The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report

Submerged Culvert S-1 (Lennon Drain, Sta. 10+425 LaSalle)

PARKWAY
INFRASTRUCTURE CONSTRUCTORS
Aug 1, 2012
PROCESSED
PROJECT DOCUMENT AND DATA MANAGEMENT

Geocres No. 40J6-40

Revision History					
Revision	**Date**	**Status**	**Prepared By**	**Checked By**	**Reviewed By**
0	07/25/2012	Issued for Construction	GN	DD	NSV

	Name, Title	Signature	Date
Prepared By	Ganan Nadarajah, M.A.Sc., P.Eng., Geotechnical Engineer		07/25/2012
Reviewed By	Narendra S. Verma, Ph.D., P.Eng., F.ASCE, D.GE. Principal Geotechnical Engineer (Designated MTO RAQS Contact)	Narendra	07/25/2012
Approved By	Brian Lapos, M.Sc., P.Eng Geotechnical Engineer / Project Manager, AMEC	Brian Lapos	07/25/2012

List of Contents and Appendices Page

List of Tables

Drawings

285380-03-060- WIP1-4101	Hwy. 401 Lennon Submerged Culverts S-1 General Arrangement
285380-04-090- WIP1-4101	Location Plan & Interpreted Stratigraphic Profile STA 14+700W to STA 10+500L
285380-04-090-WIP1-4102	Hwy. 401 Lennon Submerged Culverts S-1 Borehole Locations and Soil Strata
285380-04-091-WIP1-4103	Hwy. 401 Lennon Submerged Culverts S-1 Soil Stratigraphy

List of Applicable OPSDs

OPSD 802.31: Rigid Pipe Bedding, Cover and Backfill. Type 3 Soil – Earth Excavation
OPSD 3090.101: Foundation Frost Depths for Southern Ontario

List of Figures

List of Appendices

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

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

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

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

On December 17, 2010 Infrastructure Ontario and Ministry of Transportation 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. This combination brings a wide range of local and international experience to the project.

1.2 Report Introduction

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

This report presents the geotechnical design of the Submerged Culvert S-1, located on Lennon drain in LaSalle sector of the proposed Windsor-Essex Parkway (WEP) project. The culvert will be located between Highway 401 Sta. 10+400L and 10+450L (LaSalle). The culvert includes three submerged concrete pipes which will traverse under Highway 401 and realigned Highway 3 and located approximately 6 m and 13 m below the finished grades, respectively.

The design presented in this report was generally advanced from the preliminary geotechnical design developed for the WEMG (Windsor-Essex Mobility Group) proposal in June 2010 (ref. R-45)[1] and the results of the additional investigation. The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines, and the Parkway Infrastructure Constructors (PIC).

This report is issued for construction (IFC) and includes the results of the additional geotechnical investigation carried out to support the design, data from previous investigations and other relevant background information, and addresses review comments from peer reviews and MTO.

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 presented in Sections 5 and 6. Other information is presented in Sections 7 to 9.

---

[1] References are listed in Section 9.

The design of the Lennon drain Submerged Culvert S-1 complies with the requirements of PA (Project Agreement) 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 described in references R-17, R-20, R-21 and R-27. 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. P.P. Hudec (ref. R-27) 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 Zonal Velocity Zv = 0 and Zonal Acceleration Za = 0. Zonal Velocity ratio, V, and Zonal Acceleration ratio, A, are both 0.

In accordance with the Canadian Highway Bridge Design Code (CHBDC) the soil profile at the site of the project meets 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 was completed during the background investigation program (ref. R-17 to R-25) at locations distributed strategically along is 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 Site Conditions

The Lennon submerged culvert S-1 site is situated near the west end of the LaSalle sector of the Parkway, close to the border between the Windsor and LaSalle Municipalities in Ontario. An existing open ditch Lennon drain runs roughly east-west and crosses the existing Highway 3 by culvert located around the intersection of Daytona Avenue and Highway 3. The proposed Lennon drain submerged culvert S-1 will replace the existing culvert and will traverse under the proposed Highway 401 and realigned Highway 3. The ground topography around the proposed location of the culvert is essentially flat with elevations ranging from approximately 182.0[2] at the inlet structure (north of Highway 3) to 180.0 at the outlet structure (south of Highway 401). Adjacent land use is typically urban residential, parkland and light commercial.

2.4 Frost Depths

In accordance with MTO–SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-40) and OPSD 3090.101[3], the frost depth below the ground surface in Windsor area is estimated to 1.0 m. This estimate is considered applicable for natural soils and/or conventional pavement materials where the ground surface is usually cleaned from the snow cover.

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

---

[2] Elevations are in meters and are referred to geodetic datum.

[3] Ontario Provisional Standard Drawings are included at the end of the report text.

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-25) as part of background information for development of the WEP proposal designs. Additional geotechnical investigation was carried out in 2011 to supplement the previously obtained (pre-bid) subsurface soil data, as required to support the detailed design of the WEP embankment and structures. The additional investigation program at and around the proposed location of the Submerged Culvert S-1 comprised a total of 3 boreholes (T7-1, PS3-1 and PS4-1), 3 cone penetration tests (CPT T7-1, CPT 38-RW and CPT 39-RW), 1 Nilcon vane profile (adjacent Borehole PS3-1) and 1 flat plate dilatometer profile (DMT T7-1). Table 3-1 lists the test holes put down at or in close proximity (within 100 m or so) from the culvert S-1 site during the previous and the current geotechnical investigations.

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

Reference	Boreholes	Nilcon Vane Tests	CPT's	DMTs
Additional Investigation (2011)	T7-1		CPT T7-1	DMT T7-1
	PS3-1	NIL PS3-1		
	PS4-1			
	CPT 39-RW(*)		CPT 38-RW	
	CPT 38-RW(*)		CPT 39-RW	
	HGMW-1(*)			
Previous Studies (2007-09)	BH-127			
	BH/CPT-128(*)		CPT-128	
	BH-323			

(*)Shallow boreholes drilled to facilitate CPT testing or for other purposes.

Drawing 285380-04-090-WIP1-4102 shows the locations of the test holes put down in close proximity to the submerged culvert S-1 site.

3.1.1 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 215 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 in boreholes was generally carried out using a 50 mm diameter split spoon sampler. At select depths, samples were also taken using 70 mm diameter x 600 mm long 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 identified and placed in airtight containers by an experienced technologist and were transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing. Rock coring of the bedrock was completed using NQ or HQ sized 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 tests listed in Table 3.1 were carried out adjacent to the Borehole PS3-1 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, Rock Coring, Nilcon Vane Tests and Instrumentation in Boreholes

Borehole	Location	GS Elevation (m)	Overburden Thickness (m)(*)	Test & Instrument Elevations			
				Rock Coring Elevation	Nilcon Vane Elevation	VWP	MHSG
T7-1 (2011)	4679413.6N 332295.2E	181.5	30.2	151.3 to 145.8	-	172.4 161.7	172.0 162.0
PS3-1 (2011)	4679421.9N 332245.3E	181.3	32.5	149.4 to 145.7	176.0 to 153.5	166.0 158.4 148.3	-
PS4-1 (2011)	4679483.2N 332301.5E	182.9	35.6	146.0 to 144.9	-	-	-
BH-127 (Pre-Bid)	4679370.9N 332251.6E	181.3	32.8	148.5 to 145.2	-	146.0	-
BH-323 (Pre-Bid)	4679521.4N 332167.6N	181.3	33.1	148.2 to 143.0	-	-	-

Legend: VWP Vibrating wire piezometer
MHSG Spider magnet heave/settlement gauge
(*) Overburden includes existing fill thickness

Rock cores were examined in the field and photographed in the laboratory. Photographs of rock core are presented in Appendix F. 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 rock cores were photographed in the laboratory. Compression strength tests were carried out on rock core samples selected from across the WEP length.

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

The Nilcon vane tests and CPTs were carried out in cohesive soil strata after augering through the stiffer/denser 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). Three of the CPTs were advanced to refusal encountered near elevations 150 to 152. The fourth CPT was terminated earlier at about elevation 157 due to equipment failure. 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 and the advancement of the blade was temporarily arrested to activate the earth pressure reading device system. The tests were conducted following the provisions of ASTM D 6635.

The locations of boreholes, Nilcon tests and CPTs executed during the most recent 2011 investigation, and the inferred soil profile along the WEP alignment (Sta. 14+700W to Sta. 10+400L), are shown on Drawings 285380-04-090-WIP1-4101. The test hole locations in plan and soil stratigraphic section at the culvert location are shown on Drawings 285380-04-090-WIP1-4102 and 285380-04-090-WIP1-4103. 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.

3.2 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. A brief description follows.

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 selected 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. Sensor elevation and details of installations are provided in Table 3-2 and applicable borehole logs.

Magnetic Settlement/Heave Gauges: Spider magnets (RST, Model SSMM100 mechanical release spider target for 25 mm pipe) were installed in boreholes at selected locations and depths to permit future measurement of heave and settlement. The magnetic torus were placed at selected elevations around a 25 mm diameter pipe, which was extended to above the ground surface. The spider legs of the magnetic torus grip into the surrounding soil, which enables the 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. Installation Ring/Gauge elevations are provided in Table 3-2 and applicable borehole logs.

The installation of all instruments and the grouting of the holes were carried out in accordance with the manufacturer specifications. Proper future decommissioning of the instrumentation holes is responsibility of WEMG / PIC.

3.2.1 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 of the silty clay and silty sand obtained from Boreholes PS3-1 and PS4-1 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.

3.3 Data Interpretation – General Discussion

Field Vane Test Data Correction: The chart in Figure 3-11[4] developed initially by Bjerrum (1972) and updated subsequently by Ladd et al (1977) based on circular arc failure analyses of embankment failures suggest correction by multiplying the field vane data by 1.05 to 1.10 for soils with plasticity index (PI) of about 15 (ref. R-6 and R-33). 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 (Figure 3-2, ref. R-1 and R-8). The field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI.

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

$$S_{u\,CPT} = \frac{Q_t - \sigma_{vo}}{N_{kt}} \qquad (1)$$

Where:

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

The CPT based S_u profiles were developed to achieve a general agreement with the S_u profiles from the nearby Nilcon vane tests. 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 Culvert S-1 site, an N_{kt} factor of 14 was used for the clay crust and transition layers. The N_{kt} factors used for the underlying grey silty clay to clayey silt stratum and the lower clayey silt stratum were 16 and 12, respectively. Figure 3.3a presents the undrained shear strength (S_u) and maximum past pressure (P_c') profiles for WEP segment between Sta. 14+700W and Sta. 10+500L developed from the recent 2011 and pre-bid investigation data.

Pre-Consolidation Pressures from Cone Penetration Tests: The approach used for estimating the pre-consolidation pressures from the estimated S_u profiles follows the Stress History and Normalized Soil Engineering Properties (SHANSEP) method developed at MIT (Ladd and Foott, 1974, ref. R-33). The following relationship was used to compute the pre-consolidation pressures:

---

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

$$OCR = \frac{\sigma'_p}{\sigma'_{vo}} = \left[\frac{S_u / \sigma'_{vo}}{S} \right]^{1/m} \qquad (2)$$

Where:

S_u is the actual undrained shear strength,

σ'_{vo} is the vertical effective stress,

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

S is the normalized strength ratio, S_u/σ'_v, of normally consolidated soil,

OCR is the overconsolidation ratio, and

m is an empirically determined exponent, typically varying between 0.7 and 1.0.

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

$$\sigma'_p = \sigma'_{vo} \times \left[\frac{\dfrac{S_{u\,CPT}}{\sigma'_{vo}}}{0.18} \right]^{1.05} \qquad (3)$$

Flat Blade Dilatometer (DMT) Test Data:

DMT tests were conducted following the ASTM D6635-01 (2007). The soil properties from the results of these tests were developed in general using the guidelines provided in ISSMGE, 2001 (ref R-29), except that the undrained shear strength values for the clay deposits were estimated using the relationship $S_u = S \times \sigma'_{vo} \times (0.5 \times K_d)^{1.25}$, where S=0.18 and K_d is the horizontal stress index represented by:

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

Where:

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

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

The constant 0.18 for S_u/σ'_{vo} for OCR=1 curve is based on average plasticity index of the silty clay to clayey silt stratum and Chandler 1988 relationship (ref. R-11).

The undrained shear strength (S_u) profile inferred from the DMT T7-1 is consistent with the average of S_u profiles inferred from CPT data within the unweathered portion of firm cohesive soils. However, the Nilcon Vane tests at Borehole PS3-1 shows somewhat higher S_u values compared to S_u profile inferred from DMT T7-1.

The undrained shear strength (S_u), pre-consolidation pressure (σ_p') and natural water content (w_N) profiles based on field and laboratory testing from boreholes and CPTs put down between Sta. 10+300L and Sta. 10+500L are presented on Figure 3.3b. These plots include moisture content data from boreholes located within the Sta.14+700W to Sta.10+500L. Also included on the figure are $0.18 \times \sigma_{vo}'$ curve (representing S_u profile for OCR=1) and a simplified soil stratigraphy to facilitate correlation of soil properties to the individual soil units.

4 Subsurface Conditions

The ground surface elevation around the location of the Submerged Culvert S-1 varies from approximately 182.0 at the inlet structure (north of Highway 3) to 180.0 at the outlet structure (south of Highway 401). The general soil stratigraphy encountered at the borehole and CPT locations consists the following successive strata: surficial layers of occasional fills, top soil, and upper granular deposit; an extensive clayey silt to silty clay deposit below about elevation 180m; and a lower granular deposit at about elevation 150 m overlying limestone bedrock below about elevation ranging from 148 to 149. The thickness of the clayey silt to silty clay deposit based on the available nearby boreholes is about 30 m.

