAY</b>				
	TITLE: <b>SOIL PROPERTIES PROFILES STA.10+900L TO 11+500L</b>				
CLIENT :	DATE: Mar 2012	JOB NO.: SW8801.1002	CAD FILE:	FIGURE NO.:	REV.:

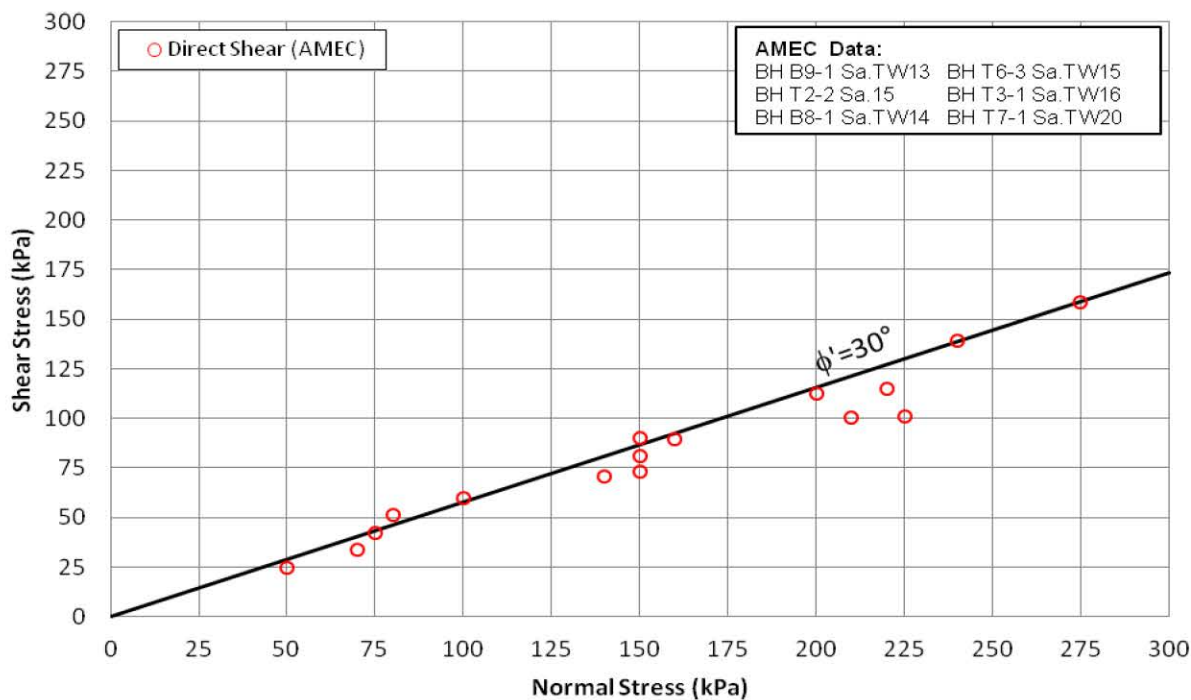
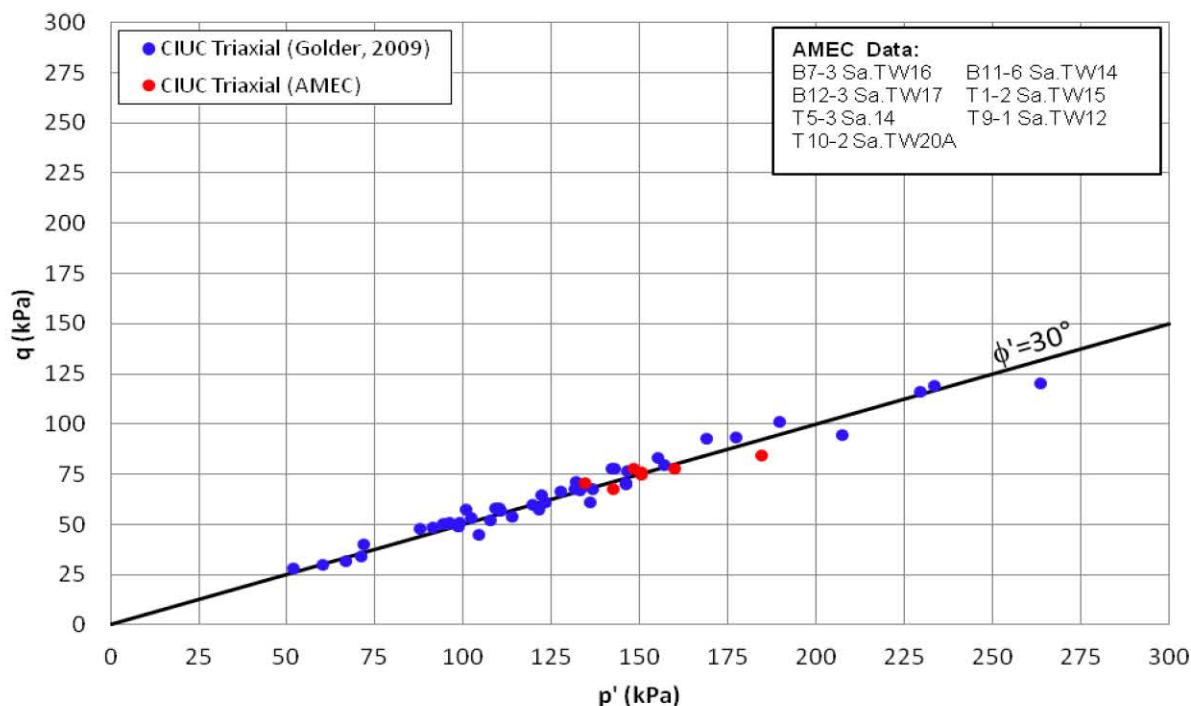
Figure 4-1: Compressibility Parameters at WEP



**Figure 4-2:  $C_c$  versus  $C_\alpha$  Relationship at WEP**

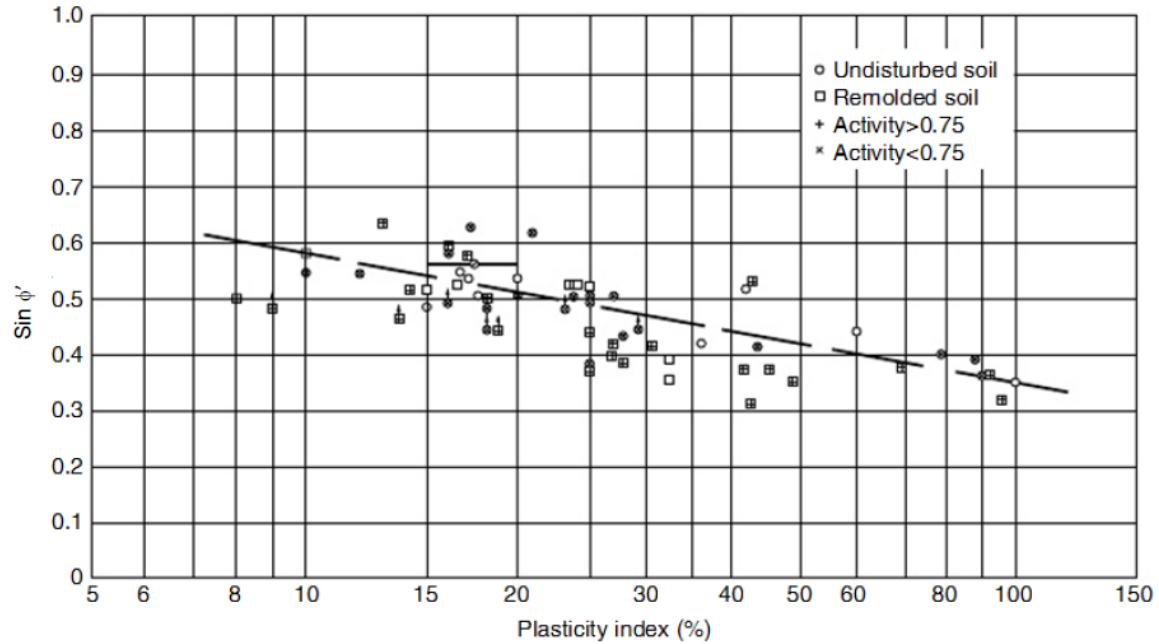


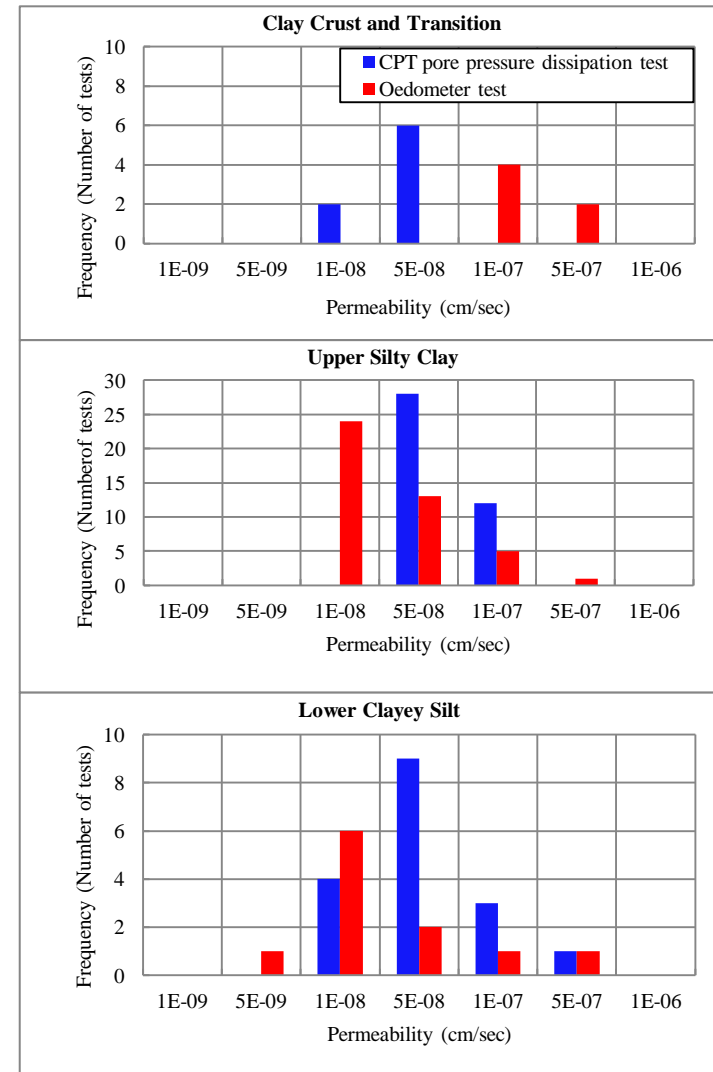
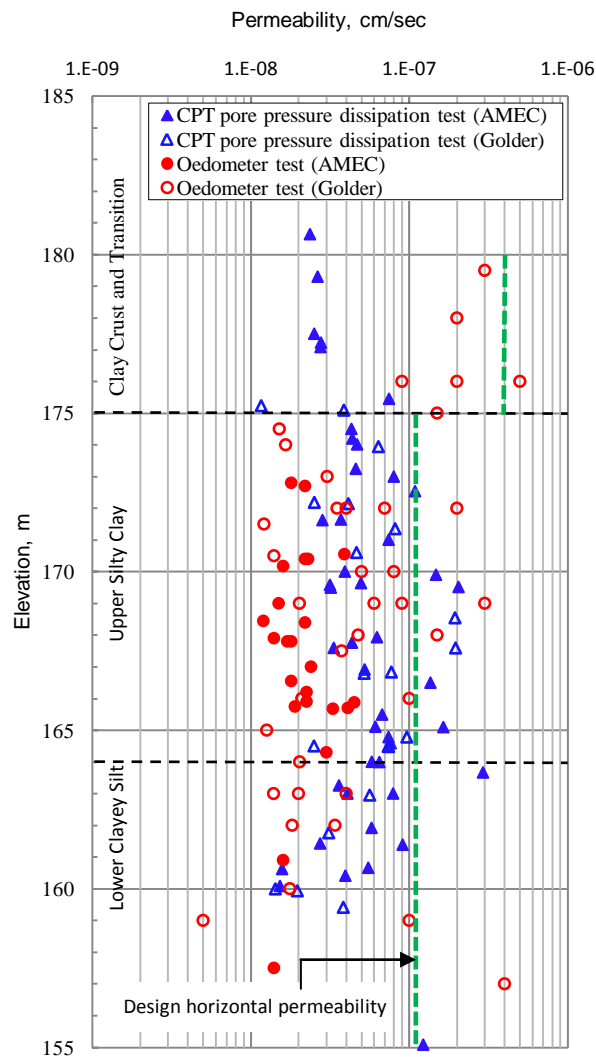
**Figure 4-3: Effective Friction Angle ( $\phi'$ ) for Silty Clay to Clayey Silt Stratum at WEP**





**Figure 4-4: Relationship between  $\sin \phi'$  and Plasticity Index for Normally Consolidated Soils (Kenney, 1959)**





**Figure 4-5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests**

## **Appendix A      Borehole and CPT Logs from Additional Geotechnical Investigation**

**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018

**Date:** March/2012  
**Rev:** 0  
**Page No.:** Appendix A

## EXPLANATION OF BOREHOLE LOG

This form describes some of the information provided on the borehole logs, which is based primarily on examination of the recovered samples, and the results of the field and laboratory tests. Additional description of the soil/rock encountered is given in the accompanying geotechnical report.

### GENERAL INFORMATION

Project details, borehole number, location coordinates and type of drilling equipment used are given at the top of the borehole log.

### SOIL LITHOLOGY

#### ***Elevation and Depth***

This column gives the elevation and depth of inferred geologic layers. The elevation is referred to the datum shown in the Description column.

#### ***Lithology Plot***

This column presents a graphic depiction of the soil and rock stratigraphy encountered within the borehole.

#### ***Description***

This column gives a description of the soil strata, based on visual and tactile examination of the samples augmented with field and laboratory test results. Each stratum is described according to the *MTC Soil Classification Manual*.

The compactness condition of cohesionless soils (SPT) and the consistency of cohesive soils (undrained shear strength) are defined as follows (*Ref. MTC Soil Classification Manual*):

Compactness of	
<u>Cohesionless Soils</u>	<u>SPT N-Value*</u>
Very loose	0 to 5
Loose	5 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

Consistency of	
<u>Cohesive Soils</u>	<u>Undrained Shear Strength</u>
	<u>kPa</u>
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very stiff	100 to 200
Hard	Over 200

\* For penetration of less than 0.3 m, N-values are indicated as the number of blows for the penetration achieved (e.g. 50/25: 50 blows for 25 centimeter penetration).

### Soil Sampling

Sample types are abbreviated as follows:

SS	Split Spoon	TW	Thin Wall Open (Pushed)	RC	Rock Core	GS	Grab Sample
AS	Auger Sample	TP	Thin Wall Piston (Pushed)	WS	Washed Sample	AR	Air Return Sample

Additional information provided in this section includes sample numbering, sample recovery and numerical testing results.

### Field and Laboratory Testing

Results of field testing (e.g., SPT, pocket penetrometer, and vane testing) and laboratory testing (e.g., natural moisture content, and limits) executed on the recovered samples are plotted in this section.

### Instrumentation Installation

Instrumentation installations (monitoring wells, piezometers, inclinometers, etc.) are plotted in this section. Water levels, if measured during fieldwork, are also plotted. These water levels may or may not be representative of the static groundwater level depending on the nature of soil stratum where the piezometer tips are located, the time elapsed from installation to reading and other applicable factors.

### Comments

This column is used to describe non-standard situations or notes of interest.

# MTC SOIL CLASSIFICATION

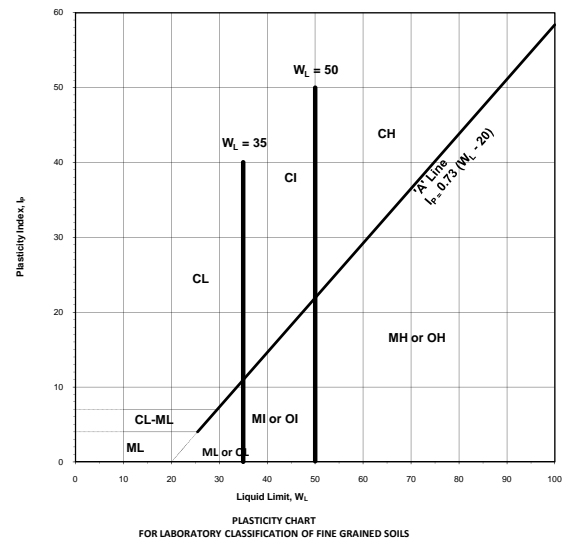
## Based on MTC Soil Classification Manual



MAJOR DIVISION					GROUP SYMBOL	TYPICAL DESCRIPTION	INFORMATION REQUIRED FOR DESCRIBING SOILS	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (LITTLE OR NO FINES)	WIDE RANGE IN GRAIN SIZE & SUBSTANTIAL AMOUNTS OF ALL INTERMEDIATE PARTICULAR SIZE	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GIVE TYPE, NAME, IF NECESSARY, INDICATE APPROX % OF SAND & GRAVEL ; MAX SIZE; ANGULARITY, SURFACE CONDITION, & HARDNESS OF THE COARSE GRAINS, LOCAL OR GEOLOGICAL NAME & OTHER PERTINENT DESCRIPTIVE INFORMATION, & SYMBOL IN PARENTHESIS.	$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN 1 AND 3		
			PREDOMINANTLY ONE SIZE OF A RANGE OF SIZES WITH STONE INTERMEDIATE SIZES MISSING	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES				
		GRAVEL WITH FINES (APPLICABLE AMOUNT OF FINES)	NON PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND- SILT MIXTURES				
			PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES				
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (LITTLE OR NO FINES)	WIDE RANGE IN GRAIN SIZE & SUBSTANTIAL AMOUNT OF ALL INTERMEDIATE PARTICLE SIZES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	FOR UNDISTURBED SOILS ADD INFORMATION ON STRATIFICATION, DEGREE OF COMPACTNESS, CEMENTATION, MOISTURE CONDITION & DRAINAGE CHARACTERISTICS	NOT MEETING ALL GRADATION REQUIREMENTS FOR GW		
			PREDOMINANTLY ONE SIZE OR A RANGE OF SIZES WITH SOME INTERMEDIATE SIZE MISSING	SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES				
		SANDS WITH FINES (APPLICABLE AMOUNT OF FINES)	NON PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES				
			PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES				
	FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	IDENTIFICATION PROCEDURE ON FRACTION SMALLER THAN 425µm					USE GRAIN SIZE CURVE IN IDENTIFYING THE FACTORS AS GIVEN UNDER FIELD IDENTIFICATION	DETERMINE PERCENTAGE OF GRAVEL & SAND FROM GRAIN SIZE CURVE, DEPENDING ON PERCENTAGE OF FINES (FRACTION SMALLER THAN 75 µm) COARSE GRAINED SOILS ARE CLASSIFIED AS FOLLOWS:  LESS THAN 5% GW, GP, SW, SP MORE THAN 12% GM, GC, SM, SC 5% TO 12% BORDER LINE CASES REQUIRE USE OF DUAL SYMBOL	$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN 1 AND 3
		LIQUID LIMIT LESS THAN 35 AND 50	DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)				
NONE			QUICK	NONE	ML				
MEDIUM TO HIGH			NONE TO VERY SLOW	MEDIUM	CL				
SLIGHT TO MEDIUM			SLOW	SLIGHT	OL				
LIQUID LIMIT BETWEEN 35 AND 50		NONE TO SLIGHT	SLOW TO QUICK	SLIGHT	MI				
		HIGH	NONE	MEDIUM TO HIGH	CI				
		SLIGHT TO MEDIUM	VERY SLOW	SLIGHT	OI				
		SLIGHT TO MEDIUM	SLOW TO NONE	MEDIUM	MH				
LIQUID LIMIT GREATER THAN 50		HIGH TO VERY HIGH	NONE	HIGH	CH				
		MEDIUM TO HIGH	NONE TO VERY SLOW	SLIGHT TO MEDIUM	OH				
		ORGANIC CLAYS OF HIGH PLASTICITY							
		PEAT AND OTHER HIGHLY ORGANIC SOILS				Pt			

USE GRAIN SIZE CURVE IN IDENTIFYING THE FACTORS AS GIVEN UNDER FIELD IDENTIFICATION

FRACTION	U.S STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS		
GRAVEL	COARSE	PASSING	RETAINED	PERCENT	DESCRIPTOR
		75 mm	26.5 mm	40-50 30-40 20-30 10-20 1-10	AND Y/EY WITH SOME TRACE
	FINE	26.5 mm	4.75 mm		
SAND	COARSE	4.75 mm	2.00 mm		
	MEDIUM	2.00 mm	425 µm		
	FINE	425 µm	75 µm		
FINES (SILT OR CLAY BASED ON PLASTICITY)		75 µm			
OVERSIZED MATERIAL					
ROUNDED OR SUBROUNDED: COBBLES 75 mm TO 200 mm BOULDERS > 200 mm				NOT ROUNDED: ROCK FRAGMENTS > 75 mm ROCKS > 0.76 CUBIC METRE IN VOLUME	



**BOUNDARY CLASSIFICATION:** BOUNDARY CLASSIFICATION: SOILS POSSESSING CHARACTERISTICS OF TWO GROUPS ARE DESIGNATED BY COMBINATIONS OF GROUP SYMBOLS FOR EXAMPLE GW-GC WELL GRADED GRAVEL-SAND MIXTURE WITH CLAY BINDER



AMEC Earth & Environmental,  
a Division of AMEC American

[www.amec.com](http://www.amec.com)

**MTC SOIL CLASSIFICATION MANUAL  
ENGINEERING PROPERTIES OF SOIL**



TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	PERMEABILITY WHEN COMPACTED	STRENGTH WHEN COMPACTED	COMPRESSIBILITY WHEN COMPACTED	WORKABILITY AS A CONSTRUCTION MATERIAL	SCOUR RESISTANCE	SUSCEPTIBILITY TO SURFICIAL EROSION	SUSCEPTIBILITY TO FROST ACTION	DRAINAGE CHARACTERISTICS
WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GW	PERVIOUS	EXCELLENT	NEGLECTIBLE	EXCELLENT	MEDIUM	NEGLECTIBLE	NEGLECTIBLE	EXCELLENT
POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GP	VERY PERVIOUS	GOOD	NEGLECTIBLE	GOOD	MEDIUM	NEGLECTIBLE	NEGLECTIBLE	EXCELLENT
SILTY GRAVELS, POORLY GRADED GRAVEL- SAND-SILT MIXTURES	GM	SEMI-PERVIOUS TO IMPERVIOUS	GOOD	NEGLECTIBLE	GOOD	LOW TO MEDIUM	SLIGHT	SLIGHT	FAIR TO SEMI IMPERVIOUS
CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	GC	IMPERVIOUS	GOOD TO FAIR	VERY LOW	GOOD	MEDIUM	SLIGHT	NEGLECTIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SW	PERVIOUS	EXCELLENT	NEGLECTIBLE	EXCELLENT	LOW TO MEDIUM	SLIGHT	NEGLECTIBLE	EXCELLENT
POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SP	PERVIOUS	GOOD	VERY LOW	FAIR TO GOOD	LOW TO MEDIUM	MODERATE	NEGLECTIBLE TO SLIGHT	EXCELLENT
SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES	SM	SEMI-PERVIOUS TO IMPERVIOUS	GOOD	LOW	FAIR	LOW	MODERATE	SLIGHT TO MODERATE	FAIR TO SEMI IMPERVIOUS IMPERVIOUS
CLAYEY SANDS, POORLY GRADED SAND WITH SOME CLAY MIXTURES	SC	IMPERVIOUS	GOOD TO FAIR	LOW	GOOD	VERY LOW TO LOW	MODERATE TO SLIGHT	NEGLECTIBLE	PRACTICALLY IMPERVIOUS
INORGANIC SILTS AND SANDY SILTS OF SLIGHT PLASTICITY, ROCK FLOUR	ML	SEMI-PERVIOUS TO IMPERVIOUS	FAIR	MEDIUM	FAIR	VERY LOW	SEVERE	SEVERE	FAIR TO POOR
INORGANIC CLAYEY SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS	CL	IMPERVIOUS	FAIR	MEDIUM	GOOD TO FAIR	LOW TO MEDIUM	SLIGHT TO MODERATE	MODERATE TO SEVERE	PRACTICALLY IMPERVIOUS
ORGANIC SILTS OF LOW PLASTICITY	OL	SEMI-PERVIOUS TO IMPERVIOUS	POOR	MEDIUM	FAIR TO POOR	VERY LOW TO LOW	SEVERE	SEVERE	POOR
INORGANIC COMPRESSIBLE SILTS OF MEDIUM PLASTICITY	MI	SEMI-PERVIOUS TO IMPERVIOUS	FAIR	MEDIUM TO HIGH	FAIR TO POOR	LOW	MODERATE	MODERATE TO SEVERE	FAIR TO POOR
INORGANIC SILTY CLAYS OF MEDIUM PLASTICITY	CI	IMPERVIOUS	FAIR TO POOR	HIGH	FAIR	LOW TO MEDIUM	SLIGHT	MODERATE TO SEVERE	SEMI IMPERVIOUS TO PRACTICALLY
ORGANIC SILTY CLAY OF MEDIUM PLASTICITY	OI	SEMI-PERVIOUS TO IMPERVIOUS	POOR	HIGH	POOR	VERY LOW TO LOW	SEVERE	MODERATE TO SEVERE	POOR TO PRACTICALLY IMPERVIOUS
INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	MH	SEMI-PERVIOUS TO IMPERVIOUS	FAIR TO POOR	HIGH	POOR	VERY LOW	MEDIUM	SEVERE	POOR
INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	CH	IMPERVIOUS	POOR	HIGH	FAIR TO POOR	LOW TO MEDIUM	SLIGHT TO NEGLECTIBLE	NEGLECTIBLE	PRACTICALLY IMPERVIOUS
ORGANIC CLAYS OF HIGH PLASTICITY	OH	IMPERVIOUS	POOR	HIGH	POOR	LOW	MODERATE	NEGLECTIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
PEAT AND OTHER HIGHLY ORGANIC SOILS	Pt	-	-	-	-	LOW	SEVERE	-	FAIR TO GOOD



## METRIC

Continued Next Page

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

# RECORD OF BOREHOLE No B10-1

2 OF 4

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4679159.2N, 332754.5E ORIGINATED BY LC  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE May 26, 11 - Jun 1, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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						
	-Sandy clay seams		14	TW	PH	167			×						○	19.5	14.57m below ground surface (El. 168.0 m)		
					VT							>>	1.47					-switch to NW wash bore	
					15	TW	PH	166						○					
								165											
								164			×				○		21.9		
						VT							>>	1.48					-end of drilling May 26; continue May 27 -VWP B10-1-P21 installed at 20.6m below ground surface (El. 162.0 m) -hit hard layer (rocks) -damaged shelby tip
					17	TW	PH	163							○				
								162											
								161							○				
						VT													
								160								○			
					19	TW	PH	159											
								158			×					⊞	1.5	20.8	2 21 44 33
						VT													
					21	TW	PH	157								○		20.6	
								156											
								155			×					○		20.5	
								154									○		
					23	TW	PH	153											

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11



**RECORD OF BOREHOLE No B10-1**

4 OF 4

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4679159.2N, 332754.5E ORIGINATED BY LC  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE May 26, 11 - Jun 1, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					w <sub>p</sub> w                      w <sub>L</sub>											
								○ UNCONFINED                      + FIELD VANE ● POCKET PEN.                      × LAB VANE					WATER CONTENT (%)											
								20	40	60	80	100		10	20	30								
	VWPB10-1-P21 (mid-depth):  June 4, 2011:   EL.   180.6m June 25, 2011:   EL.   180.7m July 10, 2011:   EL.   180.6m July 23, 2011:   EL.   180.4m Aug. 29, 2011: El. 180.4 m						137																	
							136																	
							135																	
							134																	
							133																	
							132																	
							131																	
							130																	
							129																	
							128																	
							127																	
							126																	
125																								
124																								
123																								

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11

## METRIC

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

## METRIC

Continued Next Page

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



# RECORD OF BOREHOLE No B10-2

3 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679069.6, E332866.8 ORIGINATED BY SD  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE May 30, 11 - May 31, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
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			GR   SA   SI   CL		
150.3															
32.0															
149.1															
33.2															

-end of drilling  
 May 30; continue  
 May 31  
 ROD=62%  
 TCR = 92%  
 SCR = 77%





+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B10-3

1 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679083.2, E332836.8 ORIGINATED BY LC  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE Jun 1, 11 - Jun 7, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
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
182.2	Ground Surface															GR SA SI CL		
0.0	TOPSOIL, clayey															-vibrating wire piezometers (VWP) installed in borehole -hand dug test hole for confirmation of topsoil thickness		
181.4	CLAYEY SILT Some sand, trace gravel Firm to hard, Mottled bown-grey to brown to grey trace pink nodules Trace organics		1	SS	10								○			-hand dug test hole for confirmation of topsoil thickness filled with groundwater at 0.45m below ground surface after 30 min.		
0.8			2	SS	25									○				
			3	SS	28									○				
			4	SS	32									○				
			5	SS	14									○				
			6	SS	14									○			-25mm diameter stone in tip of split spoon	
			7	SS	11									○				
			8	TW	PH					×					○			
			9	TW	PH						×				○			
			10	TW	PH					×					○			
	-Thin silt lenses/inclusions			VT														
			11	TW	PH									○			-VWP B10-3-P10 installed at 10.1m below ground surface (El. 172.1 m)	
			12	TW	PH					×				○				
				VT														
			13	TW	PH					×					○			
168.8	SILTY CLAY And thin silt partings, inclusions Firm, dark grey																	
13.4																		
167.6																		
14.6																		

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11

## METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11

## METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

## METRIC

Continued Next Page

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

# RECORD OF BOREHOLE No B10-4

2 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4679068.8N, 332867.5E ORIGINATED BY DG  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE Jun 7, 11 - Jun 8, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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	CLAYEY SILT Some sand, trace gravel Soft to stiff, grey (continued)																	
			14	TW	PH									20.4				
			15	TW	PH													
				VT														
			16	TW	PH									21.9	7 26 44 24			
			17	TW	PH													
				VT														
			18	TW	PH									21.5	-end of drilling june 7; continue june 8			
			19	TW	PH													
			20	TW	PH									21.0				
			21	TW	PH													
			22	TW	PH									21.4	2 17 44 36			
			23	TW	PH													

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11



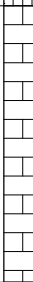


# RECORD OF BOREHOLE No B10-4

3 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION 4679068.8N, 332867.5E ORIGINATED BY DG  
DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
DATUM Geodetic DATE Jun 7, 11 - Jun 8, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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									
152.2							20	40	60	80	100							
30.2	<b>SILTY CLAY</b> Soft, grey		24	TW	PH													
151.0	<b>FINE SANDY SILT</b> Trace fine-med gravel Very dense, grey																	
31.4			25	SS	75													
149.7	<b>LIMESTONE</b> Medium to fine grained Laminated, stylolites present, white to grey		26	RC														
32.7																		
			27	RC														
			28	RC														
147.0	<b>END OF BOREHOLE</b>  No groundwater observed during drilling due to wash boring																	
35.4																		

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

# RECORD OF BOREHOLE No B10-5

1 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION 4679023.7N, 332901E ORIGINATED BY TP  
DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
DATUM Geodetic DATE May 13, 11 - May 14, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED ● POCKET PEN.	+ FIELD VANE × LAB VANE									
182.3	Ground Surface														GR SA SI CL			
0.0 182.0 0.3	300mm Black Organic Clay <b>TOPSOIL</b> <b>CLAYEY SILT</b> Some sand, trace gravel Stiff to very stiff, mottled brown-grey to brown to grey														-Observation Well (OW) and Vibrating Wire Piezometers (VWP) installed in borehole			
			1	SS	16													
			2	SS	21													
			3	SS	22													
			4	SS	21										-bulk auger sample retained			
			5	SS	14													
			6	SS	14													
			7	SS	13													
			8	SS	PH										-no recovery with shelby tube; sample retrieved by pushing split spoon			
			9	SS	PH										-no recovery with shelby tube; sample retrieved by pushing split spoon			
			10	TW	PH													
			VT															
			11	TW	PH													
			12	TW	PH										-VWP #P12 installed at 11.6m below ground surface			
			VT															
168.7 13.6	<b>SILTY CLAY</b> Grey, some pink and black nodules, sandy clay lenses		13	TW	PH													

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11

# RECORD OF BOREHOLE No B10-5

2 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4679023.7N, 332901E ORIGINATED BY TP  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE May 13, 11 - May 14, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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										
								20 40 60 80 100	20 40 60 80 100	10 20 30									
	<b>SILTY CLAY</b> Grey, some pink and black nodules, sandy clay lenses (continued)		14	TW	PH														
					VT														
165.7 16.6	<b>CLAYEY SILT</b> Some sand, trace gravel Firm to stiff, grey		15	TW	PH			×						21.0	-switch to wash bore with casing				
				16	SS	PH													
						VT													
			17	TW	PH								21.7	-no recovery with shelby tube; sample retrieved by pushing split spoon					
			18	TW	PH														
					VT														
			19	TW	PH								21.7						
			20	SS	PH														
			21	SS	PH														
155.5 26.8	<b>SILTY CLAY AND FINE SAND</b> Laminated Firm, grey		22	TW	PH														
154.1 28.2	<b>SILTY FINESAND to SANDY SILT</b> , Some clay, saturated Compact, grey																		
153.0 29.3	<b>SILTY CLAY</b> Firm to stiff, grey		23A, B	SS	7														

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11



# RECORD OF BOREHOLE No B10-5-P32

1 OF 4

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679021, E332901.4 ORIGINATED BY LC  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE May 16, 11 - May 17, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
182.3	0.0	Augered to 3.05m, then NW casing to bedrock. No sampling overburden vibrating wire piezometer (VWP) installed in borehole					182																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11

## 2 OF 4

METRIC

[illegible]

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



## METRIC

[illegible]

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11

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

# RECORD OF BOREHOLE No B10-5-P32

4 OF 4

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679021, E332901.4 ORIGINATED BY LC  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE May 16, 11 - May 17, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT  W <sub>p</sub>	NATURAL MOISTURE CONTENT  W	LIQUID LIMIT  W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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												
	July 23, 2011: EL. 177.0m						20	40	60	80	100	10	20	30						
							137													
							136													
							135													
							134													
							133													
							132													
							131													
							130													
							129													
							128													
							127													
							126													
							125													
							124													
							123													

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/11/11

# RECORD OF BOREHOLE No B10-6

1 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4678990.9N, 332930.4E ORIGINATED BY TA  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE May 14, 11 - May 16, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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						
182.0	Ground Surface																
0.0	300mm Black Organic Sandy Silt																
181.7	TOPSOIL																
0.3	Orange-Brown																
181.2	SILTY SAND																
0.8	Brown																
	CLAYEY SILT		1	SS	8		181										
	Some sand, trace gravel																
	-Layers of fine to medium sand, clayey sand																
	-Rootlets		2	SS	20		180									-bulk auger sample from 1.5m to 3.8m	
	Very stiff																
			3	SS	23												
							179									-cobble/boulder encountered augers chattering	
			4	SS	22												
	Grey		5	SS	10		178										
	Stiff																
	-Some small pink clay nodules below 4.5m		6	SS	8		177										
			7	SS	10												
							176										
			8	TW	PH												
							175										
			9	TW	PH		174							21.3	2 24 38 36		
							173										
			10	TW	PH												
							172										
				VT													
			11	TW	PH		171							20.3			
				VT													
							170										
			12	TW	PH												
				VT			169										
							168							19.4	1 10 41 48		
			13	TW	PH												
				VT													

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 11/10/11

# RECORD OF BOREHOLE No B10-6

2 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4678990.9N, 332930.4E ORIGINATED BY TA  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE May 14, 11 - May 16, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE						● POCKET PEN.	× LAB VANE
166.9 15.1	Grey CLAYEY SILT Some sand, trace gravel															
			14	TW	PH											
				VT												
			15	TW	PH				×							
			16	TW	PH											
			17	TW	PH				×							
		18	TW	PH												
		19	TW	PH												
		20	SS	PH												
		21	TW	PH												
155.2 26.8	Grey SILTY CLAY AND SILT/FINE SAND Laminated Firm															
			22	TW	PH											
		23	SS	10												

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 11/10/11

## METRIC

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 11/10/11

## METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



# RECORD OF BOREHOLE No B10-7

2 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4678978.0N, 332968.0E ORIGINATED BY SD  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 25 May 11 - 31 May 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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									20 40 60 80 100	10 20 30
	CLAYEY SILT Some sand, trace gravel Soft to very stiff (continued)		14	TW	PH		167								21.2	-end of augers; continue with N-casing		
							166			1.5								
				VT														
			15	TW	PH		165	×										
							164											
			16	TW	PH		163									21.7	-VWP B10-7-P21 & MG B10-7-SM21 installed at 19.8m below ground surface (El. 162.4 m)	
							162			×								
							161											
			18	TW	PH		160											
			19	SS	14													
							159			×						20.8	-end of drilling May 26; continue May 27	
							158											
			21	TW	PH		157											
			22	SS	13		156											
			23	TW	PH		155											
					154									18.5				
					153			×										
	25	TW	PH															

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

## METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



## METRIC

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

## METRIC

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

# RECORD OF BOREHOLE No CPT B10-2

1 OF 1

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679056.3, E332920.1 ORIGINATED BY TA  
 DIST                      HWY WEP BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE May 13, 11 - May 13, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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										
182.2	Ground Surface																		
0.0	<b>TOPSOIL</b> , 275mm thick, organic clay, black																		
181.9	<b>CLAYEY SILT</b> Some sand, trace gravel Stiff to very stiff, mottled brown-grey to brown to grey																		
0.3			1	SS	14														
			2	SS	17														
			3	SS	23														
			4	SS	22														
			5	SS	12														
	-Vertical silt seams		6	SS	14														
177.2	<b>END OF SAMPLED BOREHOLE</b> Continue with CPT from 5.0 m to refusal at 29.3 m (El. 177.2 m to El. 152.9 m)																		
5.0																			

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

# RECORD OF CONE PENETRATION TEST CPT B10-1

**METRIC**

PROJECT Windsor-Essex Parkway

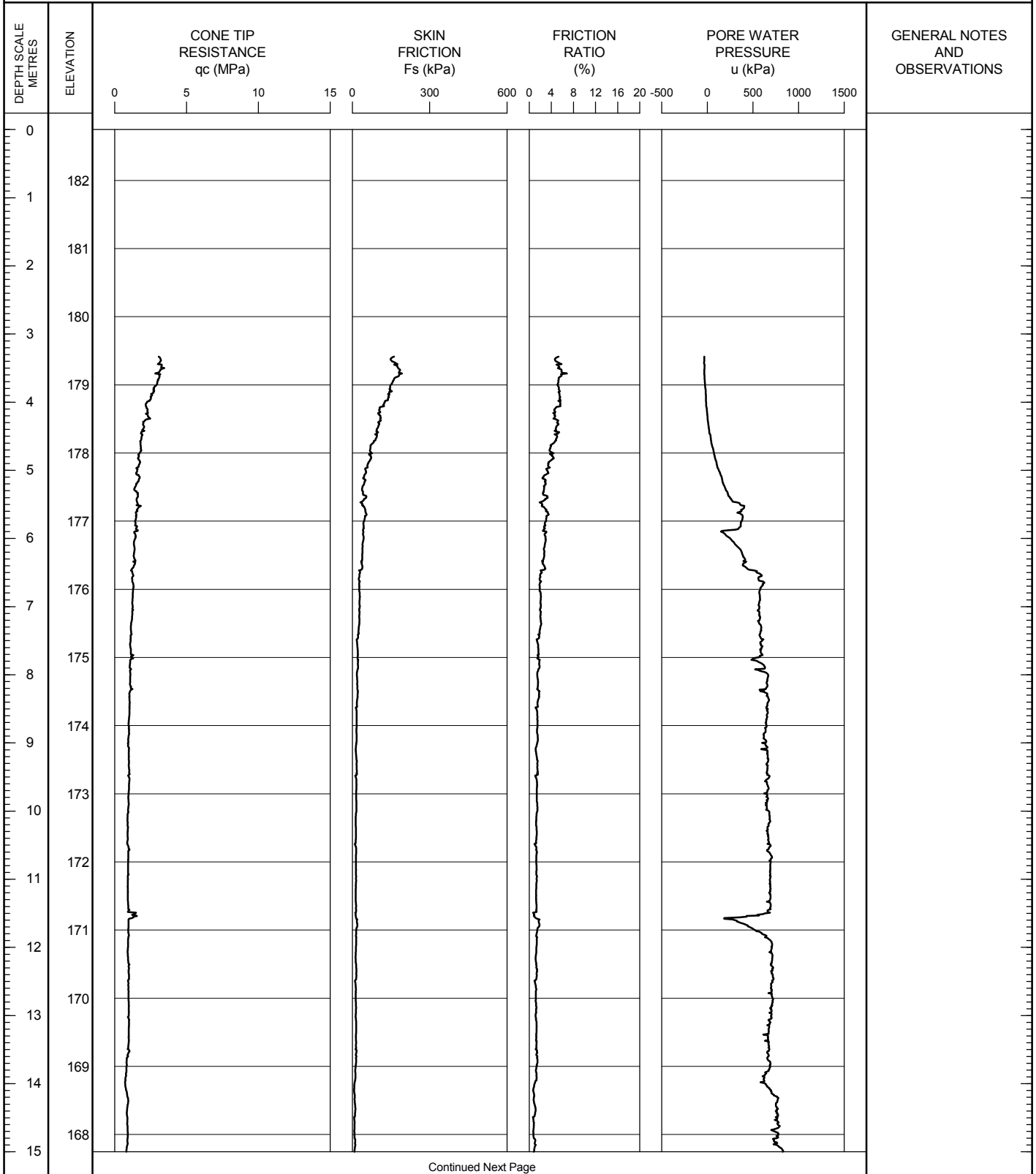
TEST DATE 7/26/2011 - 7/26/2011

SHEET 1 OF 3

LOCATION N4679147.4; E332774.5

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.7 PREDRILL DEPTH: 2.58 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEP CPT LOG CPT B10.GPJ ONTARIO MOT GDT 22/12/11