4.1 Topsoil, Surficial Fills and Upper Granular Deposit

Surficial fill was encountered in Boreholes T7-1, BH-323 and BH-127 and CPT-128. The fills at the borehole locations were variable and consisted of sand and gravel to fine sand and ranged in thickness from 1.0 to 1.5 m. A layer of upper granular deposit was encountered beneath the fill or topsoil in Boreholes T7-1, PS4-1, HGMW-1, BH-127 and CPT 39-RW. The upper granular deposit consisted of sand to silty sand and was encountered approximately between elevations 179.0 and 181.0. The thickness of the upper granular deposit varied from 0.5 to 3.0 m at the borehole locations. The Standard Penetration Test (SPT) ‘N’ values determined in the upper granular deposit varied from 3 to 16, indicating a very loose to compact state of compactness. Concrete and asphalt layers were also encountered at some borehole locations.

4.2 Silty Clay to Clayey Silt Stratum

An extensive deposit of cohesive silty clay to clayey silt was encountered directly underlying the surficial fills (or upper granular layer) at about elevation 180. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into 4 successive sub-strata as follows: brown, desiccated, stiff to hard, clay crust; transition clayey silt layer; upper grey silty clay to clayey silt deposit (referred to hereafter as upper silty clay); and then a generally coarser, lower grey clayey silt deposit (referred to as lower clayey silt). The lower clayey silt deposit contains discontinuous sandy silt / silty sand seams. The natural water content, Atterberg limits and total unit weights in the various sub-strata are summarized in Table 4-1.

Table 4-1: Summary of Index Properties of the Subsurface Soil Strata

Property	Clay Crust	Transition	Upper Silty Clay	Lower Clayey Silt
Elevation Range, m	180.0 to 177.0	177.0 to 175.0	175 to 163	163 to 151
Natural Water Content, w_N, %	6 to 28	12 to 27	15 to 40	9 to 33
Liquid Limit, w_L, %	32 to 39	30 to 35	25 to 40	23 to 41
Plastic Limit, w_P, %	19 to 20	16 to 18	12 to 19	13 to 21
Plasticity Index, PI	12 to 20	14 to 17	10 to 23	9 to 20
Liquidity Index, LI	0.0 to 0.2	0 to 0.3	0.1 to 0.9	0.1 to 0.6
Design Unit Weight, γ, kN/m3	21	21	20.0	20.5

The measured undrained shear strength (from Nilcon vane testing), versus depth profiles are shown in Figure 3.3a. The undrained shear strength of the clay stratum varied with depth generally as follows:

- Clay Crust layer: >100 kPa
- Clay Transition layer: 80±20 kPa to 65±10 kPa
- Upper silty clay: 65±10 kPa to 40±10 kPa to 55±10 kPa
- Lower clayey silt: 55±10 kPa to >90±10 kPa.

The interpreted stress-strain properties and the effective shear strength properties of the silty clay to clayey silt soils were based on published correlations in the literature (Kulhawy and Mayne, 1990, ref. R-31) and the relationships proposed in Golder's Subsurface Condition Interpretation Report (ref. R-21). These were reassessed and confirmed by laboratory oedometer tests, triaxial shear tests and direct shear tests performed during the additional geotechnical investigation carried out as part of the detailed design 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 compressibility parameters used for the silty clay / clayey silt substrata for the submerged culvert S-1 site is summarized in Table 4-2 based on average water content data. The upper silty clay stratum was sub-divided into two strata (namely, Upper Silty Clay-1 and Upper Silty Clay-2) to reflect the variation in undrained shear strength and maximum past pressure profiles.

The moduli of elasticity for undrained and drained conditions were estimated using empirical correlation based on published information (ref. R-44) and local experience (ref R-21, 22). For the unweathered silty clay to clayey silt stratum, the following empirical relationships were used;

Elastic modulus (undrained conditions)	$E_u = 300 \times S_u$
Elastic modulus (drained conditions)	$E' = 0.9 \times E_u$

Estimated elastic deformation modulus values for various soil layers are summarized in Table 4-3.

Table 4-2: Summary of Compressibility Properties

Property	Clay Crust	Transition	Upper Silty Clay 1	Upper Silty Clay 2	Lower Clayey Silt
Elevation Range	180 to 177	177 to 175	175 to 166	166 to 163	163 to 151
Average Natural Water Content, w_N, %	19	20	23	23	18
Average Total Unit Weight (kN/m^3)	21	21	20	20	20.5
Preconsolidation Pressure	550	550 to 350	350 to 230	230 to 260	260 to 400
Over-Consolidation Ratio	10	10 to 4.0	4.0 to 1.4	1.4	1.5
Virgin Compression Index, C_c	0.155	0.163	0.189	0.189	0.146
Recompression Index, C_r	0.017	0.018	0.021	0.021	0.016
Swelling Index, C_s	0.039	0.041	0.047	0.047	0.036
Secondary Compression Index, C_α	0.0043	0.0046	0.0053	0.0053	0.0041

Notes: The ranges of S_u and σ_p' values indicate variation top to bottom with depth.

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	30	0.49	27	0.35
Transition	21	0.49	19	0.35
Upper Silty Clay 1	16	0.49	14.5	0.35
Upper Silty Clay 2	14	0.49	13	0.35
Lower Clayey Silt	20	0.49	18	0.35

* - Assumed values (ref R-44)

The recommended effective shear strength properties applicable to the native silty clay to clayey silt stratum are summarized as follows:

Effective cohesion, c'	0 kPa
Peak angle of internal friction	30 degrees
Critical state friction angle	25 to 27 degrees (**)

(**) Based on triaxial tests (ref R-21)

Effective cohesion (which may be potentially present) in the upper zones of over-consolidated clayey silt has been neglected for engineering design in consideration of potential 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 the silty clay to clayey silt, very dense lower granular deposit was encountered in Boreholes T7-1, BH-323 and BH-127. This lower granular deposit consisted of sand and gravel to sandy silt. Based on the SPT "N" values of greater than 100, this material is considered to be in a very dense state of compactness. This layer was encountered at about elevations ranging between 149 and 151 and varies in thickness from 1.5 to 2.0 m at the borehole locations.

4.4 Bedrock

Where rock coring was undertaken, a white to grey, limestone bedrock was encountered. The bedrock was generally fresh, medium strong, thinly laminated, fine grained, faintly to moderately porous and moderately fractured. The Rock Quality Designation (RQD) of the recovered rock cores varied on average between 50 to 90 per cent, indicating a fair to good quality. Based on this core logging the rock mass classification was estimated to range from 2.8 to 5 for the Q-System (Barton *et. al.*, 1974, ref. R-3) and 53 to 58 for the Rock Mass Rating (RMR) based on Bieniawski (1976, ref. R-5) and indicates that the rock mass can be considered as a Fair quality rock mass based on the later system. The rock quality generally increases with depth. Bedrock was encountered at elevations ranging from 148.0 to 149.0 in the vicinity of Submerged Culvert S-1. Photographs of rock cores recovered from the additional investigation are provided in Appendix H.

It was found during the preliminary investigations reported in Golder's Subsurface Condition Interpretation Report (ref. R-21) 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 4-4. The average strength of the limestone is determined to be 85.5 MPa and is 'strong rock' based on the ISRM (1978, ref.25). Additionally, based on the coefficient of variation, enough tests have been performed to characterise the compressive strength.

Table 4-4: Summary of Intact Rock Properties

Item	**Density** (kg/m^3)	**Unit Weight** (kN/m^3)	**UCS** (MPa)
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
Number of Samples, N	12	12	16

Based on the rock mass classification and the strength properties assuming an mi = 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 (Wyllie, 1999, ref. R-46).

4.5 Groundwater Conditions

The piezometric water levels within the overburden and the bedrock (including lower granular deposit) were measured to be at about elevations 180.0 and 177.0, respectively. These observations suggest a downward gradient between the overburden and the bedrock. However, based on the general trend in the Windsor area, the occurrence of artesian conditions in the bedrock cannot be ruled out.

Perched groundwater is known to accumulate seasonally within the upper deposits of fill and within the fissures in the silty clay crust. In adverse conditions, the perched groundwater levels can rise to near the original ground surface at about elevation.

Table 4-5: Summary of Measured Water Levels

Borehole	Ground Surface Elevation, m	Piezometer Type	Screen / Sensor Elevation, m	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	Elevation
BH T7-1	181.5	VWP	172.4	Silty Clay	July 24, 2011	180.4
					Aug. 6, 2011	180.5
		VWP	161.7	Silty Clay	July 24, 2011	180.4
					Aug. 6, 2011	180.4
PS3-1	181.3	VWP	166.0	Silty Clay	Nov. 03, 2011	179.5
					Nov. 11, 2011	179.5
		VWP	158.4	Silty Clay	Nov. 03, 2011	176.8
					Nov. 11, 2011	176.7
		VWP	148.3	Lower Granular	Nov. 03, 2011	176.8
					Nov. 11, 2011	176.8
BH 127	182.3	SP	146.0	Bedrock	Nov. 11, 2008	177.7
					Jan. 26, 2009	177.3
HGMW-1	183.0	SP	180.0-181.5	Silty Sand	July 29, 2011	180.0

Legend:		
	VWP	Vibrating Wire Piezometer
	SP	Standpipe

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 presence of the gas was not noted by odour during the current and previous investigations around the culvert site.

Although the presence of the H_2S and CH_4 gases was not observed during the current and previous investigations around Culvert S-1 site, their presence cannot be ruled out. Pumping tests were conducted at three locations across the proposed parkway to determine concentration levels of hydrogen sulphide gas in the bedrock aquifer of the area. Of these tests, TOW-2, located north of Culvert S-1, indicated a concentration of 20.0 mg/L of H_2S gas and at TOW-3, located south of Tunnel T-4, indicated a concentration of 7.0 mg/L of H_2S gas. As Culvert S-1is located between TOW-2 and TOW-3, H_2S gas may be present in this area.

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 (ΔU) to total stress changes can be very low. This phenomena leads to reduction in effective stress and hence shear strength (ref. R-26 and R-43). 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).

5 Development of Geotechnical Designs

5.1 Geotechnical Design Criteria and Considerations

The geotechnical design has been completed in compliance with the requirements of the executed 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 retaining walls.

5.2 Design Soil Properties

The design soil properties for the silty clay to clayey silt deposit were interpreted from the CPT and Nilcon vane test profiles and the laboratory test results. The undrained shear strength, S_u, profiles were estimated from the CPTs based on the calibration described in Section 3.2. The S_u profiles inferred from the CPTs advanced around submerged culvert S-1 site are shown in Figure 3.3b. Selected design values obtained from the profiles are summarized in Table 5-1. The upper silty clay stratum was sub-divided into two layers (namely, Upper Silty Clay-1 and Upper Silty Clay-2) to reflect the variation in undrain shear strength. Effective cohesion (where present) in the upper clay crust and transition zone has been neglected due to long term weathering, moisture ingress and fissuring effects.

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

Clay Substratum	Elevation Range, m	Undrained Shear Strength (Su), kPa	ϕ'_{max} (degrees)	ϕ'_{cs} (degrees)	Preconsolidation Pressure (σ_p'), kPa
Clay Crust	180 to 177	75(*)	30	26	550
Transition	177 to 175	75 to 65	30	26	550 to 350
Upper Silty Clay-1	175 to 166	65 to 44	30	26	350 to 230
Upper Silty Clay-2	166 to 163	44 to 50	30	26	230 to 260
Lower Clayey Silt	163 to 151	50 to 65	30	26	260 to 400

Note: The ranges of S_u and σ_p' values indicate variation top to bottom with depth.
(*) For the purpose of global stability analyses only

Legend: ϕ'_{max} = peak effective friction angle (drained)
ϕ'_{cs} = critical state friction angle at large strain

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 5-2. These values are slightly (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: Summary of Other Interpreted Design Parameters

Clay Substratum	Horizontal Permeability, cm/sec	Anisotropy Ratio, k_h/k_v	Initial Void Ratio, e_0
Clay Crust	6.8×10^{-7}	1	0.50
Transition	3.9×10^{-7}	2	0.55
Upper Silty Clay - 1	1.1×10^{-7}		0.60
Upper Silty Clay - 2	1.1×10^{-7}		0.60
Lower Clayey Silt	2.0×10^{-7}	1	0.50

5.3 Excavations and Temporary Cut Slopes

5.3.1 General

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 temporary excavations shown on figures and drawings do not constitute the actual design of the temporary slopes. The Contractors are fully responsible for the design, construction methods and performance (stability, deformability and deterioration) of the temporary slopes. The Contractors also must ensure that the temporary slopes meet the Project Agreement criteria and the needs to accommodate the construction of the structure as per design.

The Contractor should be aware that the analytical assessment presented in this report may not be sufficient to assess all factors that may affect the construction. The following comments and recommendations are considered applicable:

- Excavations are expected to encounter surficial granular soils and top soil, the clay crust and transitional layers, and will be extended into the upper silty clay. The excavations may intersect seams of saturated granular layers and/or water bearing backfill within trenches of active and/or abandoned utilities. Groundwater control will be required based on timing of construction and prevailing weather conditions.
- The stress deformation assessments referenced in this report assume that the bulk of the general excavation is initially conducted to about 0.3m of the underside of the submerged culvert. If other staging of the excavation is intended, a revision of the stress deformation analyses will be required.
- The temporary slopes should be properly protected at all times against surface erosion due to runoff, desiccation, freeze-thaw effects, gas releases etc. The duration of the slope exposure should be limited to the shortest practical time possible to minimize slope deterioration or instability.
- To protect the subgrade integrity, excavations should cease 0.5 m above final subgrade elevation. This 0.5 m protective layer shall not be removed until the bedding is ready to be placed.