OPERATOR: TA

CHECKED: DD



# RECORD OF CONE PENETRATION TEST CPT B10-1

METRIC

PROJECT Windsor-Essex Parkway

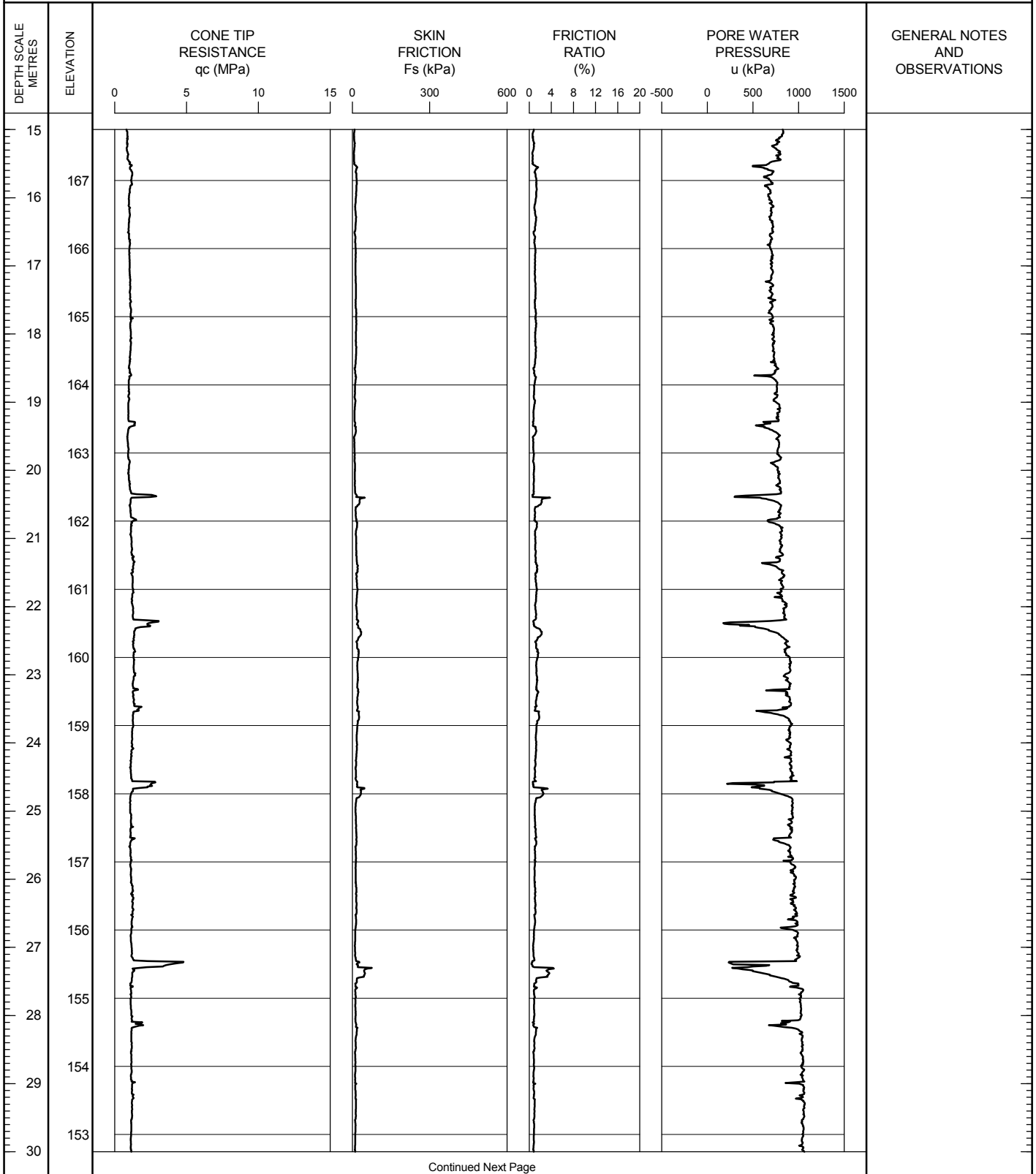
TEST DATE 7/26/2011 - 7/26/2011

SHEET 2 OF 3

LOCATION N4679147.4; E332774.5

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.7 PREDRILL DEPTH: 2.58 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT B10-1

**METRIC**

PROJECT Windsor-Essex Parkway

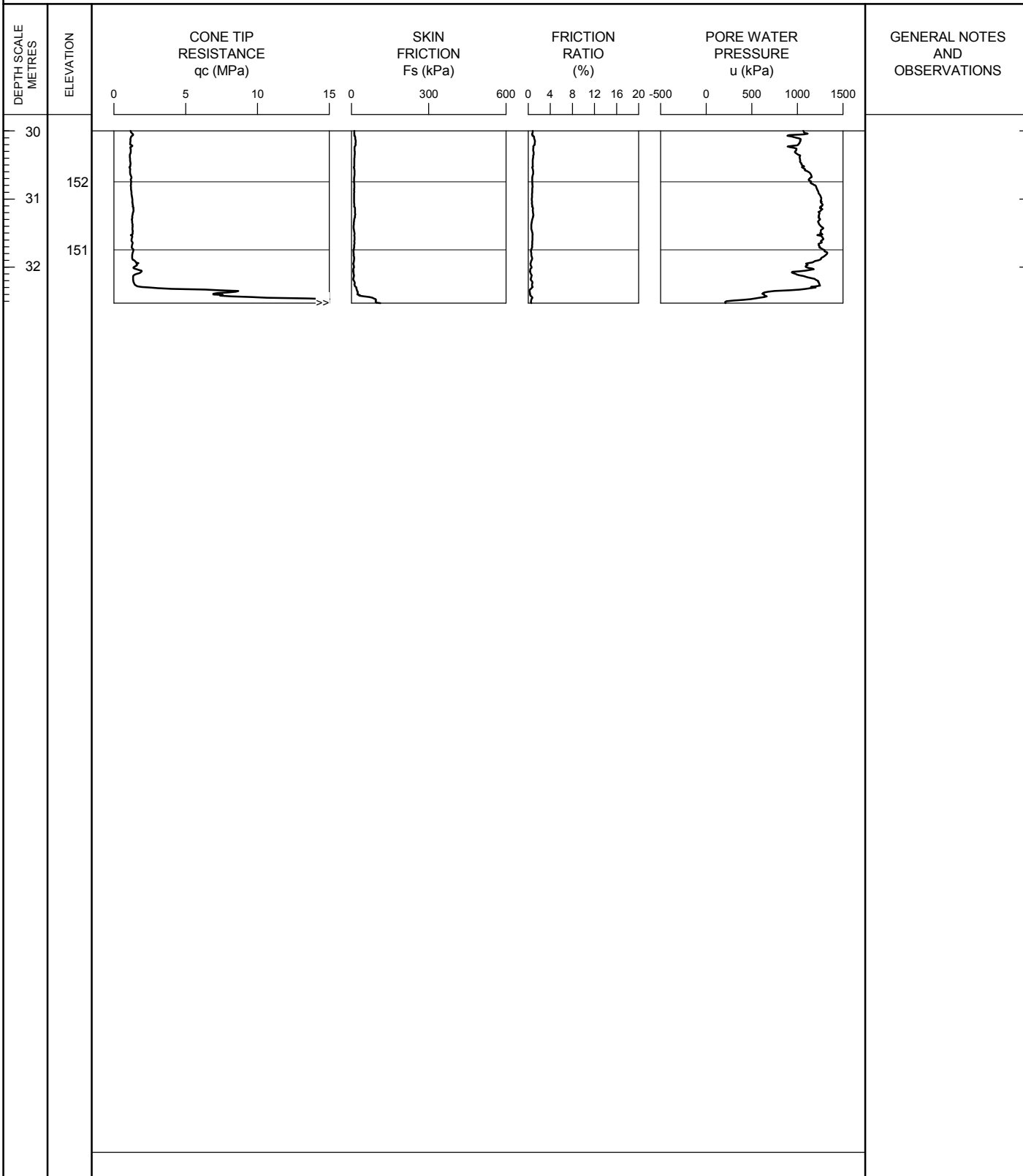
TEST DATE 7/26/2011 - 7/26/2011

SHEET 3 OF 3

LOCATION N4679147.4; E332774.5

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.7    PREDRILL DEPTH: 2.58    CORRECTION FACTOR A: 0.8    CORRECTION FACTOR B: 0



WEP CPT LOG CPT B10.GPJ ONTARIO MOT.GDT 22/12/11

OPERATOR: TA

CHECKED: DD

# RECORD OF BOREHOLE No CPT B10-3

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION 4678983.4N, 332941.1E ORIGINATED BY TA  
DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
DATUM Geodetic DATE May 14, 11 - May 14, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE						WATER CONTENT (%)	
								20	40	60	80						100	10
182.1	Ground Surface						182											
0.0	300mm Black organic clay																	
181.7	TOPSOIL																	
0.4	CLAYEY SILT Some sand, trace gravel Stiff to very stiff, mottled brown-grey to brown to grey		1	SS	16		181											
			2	SS	19		180											
			3	SS	22		179											
			4	SS	22		178											
			5	SS	17													
			6	SS	11													
177.1	END OF SAMPLED BOREHOLE Continue with CPT from 4.6 m to refusal at 31.3 m (El. 177.5 m to El. 150.8 m)						177											
5.0	No groundwater observed						176											
							175											
							174											
							173											
							172											
							171											
							170											
							169											
							168											

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No NIL B10-3

1 OF 1

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679084.3, E332836.8 ORIGINATED BY SD  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE Jun 8, 11 - Jun 8, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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		
182.2	Ground Surface						182												
0.0																			
181.9	Black TOPSOIL																		
0.3	Mottled Brown-Grey CLAYEY SILT Some sand, trace gravel Very stiff		1	SS	17														
	Brown		2	SS	19														
	Hard		3	SS	36														
			4	SS	32														
	Grey Very stiff		5	SS	16														
			6	SS	15														
177.0	END OF SAMPLED BOREHOLE (Continue with Nilcon Vane to refusal)						177												
5.2																			
							176												
							175												
							174												
							173												
							172												
							171												
							170												
							169												
							168												

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

# RECORD OF CONE PENETRATION TEST CPT B10-2

**METRIC**

PROJECT Windsor-Essex Parkway

TEST DATE 5/13/2011 - 5/13/2011

SHEET 1 OF 2

LOCATION N4679056.3; E332920.1

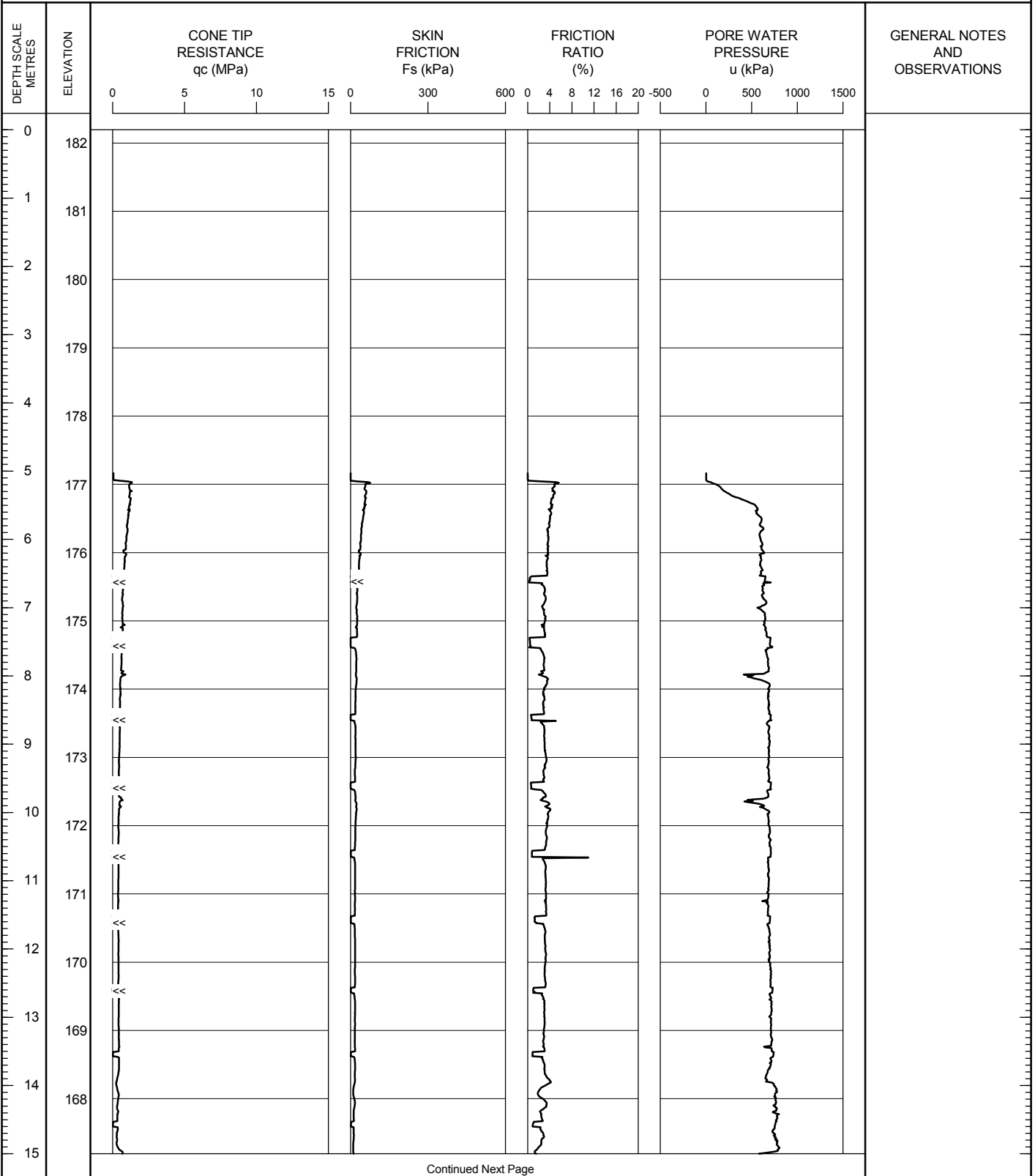
DATUM Geodetic

GROUND SURFACE ELEVATION: 182.2

PREDRILL DEPTH: 4.92

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT B10-2

**METRIC**

PROJECT Windsor-Essex Parkway

TEST DATE 5/13/2011 - 5/13/2011

SHEET 2 OF 2

LOCATION N4679056.3; E332920.1

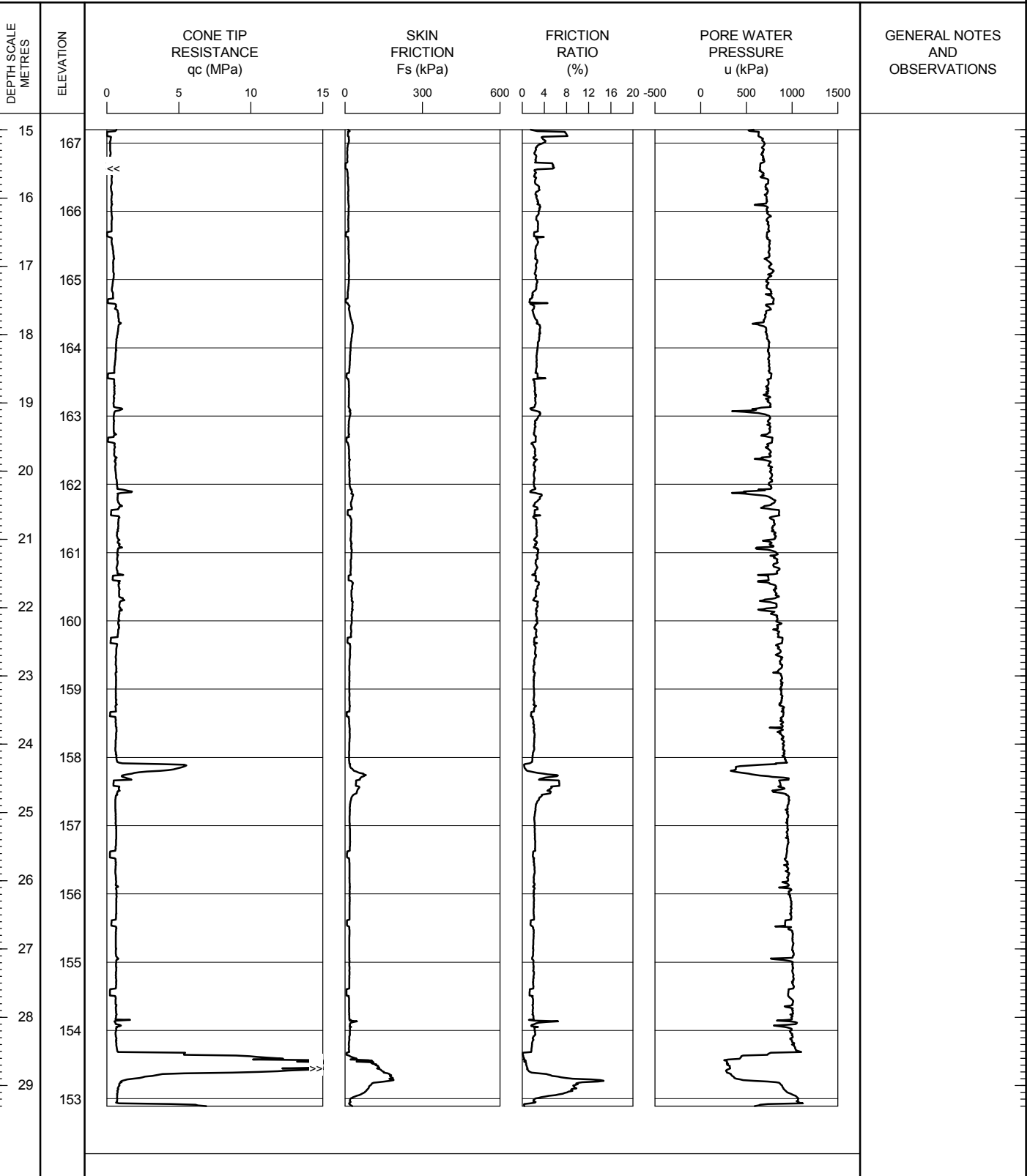
DATUM Geodetic

GROUND SURFACE ELEVATION: 182.2

PREDRILL DEPTH: 4.92

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT B10-3

**METRIC**

PROJECT Windsor-Essex Parkway

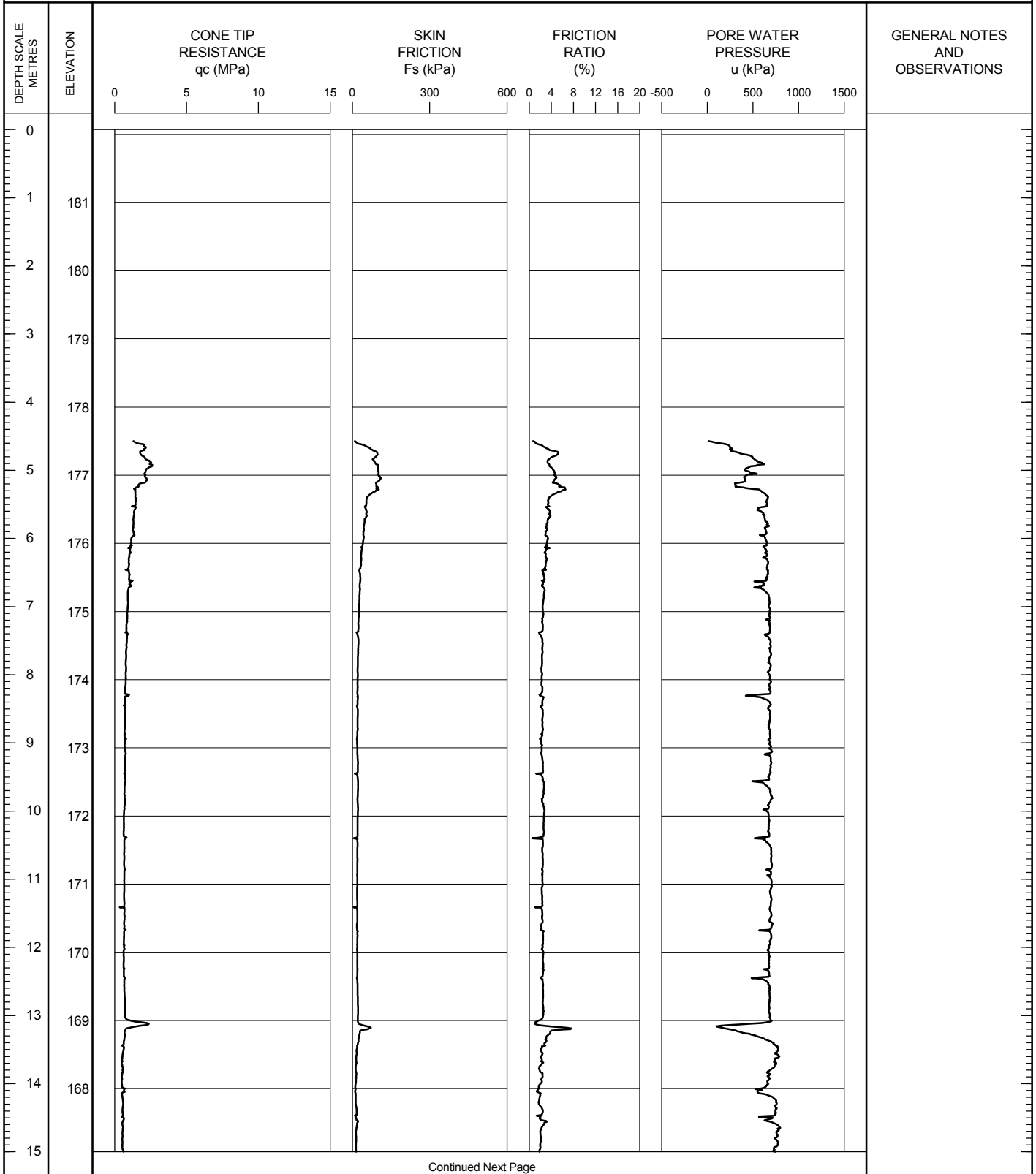
TEST DATE 5/13/2011 - 5/13/2011

SHEET 1 OF 3

LOCATION N4678983.4; E332941.1

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.1 PREDRILL DEPTH: 4.48 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD



# RECORD OF CONE PENETRATION TEST CPT B10-3

**METRIC**

PROJECT Windsor-Essex Parkway

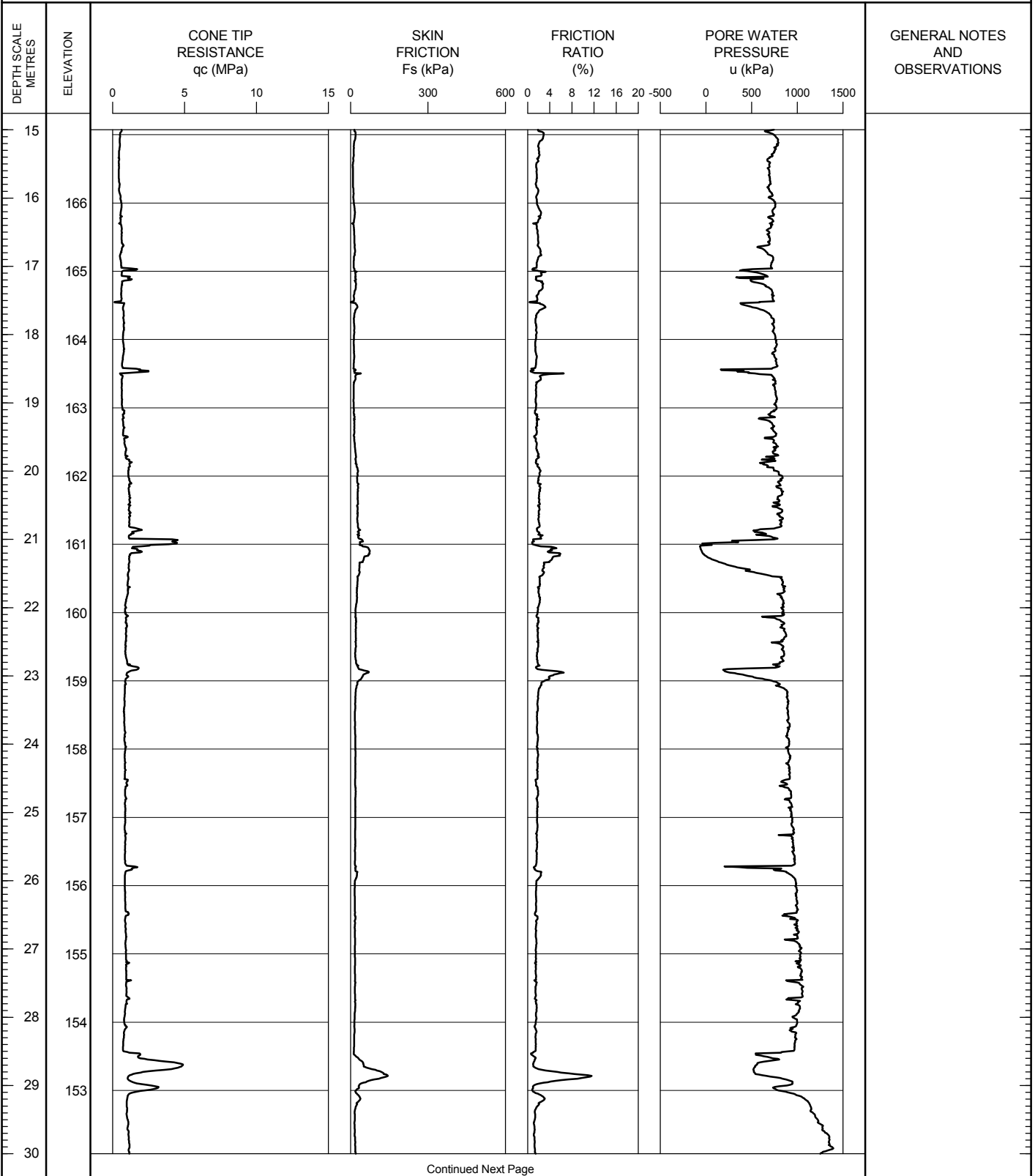
TEST DATE 5/13/2011 - 5/13/2011

SHEET 2 OF 3

LOCATION N4678983.4; E332941.1

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.1 PREDRILL DEPTH: 4.48 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT B10-3

**METRIC**

PROJECT Windsor-Essex Parkway

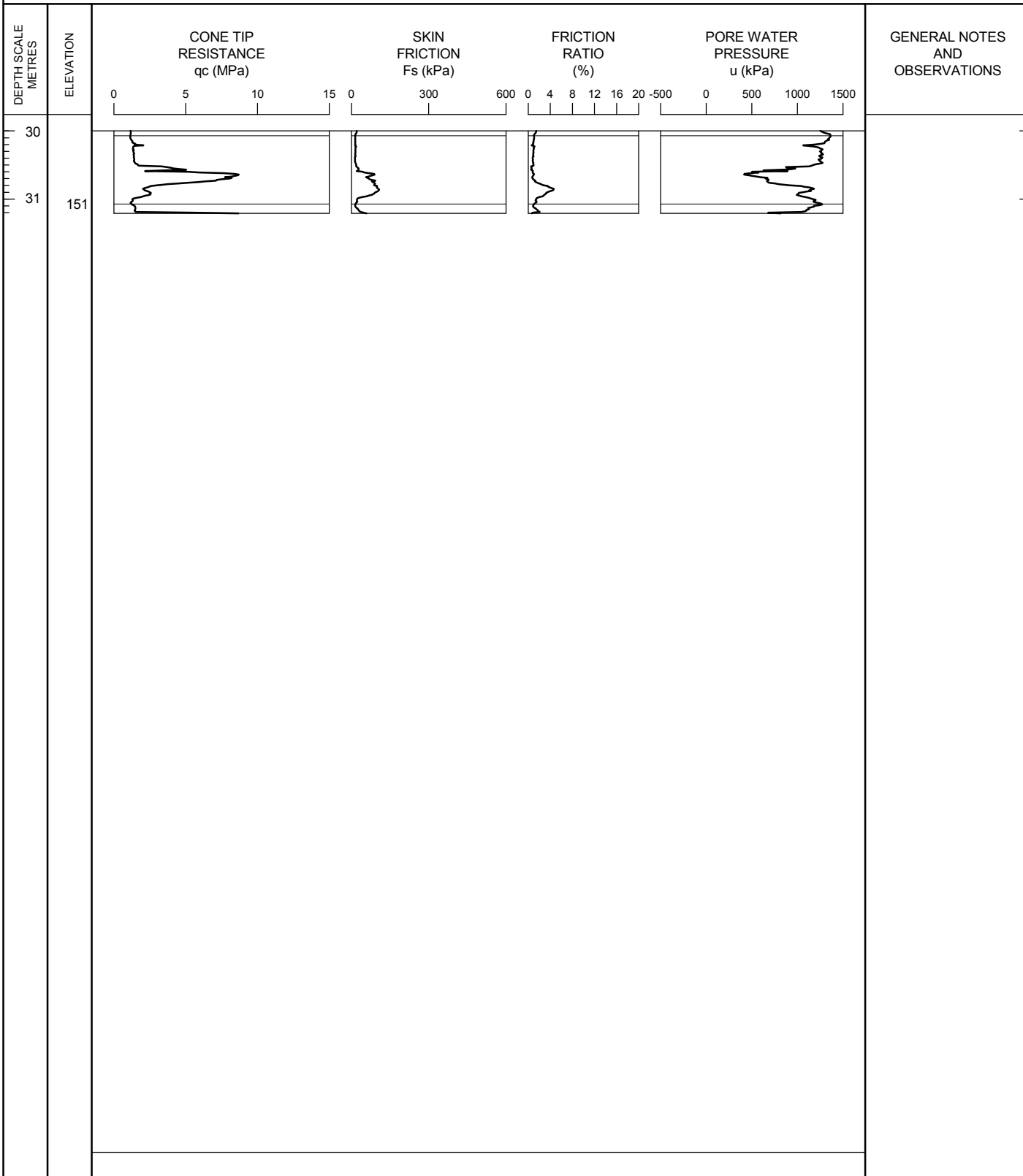
TEST DATE 5/13/2011 - 5/13/2011

SHEET 3 OF 3

LOCATION N4678983.4; E332941.1

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.1    PREDRILL DEPTH: 4.48    CORRECTION FACTOR A: 0.8    CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT 40-RW

**METRIC**

PROJECT Windsor-Essex Parkway

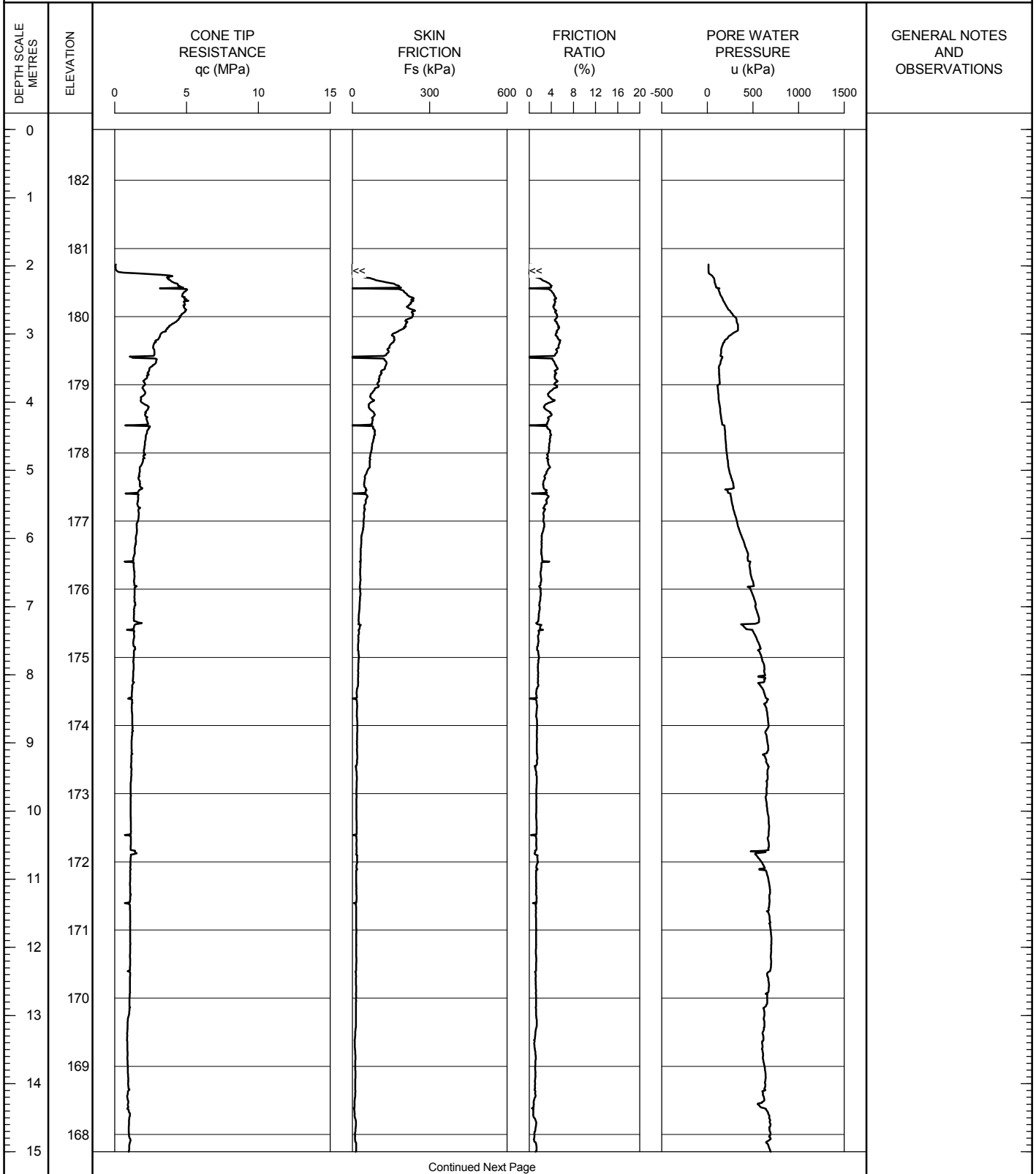
TEST DATE 8/9/2011 - 8/9/2011

SHEET 1 OF 3

LOCATION N4679160.3; E332817.4

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.7 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

WEP CPT LOG CPT-RW/GPJ ONTARIO MOT GDT 06/01/12

# RECORD OF CONE PENETRATION TEST CPT 40-RW

METRIC

PROJECT Windsor-Essex Parkway

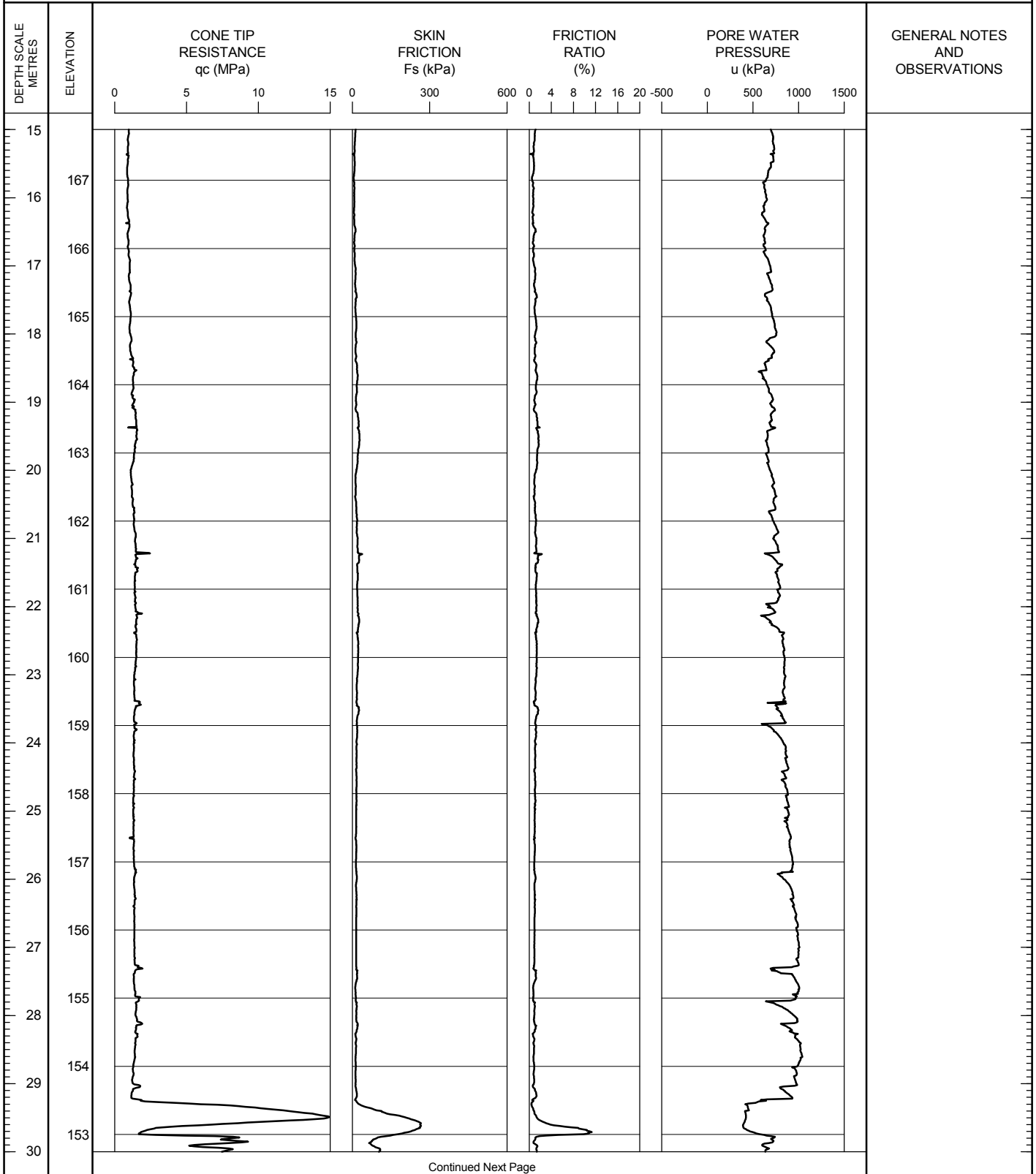
TEST DATE 8/9/2011 - 8/9/2011

SHEET 2 OF 3

LOCATION N4679160.3; E332817.4

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.7 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