- Regular inspection of the slope condition by experienced personnel along with monitoring of the ground movement at strategic locations should be carried out. Mitigation and remedial measures should be implemented promptly as required.
- Temporary support of the excavation (bracing) is the responsibility of the contractor and should be designed and constructed in accordance with OPSS 539. The support system should be designed to Performance Level 2 (OPSS 539.04.01.01) or better. Should the ground support system be allowed to remain in place after construction, SSP 539S02 should be included in the contract documents. Modification of allowable depths may be provided using a similar NSSP.
- 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.

5.3.2 Stability of Excavations and Temporary Cut Slopes

Preliminary stability analyses of excavations and temporary cut slopes were performed using the material properties summarized in Table 5.1. These analyses indicate that for conventional open excavation from existing grade the top width of the temporary excavation trench would be in excess of 100 m (assuming overall temporary excavation slope of 3H:1V) An open cut from the proposed level of Highway 401 subgrade would be approximately 25 to 30 m wide. Likely some combination of conventional excavation with a braced cut will be most practical if excavations from existing grade are considered.

Basal hydrostatic uplift was calculated based on the highest measured water level in the lower granular deposit (elevation 178), anticipated deepest excavation depth (base of pipe at elevation 167.3), and a silt-clay layer thickness of 16 m below the deepest excavation. The estimated factor of safety against hydrostatic uplift was 1.24 based on the weight of the clay cap only.

For shored excavations scenario, the factor of safety (FS) against basal stability (a global stability number defined by depth of excavation and undrained shear strength) were estimated to be 1.1 and 2.5 for excavations from existing ground surface and the proposed Highway 401 grade. The shoring system should be developed on the basis of an engineered design. It should be noted that risks of pipe settlements are associated with the extraction of deep sheet piles used for temporary shoring.

As described in Section 4.6, the presence of gassy soils near the bedrock surface could potentially be encountered. Their presence could impact the pore pressure and undrained shear strength condition of the lower part of the silty clay deposit. It is therefore recommended that if presence of gassy soils is evidenced, temporary piezometers be installed below the final excavation subgrade level to monitor the pore pressures during and following excavation. The excavation should be carried out in small depth (say 1 m) increments, particularly as of 5 m depth below existing ground surface, and sufficient time to dissipate the pore pressures should be allowed at each excavation stage. The excavation guidelines can be revised based on on-site experience.

5.4 Submerged Concrete Pipe Culvert

5.4.1 General

The general arrangement of Culvert S-1 is shown on Drawing 285380-03-060-MST1-S4101 and Figure 5.1. The Submerged Culvert S-1 will consist of three 2.4 m diameter pipes which will be laid down in parallel array near Sta. 10+415L to 10+435L. Culvert S-1 will be installed across the large excavation for Highway 401 and will be routed under Retaining Wall HRW-20L and Highway 3. The HRW-20L is about 25 m long with the exposed height of about 8 m. It consists of reinforced light weight backfill for about 13 m behind the wall. Design details of HRW-20L can be found in the retaining wall design report. Construction of the submerged culvert will involve variable depths of excavation below existing ground surface under these structures as indicated below:

- Highway 401: 13.5 m (±6.5 m below highway grade)
- Highway 3 : 13.5 m (varies from 13.5 m to 7 m)
- Retaining Wall HRW-20L: 13.5 m

Relevant configuration information for Culvert S-1 and the ground/soil conditions are summarized in Table 5.3.

Table 5-3: Summary of Design Control Elevations of Submerged Culvert S-1

Approximate Elevation, m	Subsurface Soil and Groundwater Conditions	Proposed Highway and Submerged Culvert Elements
±182		Highway 3, Top of HRW-20L
182 to 180	Existing ground surface	
180	Average piezometric level in shallow soils	
177	Average piezometric level in bedrock	
±180 to 177	Clay Crust	
177 to 175		Base of Inlet Structure
178 to 174		Base of Outlet Structure
177 to 175	Transitional Clay	
175 to 166	Upper Silty Clay-1	
±174		Highway 401 Pavement Surface
±167.5		Approximate Pipe invert elevation
±167.2		Approximate base of excavation
166 to 163	Upper Silty Clay-2	
163 to 151	Lower Clayey Silt	
151 to 149	Compact to dense Sand and Gravel	
149 to 148	Bedrock surface	

The large excavation for Highway 401, installation of Culvert S-1, construction of Retaining wall HRW-20L, temporary excavations and backfilling will induce short-term and long-term deformation in the soils along the submerged culvert profile. As such the submerged culvert must be designed to accommodate these movements.

5.4.2 Stress Deformation Analyses

Finite element stress-deformation analyses (SDA) were carried out using the SIGMA/W software to assess the ground movements at the base of the submerged culvert (below Highway 401) in short and long term. The deformation analyses were also used to evaluate settlement/heave below Highway 3 and the high retaining wall HRW-20L.

For the purposes of modeling the longitudinal configuration, the facility was assumed to be a symmetrical structure. The northern half of the pipe culvert comprising Highway 401, HRW-20L and Highway 3 was chosen for the SDA along the culvert. A separate SDA model was developed at a Section across Culvert S-1 (along HWY 401 centreline) to examine the effect of culvert installation along HWY 401.

The configuration of the Sigma/W model, section along the submerged culvert, is presented in Figure E.1. The model is based on the following loading steps:

a) Generation of the initial (in-situ) stress condition for level ground;

b) Excavation for Highway 401 and excavation for HRW-20L;

c) Installation of culvert, Inlet structure, HRW-20L and backfilling and

d) Calculation of long-term settlement and heave.

The soil stratigraphy and selection of the soil properties were based on the design soil properties discussed in Section 5.2.

The SDA were carried out using an effective stress-based model incorporating coupled stress-flow models of soil-pore water response. The initial phreatic surface was assumed to correspond to the measured groundwater level at elevation 180m.

The long term phreatic surface (in the area surrounding the concrete pipes) was assumed to follow the excavation surfaces and then stabilizes at 1m below the Highway 401 grade (i.e., elevation 173m). In this regard relief trenches for the pipe bedding and cover should be implemented.

The ground water in the granular backfill will be connected to the 100 mm Slope Subdrain and 150 mm Wall Subdrain through relief trenches. Estimated transmissivity of the relief trenches is about one order of magnitude higher than estimated ground water seepage into the granular bedding and backfill around the culvert pipes. Therefore, the relief trenches have adequate hydraulic capacity to prevent the build-up of excess hydrostatic pressures within the granular backfill around the culvert pipes. Details of the Subdrains can be found in Highway deign drawings.

Elastic-plastic Mohr-Coulomb models were used for all soil layers except for the unweathered firm and stiff silty clay to clayey silt layers below the transition zone. These unweathered layers were assigned with the Modified Cam-Clay model. Hydraulic conductivity properties described in Table 5-2 were assigned to the different silty clay soil layers.

Construction Stage (b) described above was assumed to occur over a period of three weeks implying insufficient time for any substantial dissipation of the excess pore water pressures generated by the soil unloading. Hence, the state of stress and deformations at the end of "three weeks" largely correspond to undrained conditions. Then, Stage (c) was assumed to occur over a period of "one week". After numerical simulation of the entire construction, the model was allowed to dissipate the excess pore pressures over a period of time until a steady-state condition of pore water pressure is achieved.

Calculated cumulative settlement/heave at the end of Excavation (Stage b) and end of construction (EOC, Stage c) is presented in Figures E-2 and E-3, respectively. Estimated long-term heave/settlement is presented in Figure E-4. Figure E-5 illustrates the stabilized pore water pressure contours within the natural soil layers at the end of dissipation (long-term) period.

An additional SDA model was examined for a section across Culvert S-1 (along Highway 401 centreline) to evaluate deformations caused by temporary excavation, culvert installation and backfilling. This model assumes conventional open excavations installation. These analyses should be revised if other types of excavations (braced cut or combination of conventional and braced excavation) are employed.

Configuration of the Sigma/W model (across Culvert S-1, along Highway 401 centreline) is presented in Figure E-8. The SDA modelling to estimate deformations during temporary excavation and backfilling was performed based on the following loading stages:

a) Generation of the initial (in-situ) stress condition for level ground assuming an average bulk soil unit weight of 21 kN/m^3 and a K_o factor of 0.75 (based on publications (ref. R-44) and confirmed by DMTs in the area) for the soil deposit.

b) Excavation for Highway 401.

c) Temporary excavation to elevation 167.0 m (~0.3m below Pipe base).

d) Installation of concrete pipes and backfilling to highway 401 elevation.

Estimated undrained (immediate) deformations due to excavation and backfilling are presented in Figures E-9 and E-10 respectively.

5.4.3 Serviceability Limit State (SLS) Performance

The magnitude of vertical deformations of the ground surface and culvert determined from the SDA analyses are summarized in Table 5-4 for the end of excavation, the end of construction (short-term) and the long-term steady state conditions. The deformation trends are illustrated on Figures E-2 through E-11. These analyses results suggest that the ground movements generated by the construction loads are anticipated to stabilize within approximately 10 to 15 years following completion of construction (Figure E-7).

Table 5-4: Summary of Calculated Deformations

Parameter	End of Excavation (mm)	End of Construction (mm)	Post-Construction (Long-term) (mm)	Remarks
Maximum Heave/Settlements at Pipe Invert				
Below Highway 401	+55	-35(b)	+25	Figures E-2, E-6 to E-10
Below RSS wall HRW-20L and Eastbound lanes of Highway 3	+65	-35	+10	
Below Highway 3 Centreline	+65	-55	+5	
Below Highway 3 Westbound lanes	+60	-55	-5	
Inlet/Outlet Structure	-25	±10	-10	
Maximum Heave Settlement along Highway 401 Pavement(a)				
0 m	N/A	-55(c) mm	+45(d)	Figure E-11
5 m	N/A	-50(c) mm	+45(d)	
15 m	+5	+5 mm	+60(d)	
Beyond 15 m	-5	-5 mm	+60(d)	

(a) Distances measured from centre of culvert pipes.
(b) Positive (+) sign indicates heave movement and negative (‘-’) sign indicates settlement.
(c) Movement corrected during construction.
(d) Majority of long term heave mostly caused by main excavation for Highway 401 Corridor.
(e) N/A – Not available

As mentioned earlier, a second SDA model was developed to examine the effect of the trenching and installation of the drain pipes along HWY 401. This model is represented by the cross section of the assumed open cut from the level of major permanent cut for HWY 401. The model indicate temporary heave of up to 35 mm during trenching at the base of trench. The model also indicates that the pipe invert would settle by up to 35 mm during backfilling to Highway 401 elevation. No tangible ground deformations were obtained (Figures E-9 through E-11) away from the open trench at the HWY 401 grade.

The estimated ground movements along Highway 401 centreline at pavement elevation are shown on Figure E-11 and summarized in Table 5-4. These results indicate minimal influence beyond the crest of temporary trench excavation (below Hwy 401 excavation) for installation of pipes. Similar behaviour is anticipated under Highway 3 providing that general excavation for the RSS retaining wall HRW-20L is carried out before local trenching for culvert pipe installation.

All the ground movements and deformations calculated and presented in this report are estimates based on soil deformation and compressibility properties interpreted from laboratory tests and empirical correlations. In this regard, the reported values are approximate and should be considered only as an indication of the magnitude of the soil response. These estimates will be verified and refined with respect to the actual performance monitoring in the field.

The settlements discussed above do not include deformations caused by seasonal temperature and moisture variations. Also, they do not include the effects of the long-term compression of the backfill materials that may occur further to inadequate compaction. To minimize these, the compaction specifications should be rigorously adhered to during construction in order to minimize these risks.

Outlet and Inlet Structures:

Net soil stress increase at the base of inlet and outlet structures is expected to be nominal. However, the phreatic surface around inlet and outlet structures is expected to be lowered from elevation 180 to 175 due to pressure relief drains around the culvert. Accordingly inlet and outlet structures are expected to experience minor long-term post-construction movements. One dimensional heave/settlement calculations were performed to estimate long-term settlement/heave at Inlet and Outlet structures. Based on these calculations and the SDA discussed in Section 5.4.3, it is anticipated that inlet and outlet structures would experience long-term (post-construction) settlements of less than 25 mm.

5.4.4 Ultimate Limit State (ULS) Bearing Resistance

A factored geotechnical resistance of 120 kPa at Ultimate Limit States (ULS) was determined for the native undisturbed silty clay subgrade soils supporting the concrete pipe near elevation 167.5m. For the concrete pipe installed along the sloped excavation (to connect to inlet or outlet structures), a factored geotechnical resistance of 120 kPa (elevation 167.5) to 165 kPa (elevation 175) was determined. A net factored geotechnical resistance of 200 kPa was estimated for the inlet and outlet structures.

Retaining walls LRW1 and LRW2 extend from the Inlet structure on either side of the Lennon drain and are structurally connected to the Inlet structure. The height of the retaining wall LRW1 and LRW2 are 1.80 to 2.80 m and 3.85 m respectively. A net factored geotechnical resistance of 225 kPa at ULS and 160 kPa at SLS were determined for the native undisturbed silty clay subgrade soils above elevation 175 m.

5.4.5 Earth Pressures on Retaining Structures

Temporary Braced Excavation Walls:

Temporary shoring for the deep cuts in excess of 6 m should be based on an engineered support system complying with Ontario Occupational Health and Safety Act.