## METRIC

SHEET 3 OF 3

DATUM            Geodetic

GROUND SURFACE ELEVATION: 182.7    PREDRILL DEPTH: 1.98    CORRECTION FACTOR A: 0.8    CORRECTION FACTOR B: 0

# RECORD OF CONE PENETRATION TEST CPT 41-RW

**METRIC**

PROJECT Windsor-Essex Parkway

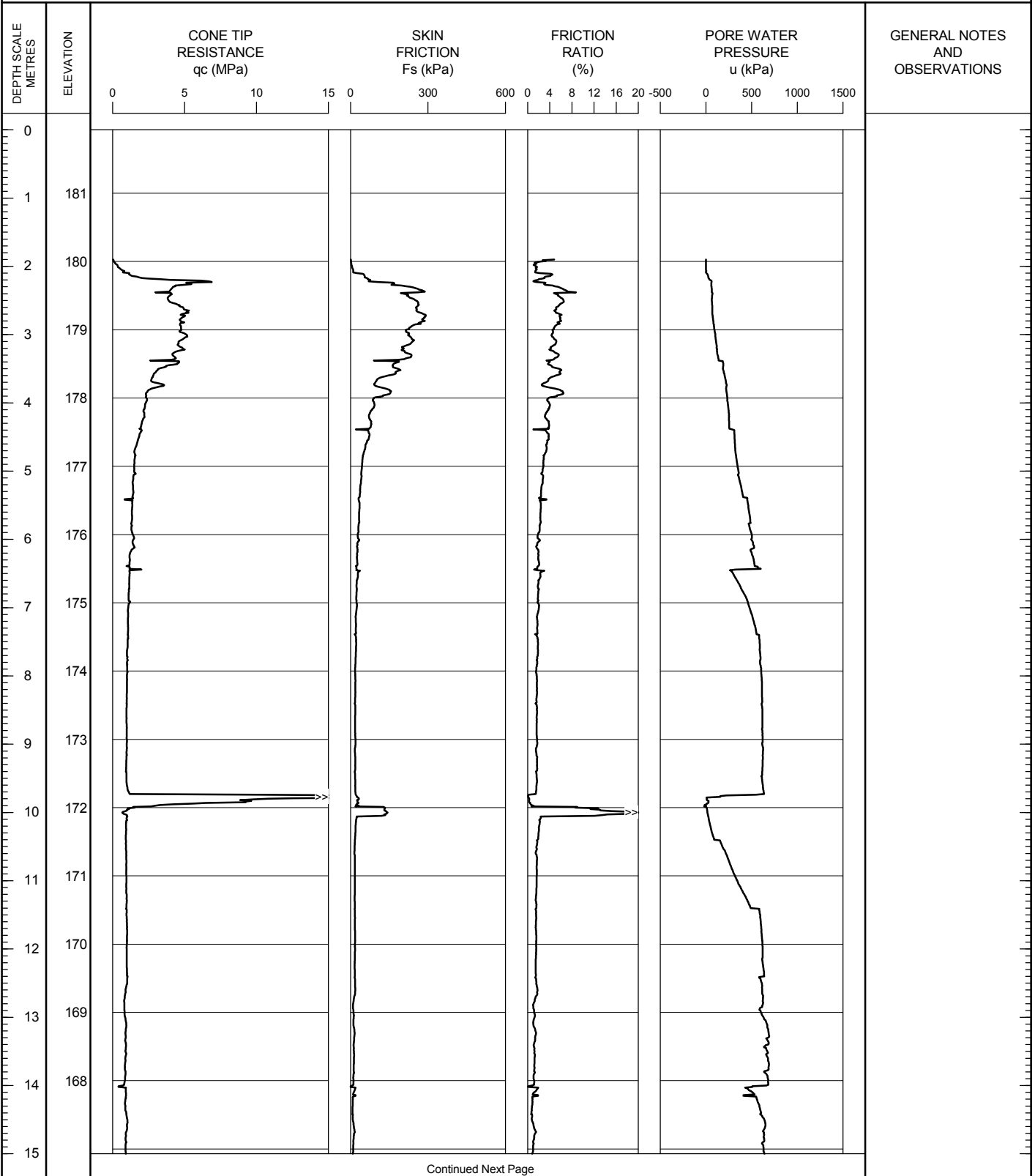
TEST DATE 7/28/2011 - 7/28/2011

SHEET 1 OF 3

LOCATION N4679053.8; E332792.8

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.9 PREDRILL DEPTH: 1.59 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEP CPT LOG CPT-RW/GPJ ONTARIO MOT GDT 06/01/12

OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT 41-RW

**METRIC**

PROJECT Windsor-Essex Parkway

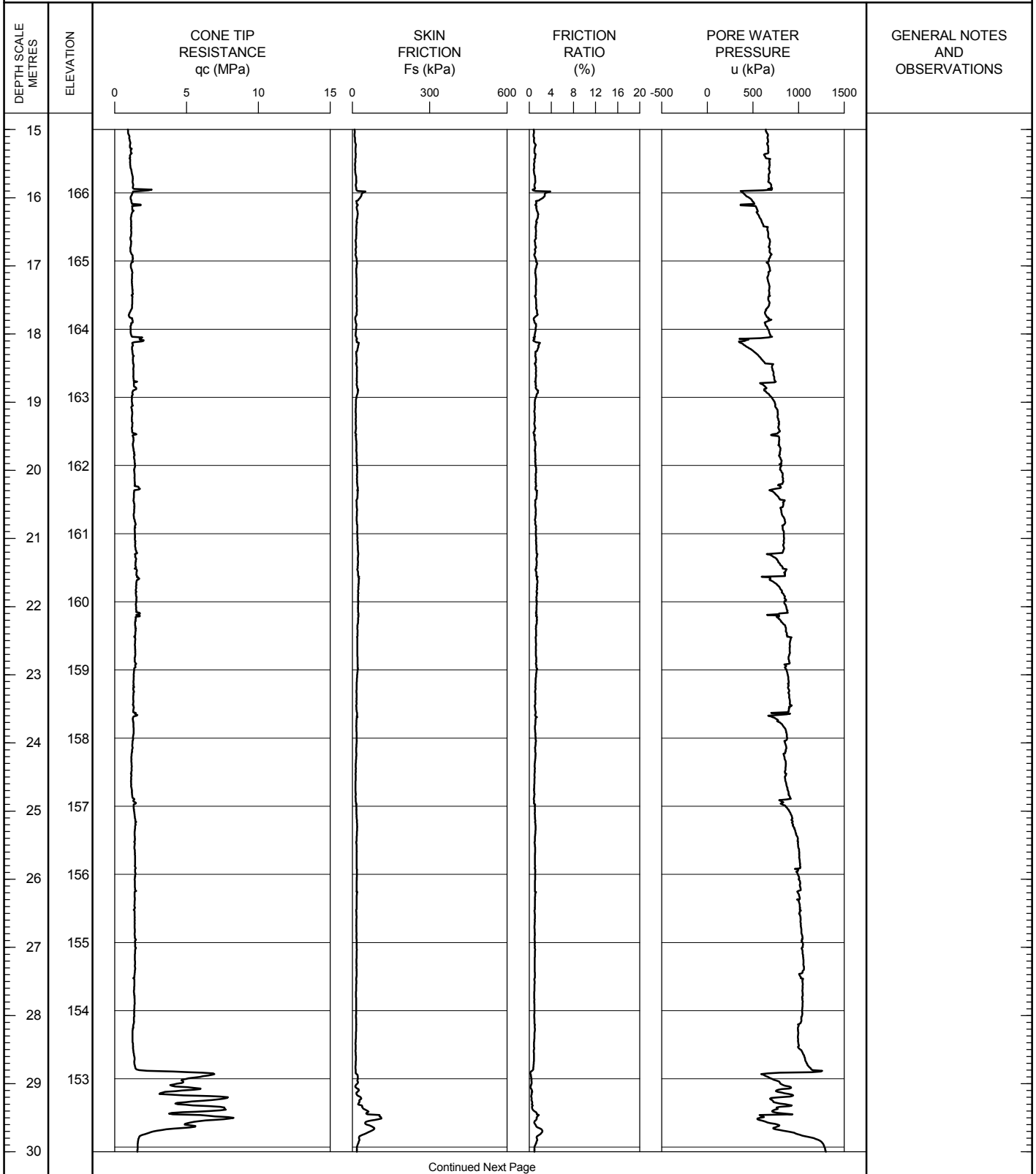
TEST DATE 7/28/2011 - 7/28/2011

SHEET 2 OF 3

LOCATION N4679053.8; E332792.8

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.9    PREDRILL DEPTH: 1.59    CORRECTION FACTOR A: 0.8    CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD



# RECORD OF CONE PENETRATION TEST CPT 41-RW

**METRIC**

PROJECT Windsor-Essex Parkway

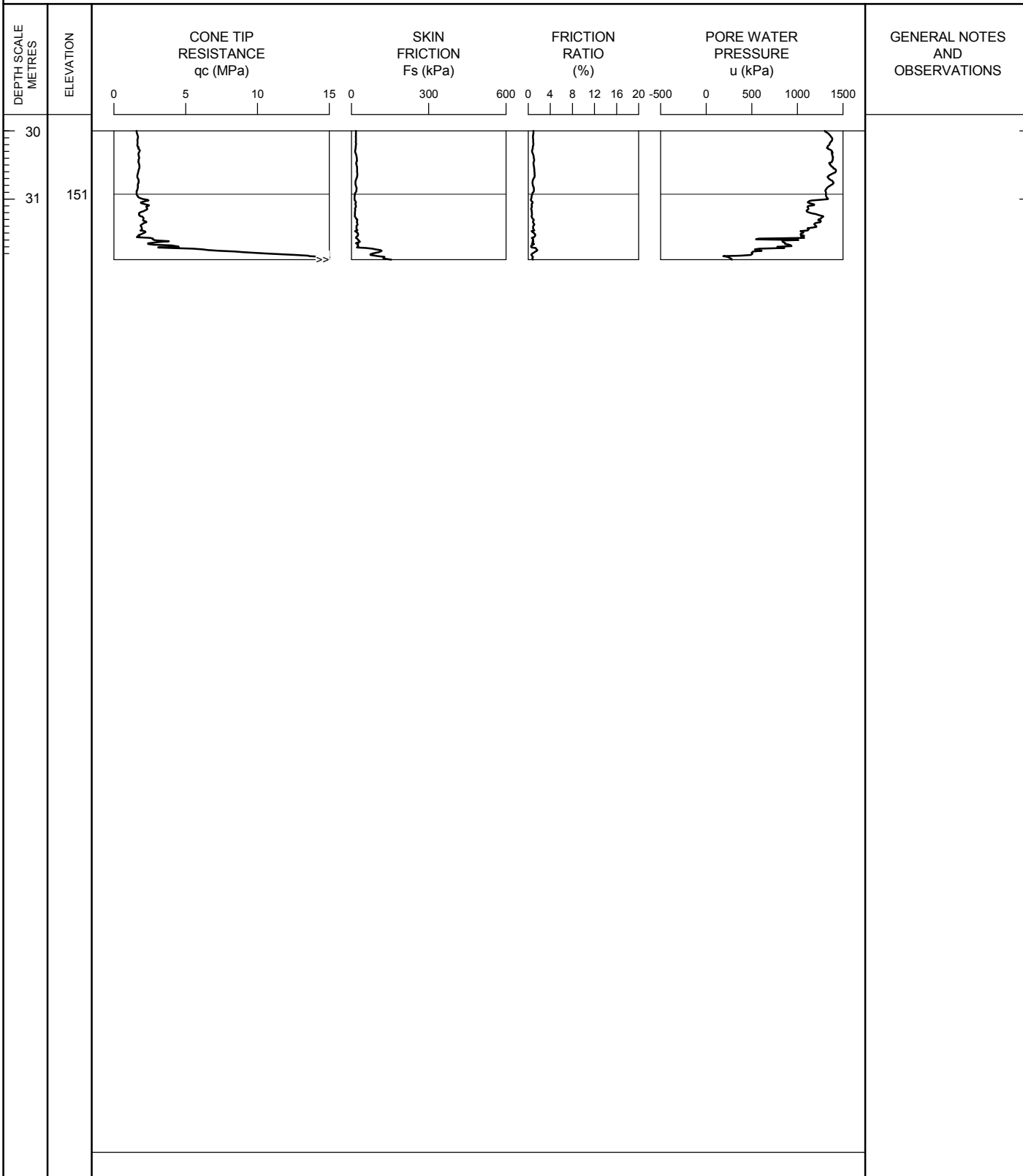
TEST DATE 7/28/2011 - 7/28/2011

SHEET 3 OF 3

LOCATION N4679053.8; E332792.8

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.9    PREDRILL DEPTH: 1.59    CORRECTION FACTOR A: 0.8    CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT 42-RW

METRIC

PROJECT Windsor-Essex Parkway

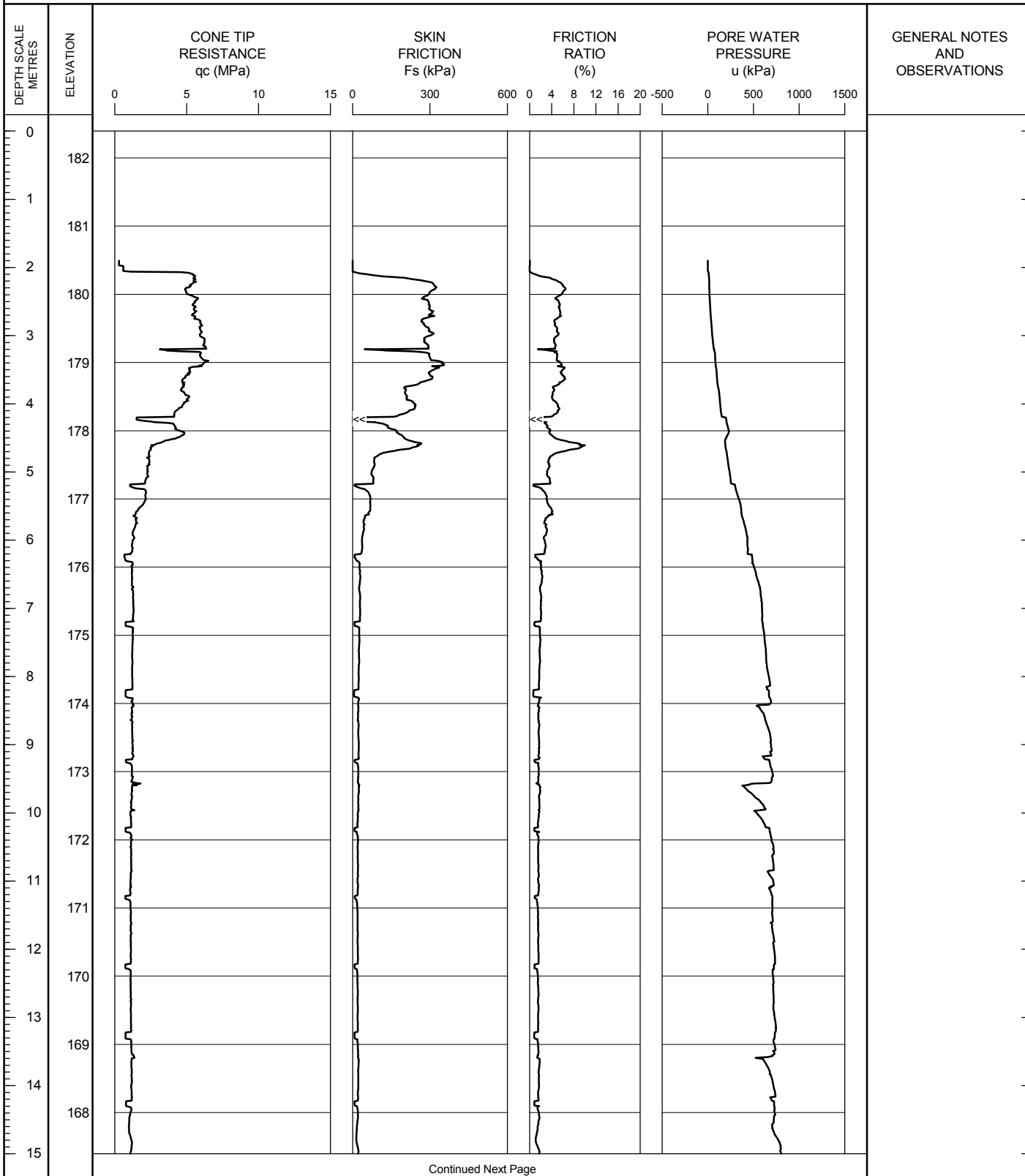
TEST DATE 7/3/2011 - 7/3/2011

SHEET 1 OF 2

LOCATION N4678892.0; E333107.4

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.4 PREDRILL DEPTH: 1.97 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEF CPT LOG CPT-RW.GPJ ONTARIO MOT.GDT 06/01/12

OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT 42-RW

**METRIC**

PROJECT Windsor-Essex Parkway

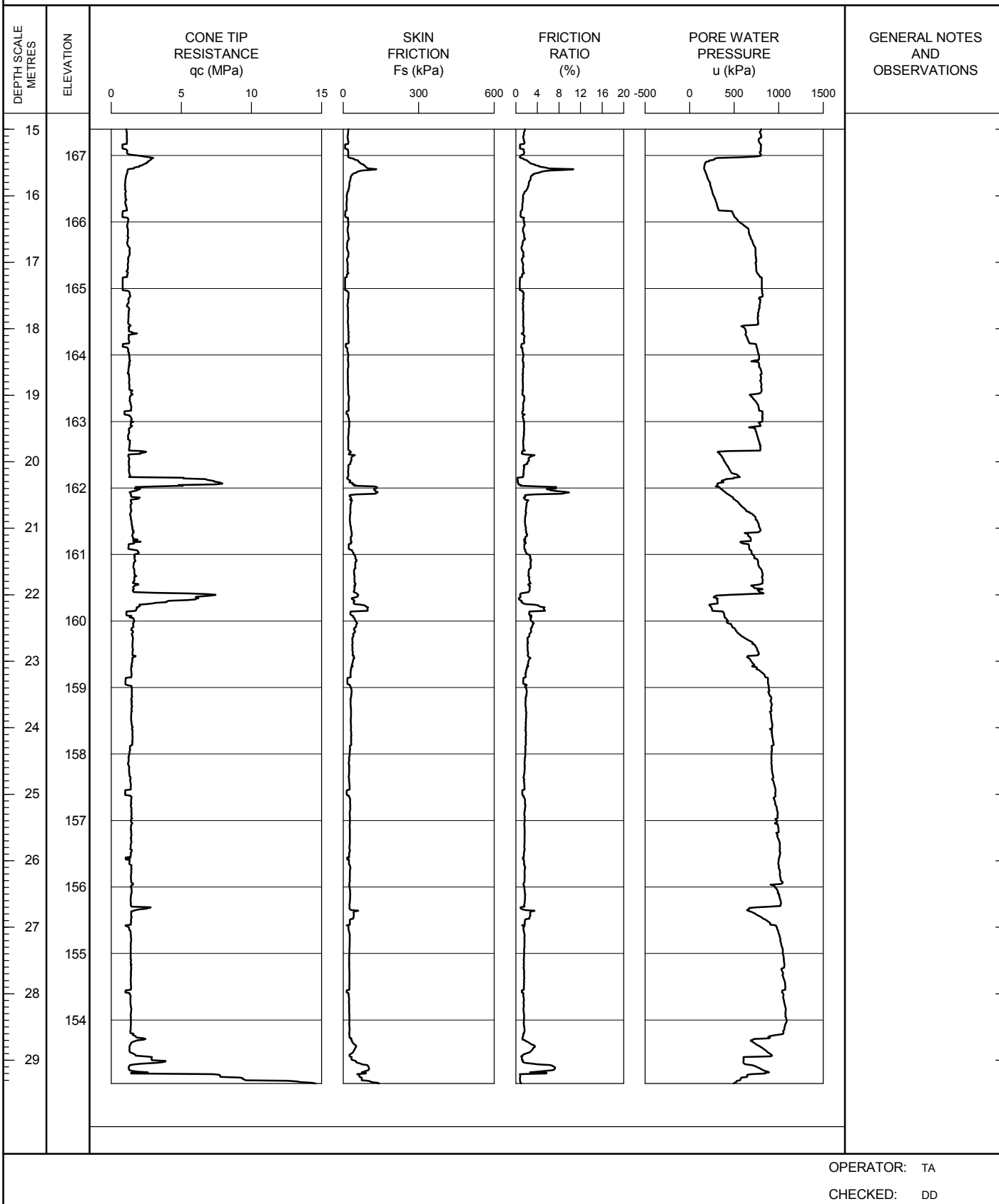
TEST DATE 7/3/2011 - 7/3/2011

SHEET 2 OF 2

LOCATION N4678892.0; E333107.4

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.4 PREDRILL DEPTH: 1.97 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEF CPT LOG CPT-RW/GPJ ONTARIO MOT GDT 06/01/12

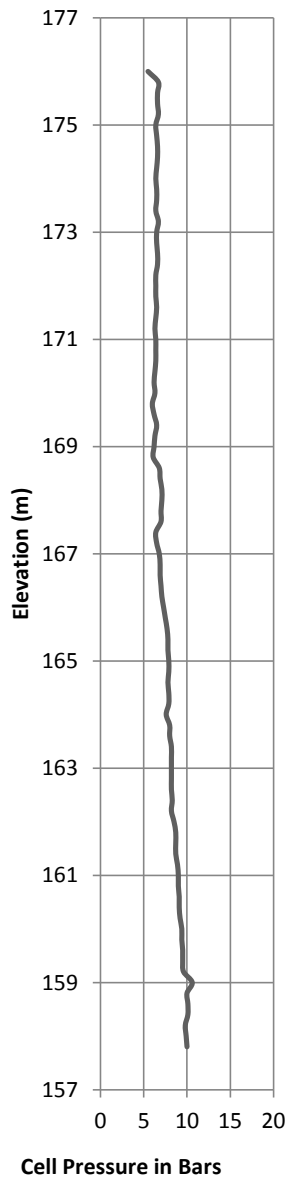
# RECORD OF DILATOMETER TEST DMT05-RW

Project : Windsor-Essex Parkway  
Location: N 4679146.3; E 332752.1  
Ground Surface Elevation : 183.8

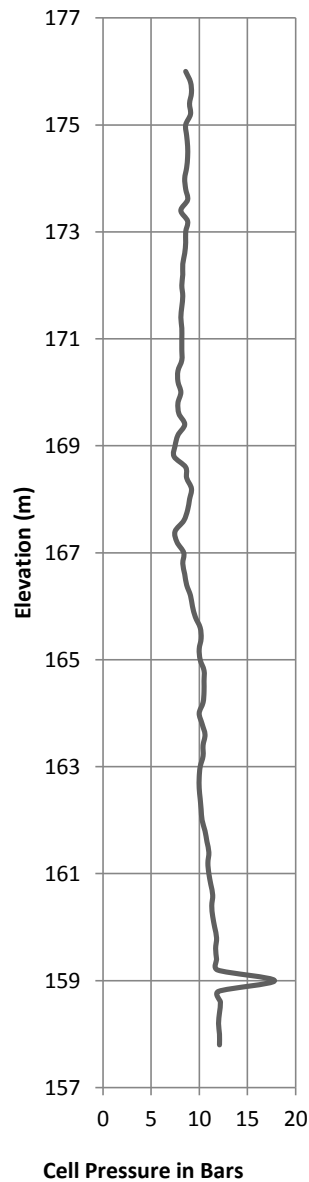
Test Date: 5/31/2011  
Predrill Depth : 7.6 m  
Delta A: 0.20 Bar

Sheet 1 of 1  
Datum Geodetic  
Delta B: 0.28 Bar

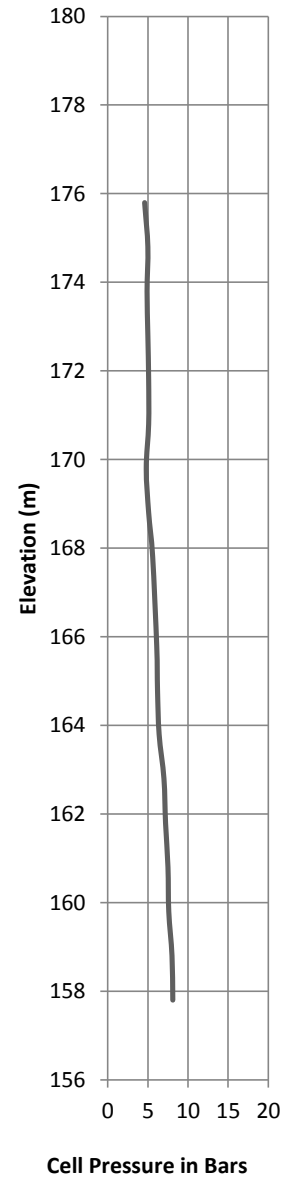
**Reading A**



**Reading B**



**Reading C**



Operator: LC



Checked: DD

## **Appendix B      Borehole Logs from Previous Investigations**

**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018

**Date:** March/2012  
**Rev:** 0  
**Page No.:** Appendix B

PROJECT <u>09-1132-0080</u>		<b>RECORD OF BOREHOLE No CPT-317</b>		1 OF 1		<b>METRIC</b>	
W.P. _____		LOCATION <u>N 4679041.7 ; E 332972.4</u>		ORIGINATED BY <u>TA</u>			
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>DMB</u>			
DATUM <u>GEODETIC</u>		DATE <u>January 26, 2010</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		w <sub>p</sub>	w	w <sub>L</sub>		WATER CONTENT (%)					
								○ UNCONFINED      + FIELD VANE	● QUICK TRIAXIAL      × LAB VANE										
182.64	GROUND SURFACE							20   40   60   80   100											
0.00	TOPSOIL, clayey Black																		
182.21	CLAYEY SILT, some sand, trace gravel, with occasional fissures, silt partings and seams Very stiff to hard Brown																		
0.43			1	SS	21														
			2	SS	36														
			3	SS	33														
			4	SS	33														
178.98	END OF BOREHOLE																		
3.66	Borehole dry during drilling on January 26, 2010.																		

LDN\_MTO\_06 09-1132-0080.GPJ LDN\_MTO.GDT 11/03/10

PROJECT <u>09-1132-0080</u>		<b>RECORD OF BOREHOLE No CPT-320</b>		1 OF 1		<b>METRIC</b>	
W.P. _____		LOCATION <u>N 4679155.5 ; E 332737.0</u>		ORIGINATED BY <u>TA</u>			
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>DMB</u>			
DATUM <u>GEODETIC</u>		DATE <u>December 21, 2009</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>p</sub>	W	W <sub>L</sub>		GR	SA	SI	CL	
								20	40	60	80	100									
183.50	GROUND SURFACE																				
0.00	TOPSOIL, sandy, some rootlets Black																				
182.89																					
0.61	SAND, fine, some silt Compact to dense Brown		1	SS	18									o							
181.62			2	SS	31																
1.88	CLAYEY SILT, some sand, trace gravel Very stiff to hard Grey																				
180.60			3	SS	18									o							
2.90	END OF BOREHOLE																				
	Groundwater encountered at about elev. 182.0m during drilling on December 21, 2009.																				

PROJECT: 09-1132-0080

# RECORD OF CONE PENETRATION TEST CPT-317

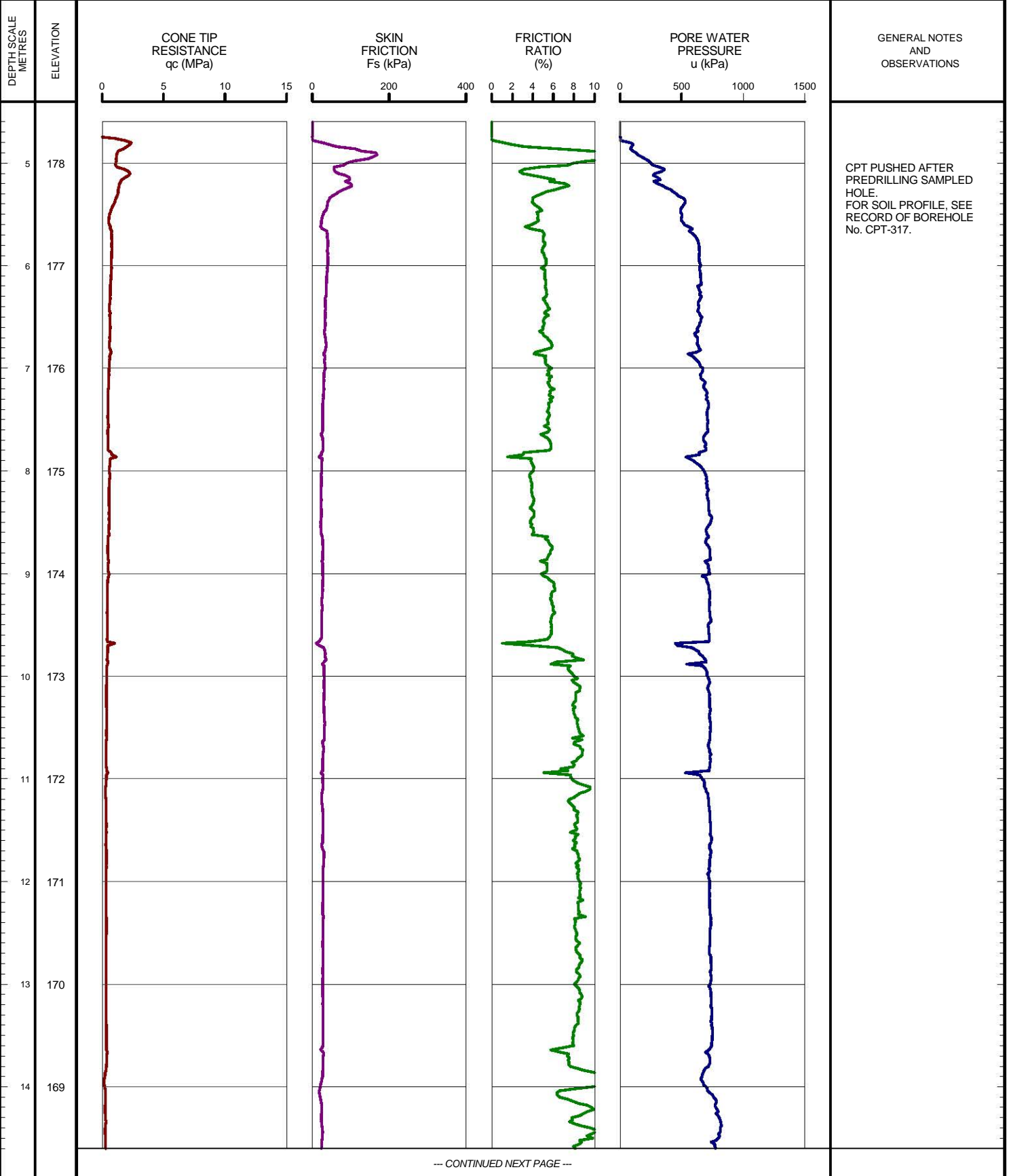
SHEET 1 OF 2

LOCATION: N 4679041.7 ;E 332972.4

TEST DATE: January 26, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 182.64m PREDRILL DEPTH: 4.60m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LON\_CPT\_01 09-1132-0080-CPT.GPJ GLDR\_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



OPERATOR: TA

CHECKED:



PROJECT: 09-1132-0080

# RECORD OF CONE PENETRATION TEST CPT-317

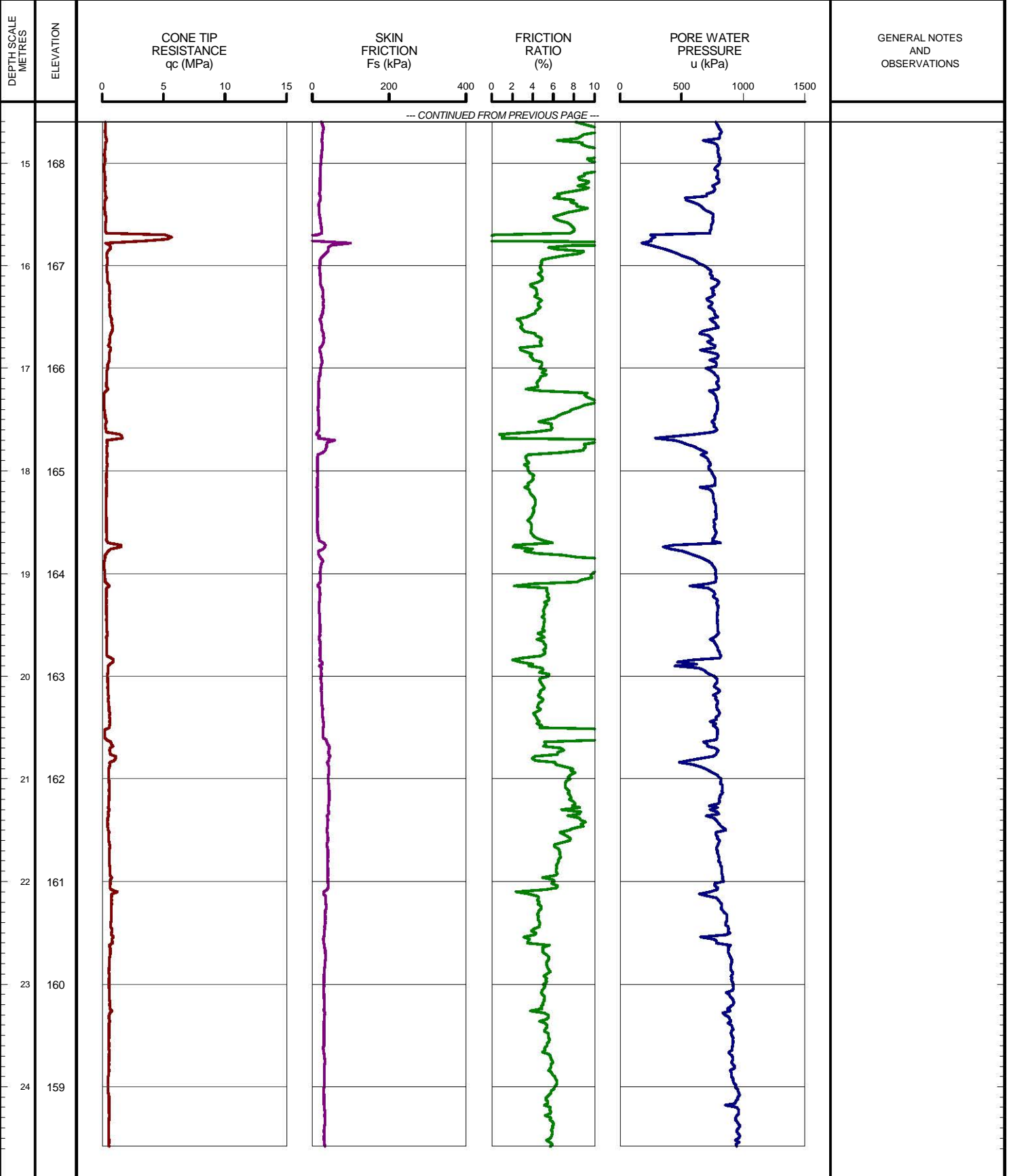
SHEET 2 OF 2

LOCATION: N 4679041.7 ;E 332972.4

TEST DATE: January 26, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 182.64m PREDRILL DEPTH: 4.60m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LON\_CPT\_01 09-1132-0080-CPT.GPJ GLDR\_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

**RECORD OF CONE PENETRATION TEST CPT-320**

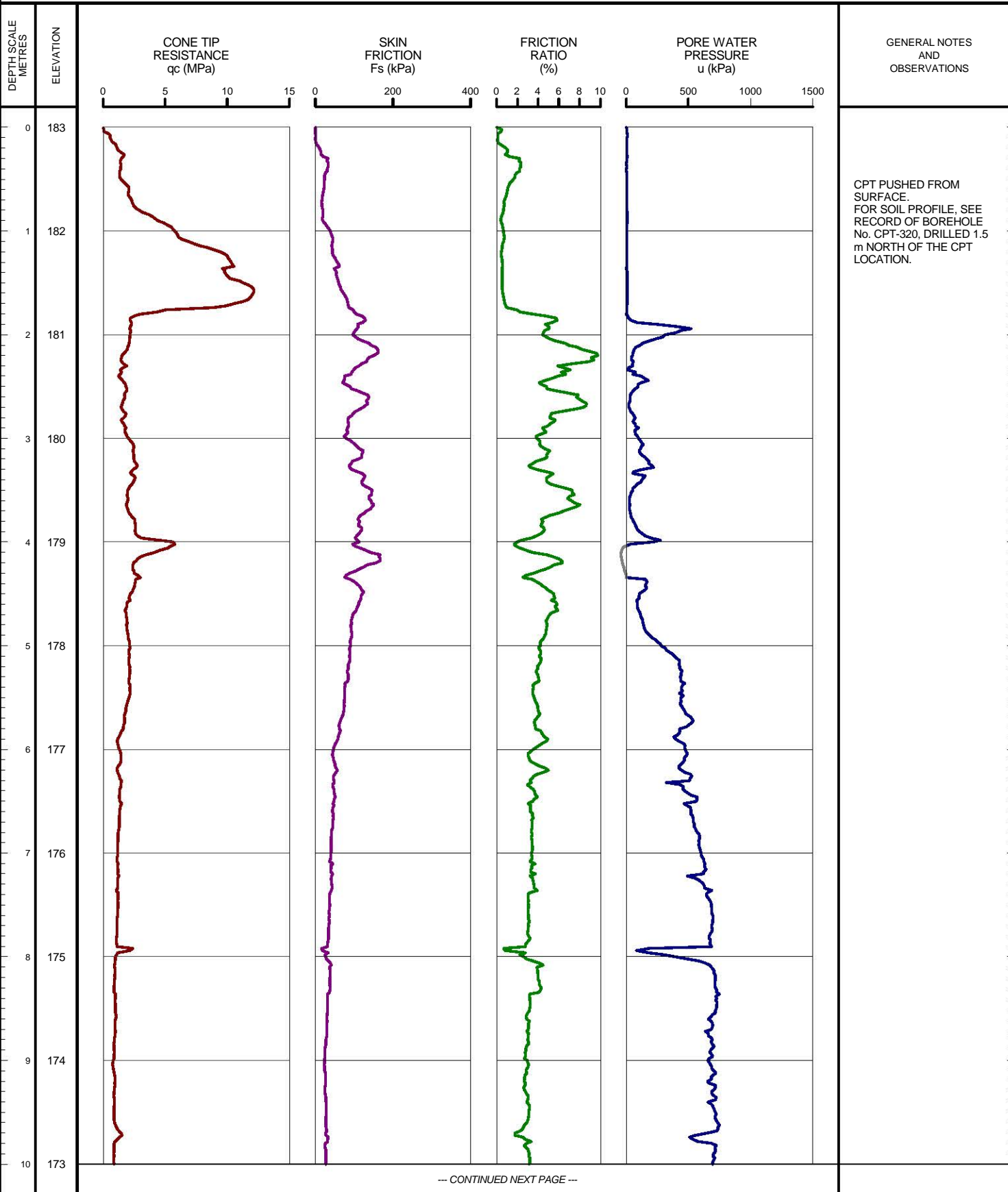
SHEET 1 OF 3

LOCATION: N 4679155.5 ;E 332737.0

TEST DATE: December 22, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 183.50m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LON\_CPT\_01 09-1132-0080-CPT.GPJ GLDR\_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

# RECORD OF CONE PENETRATION TEST CPT-320

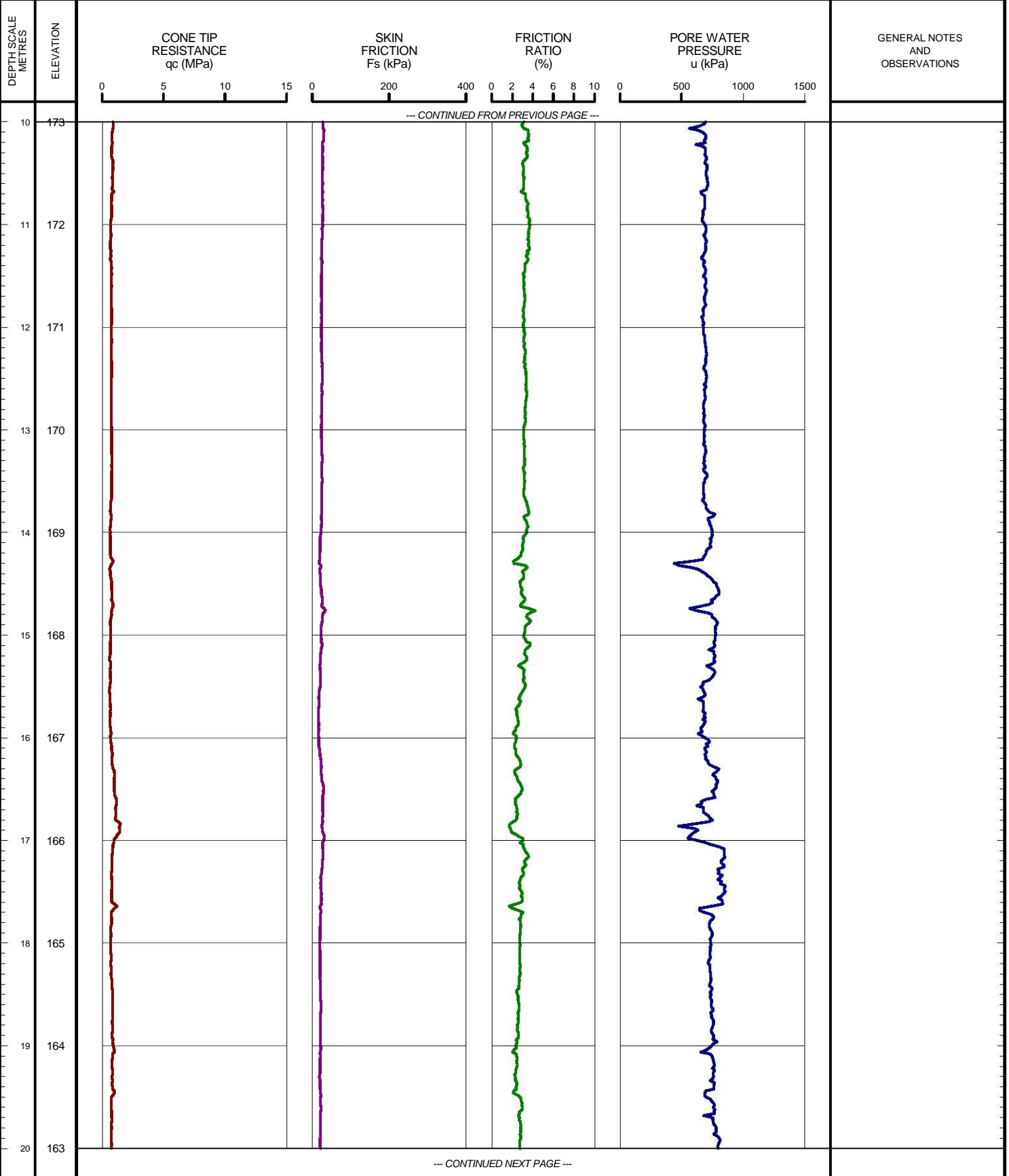
SHEET 2 OF 3

LOCATION: N 4679155.5 ;E 332737.0

TEST DATE: December 22, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 183.50m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LON\_CPT\_01 09-1132-0080-CPT.GPJ GLDR\_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

# RECORD OF CONE PENETRATION TEST CPT-320

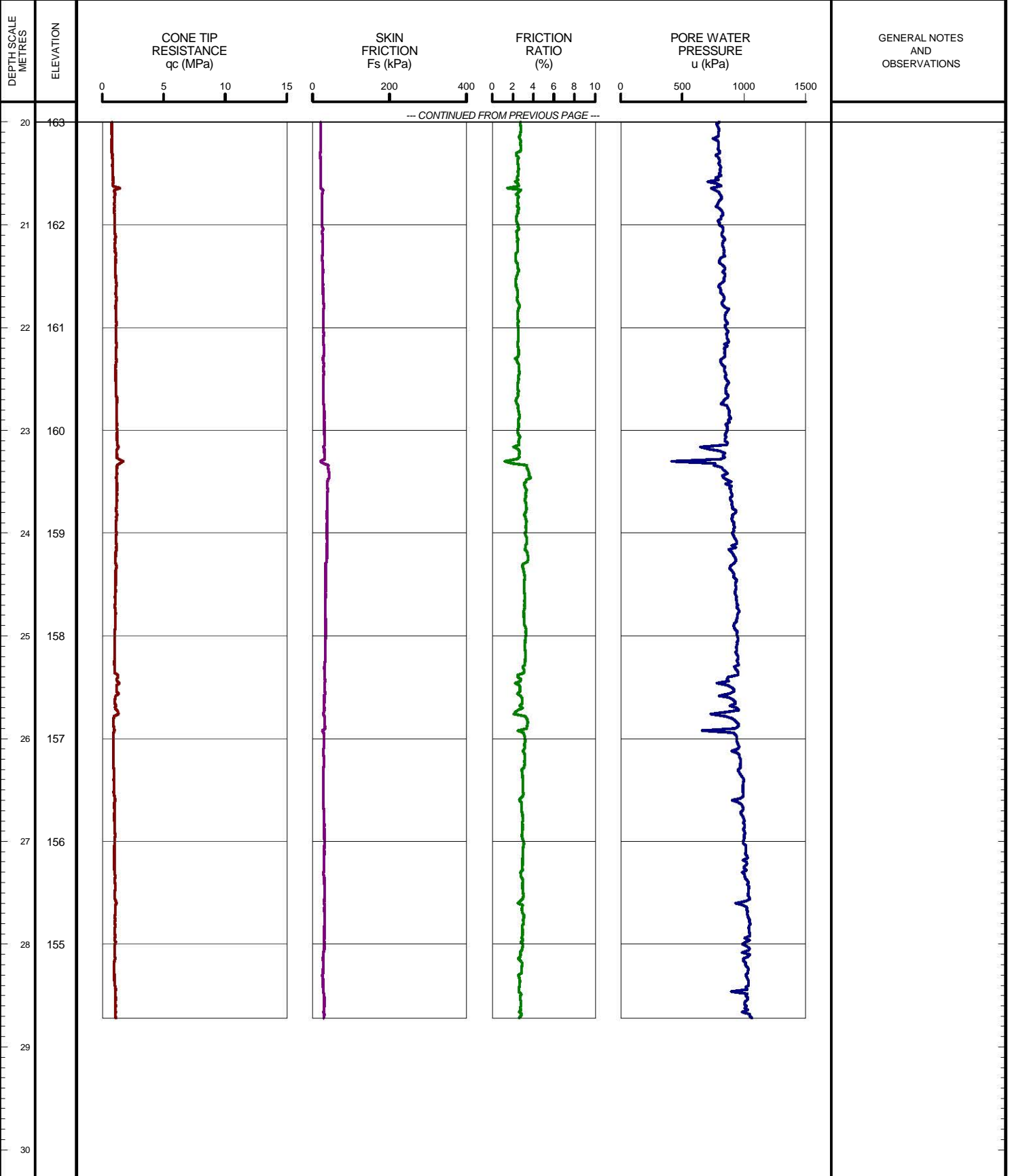
SHEET 3 OF 3

LOCATION: N 4679155.5 ;E 332737.0

TEST DATE: December 22, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 183.50m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LON\_CPT\_01 09-1132-0080-CPT.GPJ GLDR\_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



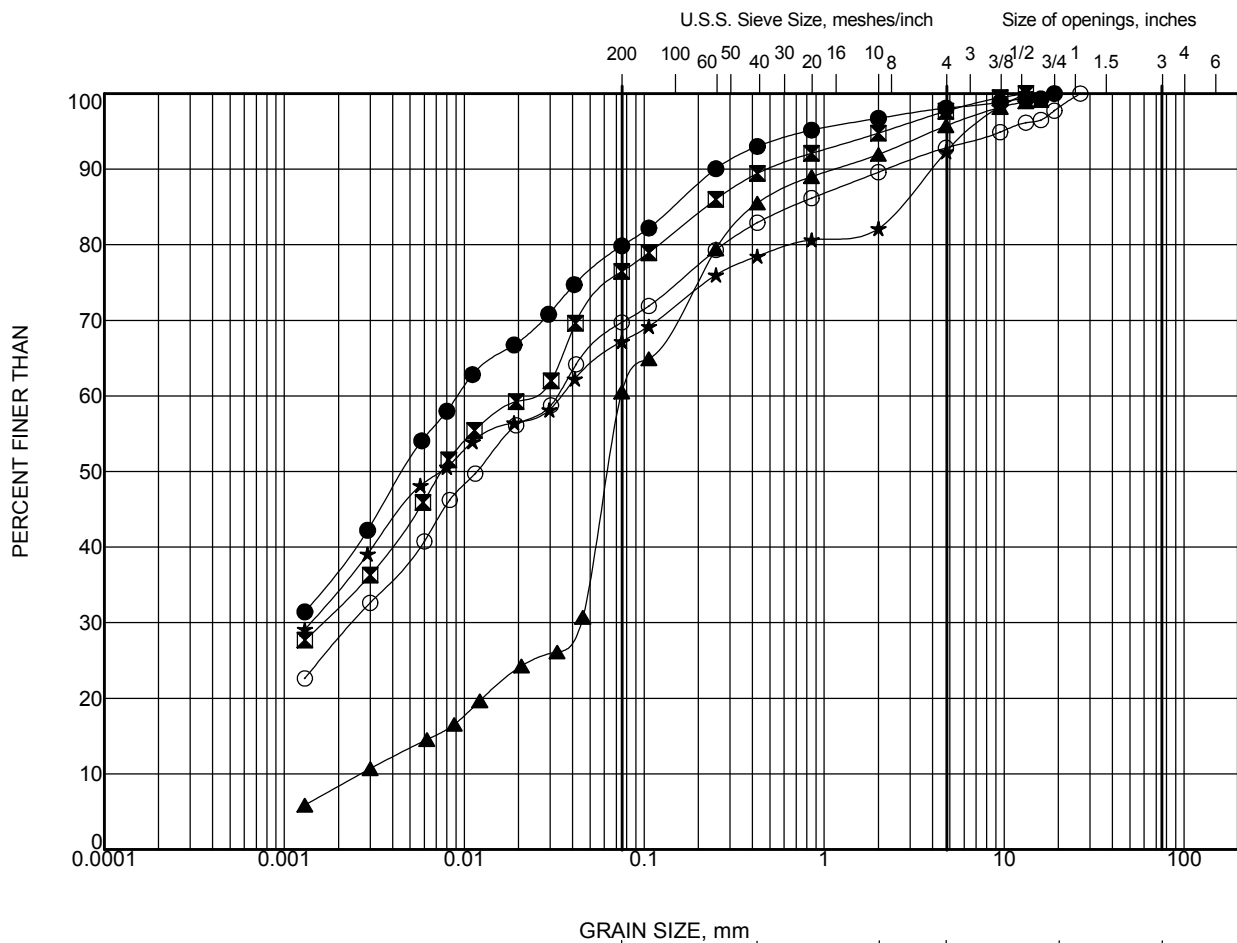
OPERATOR: TA

CHECKED:

## **Appendix C      Laboratory Test Results**

**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018


**Date:** March/2012  
**Rev:** 0  
**Page No.:** Appendix C

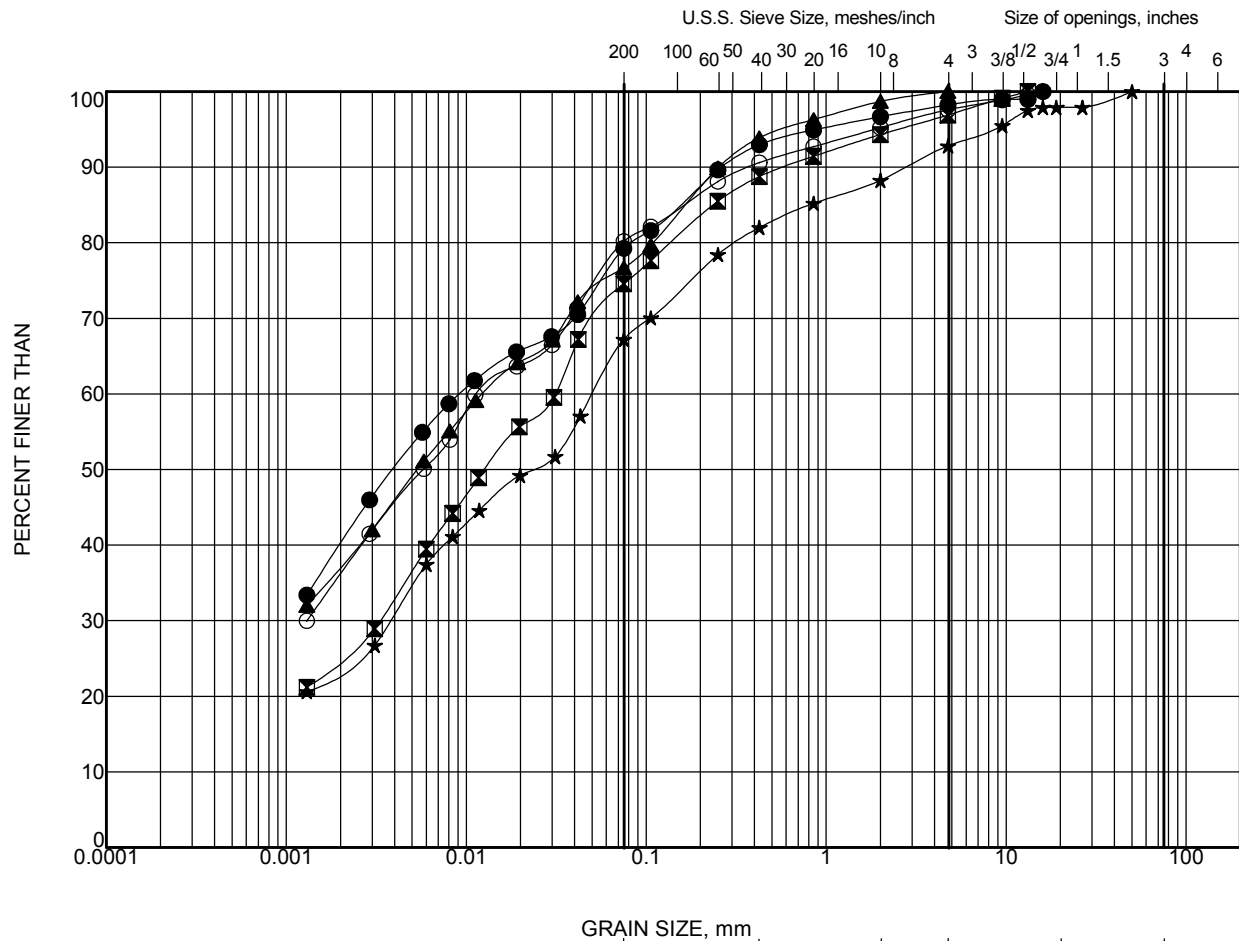


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B10-1	12	12.2
⊠	B10-1	20	24.4
▲	B10-2	8	6.1
★	B10-2	9	7.6
○	B10-2	19	22.9



PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE		<b>GRAIN SIZE DISTRIBUTION</b> <b>Silty Clay / Clayey Silt</b>		
 		PROJECT No. SW8801.1004.101	FILE No.	
DRAWN		mso	10 Nov. 2011	SCALE n/a
CHECK				REV.
		FIGURE C.1		

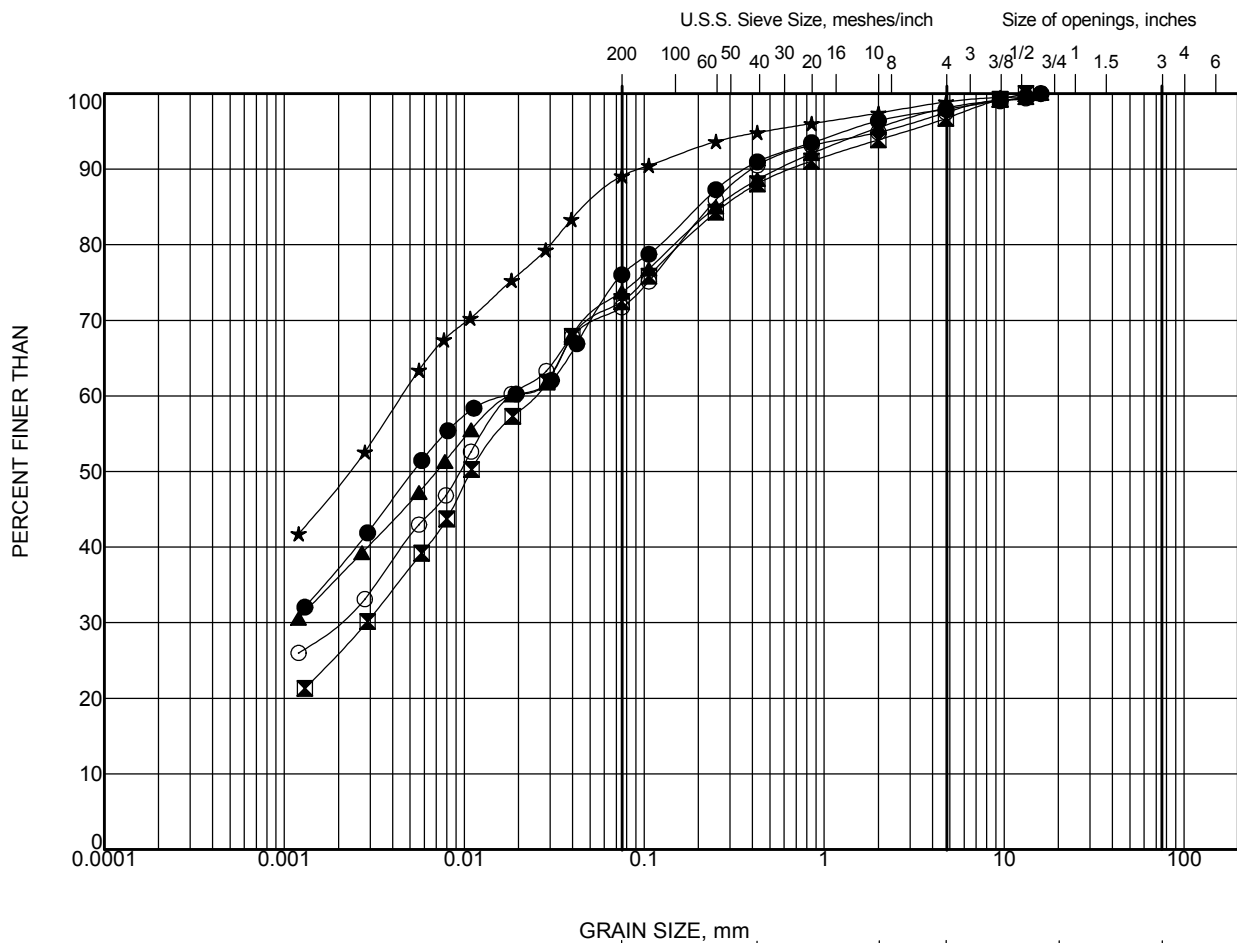


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B10-3	8	6.1
■	B10-3	22	27.4
▲	B10-4	8	6.1
★	B10-4	16	18.3
○	B10-4	22	27.4

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE		<b>GRAIN SIZE DISTRIBUTION</b> <b>Silty Clay / Clayey Silt</b>		
PROJECT No. SW8801.1004.101		FILE No.		
DRAWN mso		10 Nov. 2011		SCALE n/a
CHECK				REV.
  		FIGURE C.2		



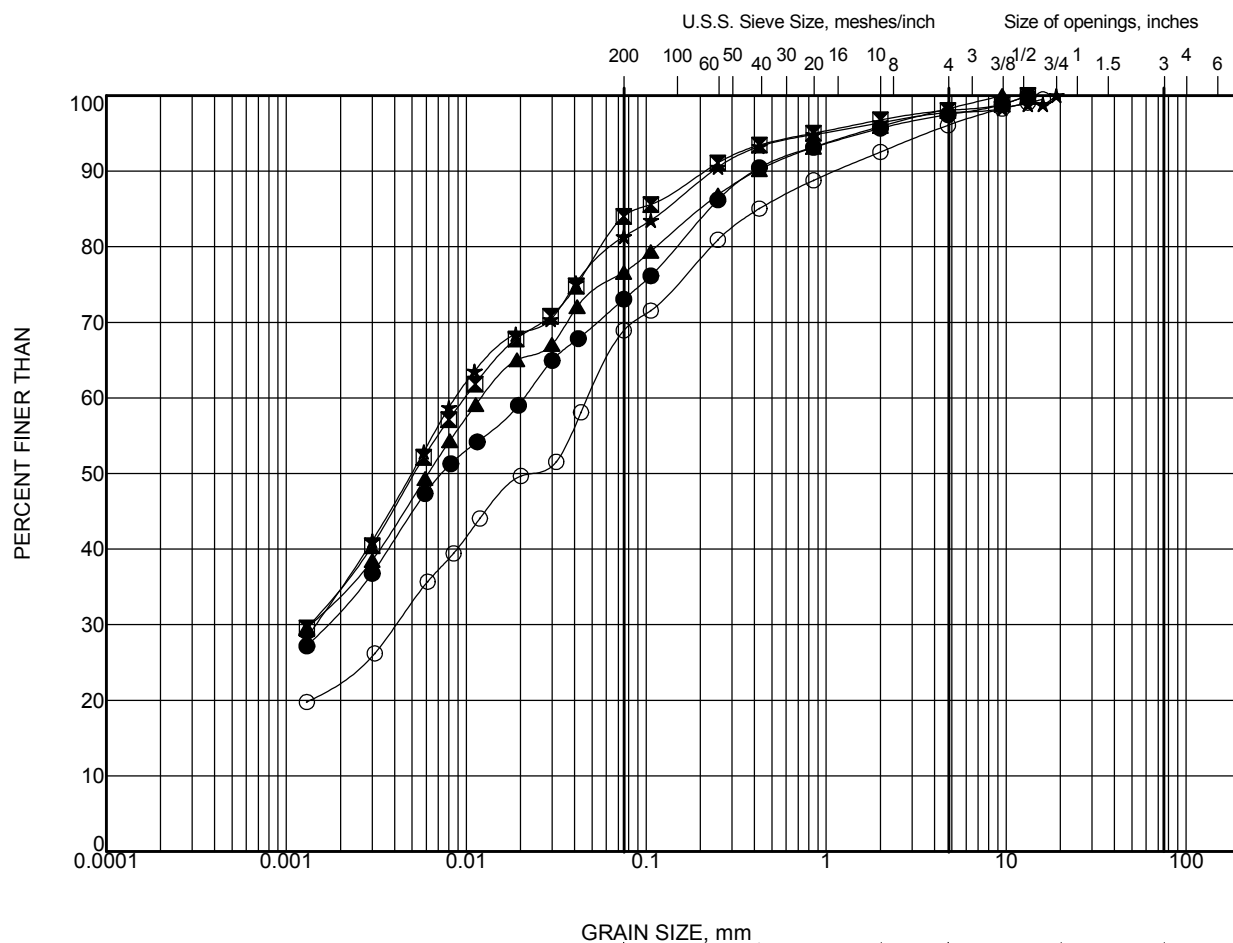
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B10-5	11	10.7
■	B10-5	17	19.8
▲	B10-6	9	7.6
★	B10-6	13	13.7
○	B10-6	17	19.8

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE		<b>GRAIN SIZE DISTRIBUTION</b> <b>Silty Clay / Clayey Silt</b>		
PROJECT No. SW8801.1004.101		FILE No.		
DRAWN mso		10 Nov. 2011		SCALE n/a
CHECK				REV.
		FIGURE C.3		



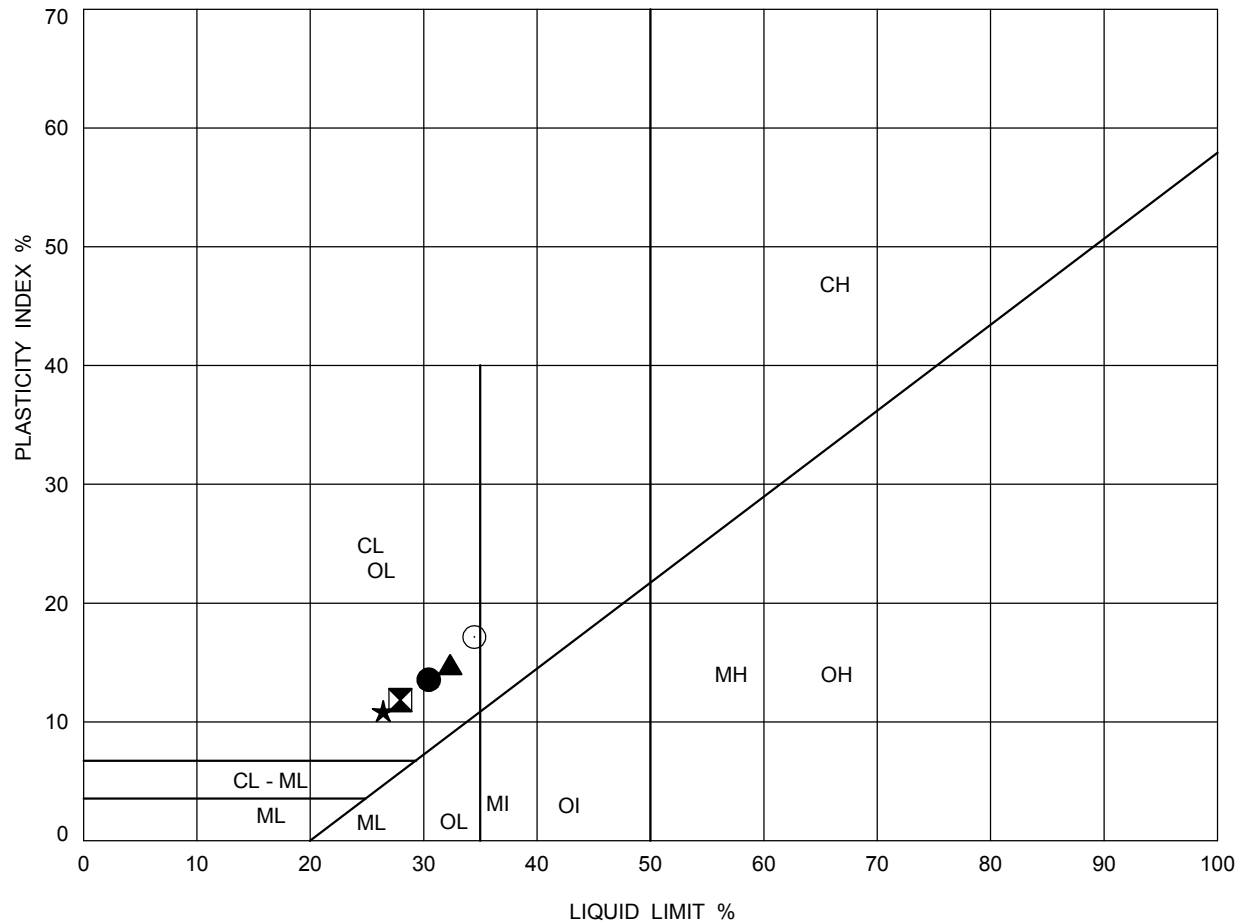


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND:



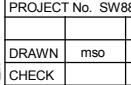
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B10-7	9	7.6
■	B10-7	13	13.7
▲	B10-7	20	22.9
★	B10-8	12	12.2
○	B10-8	17	19.8

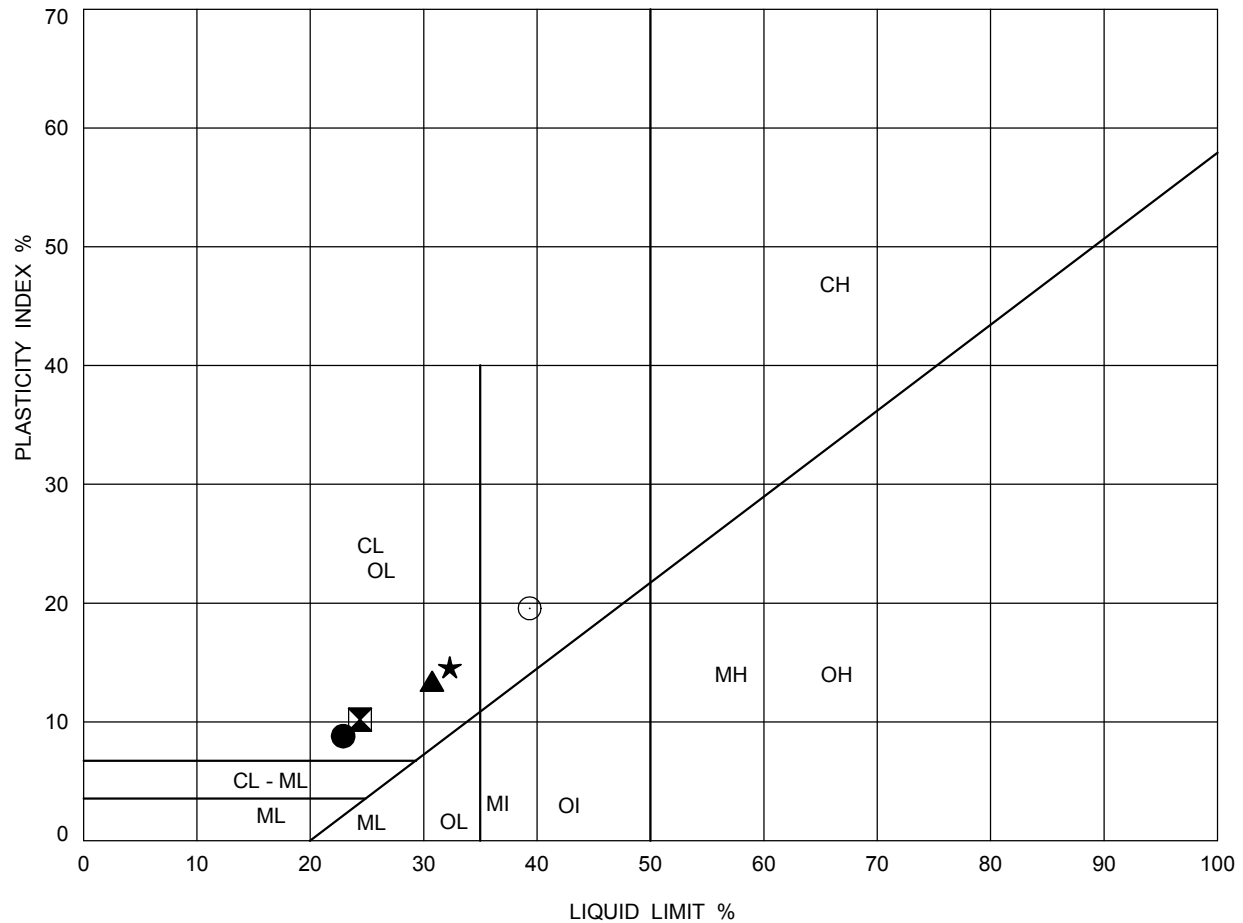
PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE		<b>GRAIN SIZE DISTRIBUTION</b> <b>Silty Clay / Clayey Silt</b>		
PROJECT No. SW8801.1004.101		FILE No.		
DRAWN mso		10 Nov. 2011		SCALE n/a
CHECK				REV.
		FIGURE C.4		



#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B10-1	12	12.2	30	17	13
⊠	B10-1	20	24.4	28	16	12
▲	B10-2	9	7.6	32	17	15
★	B10-2	19	22.9	26	16	10
○	B10-3	8	6.1	34	17	17

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART Silty Clay / Clayey Silt	
  	PROJECT No. SW8801.1004.101		FILE No.
	DRAWN	mso	10 Nov. 2011
CHECK			
			SCALE n/a
			REV.
FIGURE C.5			



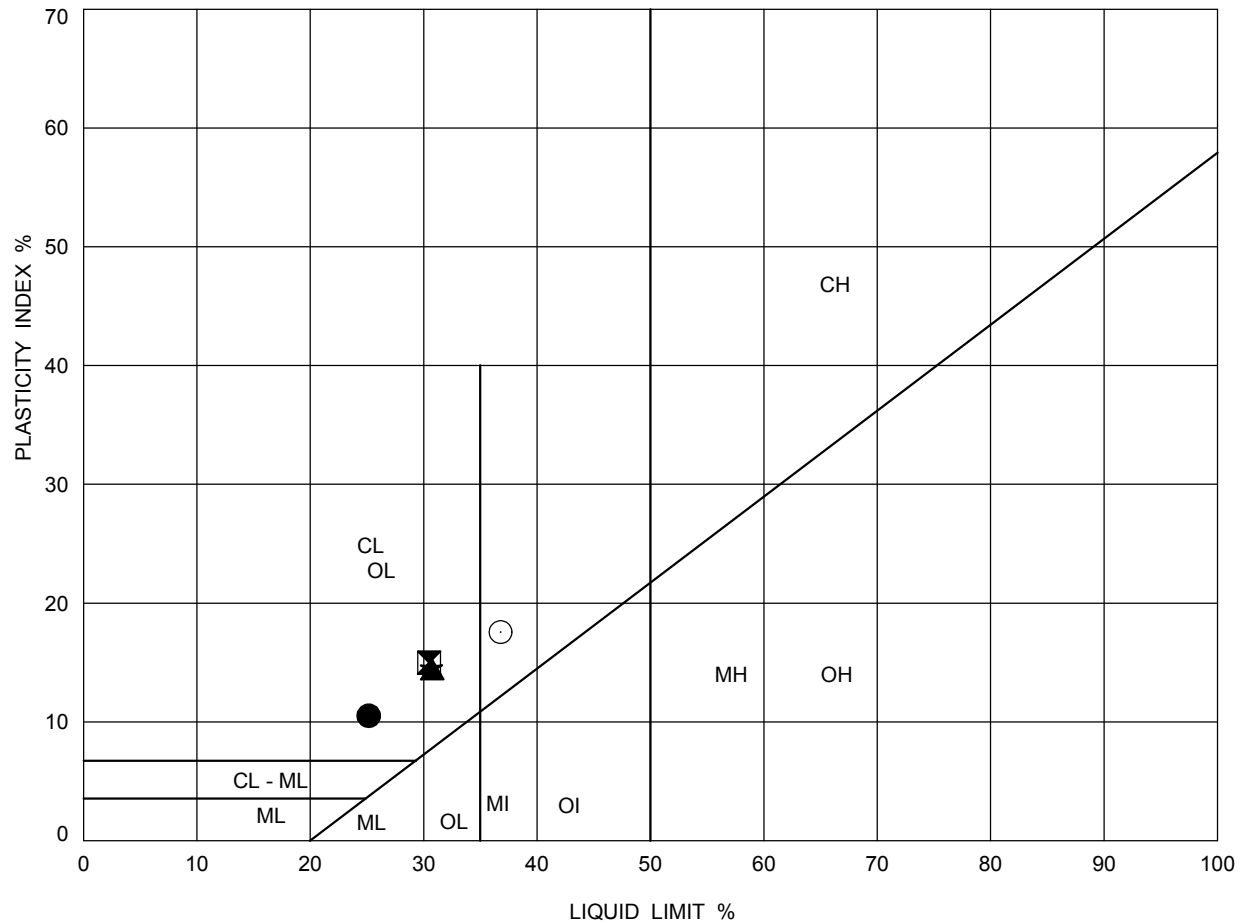
**SOIL TYPE**  
 C = Clay  
 M = Silt  
 O = Organic

**PLASTICITY**  
 L = Low  
 I = Intermediate  
 H = High

**LEGEND:**

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B10-3	22	27.4	23	14	9
⊠	B10-4	16	18.3	24	14	10
▲	B10-4	22	27.4	31	17	14
★	B10-5	11	10.7	32	18	14
○	B10-5	13	13.7	39	20	19

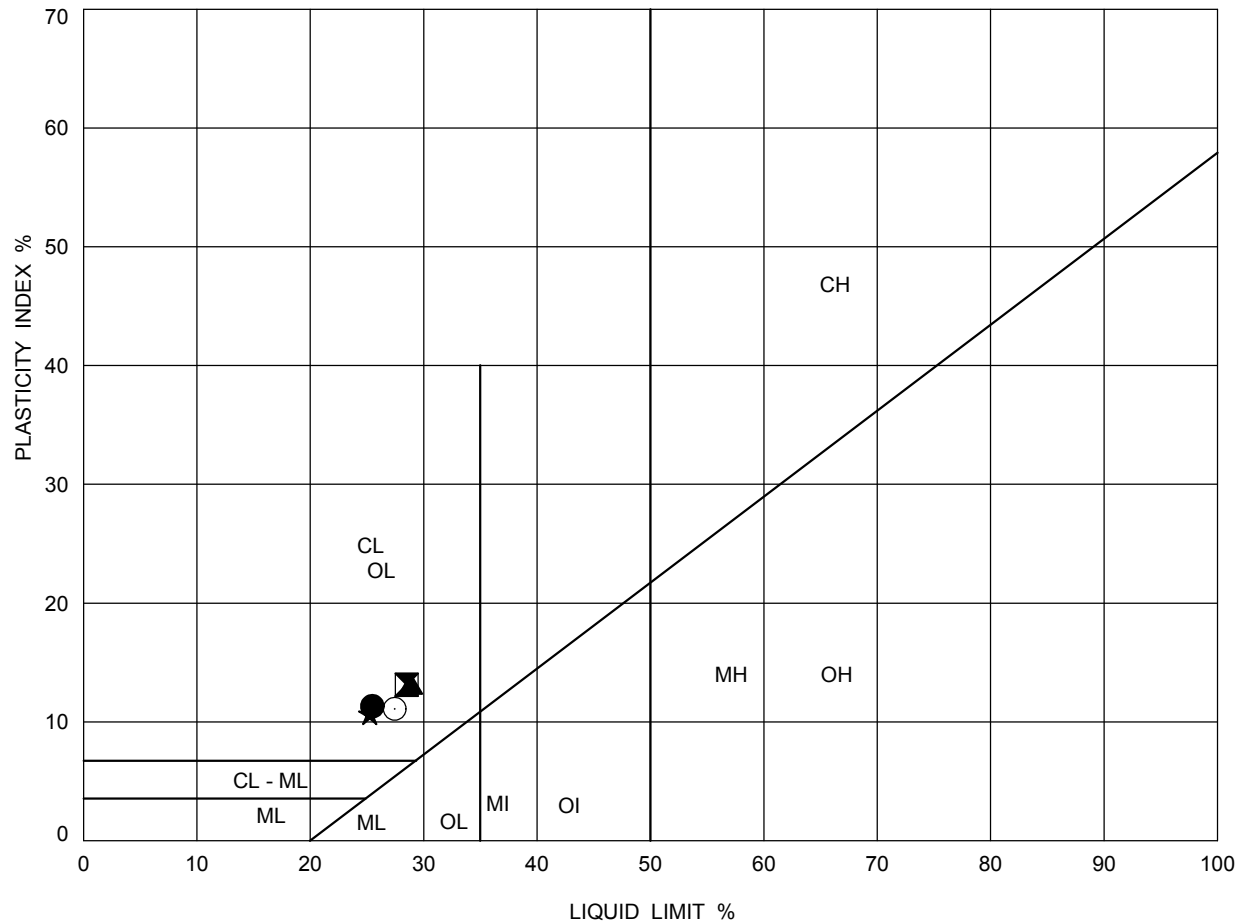
PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE		PLASTICITY CHART Silty Clay / Clayey Silt		
PROJECT No. SW8801.1004.101		FILE No.		
DRAWN mso		10 Nov. 2011		
CHECK		SCALE n/a REV.		
FIGURE C.6				



#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B10-5	17	19.8	25	15	10
⊠	B10-6	7	5.3	30	15	15
▲	B10-6	9	7.6	31	16	15
★	B10-6	11	10.7	31	16	15
○	B10-6	13	13.7	37	19	18

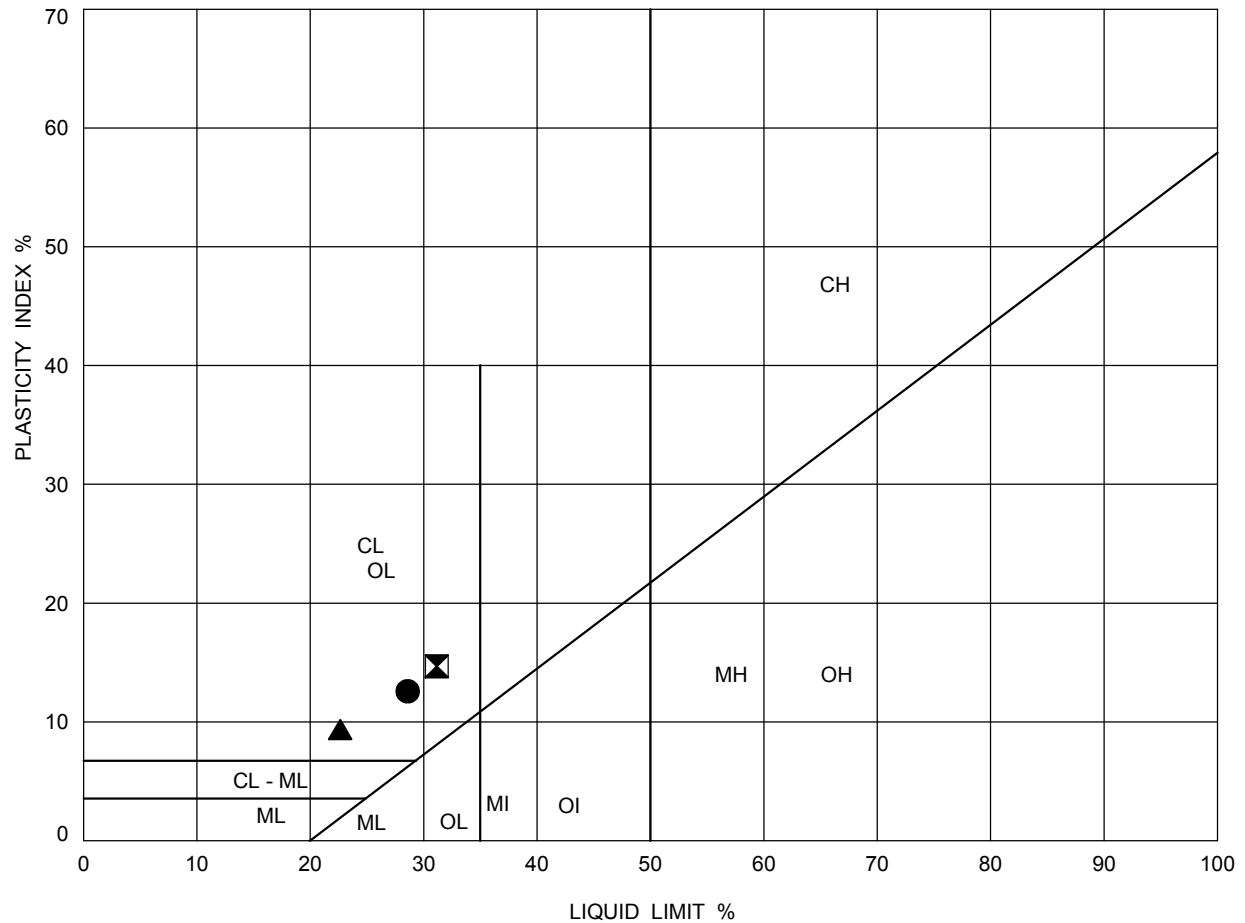
PROJECT				Windsor Essex Parkway (WEP) Windsor, Ontario			
TITLE				PLASTICITY CHART Silty Clay / Clayey Silt			
PROJECT No. SW8801.1004.101				FILE No.			
DRAWN mso 10 Nov. 2011				SCALE n/a REV.			
CHECK				FIGURE C.7			