The design earth pressures against the walls of the braced excavation should not be less than the apparent earth pressures indicated in the Canadian Foundation Engineering Manual (ref. R-8) applicable for cuts in soft-to firm or stiff cohesive soils, depending on the shear strength S_u, at the base of the excavation. Ground deformation around the deep shored excavations should be anticipated. Detailed deformation analysis should be carried out to assess ground deformations and the lateral extent of the zone of impact during temporary excavations and construction. The performance of the temporary excavations and shoring should be continuously monitored.

Permanent Retaining Structures:

The following earth pressure coefficients may be used for different type of compacted backfill at retaining structures (foundation walls):

Table 5-5: Soil Parameters for Earth Pressure Calculations

Soil Parameter	Group I Soils	Group II Soils	Group III Soils
Fill Unit Weight (kN/m^3)	22	21	20.5
Friction angle, ϕ' (degrees)	33 - 35	29 – 32	22 – 30
Coefficients of static lateral earth pressure:			
'Active' or Unrestrained, $K_a^{(*)}$	0.27 to 0.30	0.31 to 0.35	0.33 to 0.45
'At rest' or Restrained, $K_o^{(*)}$	0.44 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 compacted to > 95% Standard Proctor maximum dry density. The coefficients of lateral earth pressure should be adjusted if there is sloping ground at the back of the wall.
Notes:

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

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

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

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

$$K_0 = (1 - \sin\emptyset)(1 + \sin\beta)$$

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

The long term earth pressures against the buried structures should consider the earth pressure coefficients listed above in Table 5-4 in conjunction with the bulk unit weights listed in Table 4-2. The buoyant soil weight should be used for the submerged portion of the structure.

Where applicable, hydrostatic pressures should be added to earth pressures. Permanent and temporary surcharges at the ground surface should also be considered as appropriate.

A minimum earth pressure of 12 kPa should be considered along any section of the buried structure to account for the effects of compaction.

6 Construction Requirements

6.1 Temporary Excavation and Subgrade Preparation

The shapes and slopes of the temporary excavations shown do not constitute the actual design of the temporary slopes. The Contractors are fully responsible for the design, construction methods and performance (stability, deformability and deterioration) of the temporary slopes. The Contractors also must ensure that the temporary slopes meet the Project Agreement criteria and the needs to accommodate the construction of the structure as per design.

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 assumed compacted clay fill 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.

Excavations are expected to encounter surficial granular soils and top soil, and the clay crust and transitional layers, and will be extended into the upper silty clay. The excavations may intersect seams of saturated granular layers and/or water bearing backfill within trenches of active and/or abandoned utilities. Groundwater control will be required based on timing of construction and prevailing weather conditions.

The silty clay soils at the project site are highly susceptible to rapid deterioration when exposed to elements of weathering, water inflow and ponding, disturbance from construction traffic, and the like.

Minor seepage from runoff infiltrations or perched water within the fill is anticipated which should be controllable by conventional temporary dewatering methods.

The recommendations provided herein are based on the assumptions that the temporary slopes are properly protected at all times against surface erosion due to runoff, desiccation, freeze-thaw effects, and the duration of the slope exposure is limited the shortest practical time possible to minimize slope deterioration or instability.

To protect the subgrade integrity, the final excavation lift above the design elevation should not be less than 0.5 m 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 the subgrade without approved protective covers.

As indicated in Section 5.3.2, gassy soils are not likely to be encountered. It is however recommended that if the presence of gassy soils is evidenced (for example, dissolved gas bubbles coming out of solution and softening of the excavation face), the excavation should be carried out in small (say 1 m) depth increments and sufficient time to dissipate the pore pressures should be allowed at each excavation stage.

The final excavation layer above the design subgrade to be carried out using buckets equipped with smooth lips. Once exposed, the subgrade must be immediately inspected. Upon approval, a skim coat of lean concrete protection (mud mat) should be placed to provide also a working surface for forming and steel erection.

Regular monitoring and inspection of the condition of temporary slopes, retaining structures, ground movement at strategic locations and excavation base for signs of instability, deterioration, sloughing, etc should be carried out by qualified personnel. Appropriate mitigation measures should be implemented.

Appropriate monitoring of the nearby utilities and facilities is required. Monitoring should consist of a precondition survey along with regular surveying conducted of the nearby utilities, early works, etc.

6.2 Backfilling

Behind and around the inlet and outlet structures, non-frost susceptible and free draining Granular fill should be placed in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC). The pipe bedding, side fill, cover and backfill should be placed in accordance with Ontario Provincial Standard Specification OPSS 514 and CAN/CSA-S6-06 (CHBDC).

It is understood that the native silty clay to clayey silt from the crust zone is being considered for backfill material, where appropriate. The clay crust material is considered suitable for re-use as engineered fill but may require moisture conditioning. Well graded, 75 mm minus sand and gravel (Granular B Type 1 or approved equivalent) can also be considered for use as engineered fill since such materials are less sensitive to moisture content increases. The fill materials should not contain deleterious material such as construction debris or organics. Geotechnical engineering input is required in order to assess the suitability of fill materials for the use intended.

The fill should be placed in loose lifts not exceeding 200 mm in accordance with SP 105S10. Fill in the vicinity of the structural walls should be placed in 100 mm thick loose lifts. Longitudinal drains and weep holes should be installed to provide positive drainage of the backfill. Other aspects of the backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 and 3190.100.

Backfill shall be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD) under inlet and outlet structure footings or 98% SPMDD when used as backfill behind abutment retaining walls or wing walls. Heavy compaction equipment should not be employed near structural walls. Fill should be placed at moisture contents within ±2 percent of the Optimum Moisture Content. Lift thicknesses can be adjusted once the compaction equipment has been selected.

The pipe bedding shall consist of free-draining, well-graded granular material, and be pre-shaped in the transverse direction to accommodate the curved invert. A 300 mm thickness of the bedding layer that is in direct contact with the invert shall be left uncompacted. Bedding on each side of the pipe shall be completed in 200 mm lift thickness simultaneously. At no time should the levels on each side differ by more than the 200 mm uncompacted layer. Heavy vibratory equipments should not be used closed to the pipe. All equipment, including compaction, shall be operated parallel to the longitudinal axis of the pipe.

The minimum depth of cover should be 300 mm above the pipe crown. The cover material shall be placed in layers not exceeding 200 mm thickness and compacted to 95% of SPMDD. Backfill material shall be placed in layers not exceeding 300 mm thickness for the full width of the trench and be compacted to 95% of SPMDD. Backfill shall be placed to a minimum depth of 900 mm above the crown of the pipe before power operated tractors or rolling equipment shall be used for compacting.

Typical extent and specifications of pipe bedding, side fill, cover and backfill materials for supported and unsupported excavation conditions are provided in Ontario Provincial Standard Drawings OPSD 802.31. This reference drawing is provided with this report.

Qualified geotechnical personnel should monitor the placement and quality of the fill soils. Fill placement and compaction should be monitored by field density testing at regular frequencies. The recommended minimum test frequency should be one field density test per 500 m^2 for each lift of fill.

Heavy compaction equipment should not be used immediately adjacent to the walls of the structure. 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 Canadian Highway Bridge Design Code CAN/CSA-S6-06.

Fill placement and compaction during the winter months is not recommended since the required degree of compaction cannot be attained using frozen clay or granular fills.

In the case of shored excavations using sheet piles, after the completion of the excavation and backfill, removal of the sheet pile portion driven below the excavation base could cause significant settlements during and after extraction. Consideration should be given to leave in the embedded portions of these walls.

A permanent subdrainage system around the culvert pipes, inlet and outlet structures should be incorporated in the design. Depending on the location of the subdrainage, or in the absence of such system, the design should include provisions against buoyancy.

6.3 Dewatering

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

Due to the prevalent low permeability of the silty clay deposit, minor groundwater seepage is anticipated, which should be controllable by conventional temporary dewatering methods. Runoff and seepage into the excavations from perched groundwater from the fill and upper granular layers encountered should also be anticipated. In adverse conditions, these seepage rates can be significant. Provision should be made to deal with the seepage by pumping from properly filtered sumps located within the excavation. It is anticipated that movements of granular materials at the granular/clay interface will occur. In this area, blanketing of the excavation slopes with a geotextile and free draining granular material may be required to prevent the loss of ground.

All surface water should be directed away from all open excavations to prevent degradation of the subgrade. Water should not be allowed to pond in open excavations.

6.4 Corrosion Potential

Analytical testing was carried out on samples of the clay obtained from Boreholes PS3-1 and PS4-1 located nearby the culvert S-1. Table 6-1 provides the results of various analyses carried out on the soil samples to assess the potential for corrosion on concrete:

Table 6-1: Results of Analytical Testing on Soils

Location of Soil Samples	Elevation of Soil Sample	pH	Redox Potential, mV	Resistivity, ohm.cm	Sulphide, mg/kg	Sulphate, mg/kg
Borehole PS3-1 Sample 10	172	8.12	160	3280	<0.2	236
Borehole PS4-1 Sample 4	177.5	7.52	338	10000	<0.2	< 20

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.

6.5 Construction Quality Control

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

6.6 Instrumentation and Monitoring During Construction

As mentioned earlier in Section 3.2, 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.

Details and recommendations for additional instrumentation, monitoring program, as well as guidelines for alert levels, interpretation and contingencies are provided in a separate report 285380-04-118-0001.

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 at least 3 month 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.

6.7 Interaction between Culvert S-1 and Tunnel T-7

The Lennon submerged Culvert S-1 is to cross Highway 401 just west of Tunnel T-7 near Highway 401 Sta. 10+425L. The geotechnical aspects of Tunnel T-7 culvert are being addressed under a separate cover. The base of the submerged culvert excavation is approximately 167.0 m, which is 6 m deeper than the invert of the Tunnel T-7. The Culvert S-1 is 20-25 m away from the end of Tunnel T-7 and 10-15 m away from the end of the wing wall.

The Tunnel T-7 structure is supported on piles in the vicinity of the culvert; however the RSS walls may be impacted by the construction of the drain. Stress deformation analyses indicated that the excavation for the submerged culvert if carried out after the completion of the Highway 401 excavation would have nominal impact at a distance greater than 15 m. However, considering the proximity of the RSS abutment and wing walls, careful consideration must be given to the construction approach and adequate monitoring during the excavation and construction of the Culvert S-1 (assuming that the drain is constructed after substantial completion of the tunnel). Close consultation between the Contractors and Engineers should take place regarding the staging and construction methods intended.

7 Limitations of Report

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

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

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

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

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

The soil boundaries indicated have been inferred from non-continuous sampling, observations of drilling resistance, Nilcon vane, 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.

8 Closure

The geotechnical design for the Lennon submerged culvert S-1 was developed by Mr. Ganan Nadarajah, P.Eng. and checked by Dr. Dan Dimitriu, P.Eng. (Project Lead Designer). The project was executed under the technical direction of Dr. Narendra S. Verma, P.Eng. (Technical Director) who also provided the senior review of the report. Mr. Matt Oldewening, P.Eng. managed the geotechnical investigation and Mr. Brian Lapos, P.Eng. is the project manager.

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

LICENSED PROFESSIONAL ENGINEER
G. NADARAJAH
100183764
PROVINCE OF ONTARIO

Ganan Nadarajah, M.A.Sc., P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
D.V.S. DIMITRIU
PROVINCE OF ONTARIO

Dan Dimitriu, Ph.D., P.Eng.
Associate Geotechnical Engineer
(Project Lead Designer)

LICENSED PROFESSIONAL ENGINEER
N. S. VERMA
PROVINCE OF ONTARIO

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

9 References

R-1. Aas, G., Lacasse, S., Lunne, T. and Hoeg, K., 1986, Use of in situ tests for foundation design on clays. Proc. ASCE Spec. Conf. In Situ '86, ASCE GSP 6, 1-30.

R-2. ASTM Standards

R-3. Barton, N. R., Lien, R. and Lunde, J., (1974). Engineering Classification of Rock Masses for the Design of Tunnel Support, Rock Mech. 6(4), 189-239.

R-4. Bhusahan, Kul, Amante, Carlos V. and Saaty, Ramzi, 2000, Soil improvement by precompression at a tank farm site in Central Java, Indonesia, Feb. 14, 2000.

R-5. Bieniawski, Z.T. (1976). Rock mass classification in rock engineering. In exploration for rock engineering, Proc.. of the Symp. on Exploration for Rock Engineering (ed. Z.T. Bieniawski) A.A. Balkema, Rotterdam, 1, 97-106. Cape Town.

R-6. Bjerrum, L., 1972, Embankments on soft ground: SOA Report. Proc. Specialty Conference on Performance of Earth and Earth-Supported Structures, ASCE, Purdue, 2, 1-54.

R-7. Campanella, R.G. and Howie, J.A., 2005, Guidelines for the Use, Interpretation and application of seismic piezocone test data, A Manual on Interpretation of Seismic Piezocone Test Data for Geotechnical Design, June 2005.

R-8. Canadian Geotechnical Society, 2006, Canadian Foundation Engineering Manual (CFEM), 4th Edition.

R-9. Canadian Standard Association, 2006, Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-06 S6.1.06.

R-10. Canadian Standard Association, 2009, Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete CAN/CSA-A23.

R-11. Chandler, R.J., 1988, The in-situ measurement of the undrained shear strength of clays using the field vane: SOA paper." Vane Shear Strength Testing in Soils Field and Laboratory Studies, ASTM STP 1014, 13-44.

R-12. Demers, D. and Leroueil, S., 2002, Evaluation of preconsolidation pressure and the overconsolidation ratio from piezocone tests of clay deposits in Quebec. Canadian Geotechnical Journal, 39(1), 174-192.

R-13. Department of the Navy, 1986, Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2, Naval Facilities Engineering Command.

R-14. Dittrich, J.P., Rowe, R.K. Becker, D.E. and Lo, K.Y., 2010, Influence of exsolved gases on slope performance at the Sarnia approach cut to the St. Clair Tunnel, Canadian Geotechnical Journal, 47, 971-984.

R-15. Duncan, J.M. et al, 1980, Strength, Stress-Strain and Bulk Modulus Parameters for Finite Element Analyses of Stresses and Movements in Soil Masses, Report UCB/GT/80-01

R-16. Golder Associates Ltd., 2006, Structure Settlement Study, Highway 401 Reconstruction, GWP 64-00-00, MOT Ontario, SW Region, Geocres No. 40J2-79, November, 2006

R-17. Golder Associates Ltd., 2007, Preliminary foundation investigation and design report, Detroit River International Crossing Bridge Approach Corridor, Geocres No. 40J6-18, October 2007.

R-18. Golder Associates Ltd., 2009, Foundation Investigation and Design of Noise Barrier Walls, Geocres 40J2-114

R-19. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Geocres No. 40J6-27, June 2009.

R-20. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Subsurface Conditions – Baseline Report, Geocres No. 40J6-28, June 2009.

R-21. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Subsurface Conditions – Interpretation Report, Geocres No. 40J6-28, Revision December 2009.

R-22. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 1 – Soil Chemistry Data, Geocres No. 40J6-27, February 2010.

R-23. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 2 – In Situ Cross Hole and Vertical Seismic Profile Testing, Geocres No. 40J6-27, March 2010.

R-24. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 3 – Supplementary Cone Penetration Testing, Geocres No. 40J6-27, February 2010.

R-25. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 4 – Supplementary Geotechnical Investigation, March 2010.

R-26. Grozic, J.L., Robertson, P.K., and Morgenstern, N.R., 1999, The behaviour of loose gassy sand, Canadian Geotechnical Journal, 36, 482-492.

R-27. Hudec, P.P., Geology and geotechnical properties of glacial soils in Windsor, 1998.

R-28. International Society for Rock Mechanics (ISRM), 1978. Suggested methods for the quantitative description of discontinuities in rock masses. Int. J Rock Mech. Min. Sci. & Geomech. Abstr. 15, 319-368.

R-29. ISSMGE Committee TC16, 2001, The Flat Dilatometer tests (DMT) in soil investigations Report, by the International Conference on In situ Measurements of Soil Properties, Bali, Indonesia.

R-30. Kenney, T.C., 1959, "Discussion", Journal of the Soil Mechanics and Foundations Division, ASCE, vol. 85, no. SM3, pp. 67-79

R-31. Kulhawy, F.H. and Mayne, P.W., 1990, Manual on Estimating Soil Properties for Foundation Design, Report EPRI-EL6800, Palo Alto, CA, Electric Power Research Institute.

R-32. Ladd, C.C., 1971, Strength Parameters and Stress-Strain Behaviour of Saturated Clays, Research Report R71-23, Soils Publication 278, Dept. of Civil Engineering, M.I.T., Cambridge, Mass.

R-33. Ladd, C.C., and Foott, R., 1974, New design procedure for stability of soft clays, Journal of the Geotechnical Engineering Division, 100(GT7), 763-786.

R-34. Ladd, C.C., Foott, R., Ishihara, K., Schlosser, F., and Poulos, H.G., 1977, Stress-deformation and strength characteristics: SOA report. Proc., 9th Int. Conf. on Soil Mechanics and Foundation Eng., Tokyo, 2, 421-494.

R-35. Ladd, Charles C. and DeGroot, Don J., 2004, Recommended practice for soft ground site characterization: Arthur Casagrande Lecture, 12th Pan-American Conference on Soil Mechanics and Geotechnical Engineering, MIT Cambridge, MA USA, June 22-25, 2003, Revised May 9.

R-36. Leroueil, S., Magnan, J-P., and Tavenas, F., 1990, Embankments on Soft clays, Ellis Horwood.

R-37. Leroueil, S., Demers, D., and Saihi, F., 2001, Considerations on stability of embankments on clay, Soils and Foundations, Japanese Geotechnical Society, Vol. 41, No. 5, 117-127, Oct. 2001.

R-38. Lo, K.Y. and Hinchberger, S.D., 2006, Stability analysis accounting for macroscopic and microscopic structures in clays, Keynote Lecture, Proceeding 4th International Conference on Soft Soil Engineering, Vancouver, Canada, pp 3-34, Oct. 4-6, 2006.

R-39. Lunne, T., Robertson, P.K., and Powel, J., Checks, corrections and presentation of data, CPT Testing in Geotechnical Practice, 1997

R-40. Ministry of Transportation Ontario, 1990, Pavement Design and Rehabilitation Manual, SDO-90-01.

R-41. Quigley, Robert M., 1980, Geology, mineralogy, and geochemistry of Canadian soft soils: a geotechnical perspective, National Research Council of Canada, Canadian Geotechnical Journal, Vol. 17, pp. 261-285.

R-42. Schofield, A.N. and Wroth, C.P., 1968, Critical State Soil Mechanics, McGraw Hill Book Company, London, p. 157

R-43. Sobkowicz, J.C. and Morgenstern, N.R., 1984, The undrained equilibrium behaviour of gassy sediments, Canadian Geotechnical Journal, Vol. 21, pp. 439-448.

R-44. Terzaghi, K., Peck, R.B., and Mesri, G. (1990), Soil Mechanics in Engineering Practice, John Wiley and Sons, NY.

R-45. Windsor-Essex Mobility Group, 2010, Design Submission, Section 5.1.3 – Geotechnical Design.

R-46. Wyllie, D.C., 1999, Foundations on Rock, 2nd edn, Taylor and Francis, London, UK, 401 pp.

Drawings

Project:	Windsor-Essex Parkway	**Date:**	July / 2012
Document:	Geotechnical Investigation and Design Report Submerged Culvert S-1 (Lennon Drain, Sta. 10+425 LaSalle)	**Rev:**	0
Doc No.:	285380-04-119-0019 (Geocres No. 40J6-40)	**Page No.:**	Drawings

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

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | Hatch Mott MacDonald

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

NEW CONSTRUCTION
HWY 401
LENNON SUBMERGED CULVERTS S-1
GENERAL ARRANGEMENT

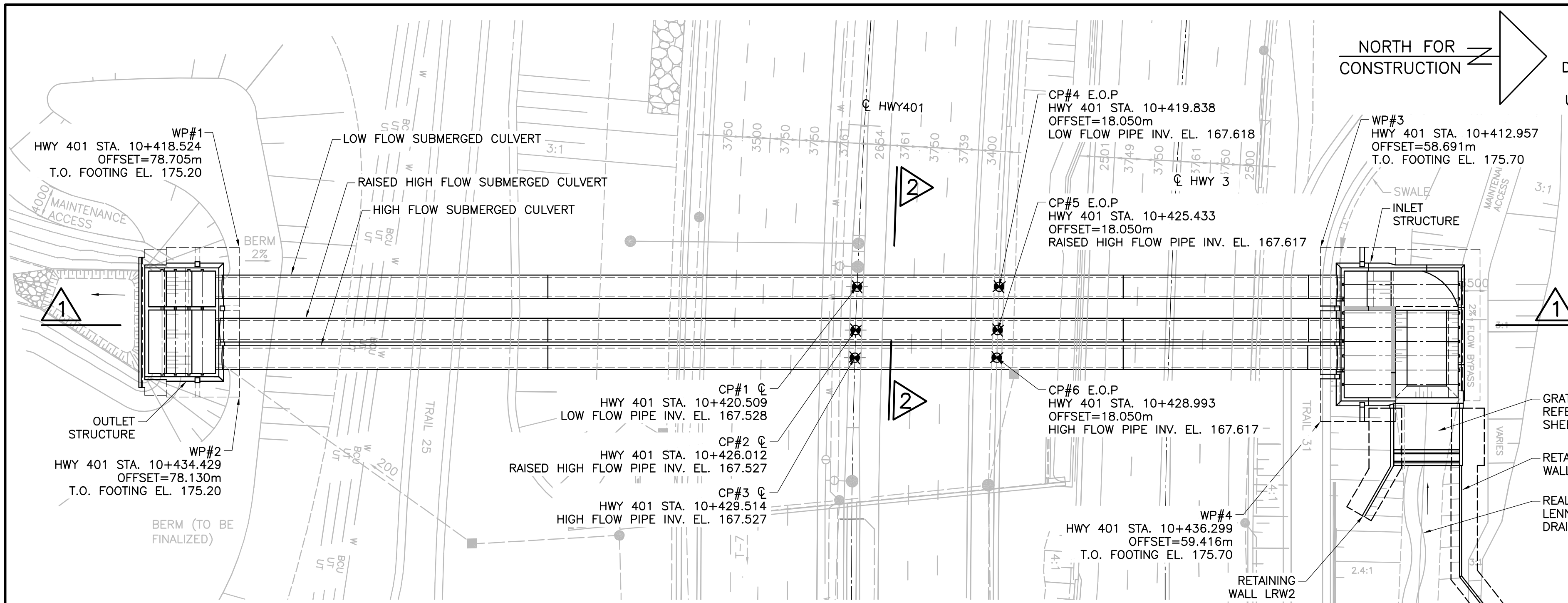
SHEET
S4101

Phase 1

IFC

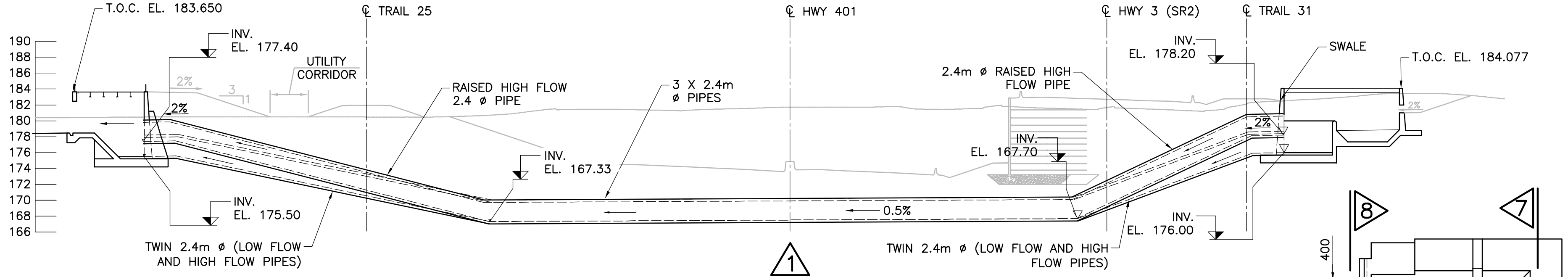
NOTES:

1. FOR GENERAL NOTES, REFER TO SHEET S4102.
2. MAX. LIVE LOAD ON ROOF GRATINGS 4kPa.

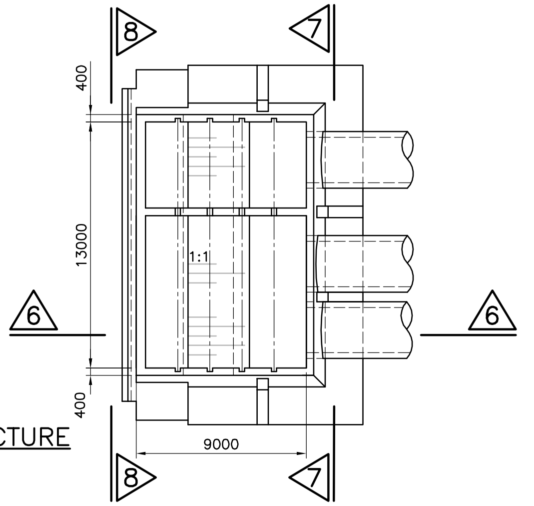


PLAN

SCALE 1:400

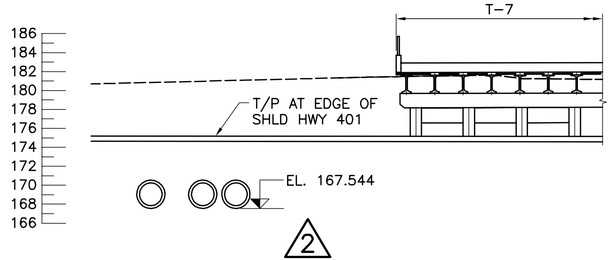


1
SCALE 1:400

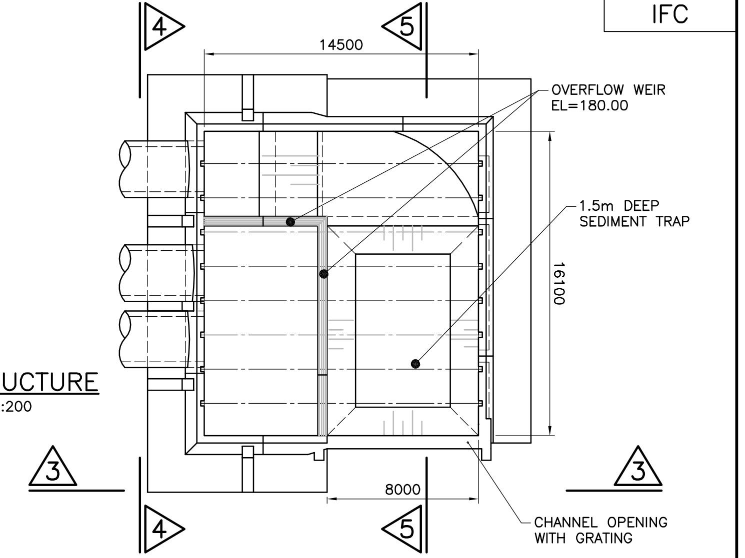


OUTLET STRUCTURE

SCALE 1:200

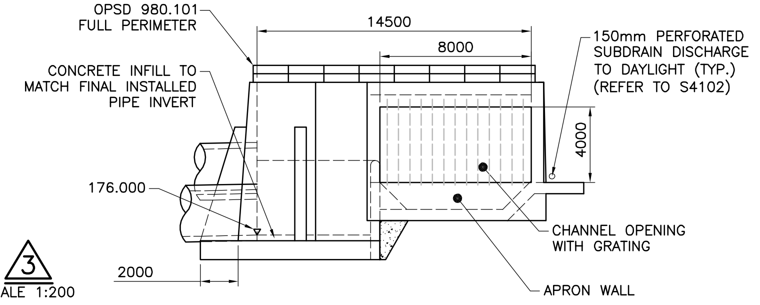


2
SCALE 1:400

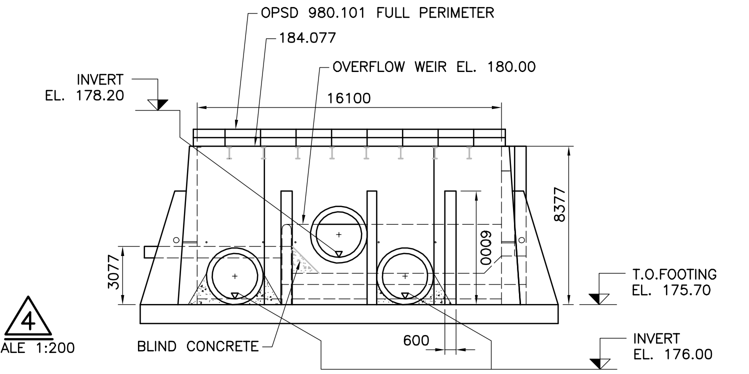


INLET STRUCTURE

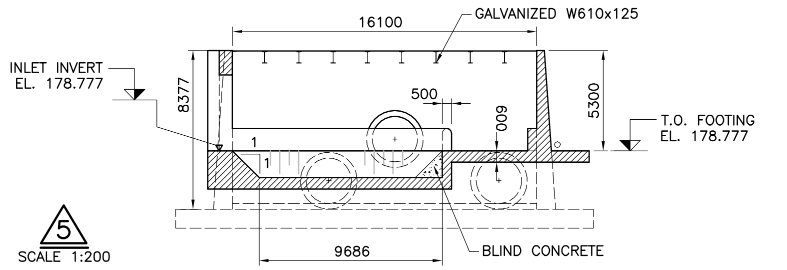
SCALE 1:200



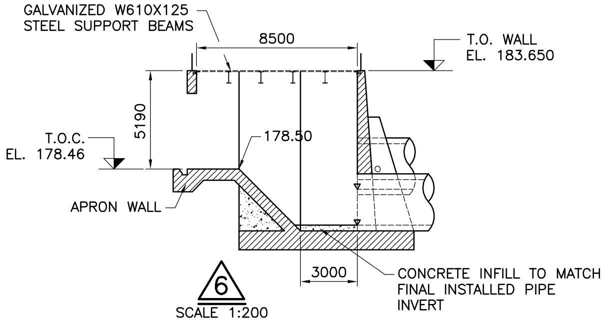
3
SCALE 1:200



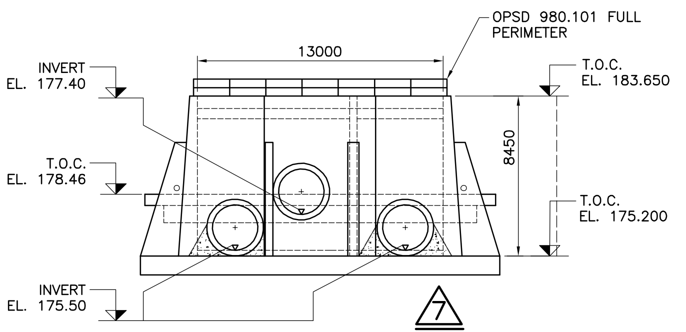
4
SCALE 1:200



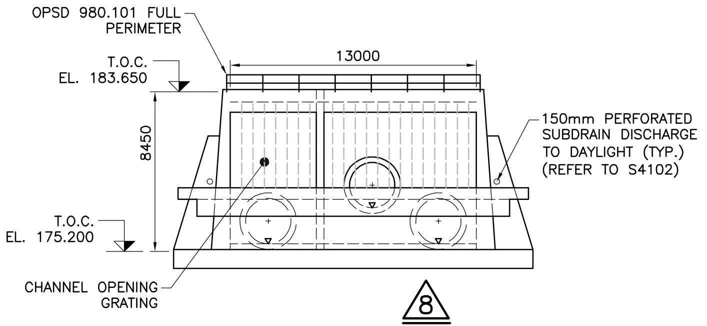
5
SCALE 1:200



6
SCALE 1:200



7
SCALE 1:200



8
SCALE 1:200

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

REVISIONS				
20-JUL-12	0	CRD	ISSUED FOR CONSTRUCTION	
DATE	REV.	BY	DESCRIPTION	
DESIGN CRD	CHK JF	CODE CAN/CSA S6-06	LOAD CL 625-ONT	
DRAWN BW	CHK BR	SITE 6-634	DATE 16-AUG-11	

DATE PLOTTED: 7/20/2012 10:39:01 AM
FILE LOCATION: C:\pwworking\hmmq_285380\stephen.labute@amec.com\dms03086\285380-03-060-WIP1-4101.dwg

METRIC

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

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

LOCATION PLAN & INTERPRETED STRATIGRAPHIC PROFILE

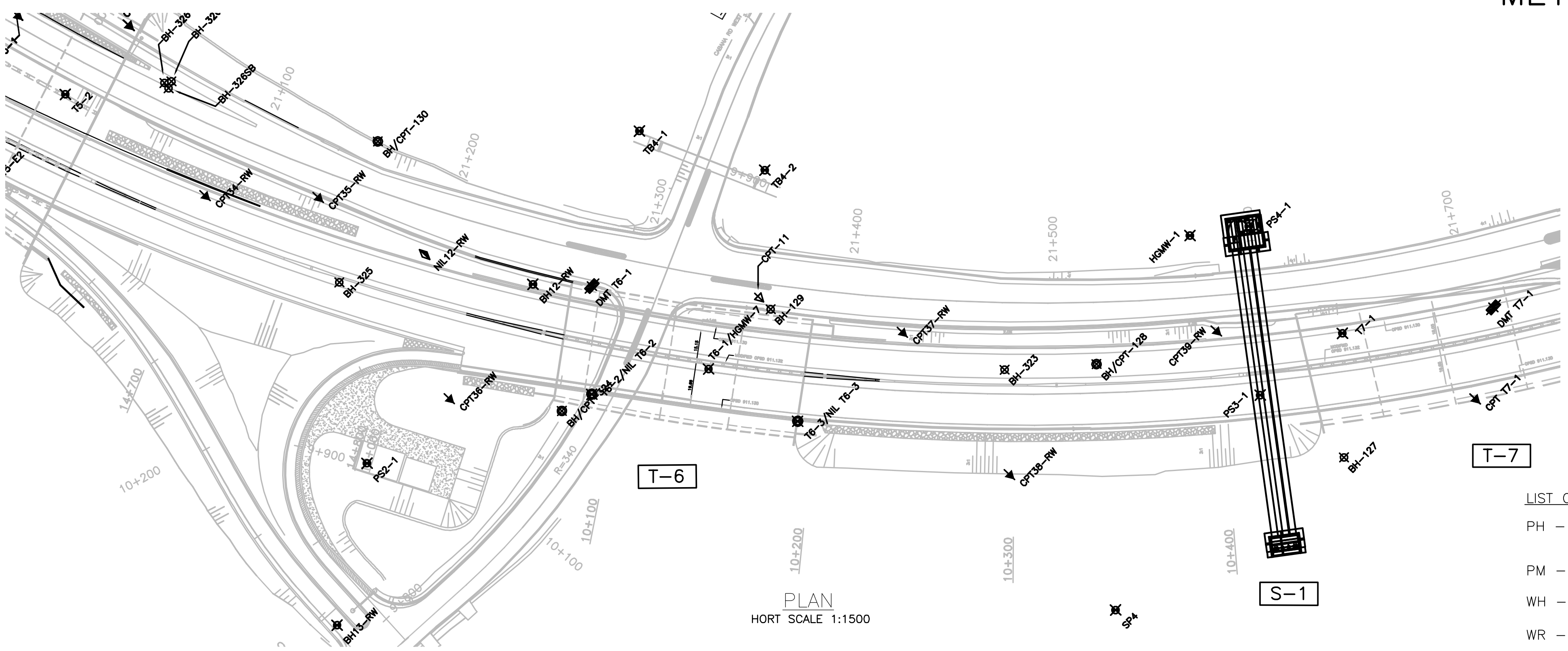
STA 14+700W TO STA 10+500L

SHEET
G4101

Phase 1

IFC

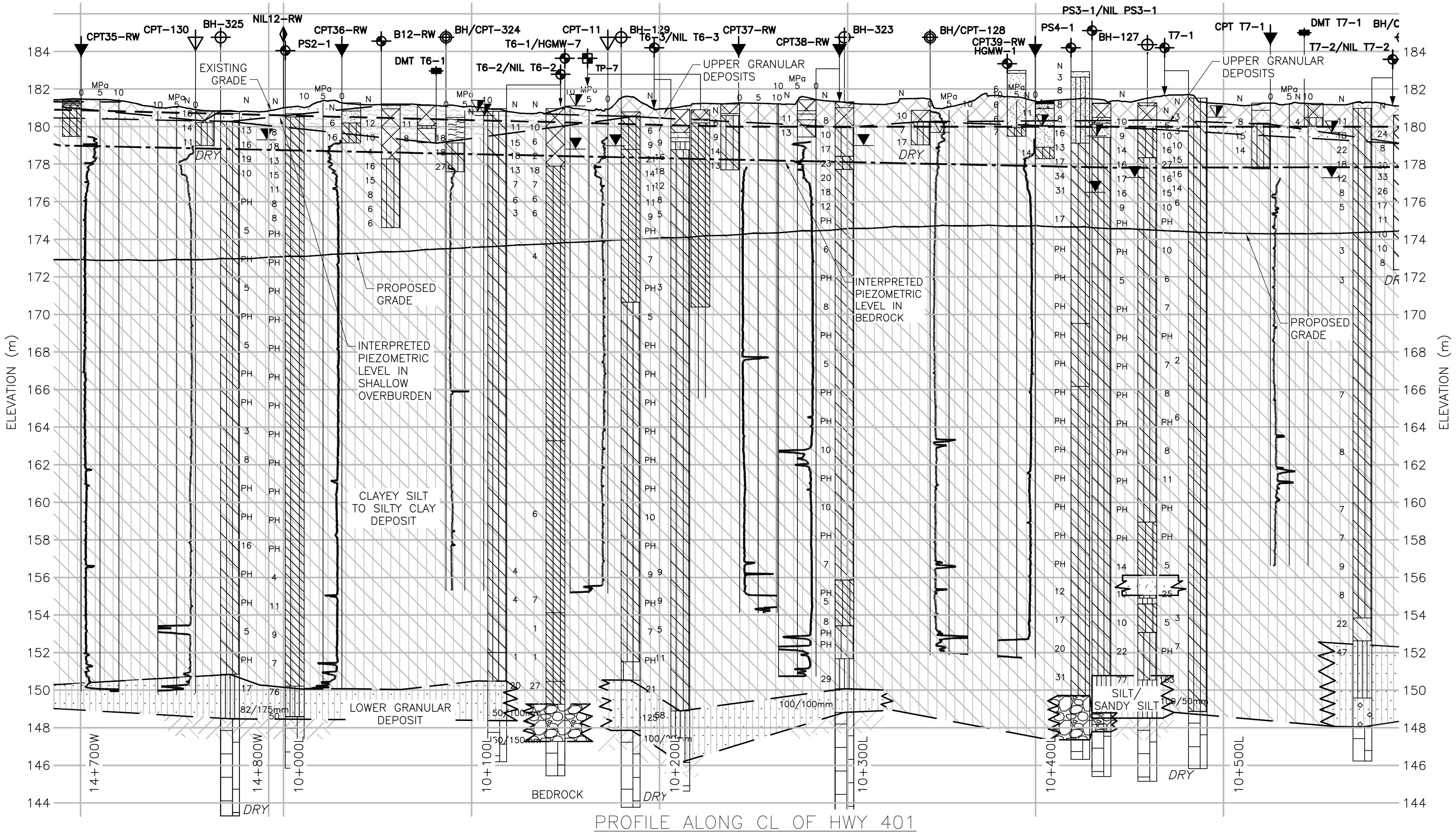
REVISIONS				
17–JUL–12	0	GN	ISSUED FOR CONSTRUCTION	
DATE	REV.	BY	DESCRIPTION	
DESIGN GN	APR NSV	DATE	30–MAY–11	