#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B10-6	15	16.8	25	14	11
⊠	B10-6	17	19.8	29	15	14
▲	B10-7	9	7.6	29	16	13
★	B10-7	13	13.7	25	15	10
○	B10-7	20	22.9	27	16	11

PROJECT				Windsor Essex Parkway (WEP) Windsor, Ontario			
TITLE				PLASTICITY CHART Silty Clay / Clayey Silt			
PROJECT No. SW8801.1004.101				FILE No.			
DRAWN mso 10 Nov. 2011				SCALE n/a REV.			
CHECK				FIGURE C.8			



#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B10-7	25	29	29	16	13
⊠	B10-8	12	12.2	31	16	15
▲	B10-8	17	19.8	23	13	10

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE		PLASTICITY CHART Silty Clay / Clayey Silt		
PROJECT No. SW8801.1004.101		FILE No.		
DRAWN mso		10 Nov. 2011		
CHECK		SCALE n/a REV.		
FIGURE C.9				

## **Appendix D      Analytical Laboratory Test Results**

**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018

**Date:** March/2012  
**Rev:** 0  
**Page No.:** Appendix D



AMEC EARTH & ENVIRONMENTAL  
ATTN: Brian Lapos  
11865 County Road 42  
TECUMSEH ON N8N 2M1

Date Received: 03-JUN-11  
Report Date: 09-JUN-11 12:42 (MT)  
Version: FINAL

Client Phone: 519-735-2499

## Certificate of Analysis

**Lab Work Order #:** L1012450  
**Project P.O. #:** NOT SUBMITTED  
**Job Reference:** SW8801.1004.101  
**Legal Site Desc:**  
**C of C Numbers:** 092978

Gayle Braun  
Senior Account Manager

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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



# ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample ID Description Sampled Date Sampled Time Client ID		L1012450-1 SOIL 27-MAY-11  B10-1 SA#25	L1012450-2 SOIL 30-MAY-11  B10-2 SA#10@30'	L1012450-3 SOIL 27-MAY-11  B10-7 SA#26@100'		
Grouping	Analyte					
SOIL						
Physical Tests	% Moisture (%)	20.7	18.5	11.0		
	pH (pH units)	7.98	8.19	7.94		
	Redox Potential (mV)	163	44.0	102		
	Resistivity (ohm cm)	2550	6540	3220		
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20	<0.20	<0.20		
Anions and Nutrients	Sulphate (mg/kg)	338	<20	172		

## 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:

092978

### 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: L1012450

Report Date: 09-JUN-11

Page 1 of 3

Client: AMEC EARTH & ENVIRONMENTAL  
11865 County Road 42  
TECUMSEH ON N8N 2M1

Contact: Brian Lapos

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
<b>MOISTURE-WT</b>	<b>Soil</b>							
Batch	R2199427							
<b>WG1290566-2</b>	<b>LCS</b>							
% Moisture			93		%		70-130	06-JUN-11
<b>WG1290566-1</b>	<b>MB</b>							
% Moisture			<0.10		%		0.1	06-JUN-11
<b>PH-WT</b>	<b>Soil</b>							
Batch	R2198896							
<b>WG1290364-1</b>	<b>CVS</b>							
pH			100		%		80-120	04-JUN-11
<b>RESISTIVITY-WT</b>	<b>Soil</b>							
Batch	R2198903							
<b>WG1290368-1</b>	<b>CVS</b>							
Resistivity			98		%		70-130	04-JUN-11
<b>SO4-WT</b>	<b>Soil</b>							
Batch	R2200711							
<b>WG1291932-3</b>	<b>LCS</b>							
Sulphate			94		%		60-140	08-JUN-11
<b>WG1291932-1</b>	<b>MB</b>							
Sulphate			<20		mg/kg		20	08-JUN-11
<b>SULPHIDE-WT</b>	<b>Soil</b>							
Batch	R2200565							
<b>WG1292239-1</b>	<b>CVS</b>							
Sulphide			84		%		50-120	08-JUN-11
<b>WG1292235-1</b>	<b>MB</b>							
Sulphide			<0.20		mg/kg		0.2	08-JUN-11

# Quality Control Report

Workorder: L1012450

Report Date: 09-JUN-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: L1012450

Report Date: 09-JUN-11

Page 3 of 3

## Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
<b>Physical Tests</b>							
Redox Potential	1	27-MAY-11	04-JUN-11 19:50	24	200	hours	EHTR
	2	30-MAY-11	04-JUN-11 19:51	24	128	hours	EHTR
	3	27-MAY-11	04-JUN-11 19:52	24	200	hours	EHTR
Resistivity	1	27-MAY-11	04-JUN-11 19:50	7	8	days	EHTL
	3	27-MAY-11	04-JUN-11 19:52	7	8	days	EHTL
<b>Leachable Anions &amp; Nutrients</b>							
Sulphide	1	27-MAY-11	08-JUN-11 14:50	7	12	days	EHTL
	2	30-MAY-11	08-JUN-11 14:51	7	9	days	EHT
	3	27-MAY-11	08-JUN-11 14:52	7	12	days	EHTL

## 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 L1012450 were received on 03-JUN-11 10:45.

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.



**ALS Environmental**

C of C # 092978  
PAGE 1 OF 1

Reg COC Rev#4 09



AMEC EARTH & ENVIRONMENTAL  
ATTN: Brian Lapos  
11865 County Road 42  
TECUMSEH ON N8N 2M1

Date Received: 30-JUN-11  
Report Date: 08-JUL-11 07:13 (MT)  
Version: FINAL

Client Phone: 519-735-2499

## Certificate of Analysis

**Lab Work Order #:** L1025377  
**Project P.O. #:** NOT SUBMITTED  
**Job Reference:** SW8801.1004.101  
**Legal Site Desc:**  
**C of C Numbers:** 092732-6

Gayle Braun  
Senior Account Manager

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

# ALS ENVIRONMENTAL ANALYTICAL REPORT

		Sample ID	L1025377-1	L1025377-2			
		Description	SOIL	SOIL			
		Sampled Date	29-JUN-11	29-JUN-11			
		Sampled Time					
		Client ID	B10-5 SA#11 35'	B10-6 SA#23			
Grouping	Analyte						
<b>SOIL</b>							
<b>Physical Tests</b>	% Moisture (%)		18.1	17.2			
	pH (pH units)		7.89	7.84			
	Redox Potential (mV)		188	218			
	Resistivity (ohm cm)		3420	1680			
<b>Leachable Anions &amp; Nutrients</b>							
	Sulphide (mg/kg)		<0.20	<0.20			
<b>Anions and Nutrients</b>							
	Sulphate (mg/kg)		186	657			



## 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:

092732-6

### 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 wwt - 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: L1025377

Report Date: 08-JUL-11

Page 1 of 3

Client: AMEC EARTH & ENVIRONMENTAL  
11865 County Road 42  
TECUMSEH ON N8N 2M1

Contact: Brian Lapos

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
<b>MOISTURE-WT</b>								
	Soil							
Batch	R2212765							
WG1305352-2	LCS							
% Moisture			92		%		70-130	30-JUN-11
WG1305352-1	MB							
% Moisture			<0.10		%		0.1	30-JUN-11
<b>PH-WT</b>								
	Soil							
Batch	R2214528							
WG1307906-1	CVS							
pH			100		%		80-120	06-JUL-11
<b>RESISTIVITY-WT</b>								
	Soil							
Batch	R2215155							
WG1308646-1	CVS							
Resistivity			100		%		70-130	07-JUL-11
<b>SO4-WT</b>								
	Soil							
Batch	R2213607							
WG1306314-3	LCS							
Sulphate			101		%		60-140	04-JUL-11
WG1306314-1	MB							
Sulphate			<20		mg/kg		20	04-JUL-11
<b>SULPHIDE-WT</b>								
	Soil							
Batch	R2213798							
WG1307079-1	CVS							
Sulphide			79		%		50-120	05-JUL-11
WG1307075-1	MB							
Sulphide			<0.20		mg/kg		0.2	05-JUL-11

# Quality Control Report

Workorder: L1025377

Report Date: 08-JUL-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: L1025377

Report Date: 08-JUL-11

Page 3 of 3

## Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
<b>Physical Tests</b>							
Redox Potential	1	29-JUN-11	07-JUL-11 17:09	24	197	hours	EHTL
	2	29-JUN-11	07-JUL-11 17:10	24	197	hours	EHTL
Resistivity	1	29-JUN-11	07-JUL-11 17:06	7	8	days	EHT
	2	29-JUN-11	07-JUL-11 17:07	7	8	days	EHT

## 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 L1025377 were received on 30-JUN-11 11:00.

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.





AMEC EARTH & ENVIRONMENTAL  
ATTN: Brian Lapos  
11865 County Road 42  
TECUMSEH ON N8N 2M1

Date Received: 16-JUN-11  
Report Date: 22-JUN-11 12:39 (MT)  
Version: FINAL

Client Phone: 519-735-2499

## Certificate of Analysis

**Lab Work Order #:** L1018322  
**Project P.O. #:** NOT SUBMITTED  
**Job Reference:** SW8801.1004.101  
**Legal Site Desc:**  
**C of C Numbers:** 092982

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

# ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample ID Description Sampled Date Sampled Time Client ID		L1018322-1 SOIL 09-JUN-11 B10-8 SA#10 32'	L1018322-2 SOIL 08-JUN-11 B10-4 SA#24	L1018322-3 SOIL 07-JUN-11 B10-3 SA#25 104'-105.5'		
Grouping	Analyte					
SOIL						
Physical Tests	% Moisture (%)	18.9	23.6	8.02		
	pH (pH units)	8.00	7.90	7.95		
	Redox Potential (mV)	87.0	123	130		
	Resistivity (ohm cm)	5380	1910	3470		
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20	<0.20	<0.20		
Anions and Nutrients	Sulphate (mg/kg)	52	567	267		

## 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:

092982

### 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 lw - 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: L1018322

Report Date: 22-JUN-11

Page 1 of 3

Client: AMEC EARTH & ENVIRONMENTAL  
11865 County Road 42  
TECUMSEH ON N8N 2M1

Contact: Brian Lapos

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
<b>MOISTURE-WT</b>								
	Soil							
Batch	R2205271							
WG1297250-2	LCS							
% Moisture			98		%		70-130	16-JUN-11
WG1297250-1	MB							
% Moisture			<0.10		%		0.1	16-JUN-11
<b>PH-WT</b>								
	Soil							
Batch	R2207258							
WG1299570-1	CVS							
pH			99		%		80-120	21-JUN-11
<b>RESISTIVITY-WT</b>								
	Soil							
Batch	R2207262							
WG1299566-1	CVS							
Resistivity			100		%		70-130	21-JUN-11
<b>SO4-WT</b>								
	Soil							
Batch	R2207781							
WG1299485-3	LCS							
Sulphate			98		%		60-140	21-JUN-11
WG1299485-1	MB							
Sulphate			<20		mg/kg		20	21-JUN-11
<b>SULPHIDE-WT</b>								
	Soil							
Batch	R2206609							
WG1298908-1	CVS							
Sulphide			85		%		50-120	20-JUN-11
WG1298740-1	MB							
Sulphide			<0.20		mg/kg		0.2	20-JUN-11

# Quality Control Report

Workorder: L1018322

Report Date: 22-JUN-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: L1018322

Report Date: 22-JUN-11

Page 3 of 3

## Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
<b>Physical Tests</b>							
Redox Potential	1	09-JUN-11	21-JUN-11 15:06	24	291	hours	EHTR
	2	08-JUN-11	21-JUN-11 15:07	24	315	hours	EHTR
	3	07-JUN-11	21-JUN-11 15:08	24	339	hours	EHTR
Resistivity	1	09-JUN-11	21-JUN-11 15:09	7	12	days	EHTL
	2	08-JUN-11	21-JUN-11 15:10	7	13	days	EHTR
	3	07-JUN-11	21-JUN-11 15:11	7	14	days	EHTR
<b>Leachable Anions &amp; Nutrients</b>							
Sulphide	1	09-JUN-11	20-JUN-11 16:03	7	11	days	EHTL
	2	08-JUN-11	20-JUN-11 16:04	7	12	days	EHTR
	3	07-JUN-11	20-JUN-11 16:05	7	13	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 L1018322 were received on 16-JUN-11 10:15.

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.



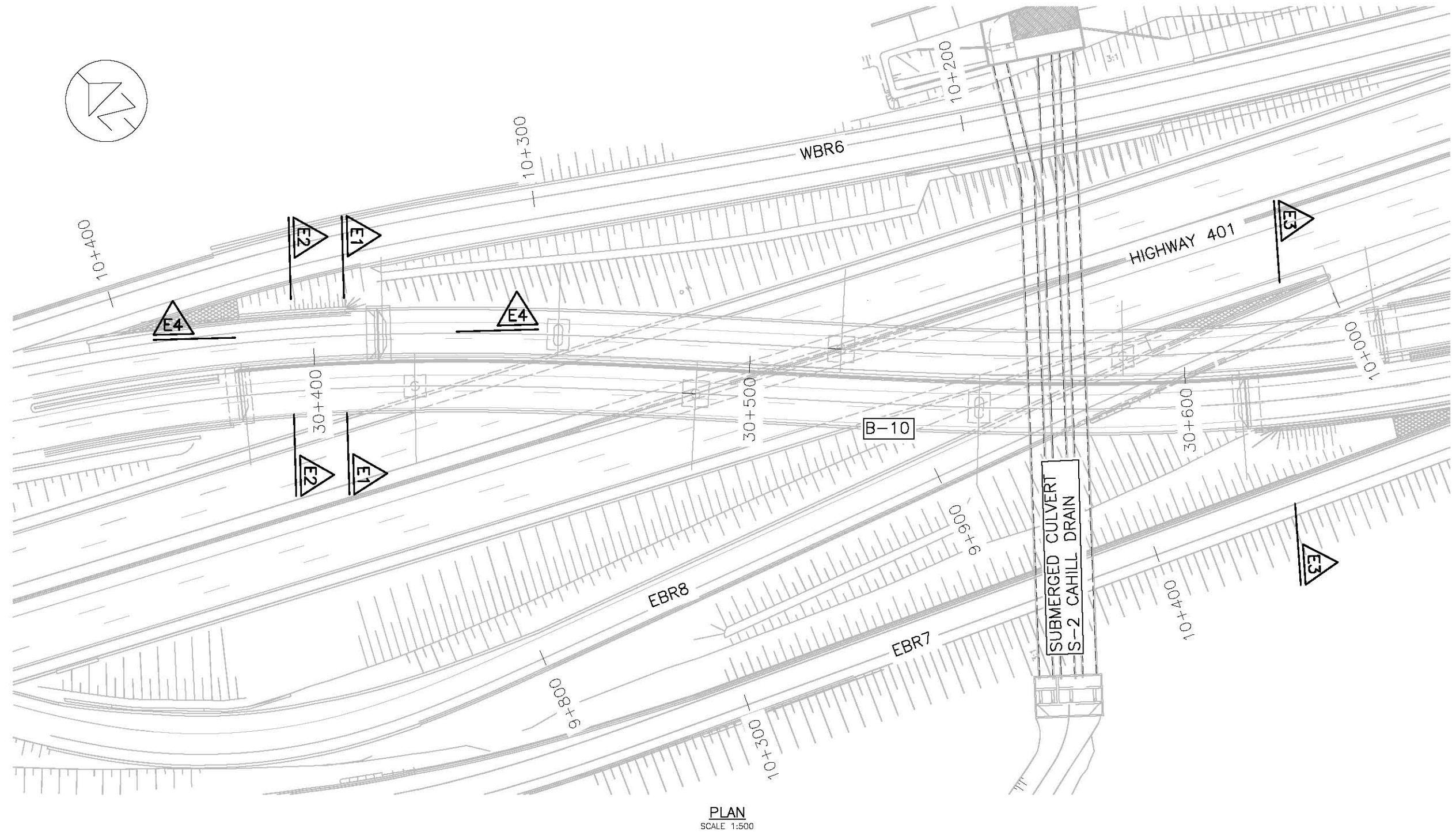
ALS  
Environmental

C of C # 092982  
PAGE 1 OF 2

## **Appendix E      Slope Stability Analyses**

**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018

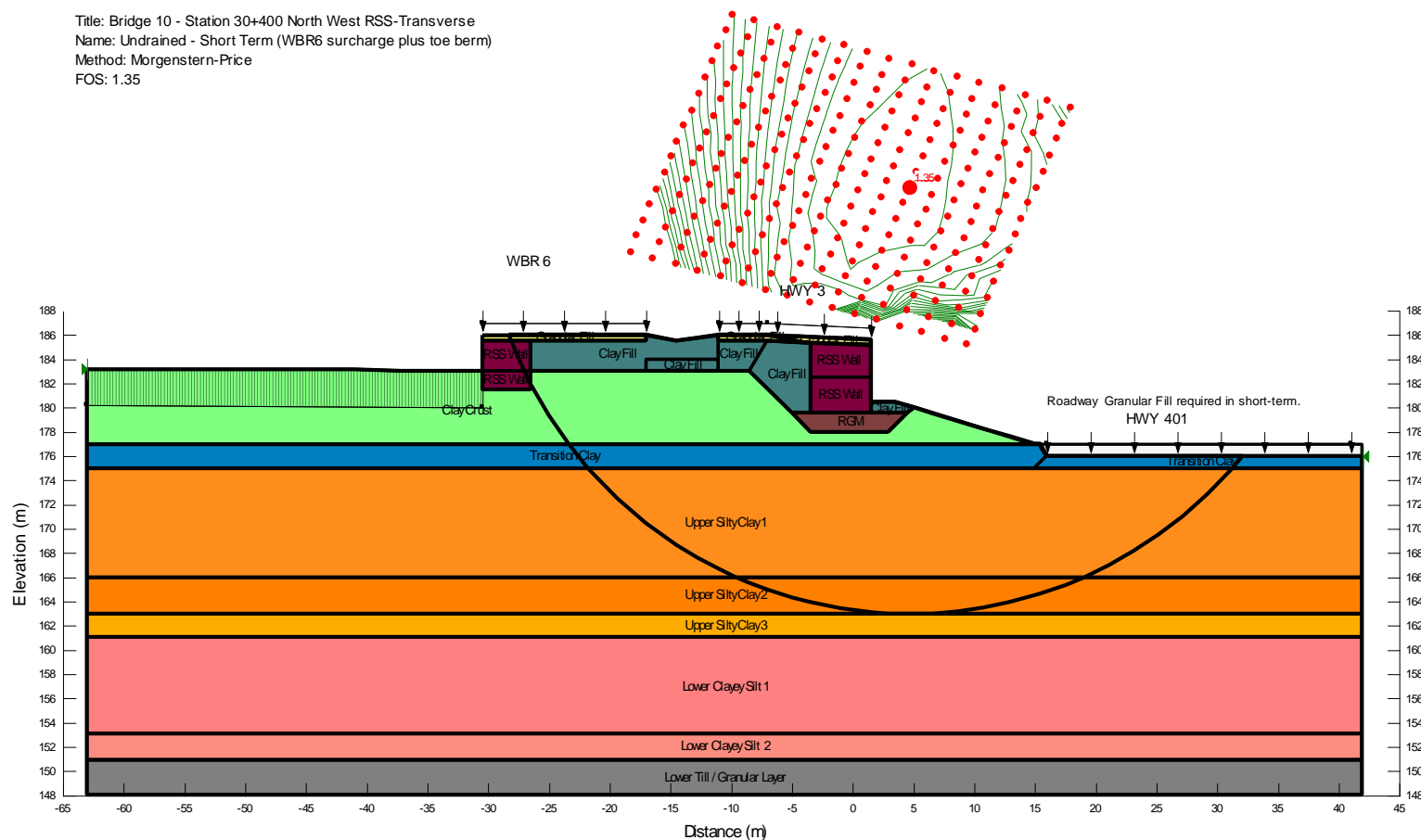
**Date:** March/2012  
**Rev:** 0  
**Page No.:** Appendix E



**NOT FOR  
CONSTRUCTION**

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

Title: Bridge 10 - Station 30+400 North West RSS-Transverse  
Name: Undrained - Short Term (WBR6 surcharge plus toe berm)  
Method: Morgenstern-Price  
FOS: 1.35



Name: Clay Crust Model: Undrained ( $\Phi=0$ ) Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 75 kPa

Name: Upper Silty Clay 1 Model: S=f(datum) Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 55 kPa C-Rate of Change: -1.33 kPa/m Limiting C: 43 kPa Elevation: 175 m

Name: Upper Silty Clay 2 Model: S=f(datum) Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 45 kPa C-Rate of Change: 2 kPa/m Limiting C: 49 kPa Elevation: 166 m

Name: Lower Clayey Silt 1 Model: Undrained ( $\Phi=0$ ) Unit Weight: 19.5 kN/m<sup>3</sup> Cohesion: 65 kPa

Name: Lower Till / Granular Layer Model: Mohr-Coulomb Unit Weight: 19 kN/m<sup>3</sup> Cohesion: 0 kPa  $\Phi=30^\circ$   $\Phi-B: 0^\circ$

Name: Clay Fill Model: Undrained ( $\Phi=0$ ) Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa

Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa  $\Phi=35^\circ$   $\Phi-B: 0^\circ$

Name: RSS Wall Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa  $\Phi=35^\circ$   $\Phi-B: 0^\circ$

Name: RGM Model: Mohr-Coulomb Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 0 kPa  $\Phi=35^\circ$   $\Phi-B: 0^\circ$

Name: Upper Silty Clay 3 Model: S=f(datum) Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 49 kPa C-Rate of Change: 8 kPa/m Limiting C: 65 kPa Elevation: 163 m

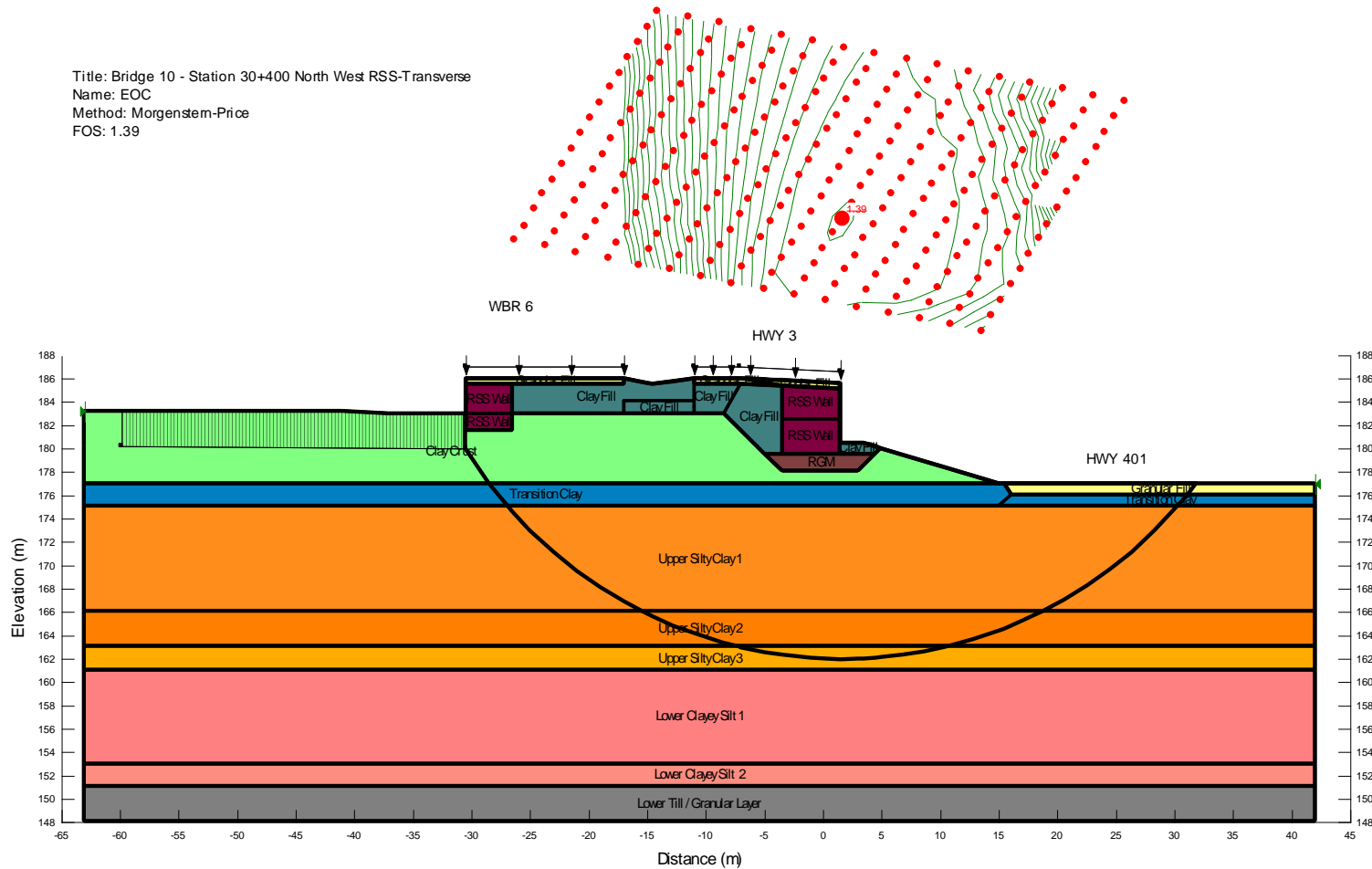
Name: Lower Clayey Silt 2 Model: S=f(datum) Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 65 kPa C-Rate of Change: 2.5 kPa/m Limiting C: 70 kPa Elevation: 153 m

Name: Transition Clay Model: S=f(datum) Unit Weight: 22 kN/m<sup>3</sup> C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m

**Figure E2. Slope Stability – Short Term (Undrained)– 30+400 West RSS Wall**



Title: Bridge 10 - Station 30+400 North West RSS-Transverse  
Name: EOC  
Method: Morgenstem-Price  
FOS: 1.39

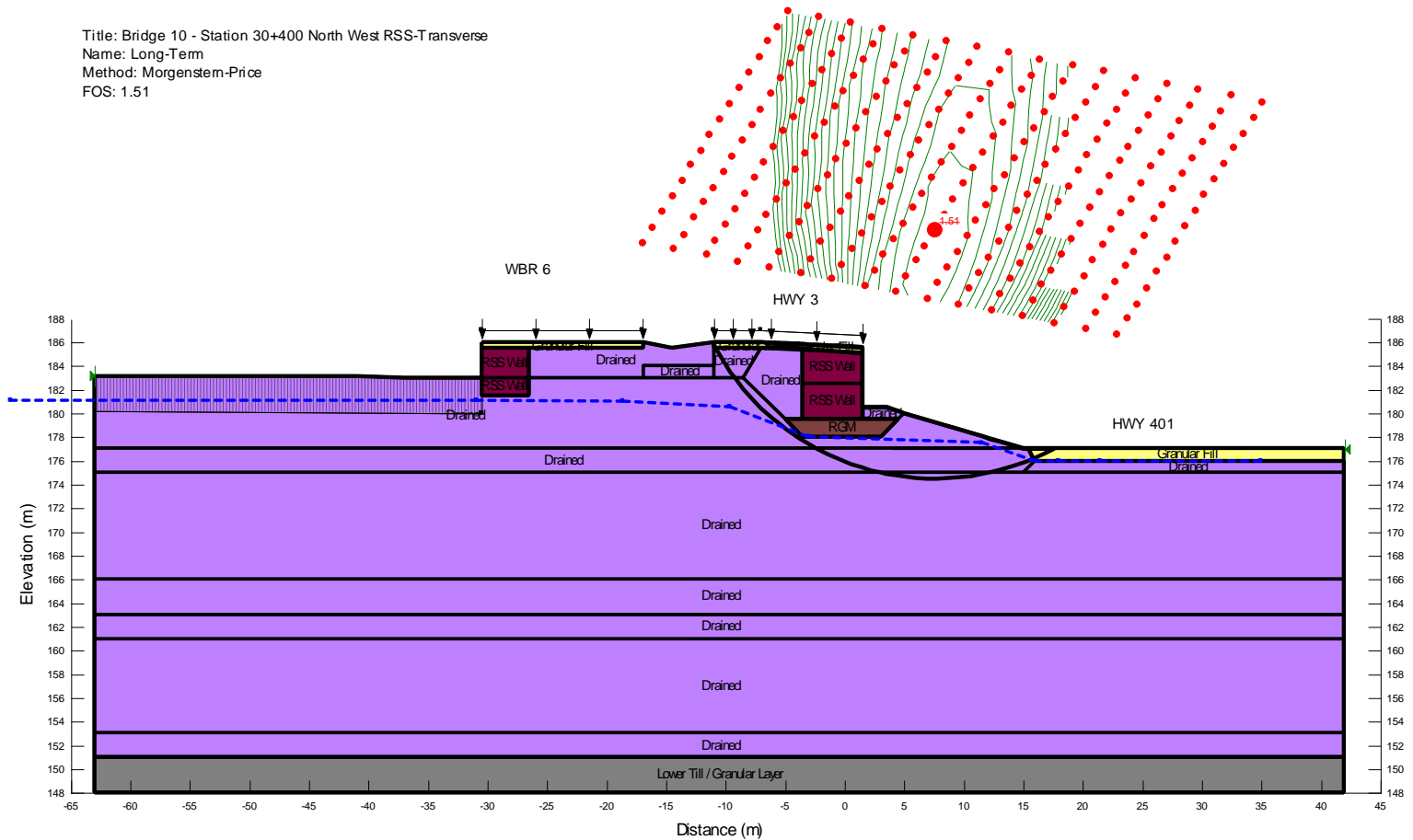


Name: Clay Crust Model: Undrained (Phi=0) Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 75 kPa  
Name: Upper Silty Clay 1 Model: S=f(datum) Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 55 kPa C-Rate of Change: -1.33 kPa/m Limiting C: 43 kPa Elevation: 175 m  
Name: Upper Silty Clay 2 Model: S=f(datum) Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 43 kPa C-Rate of Change: 2 kPa/m Limiting C: 49 kPa Elevation: 166 m  
Name: Lower Clayey Silt 1 Model: Undrained (Phi=0) Unit Weight: 19.5 kN/m<sup>3</sup> Cohesion: 65 kPa  
Name: Lower Till / Granular Layer Model: Mohr-Coulomb Unit Weight: 19 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 °  
Name: Clay Fill Model: Undrained (Phi=0) Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa  
Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 °  
Name: RSS Wall Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 100 kPa Phi: 35 ° Phi-B: 0 °  
Name: RGM Model: Mohr-Coulomb Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 °  
Name: Upper Silty Clay 3 Model: S=f(datum) Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 49 kPa C-Rate of Change: 8 kPa/m Limiting C: 65 kPa Elevation: 163 m  
Name: Lower Clayey Silt 2 Model: S=f(datum) Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 65 kPa C-Rate of Change: 2.5 kPa/m Limiting C: 70 kPa Elevation: 153 m  
Name: Transition Clay Model: S=f(datum) Unit Weight: 22 kN/m<sup>3</sup> C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m

**Figure E3. Slope Stability – End of Construction (Undrained) – 30+400 West RSS Wall**



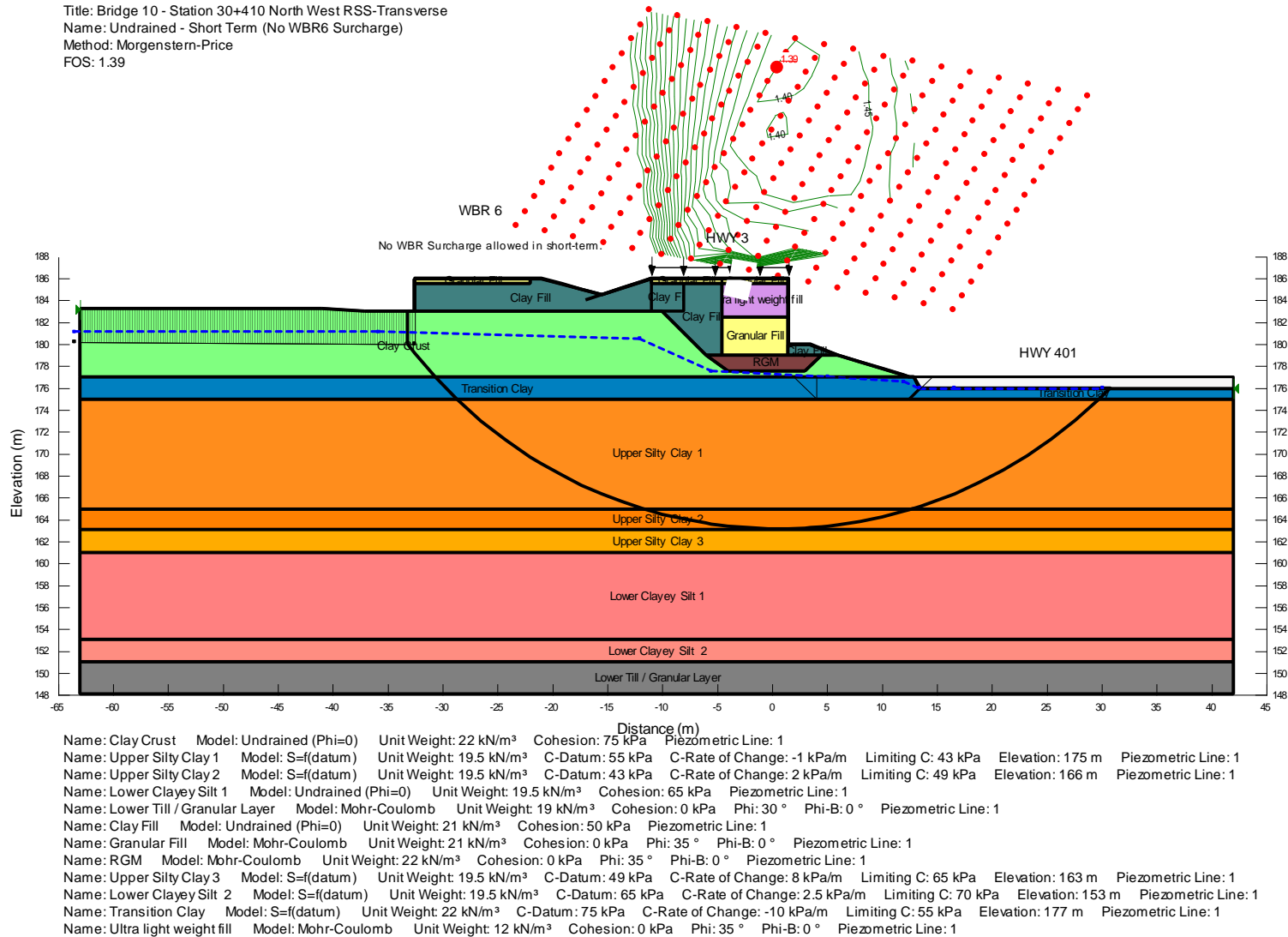
Title: Bridge 10 - Station 30+400 North West RSS-Transverse  
Name: Long-Term  
Method: Morgenstern-Price  
FOS: 1.51



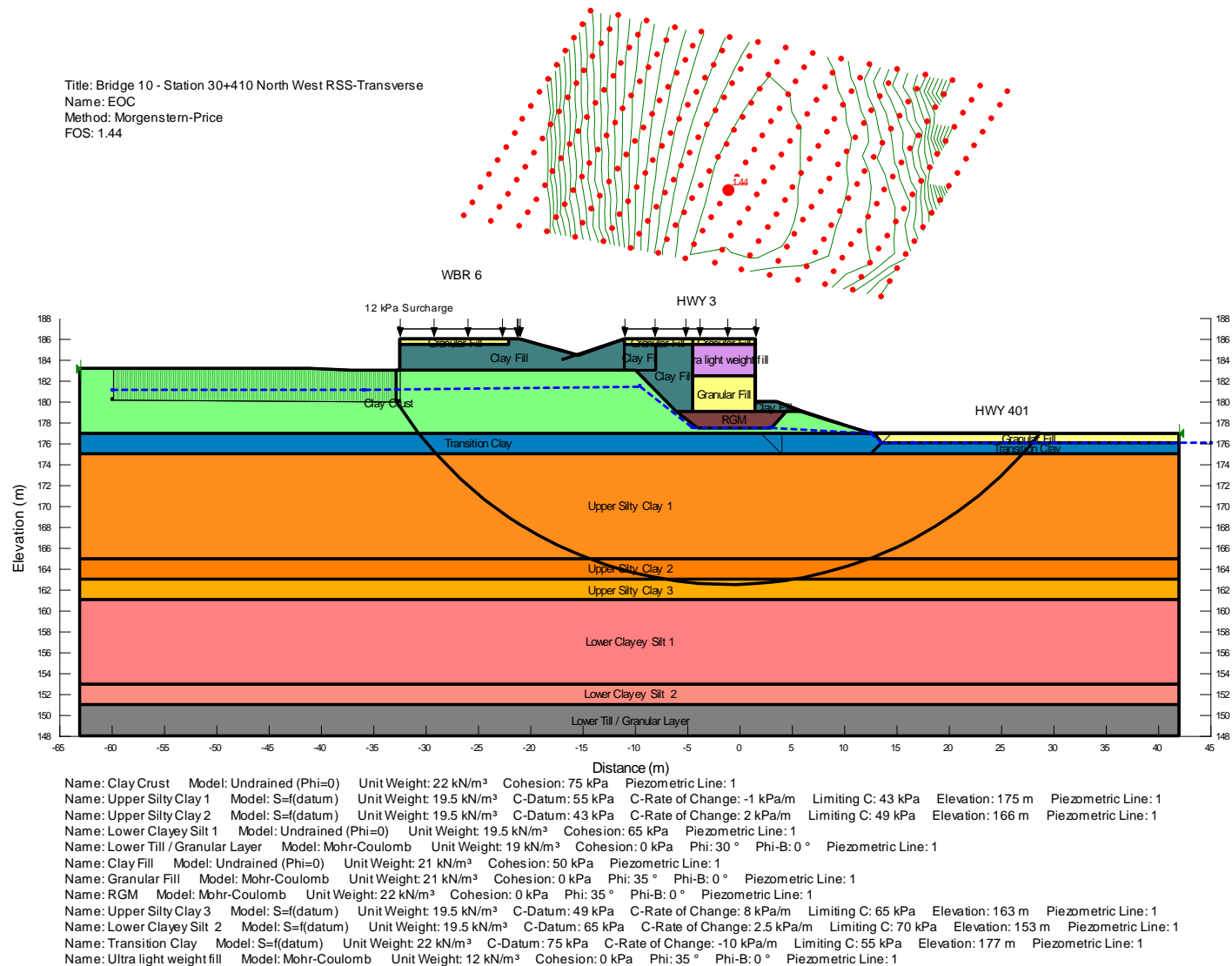
Name: Lower Till / Granular Layer Model: Mohr-Coulomb Unit Weight: 19 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1  
Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1  
Name: RSS Wall Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 100 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1  
Name: RGM Model: Mohr-Coulomb Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1  
Name: Drained Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1

**Figure E4. Slope Stability – Long-Term (Drained) – 30+400 West RSS Wall**

Title: Bridge 10 - Station 30+410 North West RSS-Transverse  
Name: Undrained - Short Term (No WBR6 Surcharge)  
Method: Morgenstern-Price  
FOS: 1.39

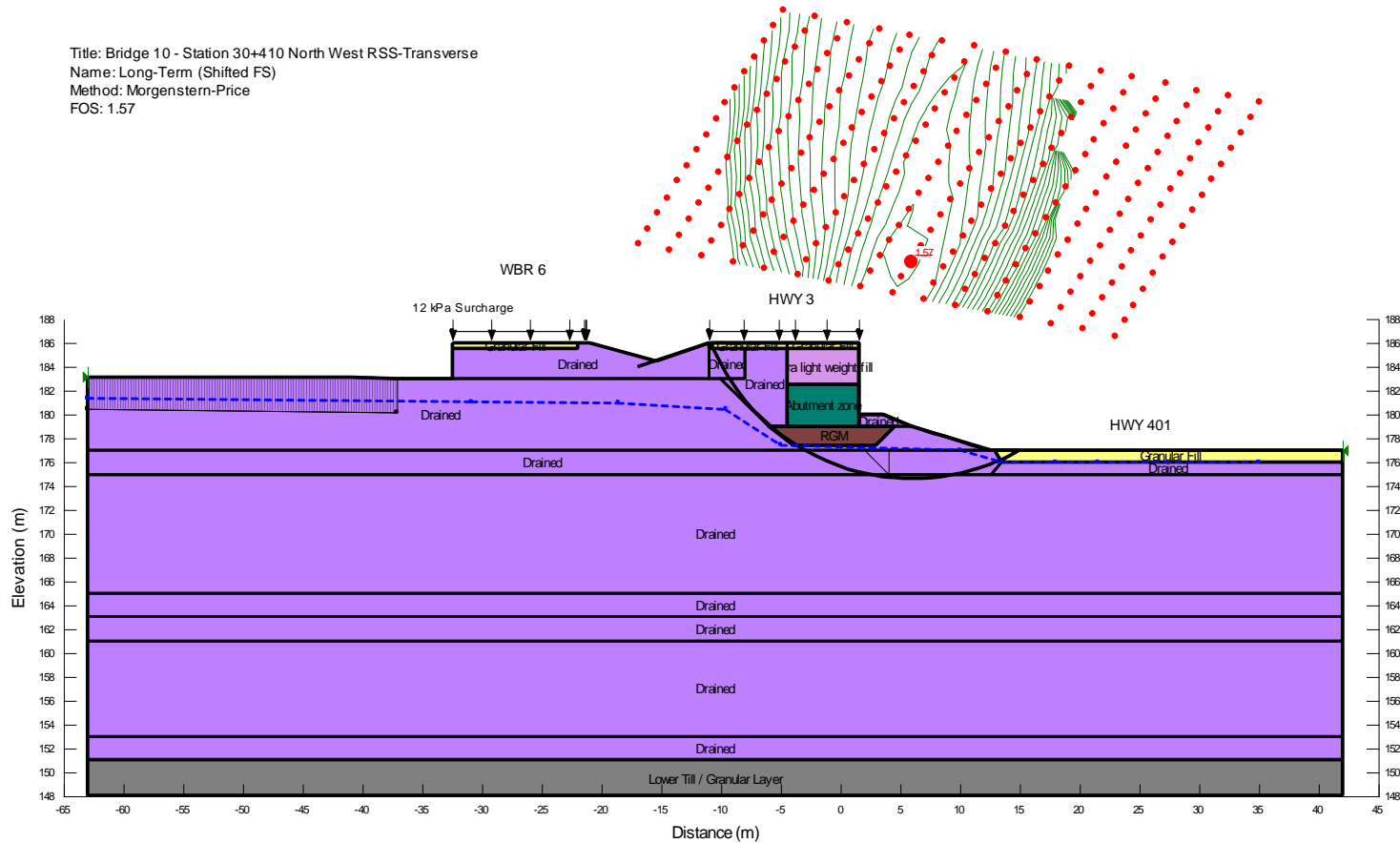


**Figure E5. Slope Stability – Short Term (Undrained)– 30+410 West RSS Wall**



**Figure E6. Slope Stability – End of Construction (Undrained)– 30+410 West RSS Wall**

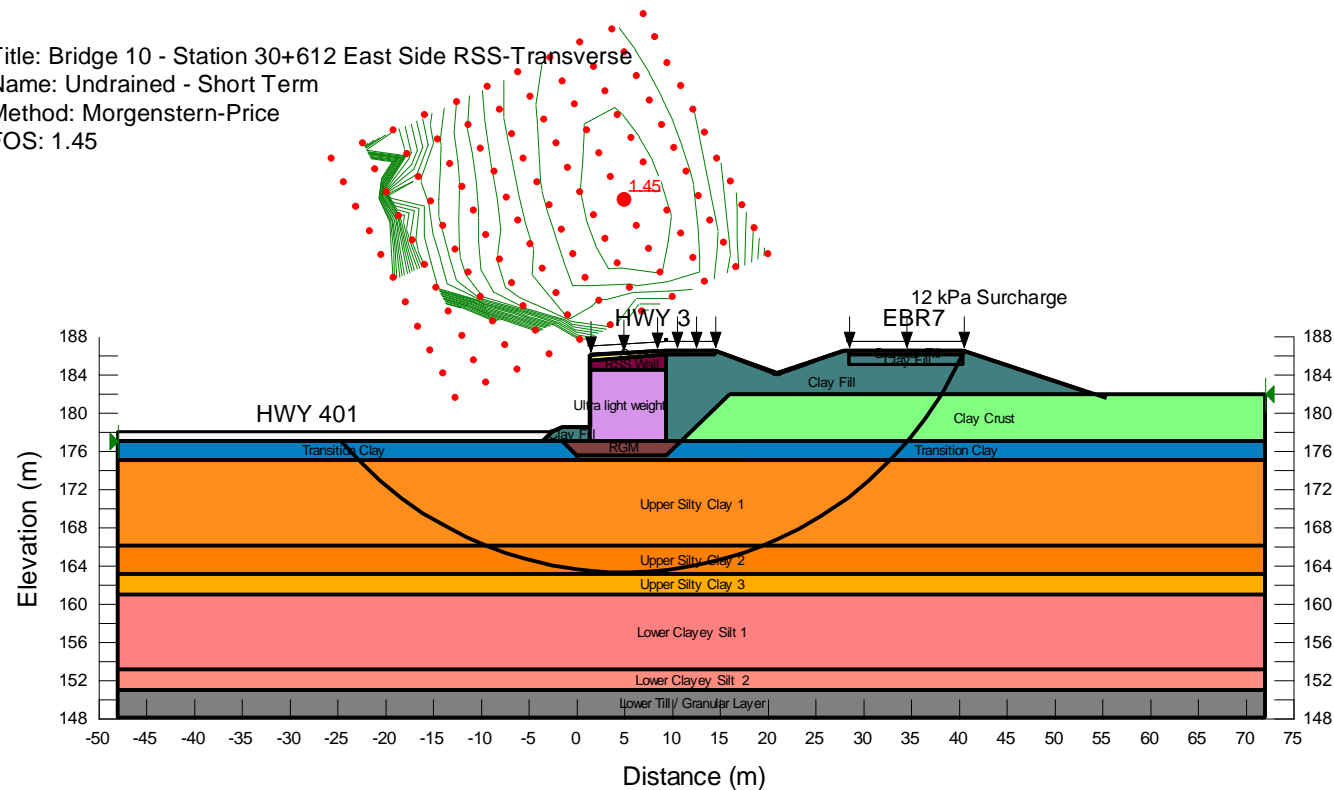
Title: Bridge 10 - Station 30+410 North West RSS-Transverse  
Name: Long-Term (Shifted FS)  
Method: Morgenstern-Price  
FOS: 1.57



Name: Lower Till / Granular Layer Model: Mohr-Coulomb Unit Weight: 19 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1  
Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1  
Name: RGM Model: Mohr-Coulomb Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1  
Name: Ultra light weight fill Model: Mohr-Coulomb Unit Weight: 12 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1  
Name: Abutment zone Model: Mohr-Coulomb Unit Weight: 0.1 kN/m<sup>3</sup> Cohesion: 500 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1  
Name: Drained Model: Mohr-Coulomb Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1

**Figure E7. Slope Stability – Long-Term (Drained) – 30+410 West RSS Wall**

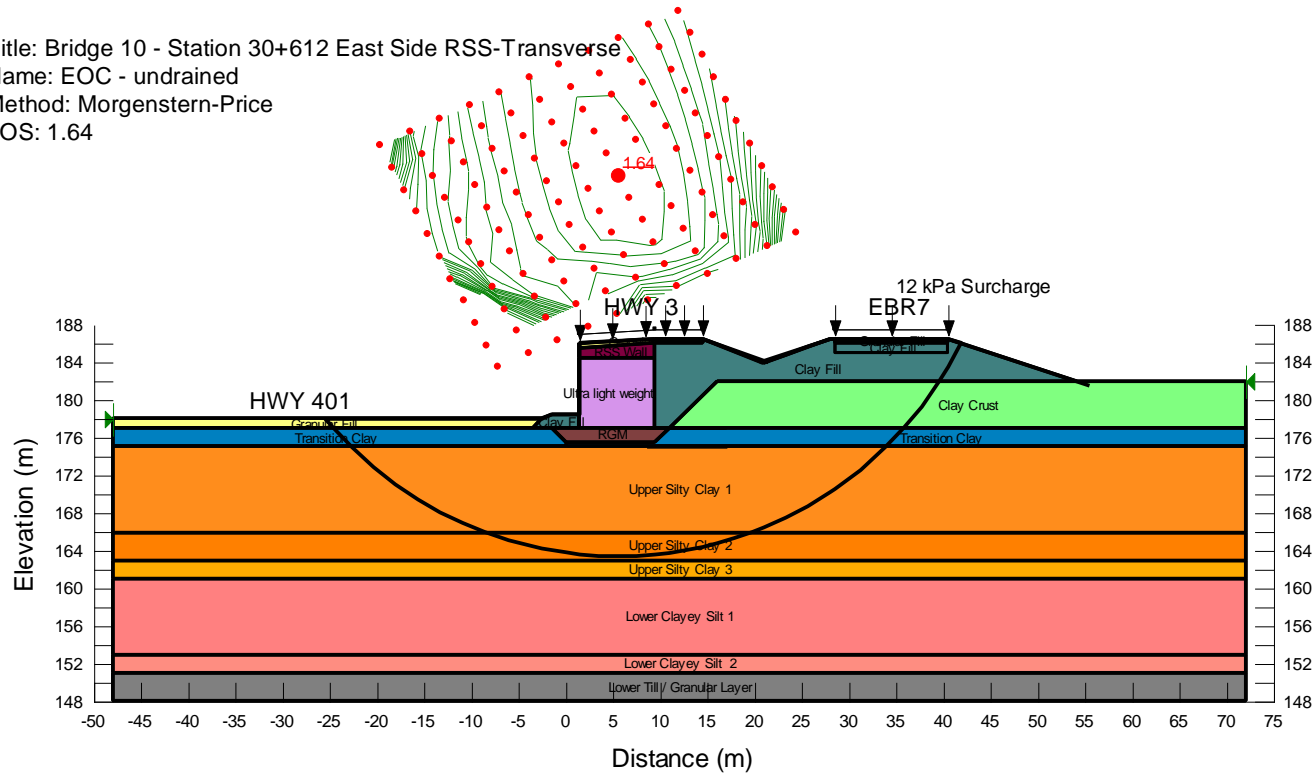
Title: Bridge 10 - Station 30+612 East Side RSS-Transverse  
Name: Undrained - Short Term  
Method: Morgenstern-Price  
FOS: 1.45



Name: Clay Crust Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 75 kPa  
Name: Upper Silty Clay 1 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 55 kPa C-Rate of Change: -1.333 kPa/m Limiting C: 43 kPa Elevation: 175 m  
Name: Upper Silty Clay 2 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 43 kPa C-Rate of Change: 2 kPa/m Limiting C: 49 kPa Elevation: 166 m  
Name: Lower Clayey Silt 1 Unit Weight: 19.5 kN/m<sup>3</sup> Cohesion: 65 kPa  
Name: Lower Till / Granular Layer Unit Weight: 19 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30 °  
Name: Clay Fill Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa  
Name: Granular Fill Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: RSS Wall Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: RGM Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: Upper Silty Clay 3 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 49 kPa C-Rate of Change: 8 kPa/m Limiting C: 65 kPa Elevation: 163 m  
Name: Lower Clayey Silt 2 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 65 kPa C-Rate of Change: 2.5 kPa/m Limiting C: 70 kPa Elevation: 153 m  
Name: Transition Clay Unit Weight: 22 kN/m<sup>3</sup> C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m  
Name: Ultra light weight fill Unit Weight: 12 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °

**Figure E8. Slope Stability – Short Term (Undrained)– 30+612 East RSS Wall**

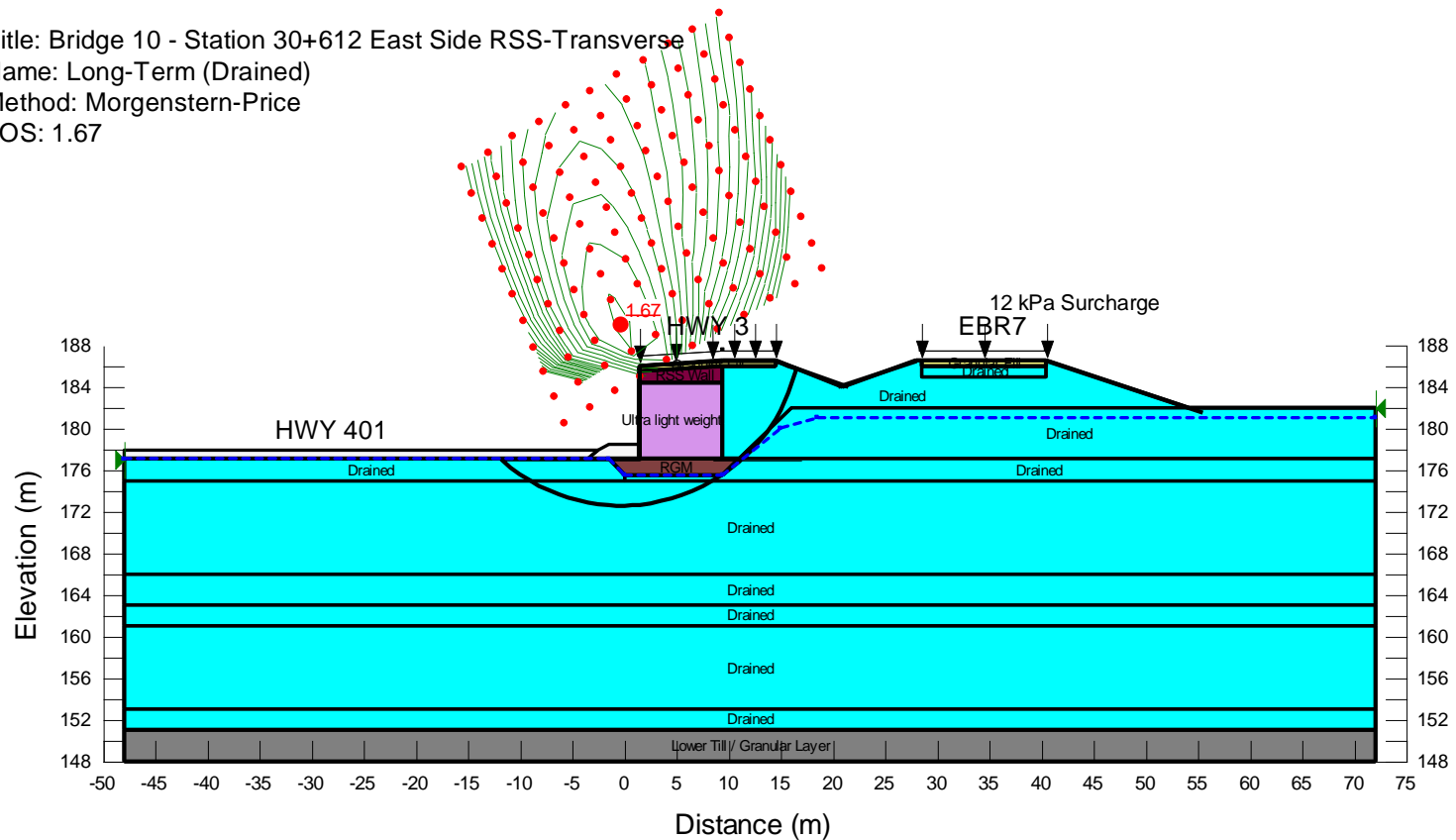
Title: Bridge 10 - Station 30+612 East Side RSS-Transverse  
Name: EOC - undrained  
Method: Morgenstern-Price  
FOS: 1.64



Name: Clay Crust Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 75 kPa  
Name: Upper Silty Clay 1 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 55 kPa C-Rate of Change: -1.333 kPa/m Limiting C: 43 kPa Elevation: 175 m  
Name: Upper Silty Clay 2 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 43 kPa C-Rate of Change: 2 kPa/m Limiting C: 49 kPa Elevation: 166 m  
Name: Lower Clayey Silt 1 Unit Weight: 19.5 kN/m<sup>3</sup> Cohesion: 65 kPa  
Name: Lower Till / Granular Layer Unit Weight: 19 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30 °  
Name: Clay Fill Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa  
Name: Granular Fill Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: RSS Wall Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: RGM Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: Upper Silty Clay 3 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 49 kPa C-Rate of Change: 8 kPa/m Limiting C: 65 kPa Elevation: 163 m  
Name: Lower Clayey Silt 2 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 65 kPa C-Rate of Change: 2.5 kPa/m Limiting C: 70 kPa Elevation: 153 m  
Name: Transition Clay Unit Weight: 22 kN/m<sup>3</sup> C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m  
Name: Ultra light weight fill Unit Weight: 12 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °

**Figure E9. Slope Stability – End of Construction (Undrained)– 30+612 East RSS Wall**

Title: Bridge 10 - Station 30+612 East Side RSS-Transverse  
Name: Long-Term (Drained)  
Method: Morgenstern-Price  
FOS: 1.67

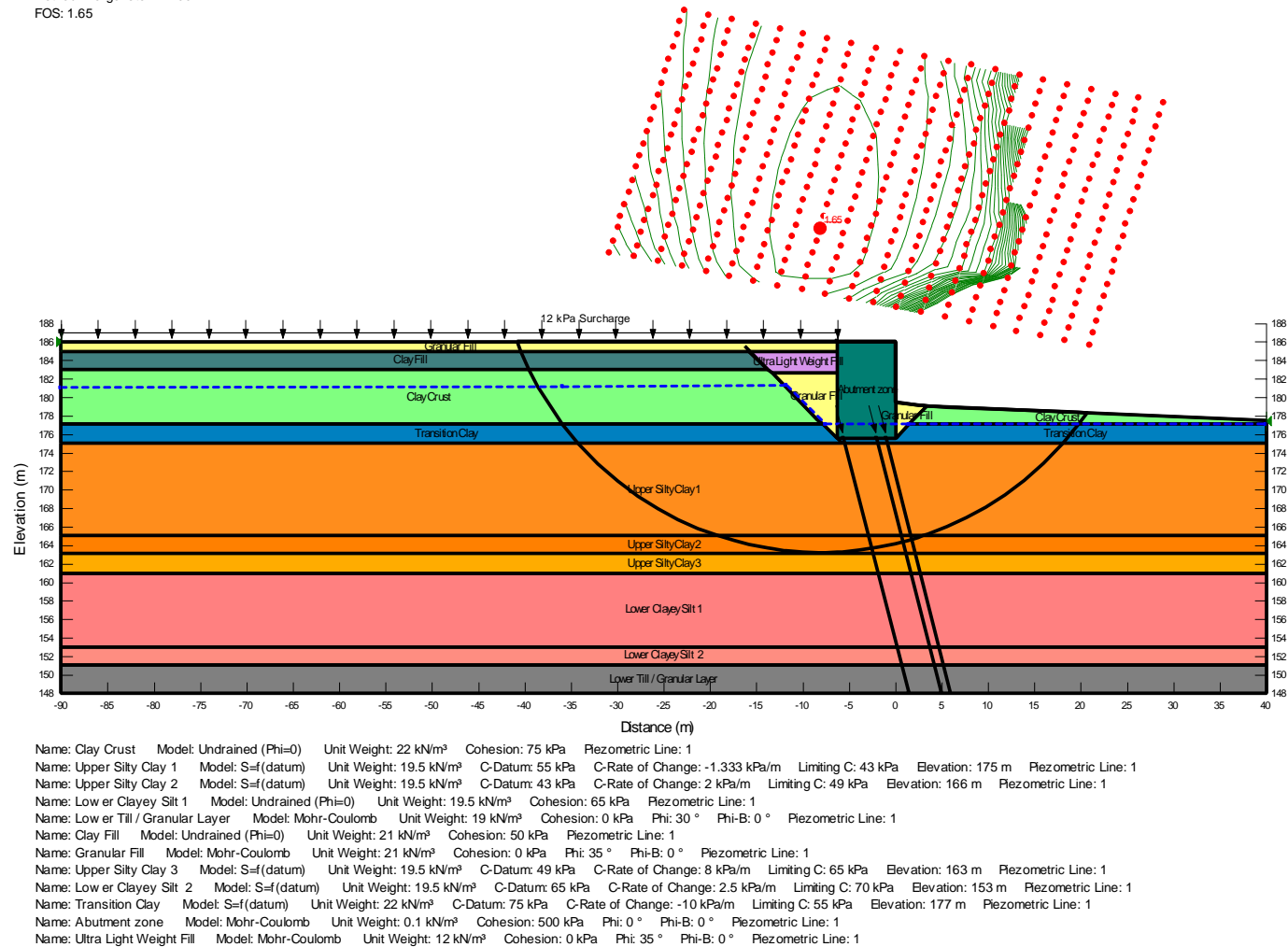


Name: Lower Till / Granular Layer Unit Weight: 19 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30 °  
Name: Granular Fill Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: RSS Wall Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: RGM Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: Ultra light weight fill Unit Weight: 12 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35 °  
Name: Drained Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30 °

**Figure E10. Slope Stability – Long-Term (Drained)– 30+612 East RSS Wall**



Title: Bridge 10 - North West Abutment (Longitudinal)  
Name: Undrained - EOC  
Method: Morgenstern-Price  
FOS: 1.65



**Figure E11. Slope Stability – End of Construction (Undrained)– NW Abutment Longitudinal**



Name: Low er Till / Granular Layer Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1  
 Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1  
 Name: Abutment zone Model: Mohr-Coulomb Unit Weight: 0.1 kN/m³ Cohesion: 500 kPa Phi: 0 ° Phi-B: 0 ° Piezometric Line: 1  
 Name: Ultra Light Weight Fill Model: Mohr-Coulomb Unit Weight: 12 kN/m³ Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 ° Piezometric Line: 1  
 Name: Drained Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 ° Phi-B: 0 ° Piezometric Line: 1

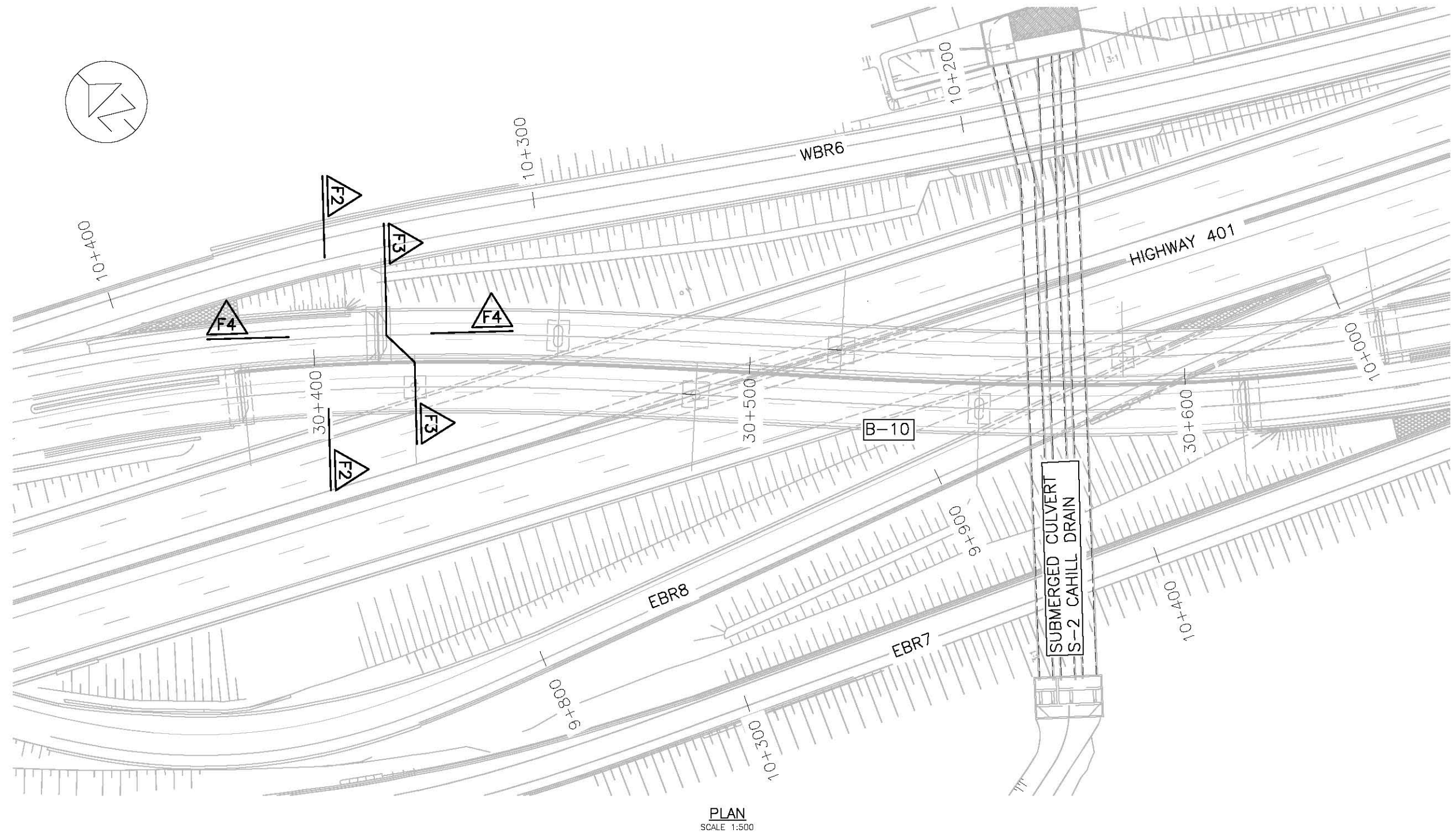
**Figure E12. Slope Stability – Long-Term (Drained)– NW Abutment Longitudinal**

## **Appendix F      Stress-Deformation Analyses**

**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018

**Date:** March/2012  
**Rev:** 0  
**Page No.:** Appendix F

DOC: STRESS DEFORMATION FOR ANALYZED SECTIONS FIG F-1

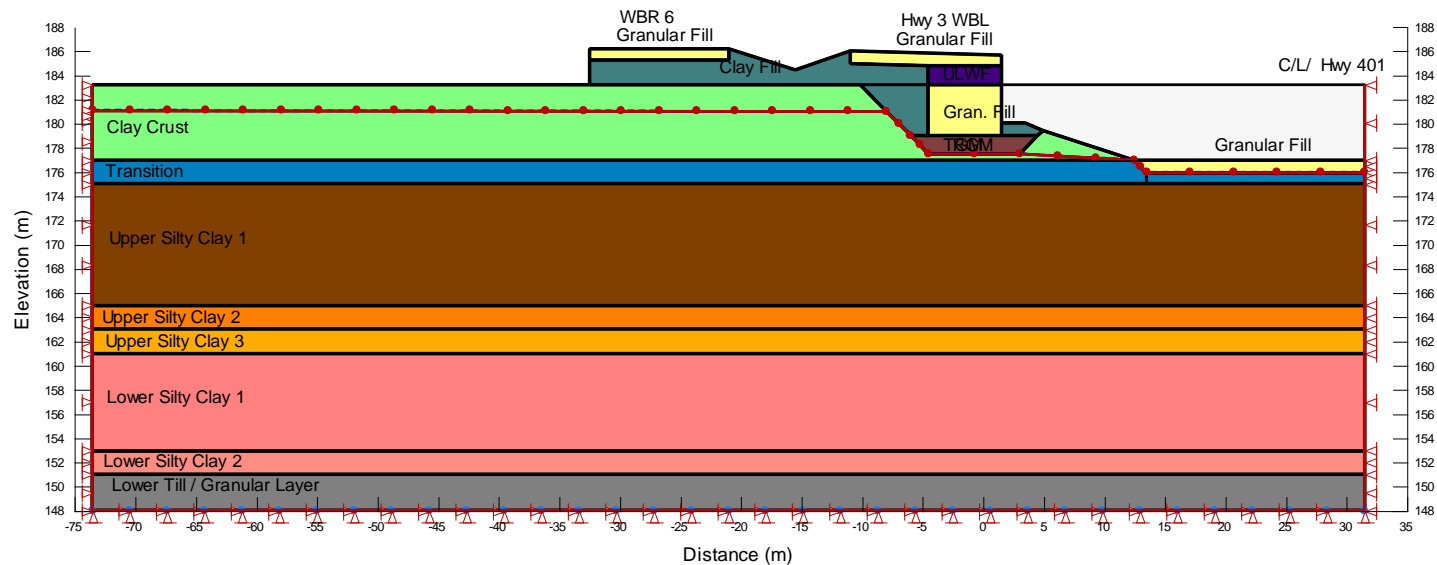


NOT FOR  
CONSTRUCTION

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

Title: Bridge 10 - West Abutment RSS Wall  
Name: Hwy 401 Road Fill  
Method: Coupled Stress/PWP

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 1.57 Poisson's Ratio: 0.35 Lambda: 0.089 Kappa: 0.0098 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.078 Kappa: 0.0086 Initial Void Ratio: 0.59 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Till / Granular Layer Model: Linear Elastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Unit Weight: 19 kN/m<sup>3</sup>  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: RGM Model: Elastic-Plastic Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 3 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.68 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.14 Kappa: 0.016 Initial Void Ratio: 1.05 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: transition Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Ultra light weight fill Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.35

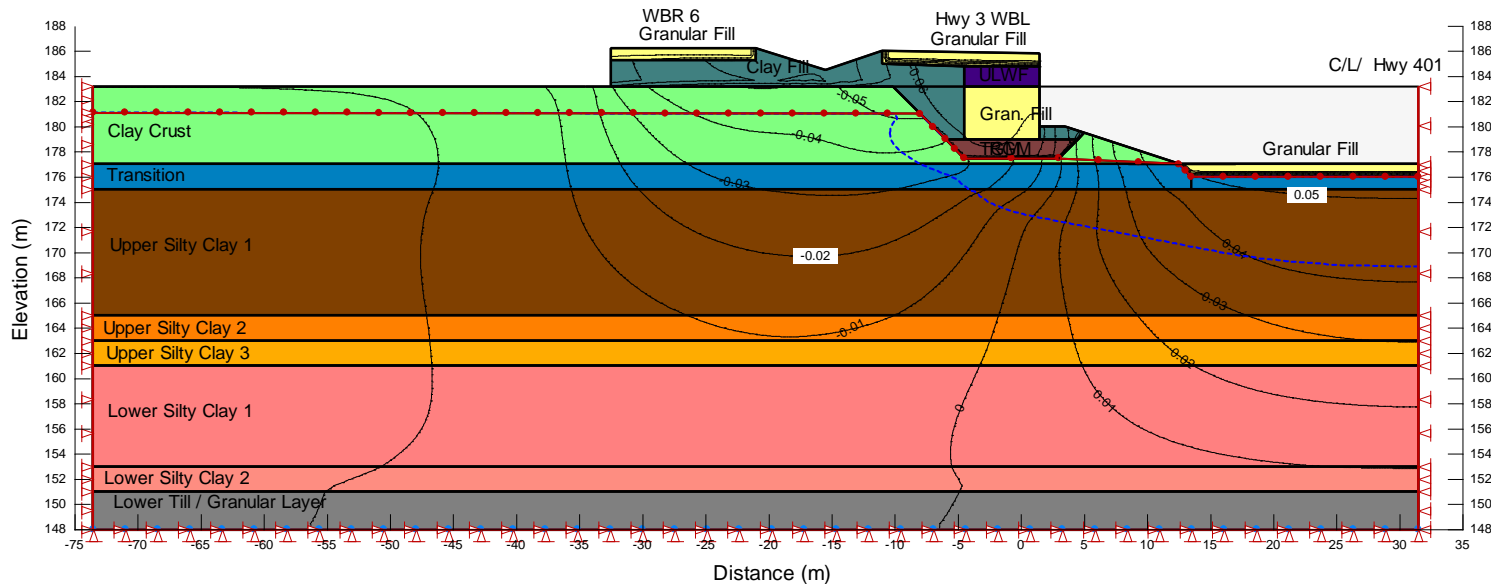


**Figure F2. West RSS Wall Finite Element Model – Drained Analysis**



Title: Bridge 10 - West Abutment RSS Wall  
Name: Hwy 401 Road Fill  
Method: Coupled Stress/PWP

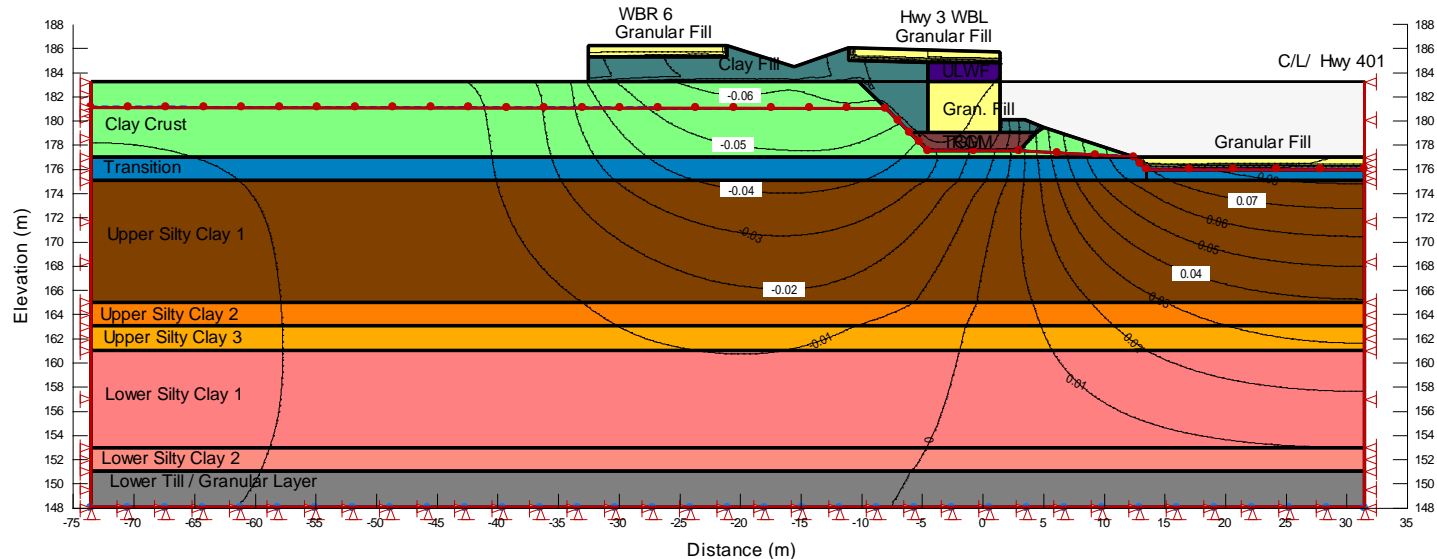
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 1.57 Poisson's Ratio: 0.35 Lambda: 0.089 Kappa: 0.0098 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.078 Kappa: 0.0086 Initial Void Ratio: 0.59 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 2 °  
Name: Lower Till / Granular Layer Model: Linear Elastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Unit Weight: 19 kN/m<sup>3</sup>  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: RGM Model: Elastic-Plastic Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 3 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.68 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.14 Kappa: 0.016 Initial Void Ratio: 1.05 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: transition Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Ultra light weight fill Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.35



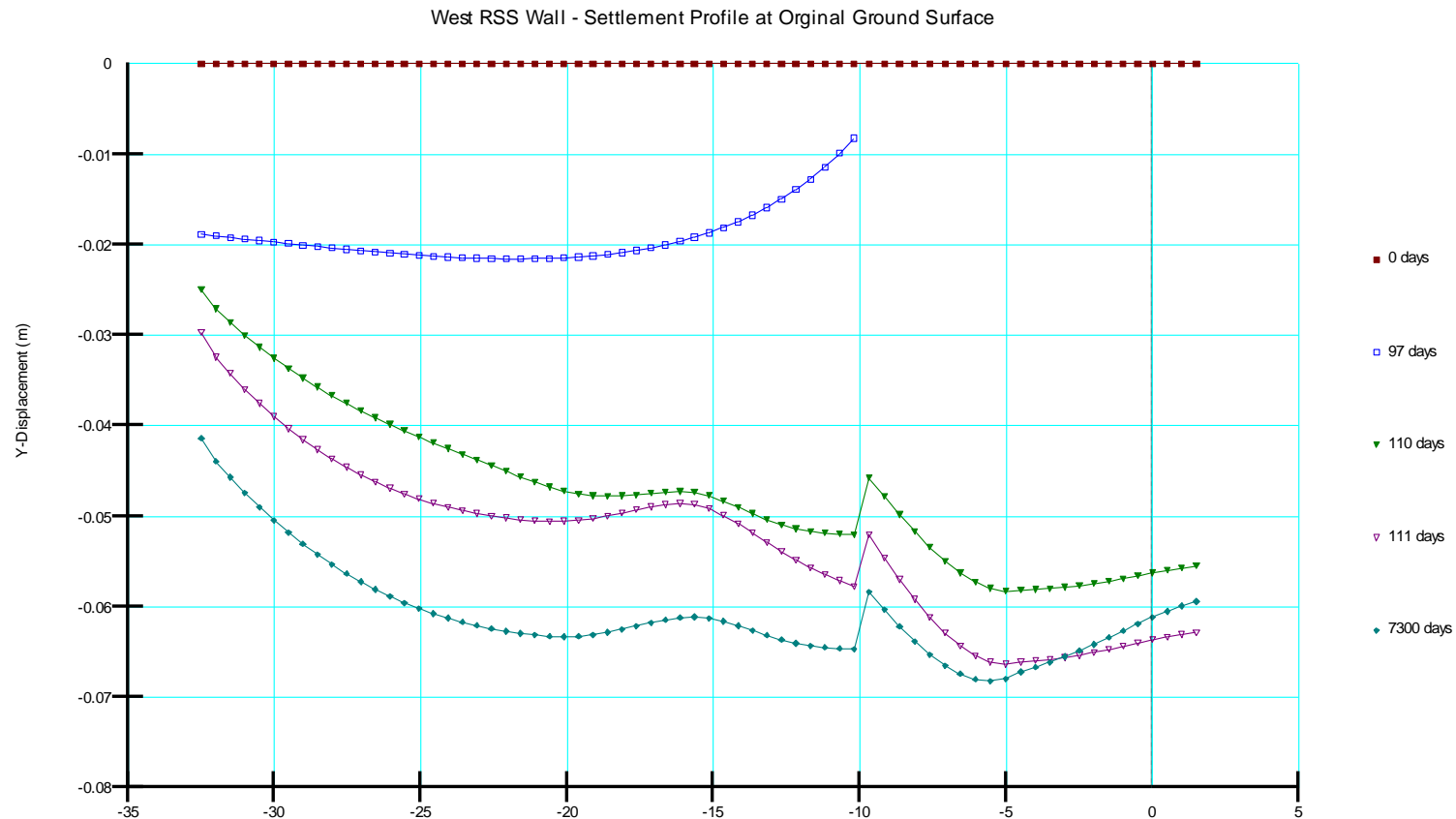
**Figure F2-1. West RSS Wall Configuration – Drained – End of Construction Cumulative Vertical Deformations**

Title: Bridge 10 - West Abutment RSS Wall  
Name: Hwy 401 Road Fill  
Method: Coupled Stress/PWP

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 1.57 Poisson's Ratio: 0.35 Lambda: 0.089 Kappa: 0.0098 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.078 Kappa: 0.0086 Initial Void Ratio: 0.59 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Till / Granular Layer Model: Linear Elastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Unit Weight: 19 kN/m<sup>3</sup>  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: RGM Model: Elastic-Plastic Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 3 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.68 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.14 Kappa: 0.016 Initial Void Ratio: 1.05 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: transition Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Ultra light weight fill Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.35