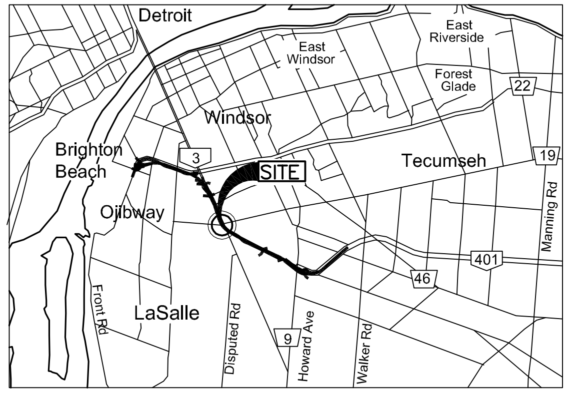
PLAN
HORT SCALE 1:1500

LIST OF ABBREVIATIONS

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



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



KEY PLAN
SCALE
1 0 2 4Km

LEGEND

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

NOTES

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

SCALES
15 0 30
Horizontal
1.5 0 3
Vertical

DATE PLOTTED: 7/20/2012 10:34:06 AM
FILE LOCATION: C:\pwworking\hmmg_285380\stephen.lobule@amec.com\dms09929\285380-04-090-WIP1-4101.dwg

METRIC

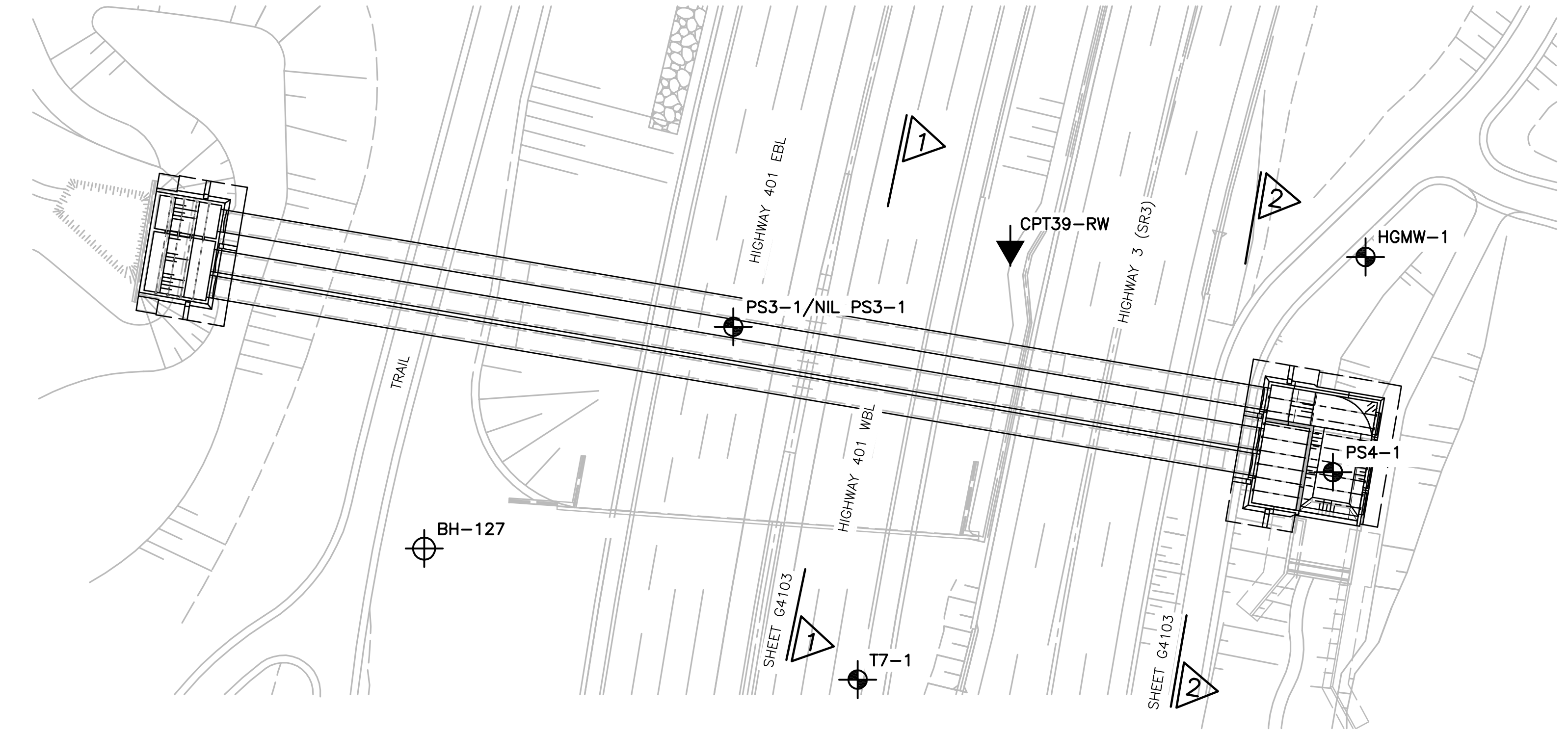
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

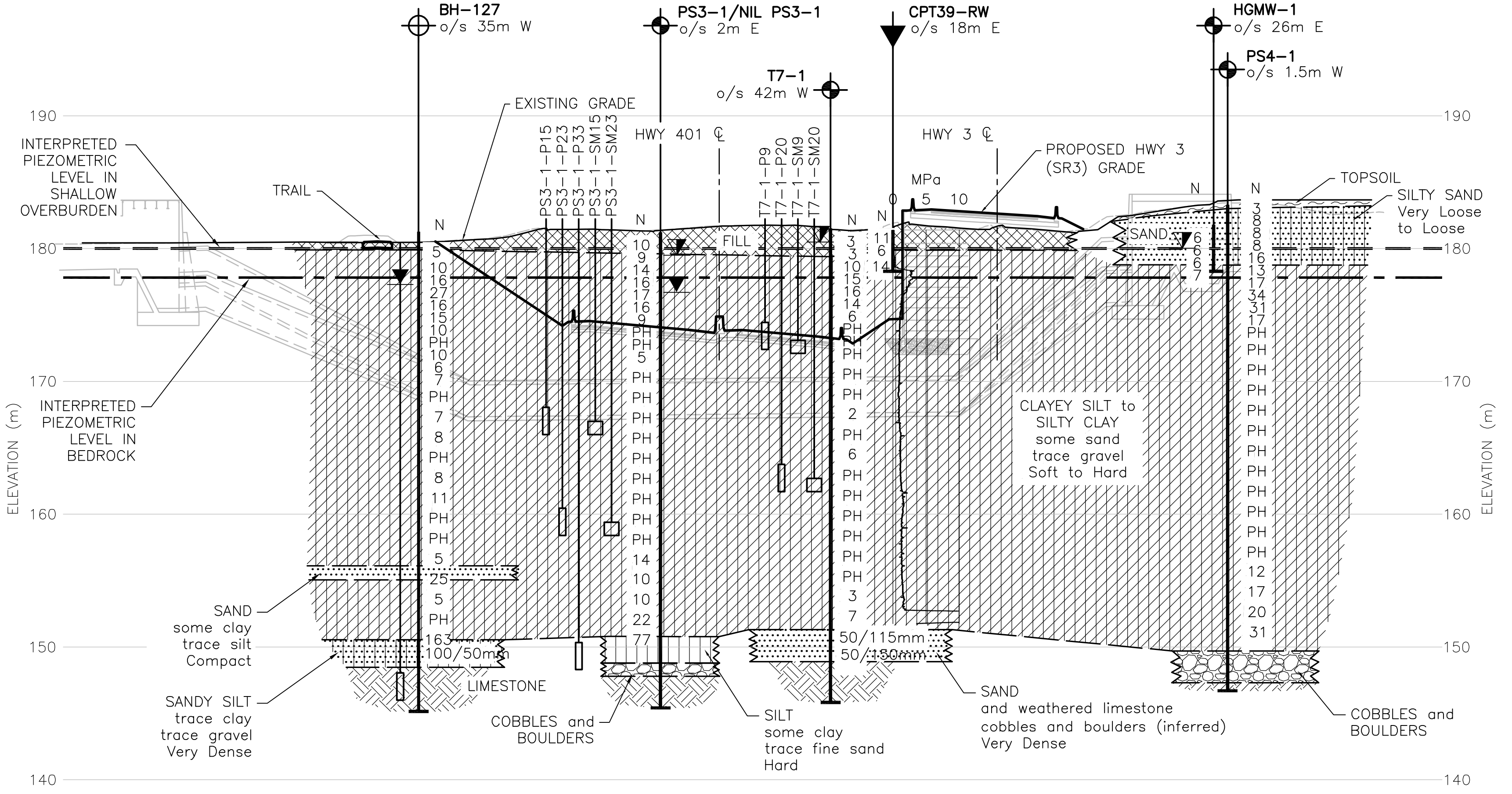
Windsor–Essex
Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
LENNON SUBMERGED CULVERTS S–1
BOREHOLE LOCATIONS & SOIL STRATA

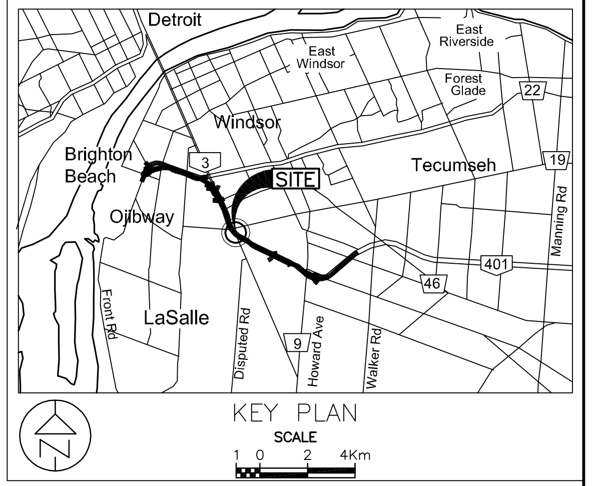
SHEET
G4102
Phase 1
IFC



PLAN
SCALE 1:500



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



KEY PLAN
SCALE
1 0 2 4Km

LEGEND

- ⊕ BOREHOLE CURRENT INVESTIGATION
- ◉ BOREHOLE AND NILCON VANE CURRENT INVESTIGATION
- ⊕ SW/SP HOLE (HYDROGEOLOGY) 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)
- □ MHSG – MAGNETIC HEAVE/SETTLEMENT GAUGE (SM)
- ▯ P – VIBRATING WIRE PIEZOMETER (VWP)
- DRY BOREHOLE DRY DURING DRILLING
- ▽ WATER LEVEL DURING DRILLING
- ▼ WATER LEVEL (SHALLOW PIEZO)
- ▼ WATER LEVEL (DEEP PIEZO)
- MPa 0 5 10 CPT–qc

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

No.	ELEVATION	CO–ORDINATES (UTM, NAD 83 ZONE 17) NORTHING	EASTING
AMEC BOREHOLES			
PS3–1/NIL PS3–1	181.3	4679421.9	332245.3
PS4–1	182.9	4679483.2	332301.5
HGMW–1	183.0	4679501.0	332278.2
T7–1	181.5	4679413.6	332295.2
CPT39–RW	181.4	4679460.1	332253.2
PREVIOUS BOREHOLES			
BH–127	181.3	4679370.9	332251.6

NOTES

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

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS DATE	REV.	BY	DESCRIPTION
17–JUL–12	0	GN	ISSUED FOR CONSTRUCTION

DESIGN GN	CHK DD	CODE CAN/CSA–S6–06	LOAD CL–625–ONT
DRAWN MM	CHK NSV	SITE 6–634	DATE 17–AUG–11

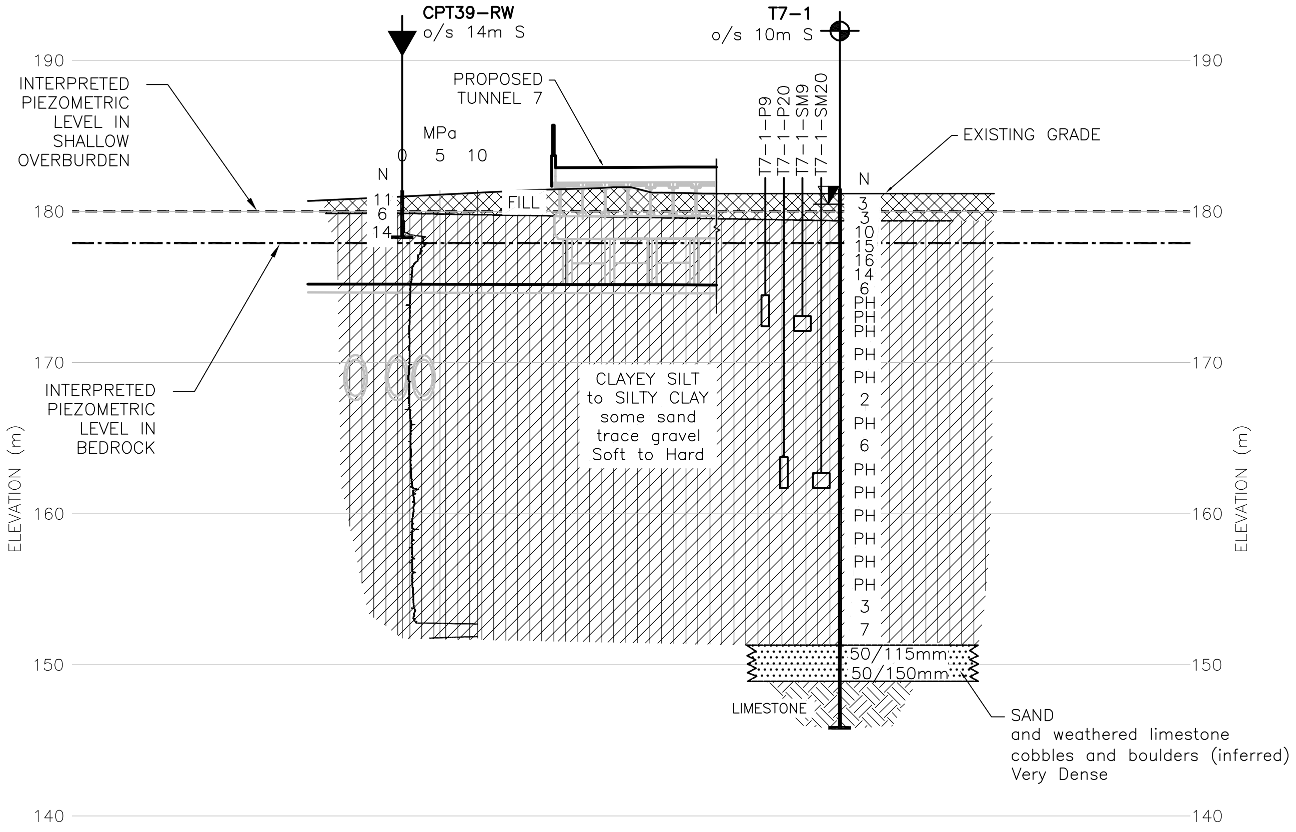
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