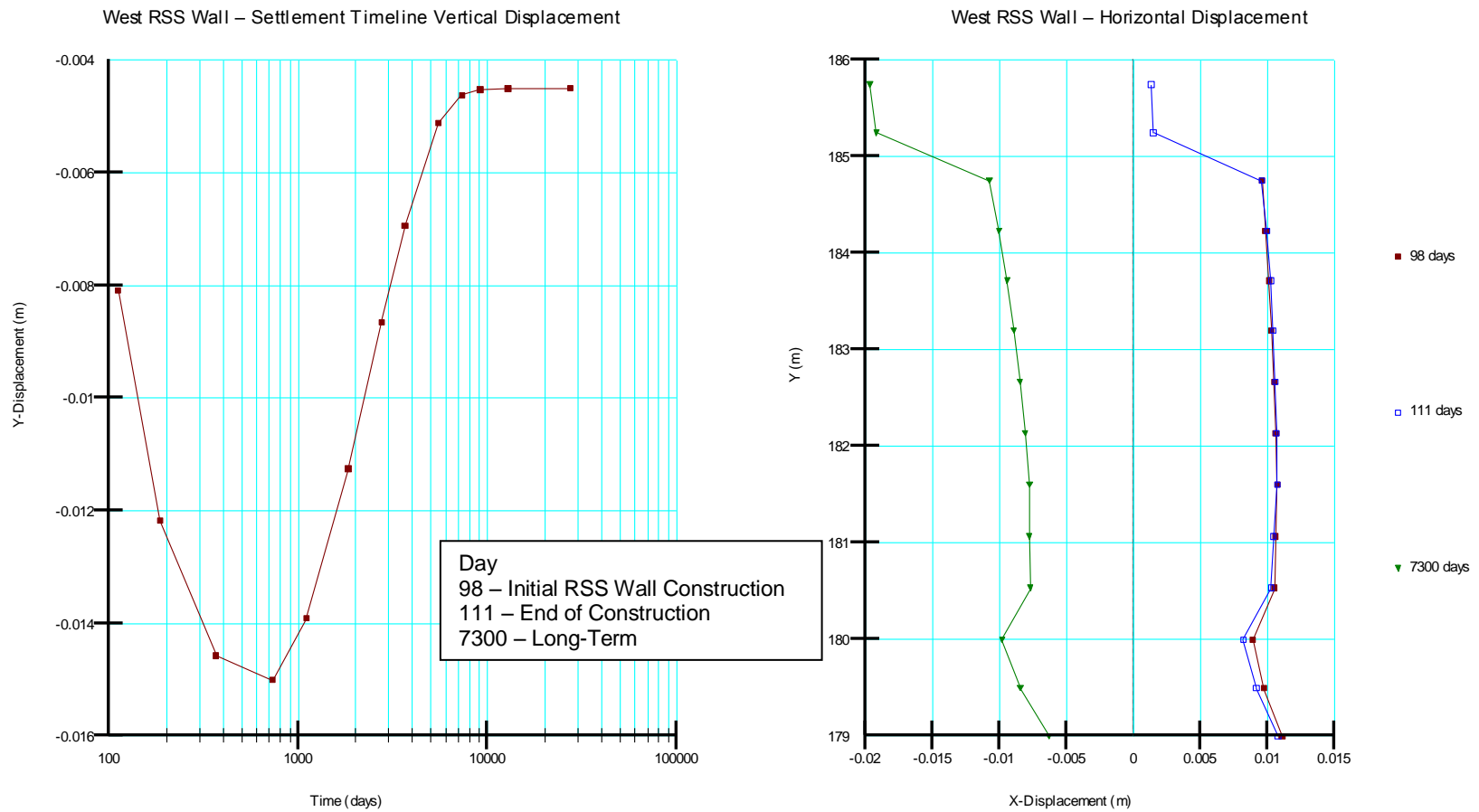


**Figure F2-2. West RSS Wall Configuration – Drained Long-Term Cumulative Vertical Deformations**



Day  
0 – Existing condition  
97 – Excavation  
110 – Final RSS Wall Construction – prior to Roadway Subgrade  
111 – End of Construction  
7300 – Long-Term

**Figure F2-3. West RSS Wall Configuration – Drained Settlement Profiles Top of Wall Vertical**

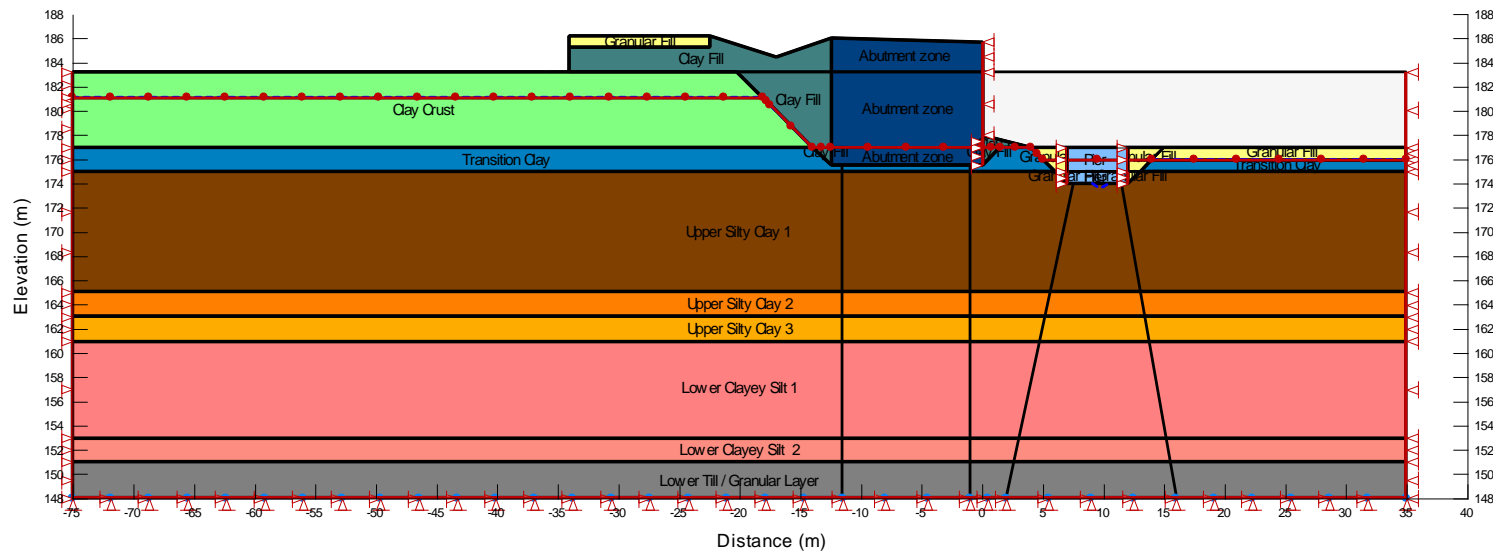


**Figure F2-4. West RSS Wall Settlement and Horizontal Displacement**



Title: Bridge 10 - Station 11+025 North West Abutment (Transverse)  
Name: Hwy 401 Road Fill  
Method: Coupled Stress/PWP  
Elapsed Time: 9125 days

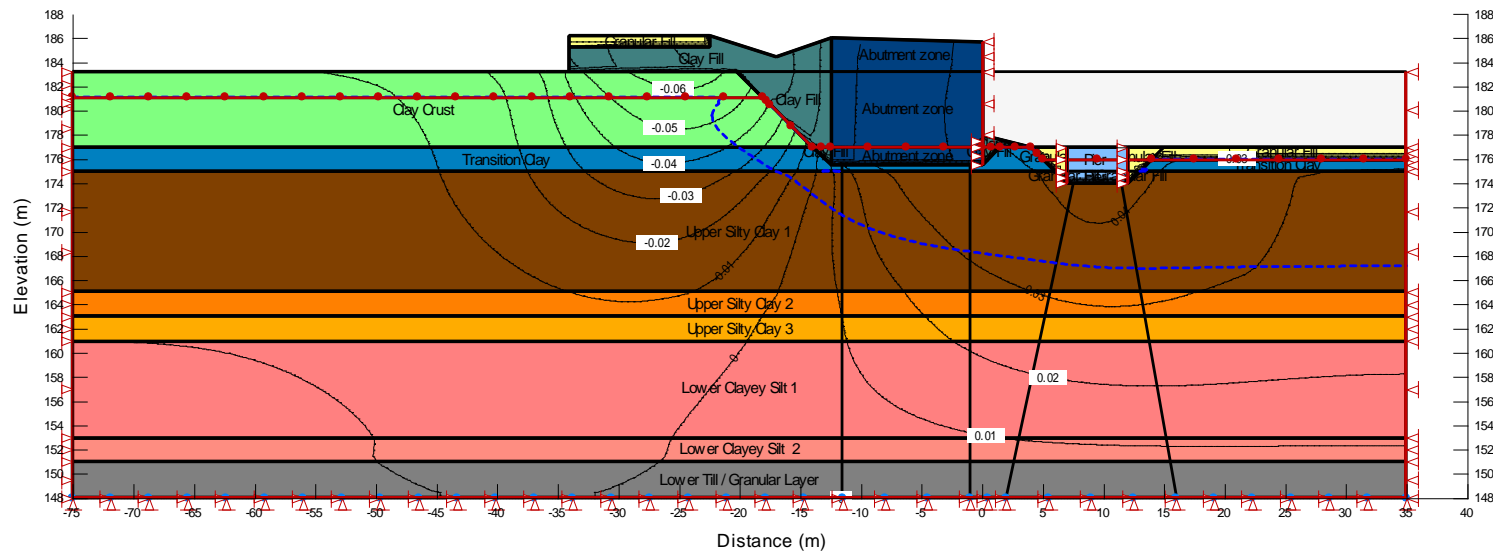
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 1.57 Poisson's Ratio: 0.35 Lambda: 0.089 Kappa: 0.0098 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.078 Kappa: 0.0086 Initial Void Ratio: 0.59 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Till / Granular Layer Model: Linear Elastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Unit Weight: 19 kN/m<sup>3</sup>  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Upper Silty Clay 3 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.68 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.14 Kappa: 0.016 Initial Void Ratio: 1.05 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: transition Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Abutment zone Model: Linear Elastic Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Pier Model: Linear Elastic Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m<sup>3</sup> Poisson's Ratio: 0.35



**Figure F3 Bridge 10 NW Abutment (Transverse) Finite Element Model – Drained Analysis**

Title: Bridge 10 - Station 11+025 North West Abutment (Transverse)  
Name: Hwy 401 Road Fill  
Method: Coupled Stress/PWP  
Elapsed Time: 111 days

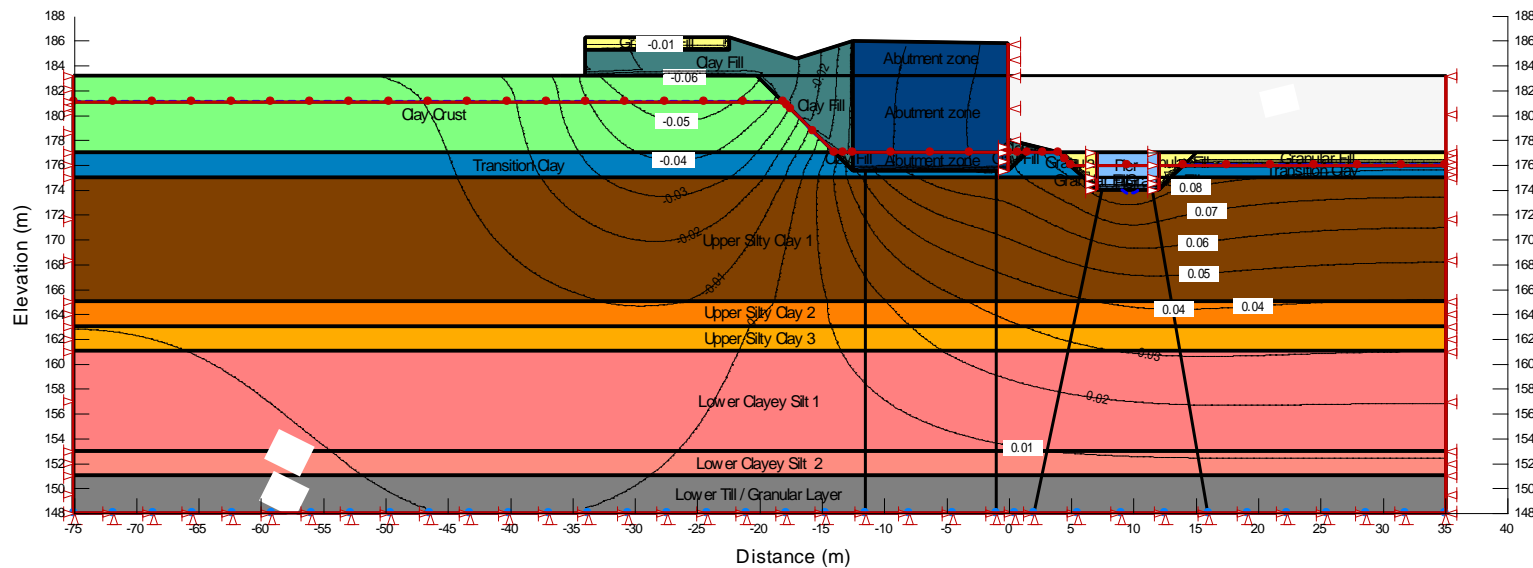
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 1.57 Poisson's Ratio: 0.35 Lambda: 0.089 Kappa: 0.0098 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.078 Kappa: 0.0086 Initial Void Ratio: 0.59 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Till / Granular Layer Model: Linear Elastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Unit Weight: 19 kN/m<sup>3</sup>  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Upper Silty Clay 3 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.68 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.14 Kappa: 0.016 Initial Void Ratio: 1.05 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: transition Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Abutment zone Model: Linear Elastic Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Pier Model: Linear Elastic Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m<sup>3</sup> Poisson's Ratio: 0.35



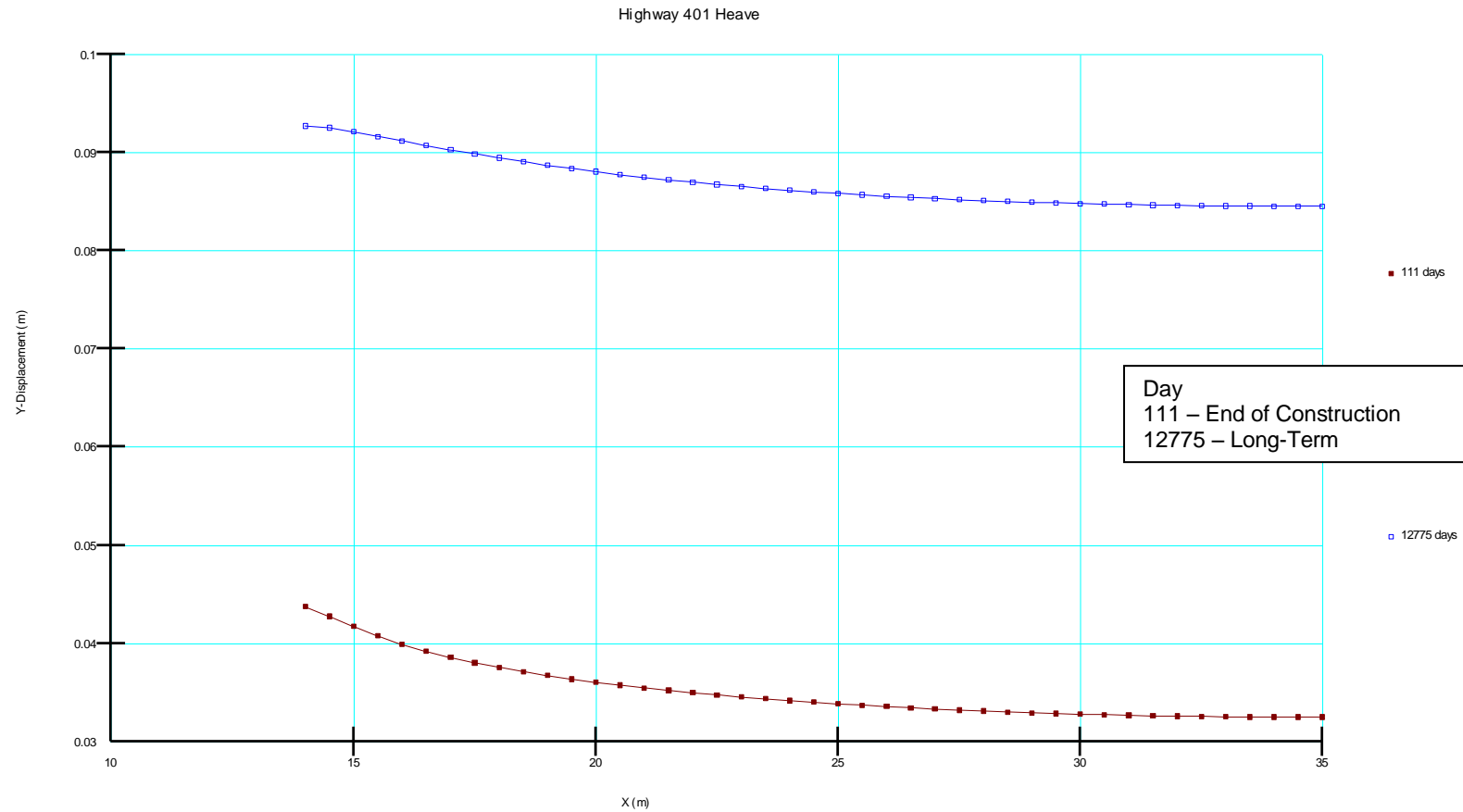
**Figure F3-1. NW Abutment - Transverse - End of Construction (111 days) Cumulative Vertical Deformations**

Title: Bridge 10 - Station 11+025 North West Abutment (Transverse)  
Name: Hwy 401 Road Fill  
Method: Coupled Stress/PWP  
Elapsed Time: 9125 days

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 1.57 Poisson's Ratio: 0.35 Lambda: 0.089 Kappa: 0.0098 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.078 Kappa: 0.0086 Initial Void Ratio: 0.59 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Till / Granular Layer Model: Linear Elastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Unit Weight: 19 kN/m<sup>3</sup>  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Upper Silty Clay 3 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.68 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.14 Kappa: 0.016 Initial Void Ratio: 1.05 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: transition Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Abutment zone Model: Linear Elastic Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Pier Model: Linear Elastic Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m<sup>3</sup> Poisson's Ratio: 0.35



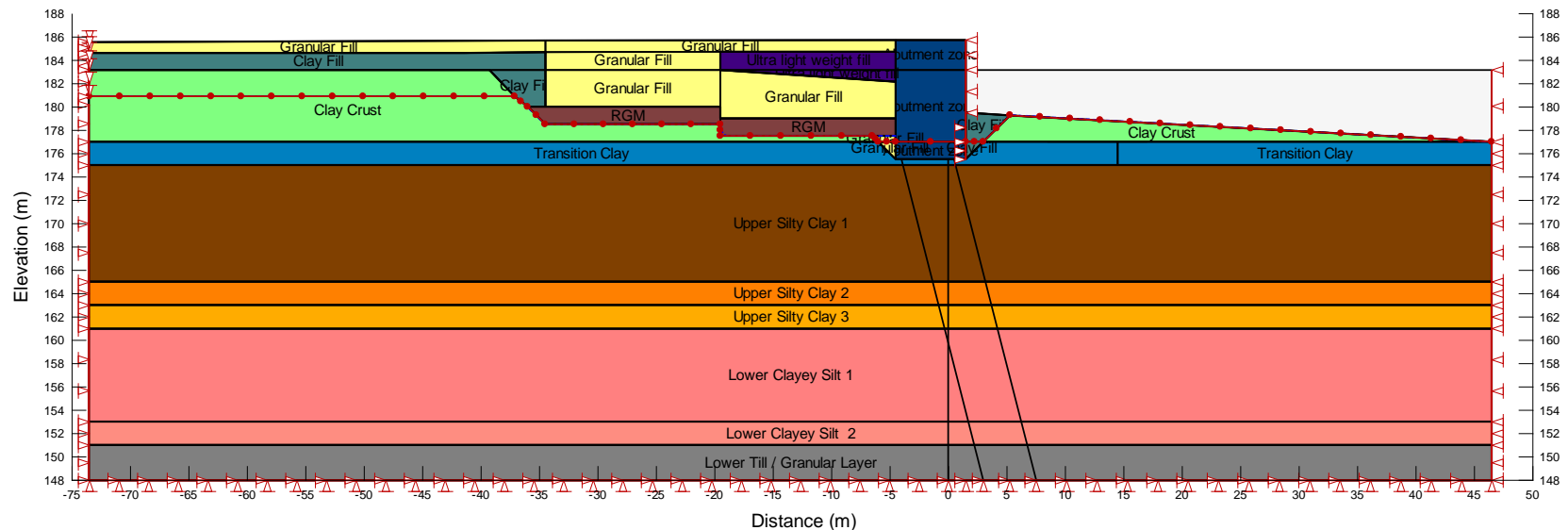
**Figure F3-2. NW Abutment - Transverse - Long-Term (9125 days) Cumulative Vertical Deformations**



**Figure F3-3 NW Abutment - Transverse - Highway 401 Heave**

Title: Bridge 10 - Station 11+025 North West Abutment (Transverse)  
Name: Hwy 401 Road Fill  
Method: Coupled Stress/PWP

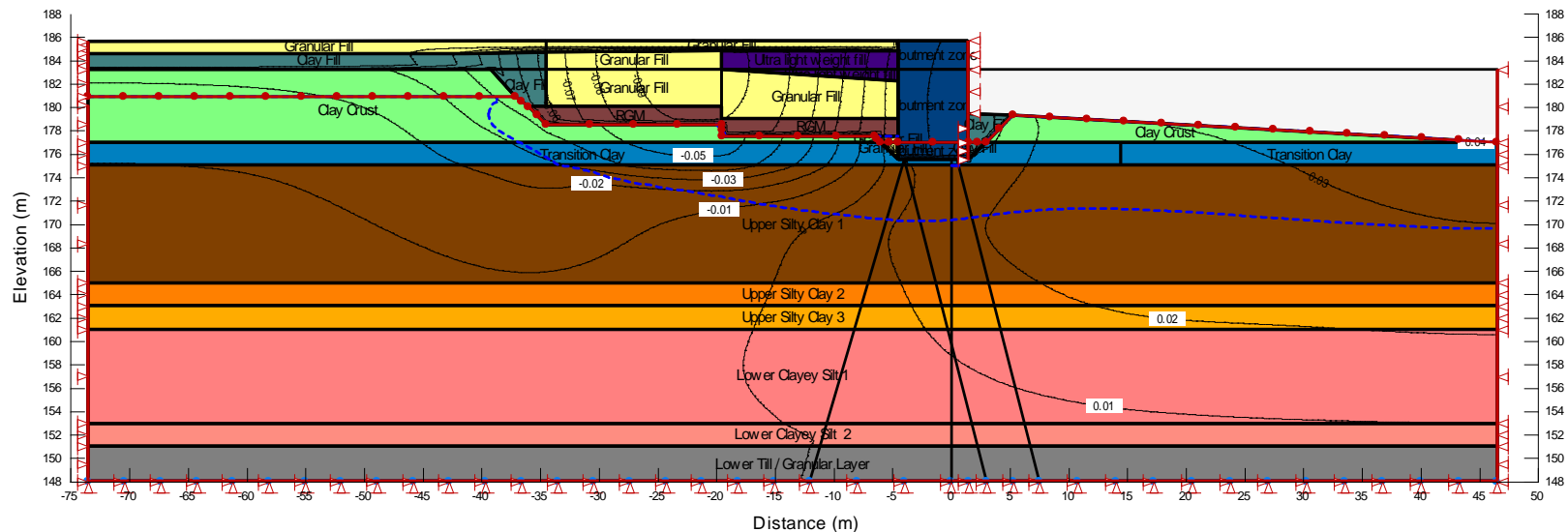
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 1.57 Poisson's Ratio: 0.35 Lambda: 0.089 Kappa: 0.0098 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.078 Kappa: 0.0086 Initial Void Ratio: 0.59 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Till / Granular Layer Model: Linear Elastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Unit Weight: 19 kN/m<sup>3</sup>  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: RGM Model: Elastic-Plastic Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 3 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.68 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.14 Kappa: 0.016 Initial Void Ratio: 1.05 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: transition Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Ultra light weight fill Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Abutment zone Model: Linear Elastic Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m<sup>3</sup> Poisson's Ratio: 0.35



**Figure F4 – NW Abutment – Longitudinal - Finite Element Model – Drained Analysis**

Title: Bridge 10 - Station 11+025 North West Abutment (Transverse)  
Name: Hwy 401 Road Fill  
Method: Coupled Stress/PWP

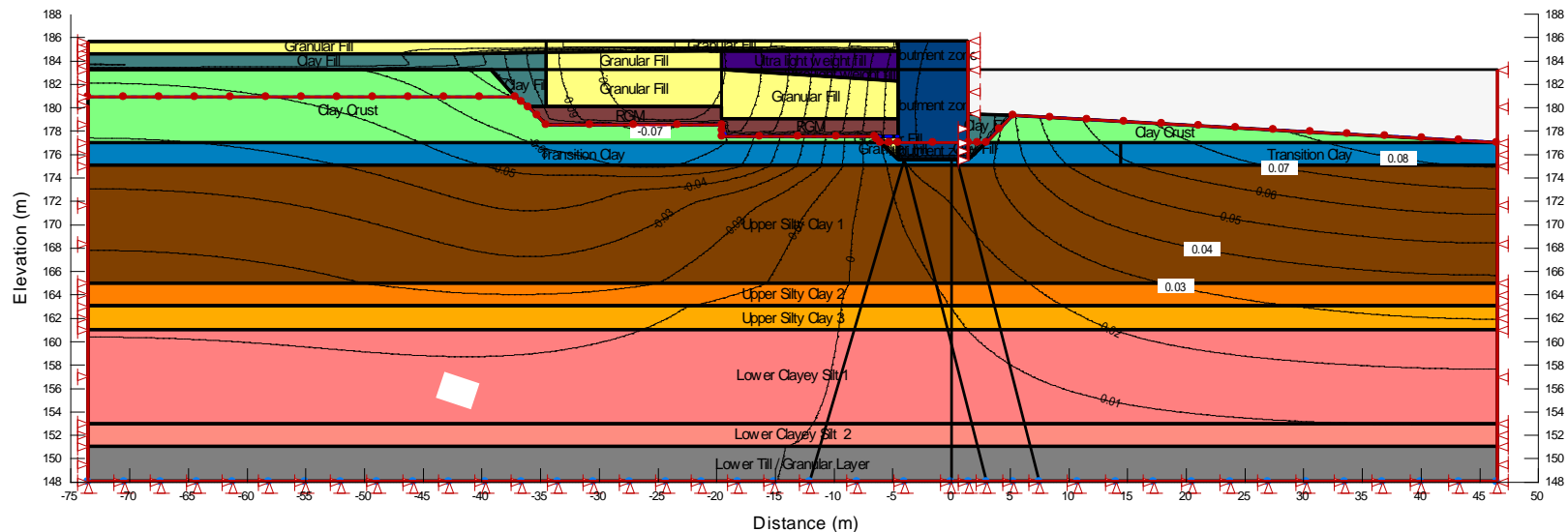
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E): 20300 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa  $\Phi_i$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 1.57 Poisson's Ratio: 0.35 Lambda: 0.089 Kappa: 0.0098 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi_i$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi_i$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.078 Kappa: 0.0086 Initial Void Ratio: 0.59 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi_i$ : 26 °  
Name: Lower Till / Granular Layer Model: Linear Elastic Effective Young's Modulus (E): 40000 kPa Poisson's Ratio: 0.35 Unit Weight: 19 kN/m<sup>3</sup>  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: RGM Model: Elastic-Plastic Effective Young's Modulus (E): 50000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa  $\Phi_i$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 3 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.68 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi_i$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.14 Kappa: 0.016 Initial Void Ratio: 1.05 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi_i$ : 26 °  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: transition Poisson's Ratio: 0.35 Cohesion: 0 kPa  $\Phi_i$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Ultra light weight fill Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Abutment zone Model: Linear Elastic Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m<sup>3</sup> Poisson's Ratio: 0.35



**Figure F4-1. NW Abutment - Longitudinal - End of Construction (111 days) Cumulative Vertical Deformations**

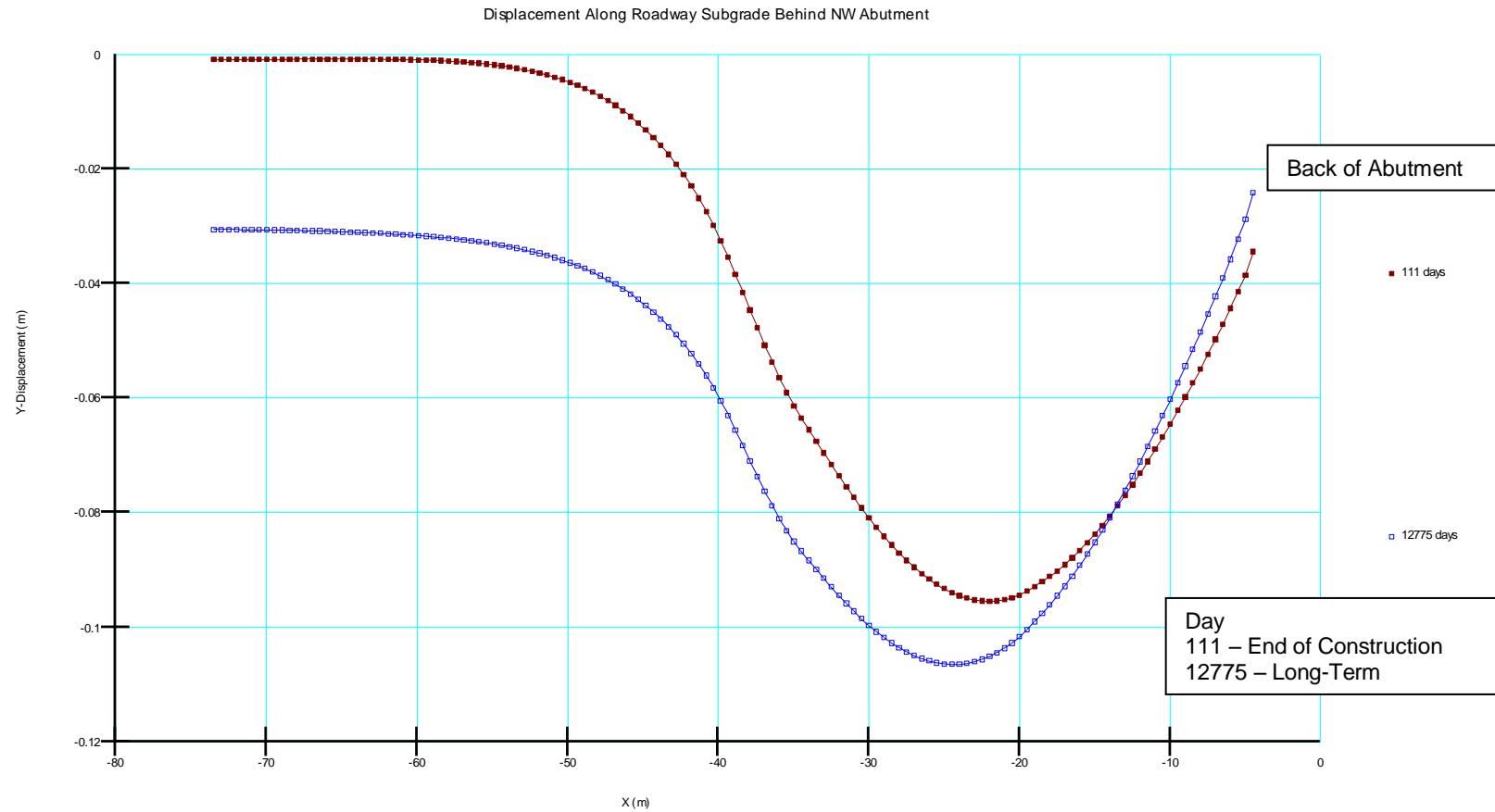
Title: Bridge 10 - Station 11+025 North West Abutment (Transverse)  
Name: Hwy 401 Road Fill  
Method: Coupled Stress/PWP

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 1.57 Poisson's Ratio: 0.35 Lambda: 0.089 Kappa: 0.0098 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.078 Kappa: 0.0086 Initial Void Ratio: 0.59 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Till / Granular Layer Model: Linear Elastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Unit Weight: 19 kN/m<sup>3</sup>  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: RGM Model: Elastic-Plastic Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 3 Model: Soft Clay (MCC) O.C. Ratio: 1.24 Poisson's Ratio: 0.35 Lambda: 0.07 Kappa: 0.0078 Initial Void Ratio: 0.68 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.14 Kappa: 0.016 Initial Void Ratio: 1.05 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: transition Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Ultra light weight fill Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Abutment zone Model: Linear Elastic Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m<sup>3</sup> Poisson's Ratio: 0.35



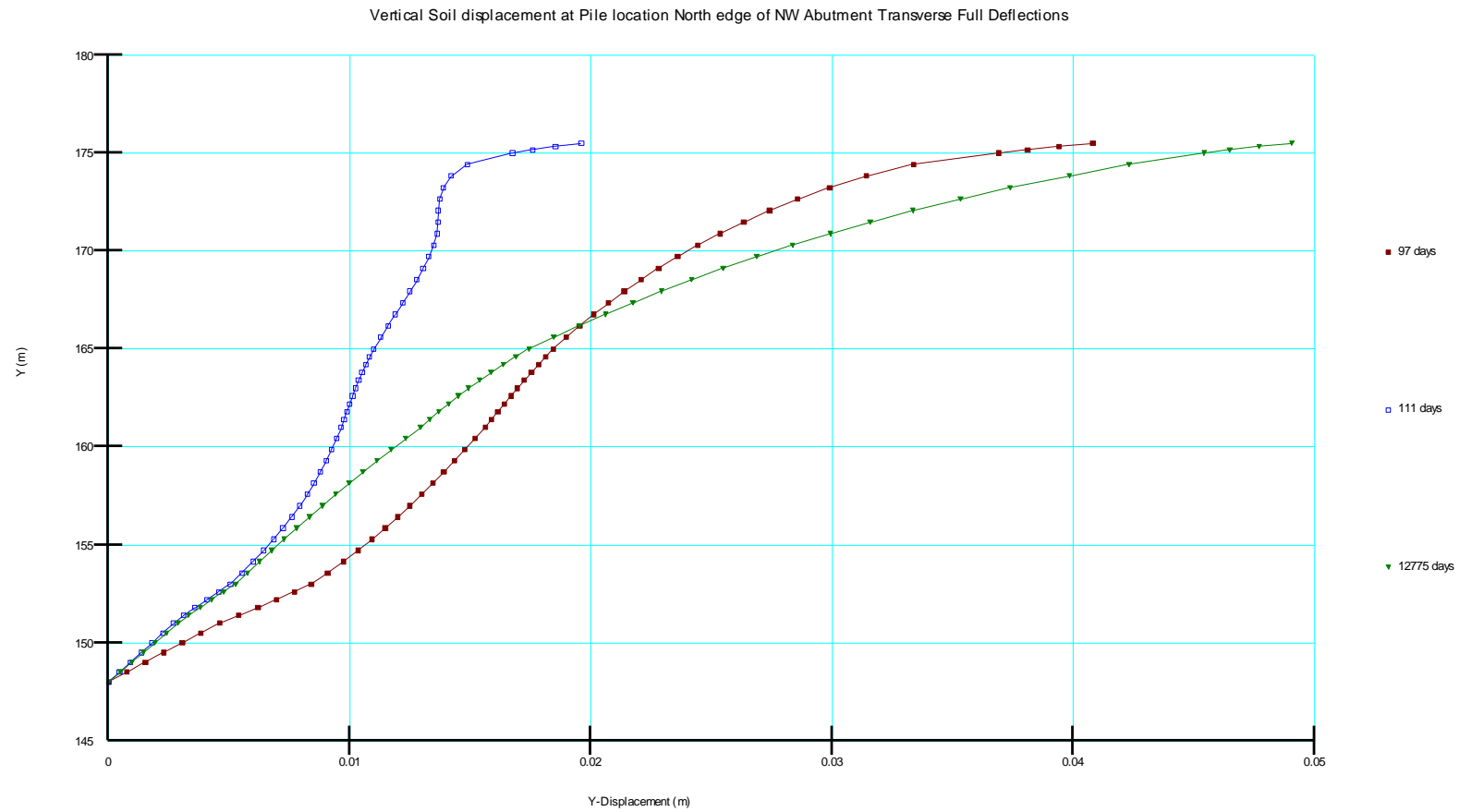
**Figure F4-2. NW Abutment - Longitudinal - Long-Term (12775 days) Cumulative Vertical Deformations**



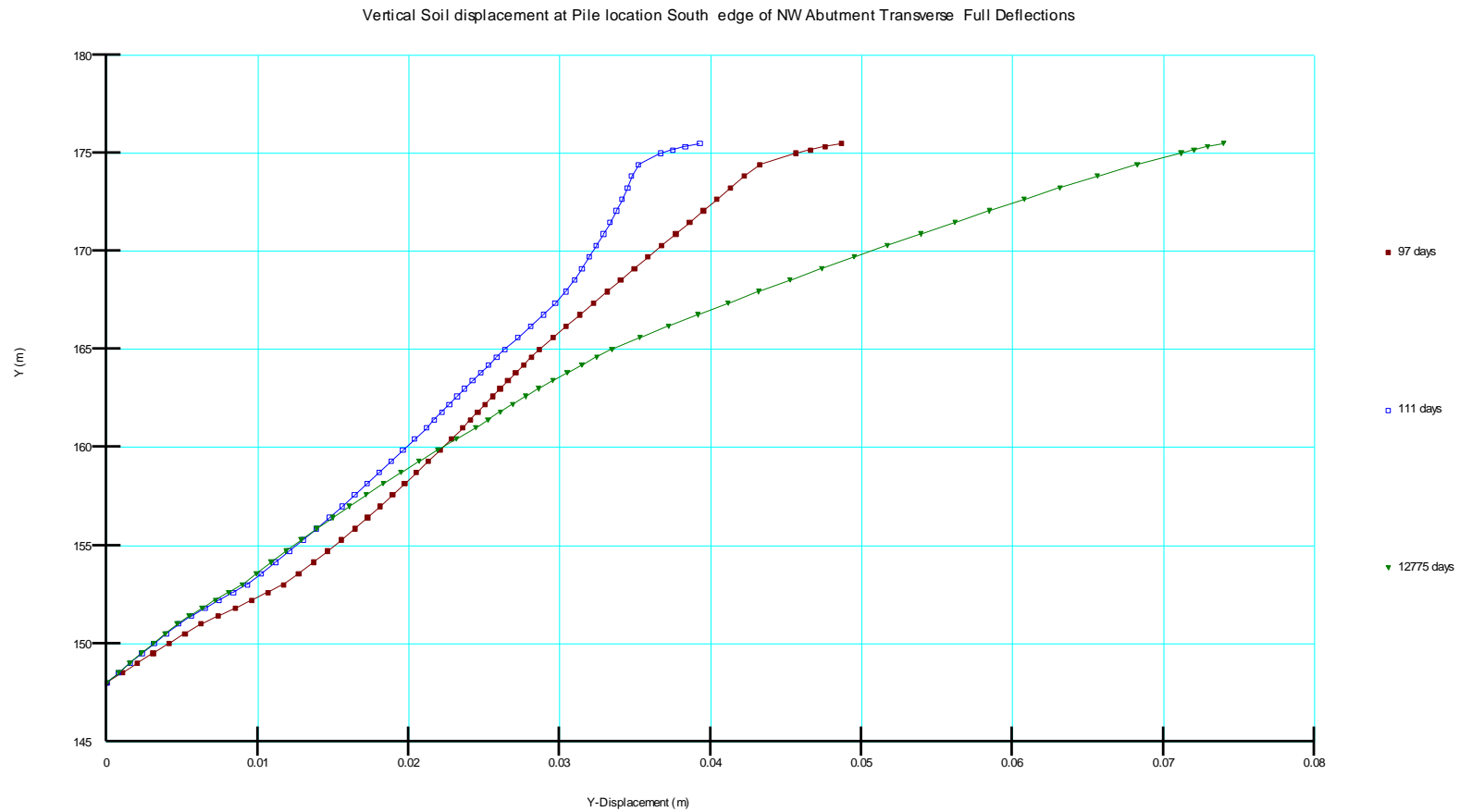


**Figure F4-3. NW Abutment - Deformation Behind Abutment**

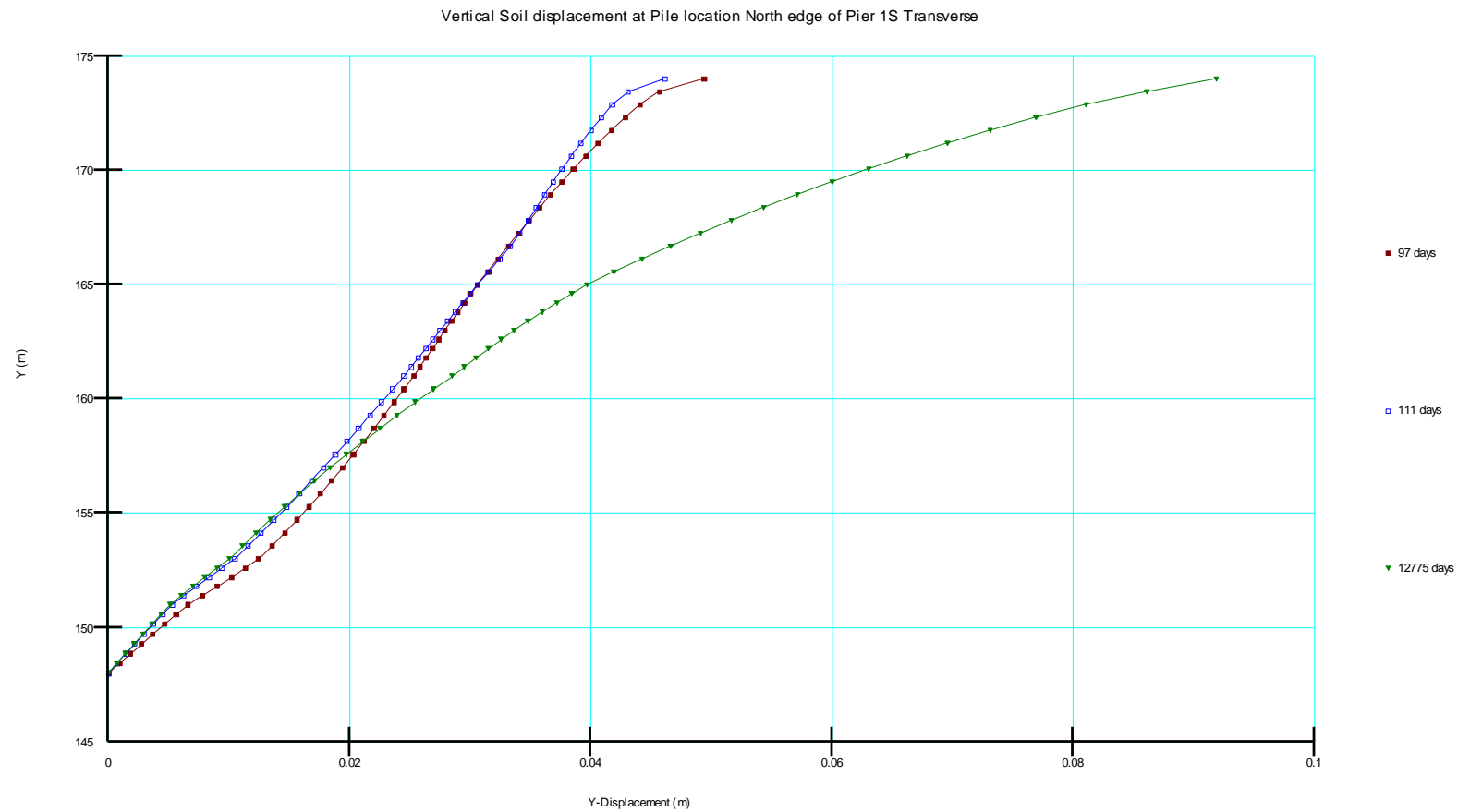




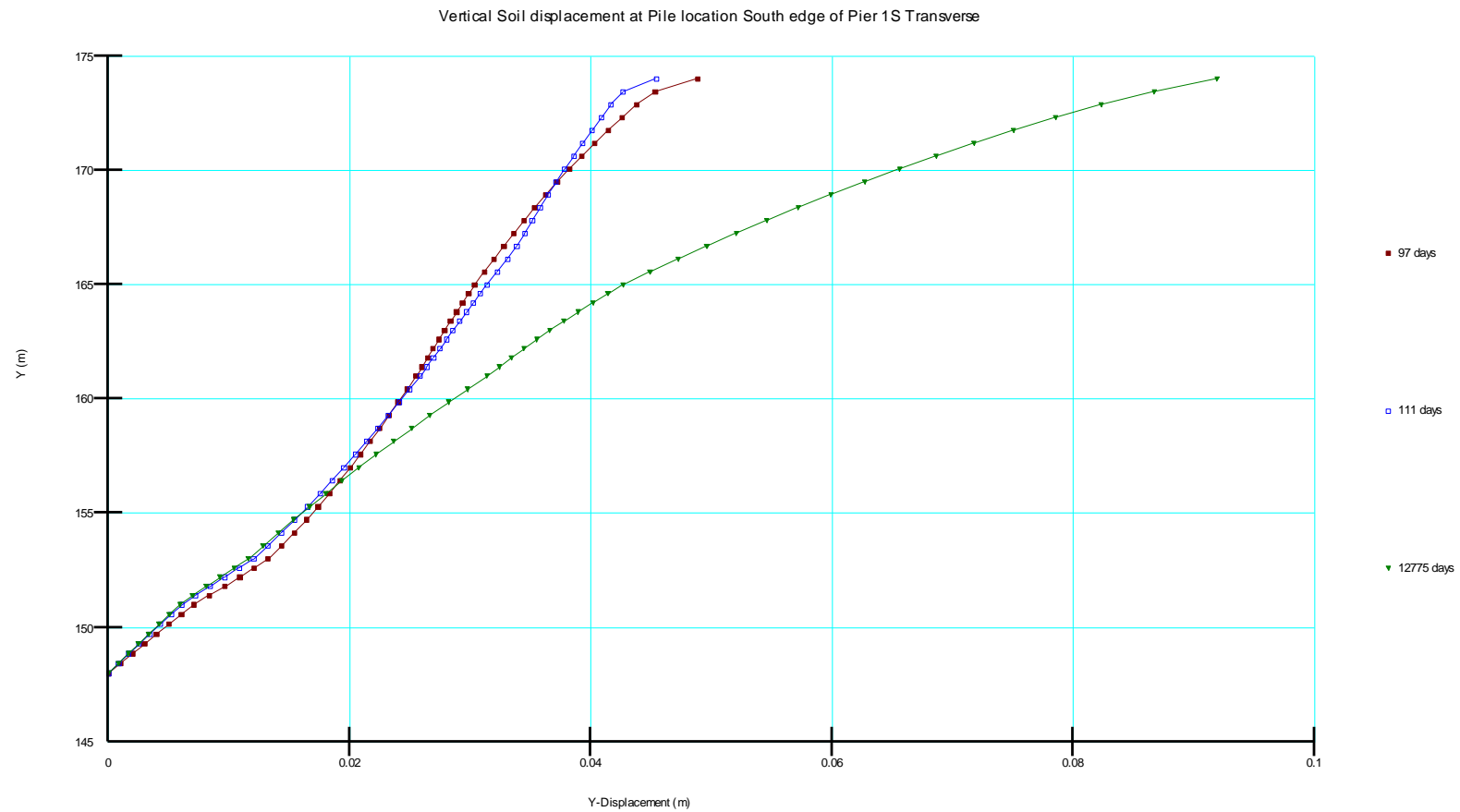
**Figure F5-1. NW Abutment Transverse Configuration – North Side Pile Cumulative Vertical Soil Deformations**



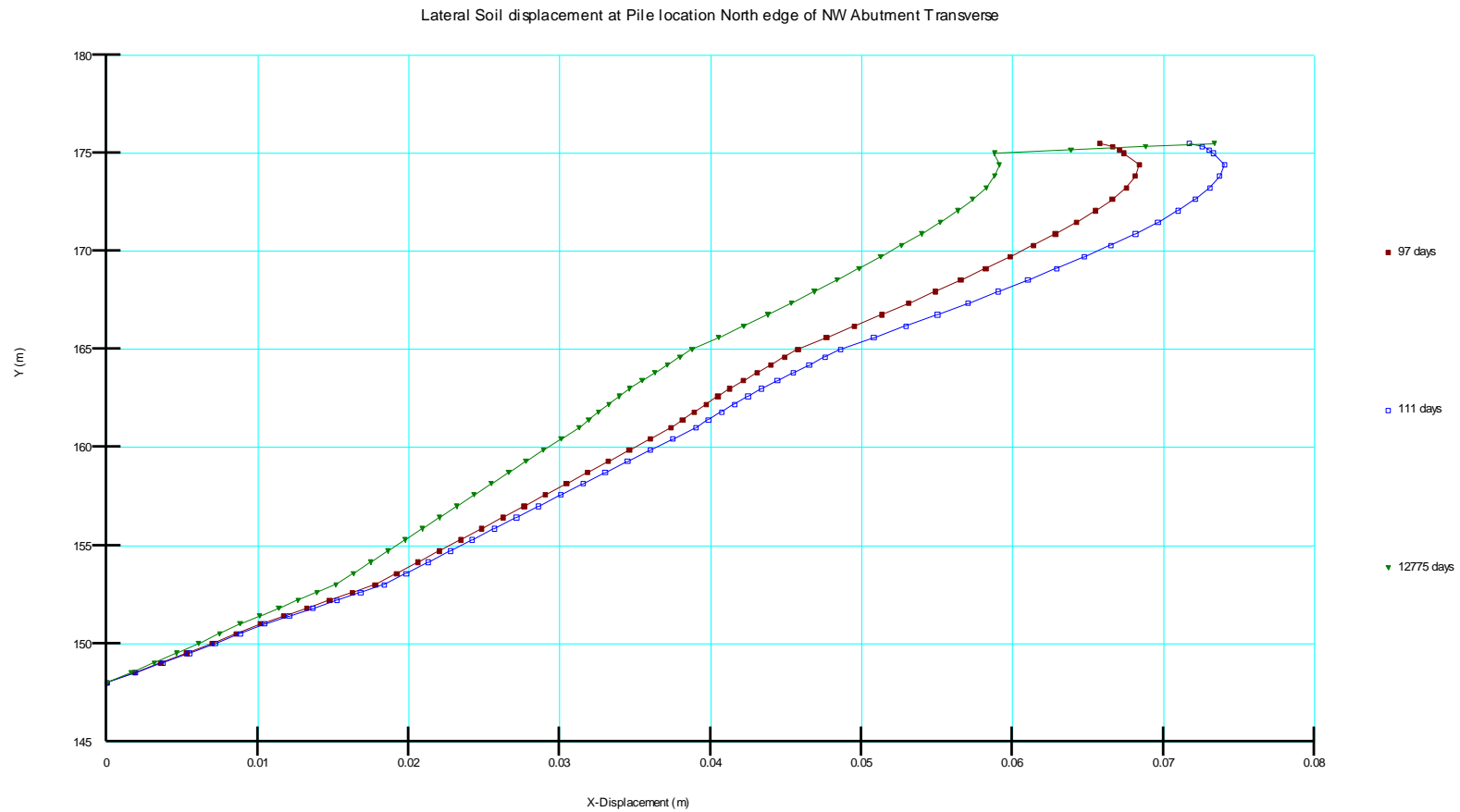
**Figure F5-2. NW Abutment Transverse Configuration – South Side Pile Cumulative Vertical Soil Deformations**



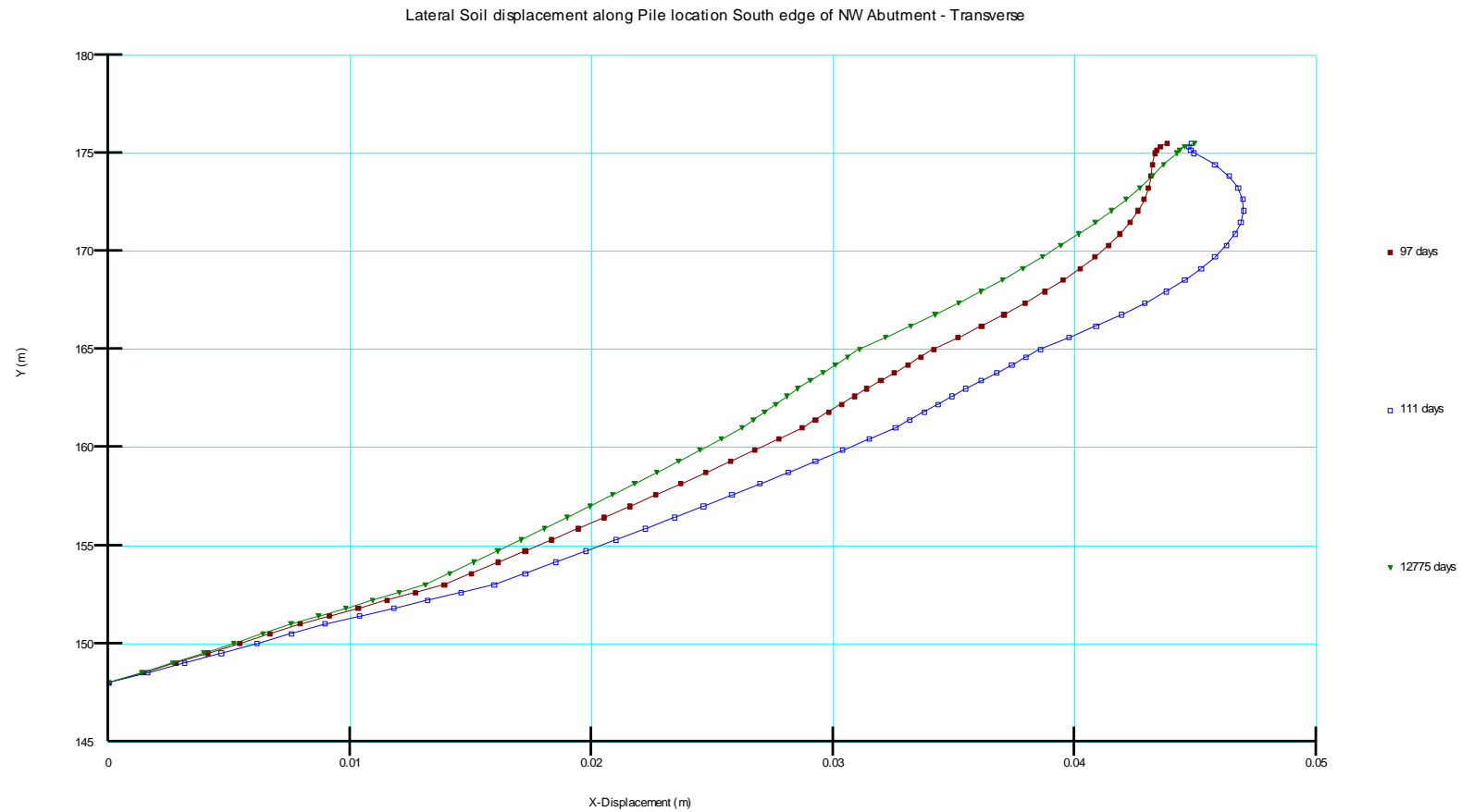
**Figure F5-3. South Pier No. 1 – North Pile Cumulative Vertical Soil Deformations**



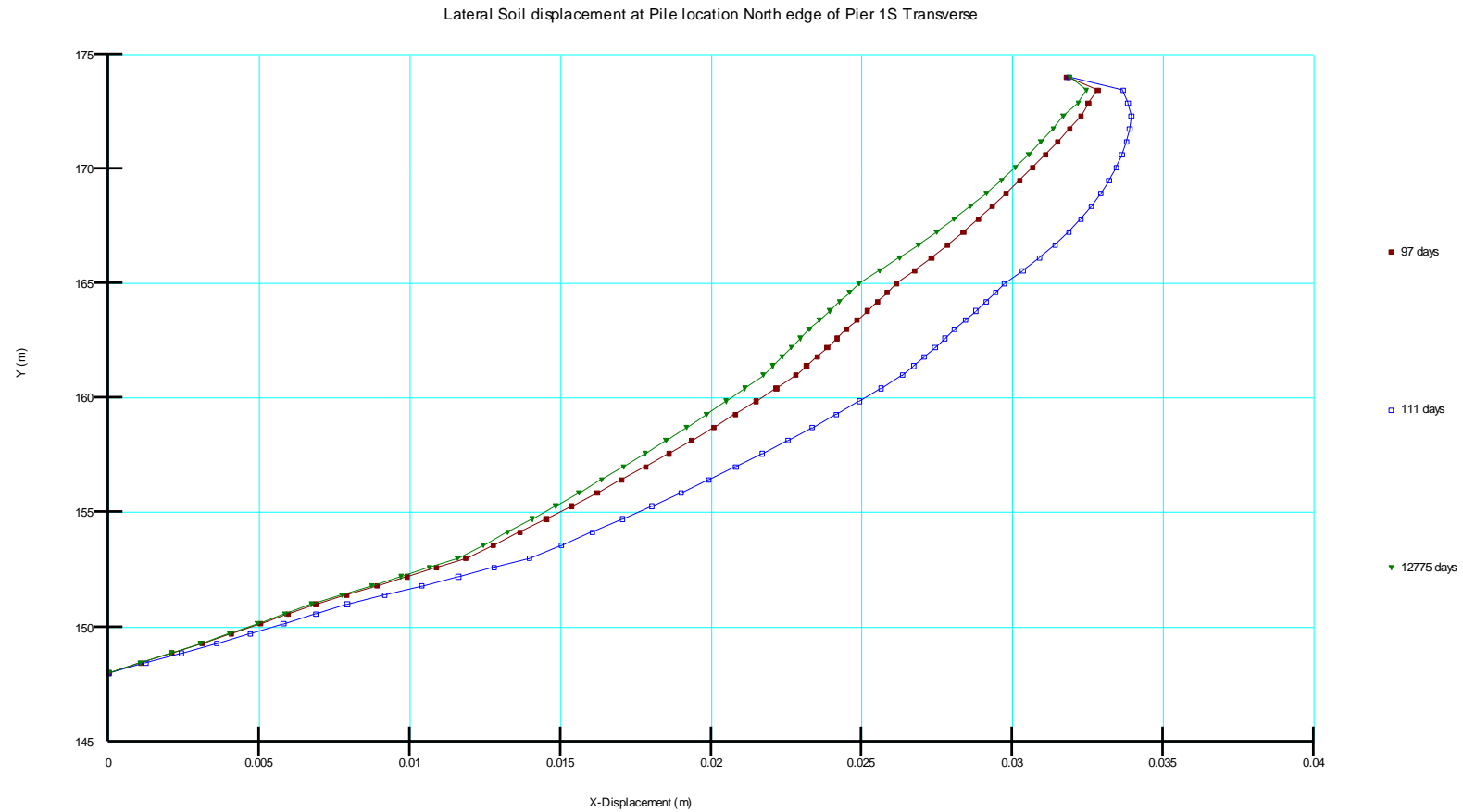
**Figure F5-4. South Pier No. 1 – South Pile Cumulative Vertical Soil Deformations**



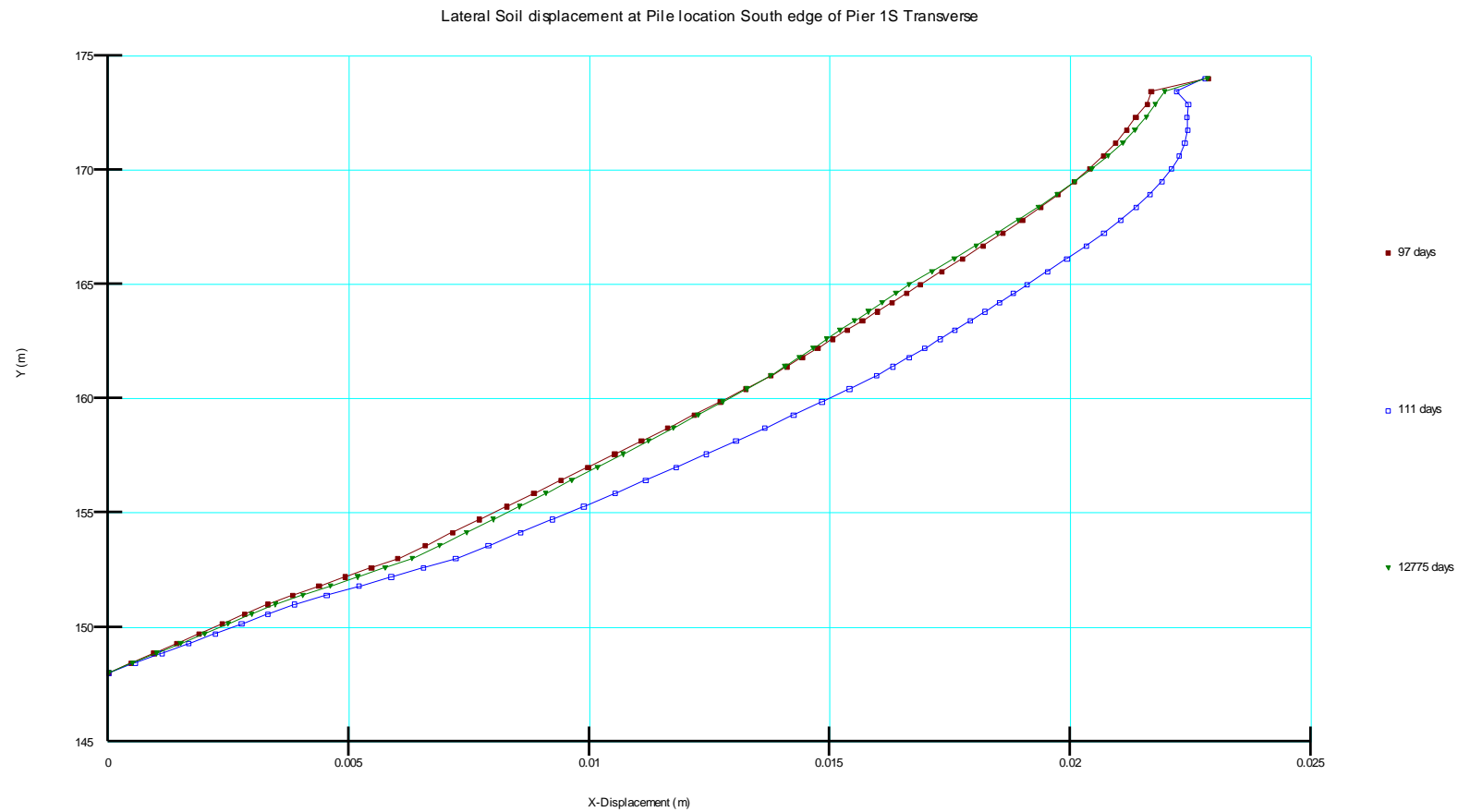
**Figure F6-1. NW Abutment Transverse Configuration – North Side Pile Lateral Cumulative Soil Deformations**



**Figure F6-2. NW Abutment Transverse Configuration – South Side Pile Lateral Cumulative Soil Deformations**



**Figure F6-3. South Pier 1 – North Side Pile Lateral Cumulative Soil Deformations**



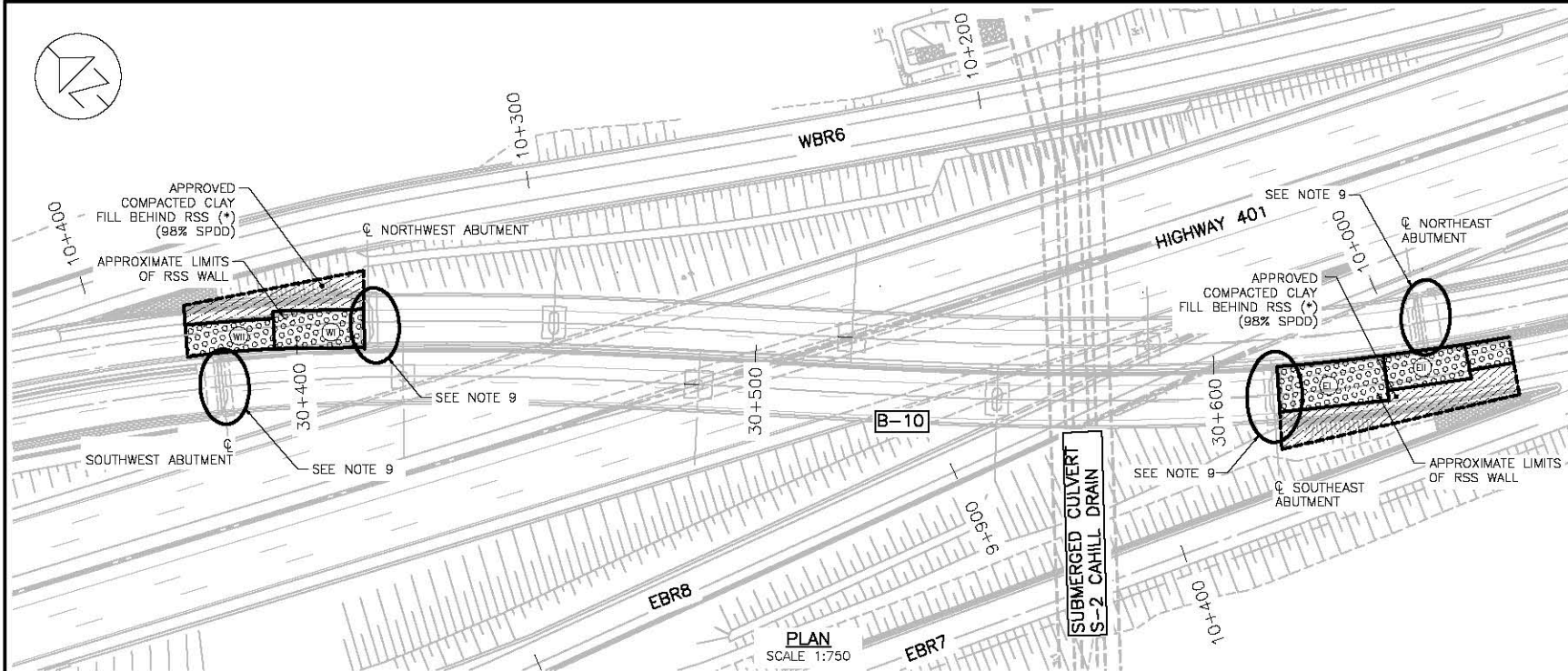
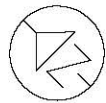
**Figure F6-4. South Pier 1 – South Side Pile Lateral Cumulative Soil Deformations**



## **Appendix G      Cross Section Conceptual Backfill Design**

**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018

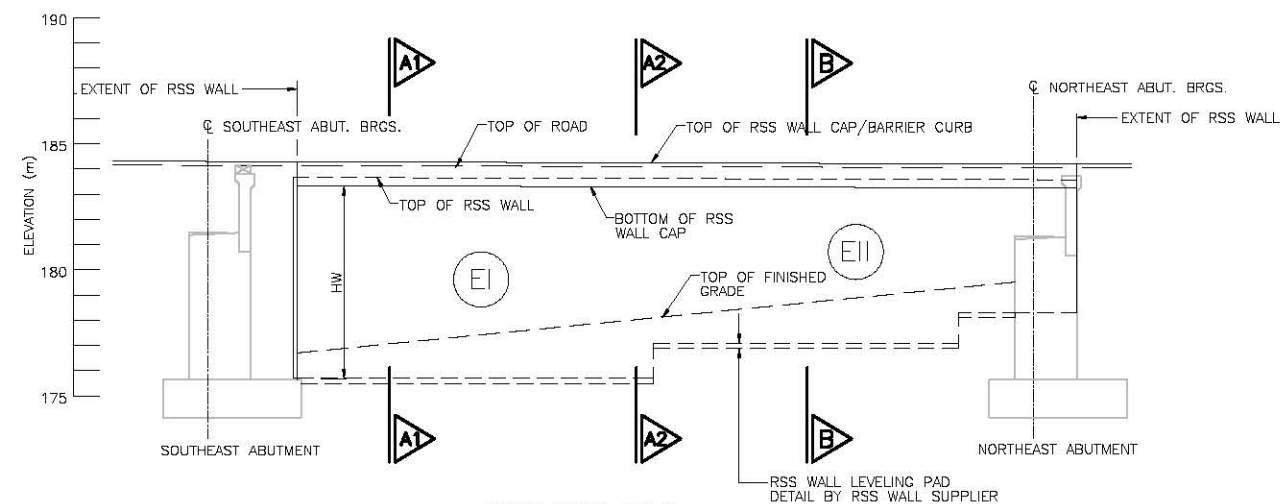
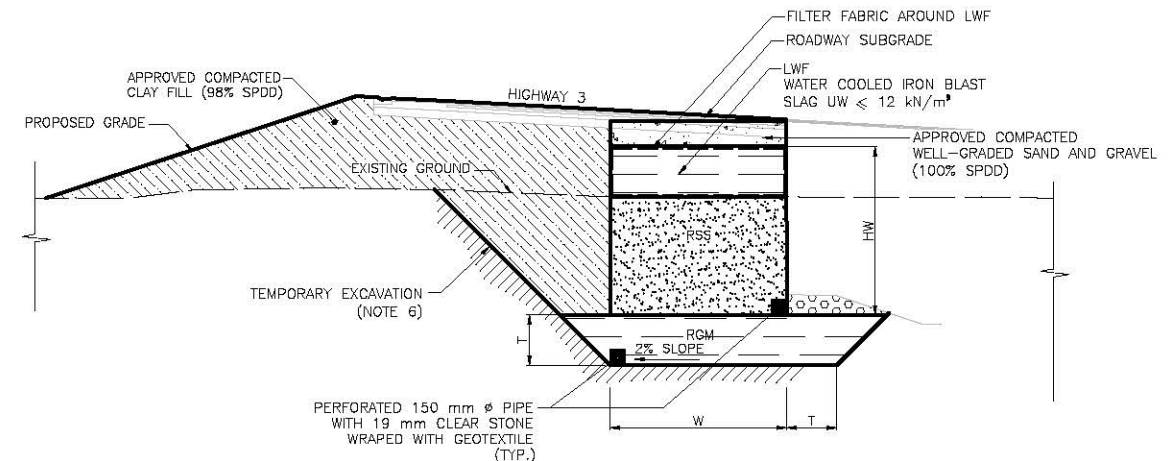
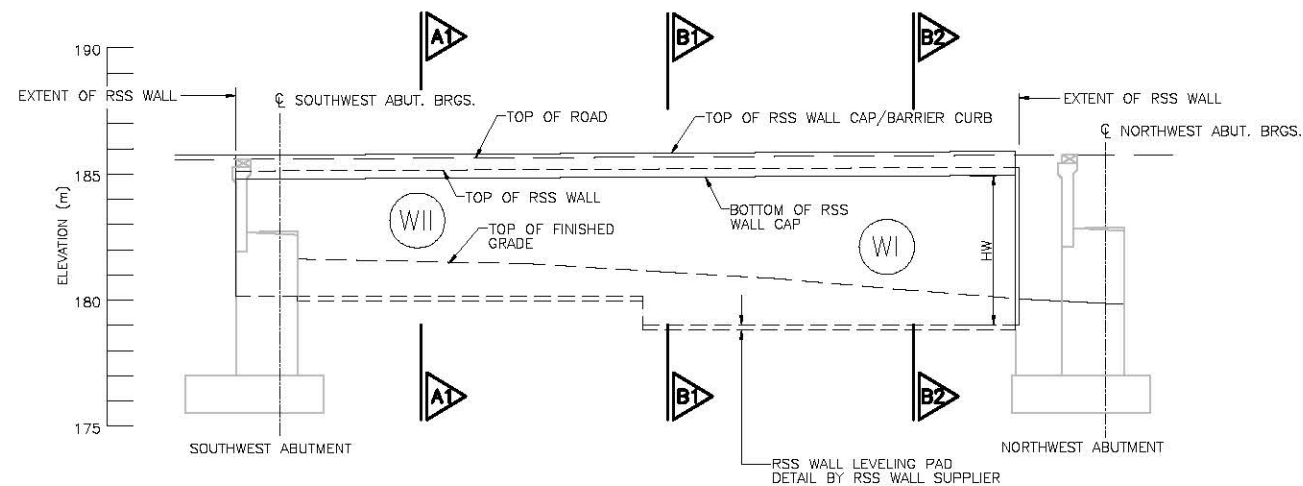
**Date:** March/2012  
**Rev:** 0  
**Page No.:** Appendix G



#### NOTES:

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
2. THE ILLUSTRATED RSS DIMENSIONS REPRESENT THE MINIMUM DIMENSIONS FOR EXTERNAL AND GLOBAL STABILITY REQUIREMENTS. THE FINAL DESIGN OF RSS, RGM AND STRUCTURAL ELEMENTS ARE TO BE DEVELOPED BY OTHERS.
3. ABUTMENT AND APPROACHWAY EMBANKMENT ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGN WERE OBTAINED FROM STRUCTURAL DRAWINGS AVAILABLE IN JANUARY 2012. ABUTMENT ELEVATIONS VARY ALONG THE APPROACHWAY.
4. CLAY SUBGRADE IS SUSCEPTIBLE TO DISTURBANCE AND LOSS OF STRENGTH DUE TO WATER INFLOW/PONDING, CONSTRUCTION TRAFFIC AND THE LIKE. SUITABLE EXCAVATION METHODS, DEWATERING AND SUBGRADE PROTECTION MUST BE EXERCISED.
5. CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS. EXCAVATED SLOPES ARE SUSCEPTIBLE TO DETERIORATION AND MAY EXPERIENCE DEFORMATIONS AND INSTABILITY. THE TEMPORARY SLOPES MUST BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED AND TREATED AS REQUIRED.
6. BACKFILL OF THE PIER FOUNDATION NEAR RSS WALL MUST BE ENTIRELY COMPLETED BEFORE BEGINNING OF THE RSS ERECTION.
7. BACKFILL IN FRONT OF RSS WALL TO BE COMPLETED TO NOT LESS THAN 500 mm OVER THE TOP OF RGM BEFORE THE CONSTRUCTION OF THE RSS WALL EXCEEDS 4 m IN HEIGHT.
8. NO TRAFFIC LOADING ALLOWED ON WBR6 UNTIL HIGHWAY 401 PAVEMENT IS IN PLACE.
9. TOE BERM REQUIRED AT WEST ABUTMENT RSS WALL FOR GLOBAL STABILITY.
10. SEE ACCOMPANYING DRAWINGS FOR APPLICABLE CONSTRUCTION NOTES.
11. GRANULAR BACKFILL BEHIND CONCRETE ABUTMENT AS PER OPSD 3101.150.

(\*) VARIES



RSS WALL	RSS STRUCTURE SIZE WIDTH, W x HEIGHT, HW (m) *	RGM THICKNESS, T (m)	LWF THICKNESS (m)
<b>WEST RSS WALL</b>			
SECTION A1	5.0 x 6.0	1.5	NONE
SECTION B1	6.0 x 7.0	1.5	1.5
SECTION B2	6.0 x 7.0	1.5	2.5
<b>EAST RSS WALL</b>			
SECTION A1	8.0 x 8.5	1.5	7.5
SECTION A2	8.0 x 8.5	1.5	5.5
SECTION B	7.0 x 7.2	1.5	3.0

\* THE RSS SUPPLIER MAY REQUIRE WIDER STRUCTURES TO MEET THE INTERNAL DESIGN REQUIREMENT.  
\* HEIGHTS VARY.

**NOT FOR  
CONSTRUCTION**

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

## **Appendix H      Rock Core Photographs**

**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018

**Date:** March/2012  
**Rev:** 0  
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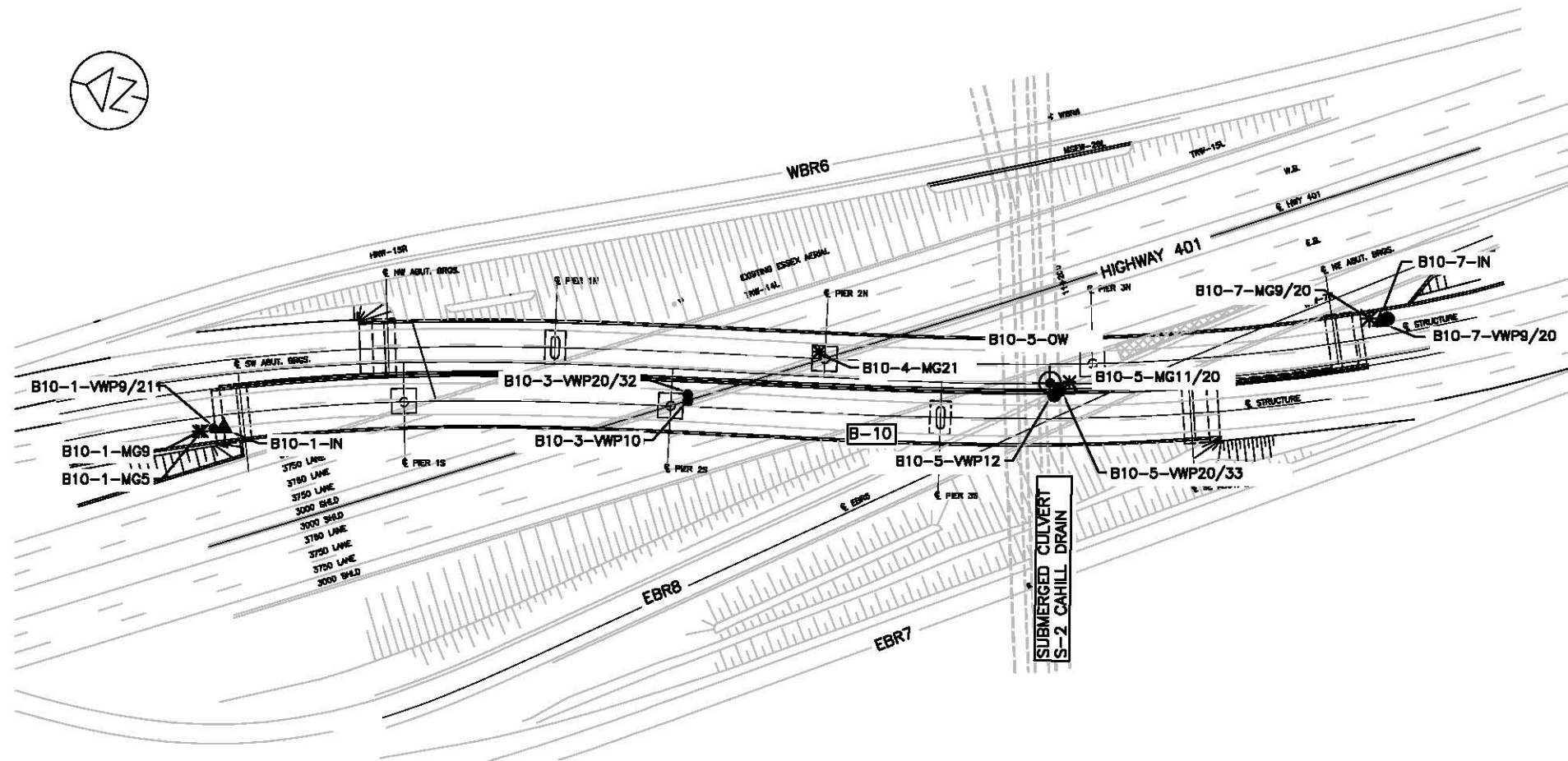
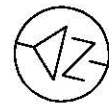


## **Appendix I      Instrumentation Plan**

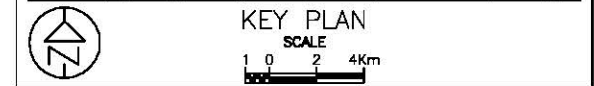
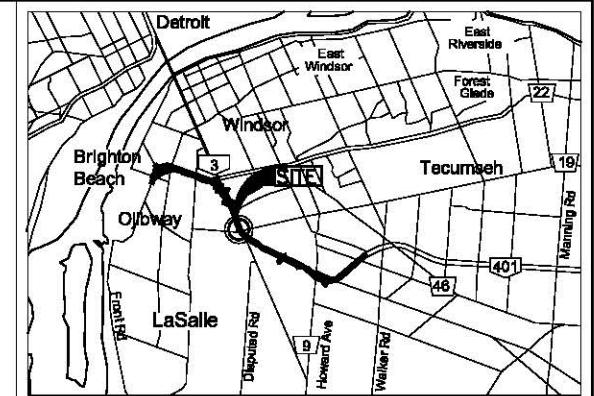
**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Bridge B-10  
(West of Geraedts Drive over Highway 401)  
**Doc No.:** 285380-04-119-0018

**Date:** March/2012  
**Rev:** 0  
**Page No.:** Appendix I





PLAN  
SCALE 1:750



KEY PLAN

SCALE  
1 0 2 4Km

LEGEND

- ▲ IN SLOPE INDICATOR (INCLINOMETER)
- VWP VIBRATING WIRE PIEZOMETER
- \* MG MAGNETIC RING SETTLEMENT GAUGE
- ⊕ OW OBSERVATION WELL

NOTES

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

No.	INSTRUMENT No.	ELEVATION	CO--ORDINATES (UTM, NAD 83 ZONE 17)	
			NORTHING	EASTING
AMEC BOREHOLES				
B10-1	B10-1-IN	182.6	4679159.2	332754.5
	B10-1-VWP9/21	182.7	4679160.5	332752.1
	B10-1-MG5	182.6	4679162.0	332749.7
	B10-1-MG9	182.6	4679162.4	332749.1
B10-3	B10-3-VWP10	182.2	4679083.2	332836.8
	B10-3-VWP20/32	182.2	4679084.3	332837.9
B10-4	B10-4-MG21	182.4	4679068.8	332867.5
B10-5	B10-5-OW	182.3	4679023.7	332901.0
	B10-5-VWP12	182.3	4679021.1	332899.6
	B10-5-VWP20/33	182.3	4679021.0	332901.4
	B10-5-MG11/20	182.3	4679020.6	332904.5
B10-7	B10-7-IN	182.2	4678977.5	332967.8
	B10-7-VWP9/20	182.2	4678976.3	332969.0
	B10-7-MG9/20	182.2	4678979.5	332966.0