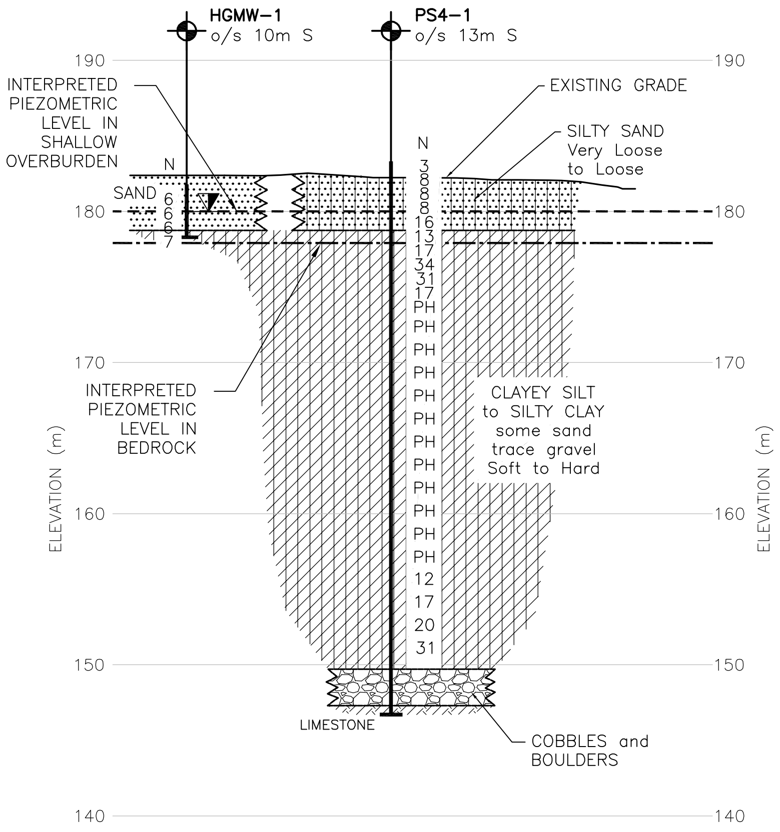
Windsor–Essex
Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
LENNON SUBMERGED CULVERTS S–1
SOIL STRATIGRAPHY

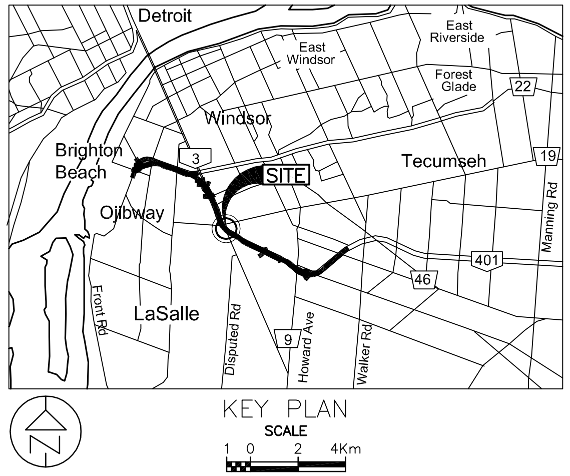
SHEET
G4103
Phase 1
IFC



1
HORT SCALE 1:500
VERT SCALE 1:250



2
HORT SCALE 1:500
VERT SCALE 1:250



KEY PLAN

LEGEND

BOREHOLE CURRENT INVESTIGATION
BOREHOLE AND NILCON VANE CURRENT INVESTIGATION
SW/SP HOLE (HYDROGEOLOGY) 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)
MHSG – MAGNETIC HEAVE/SETTLEMENT GAUGE (SM)
P – VIBRATING WIRE PIEZOMETER (VWP)
DRY BOREHOLE DRY DURING DRILLING
WATER LEVEL DURING DRILLING
WATER LEVEL (SHALLOW PIEZO)
WATER LEVEL (DEEP PIEZO)
MPa 0 5 10 CPT–qc

NOTES

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

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

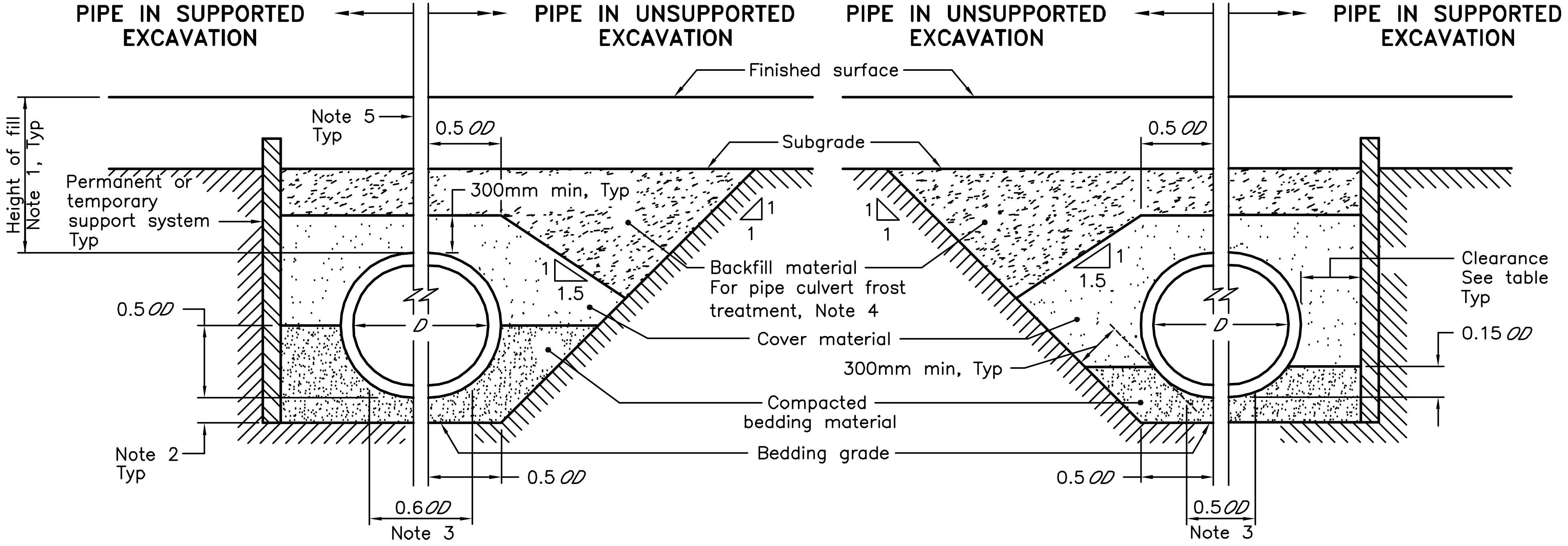
- TOPSOIL/ ORGANICS
- FILL
- SAND
- SILTY CLAY
- SILTY SAND
- COBBLES AND BOULDERS
- SILT
- SANDY SILT
- CLAYEY SILT
- SAND AND GRAVEL
- SILTY SAND AND GRAVEL
- LIMESTONE DOLOSTONE /BEDROCK

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS			
17–JUL–12	0	GN	ISSUED FOR CONSTRUCTION
DATE	REV.	BY	DESCRIPTION

DESIGN TF	CHK NSV	CODE CAN/CSA–S6–06	LOAD CL–625–ONT
DRAWN MM	CHK DD	SITE 6–634	DATE 17–AUG–11

Applicable OPSDs



CLASS B BEDDING

CLASS C BEDDING

NOTES:

1 Height of fill is measured from the finished surface to top of pipe.

2 The minimum bedding depth below the pipe shall be 0.15*D*. In no case shall this dimension be less than 150mm or greater than 300mm.

3 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.

4 Pipe culvert frost treatment shall be according to OPSD 803.030 and 803.031.

5 Condition of excavation is symmetrical about centreline of pipe.

A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.

B All dimensions are in metres unless otherwise shown.

LEGEND:

D – Inside diameter

OD – Outside diameter

CLEARANCE TABLE

Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010 | Rev 2

RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 3 SOIL – EARTH EXCAVATION

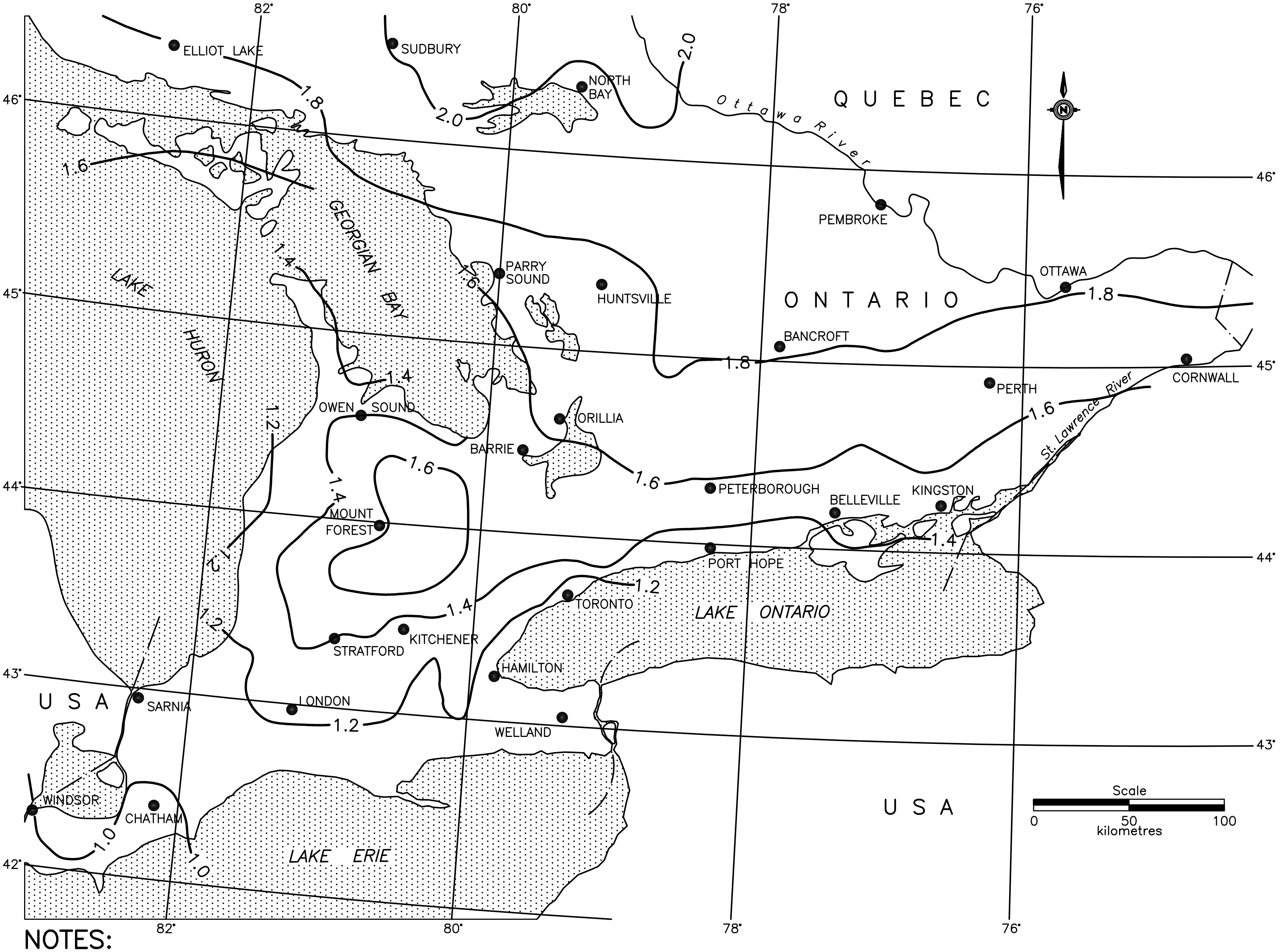
OPSD 802.031

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 | Rev 0

FOUNDATION
FROST DEPTHS
FOR SOUTHERN ONTARIO

OPSD – 3090.101



NOTES:

A These values are approximate and should only be used where the recommendations of a geotechnical engineer are not available.

B This information is based on the Ministry of Transportation and Communications Research Publication RR 225 "Aspects of prolonged exposure of pavements to sub-zero temperatures" dated December, 1981.

C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.

D Frost depths are in metres.

Figures

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

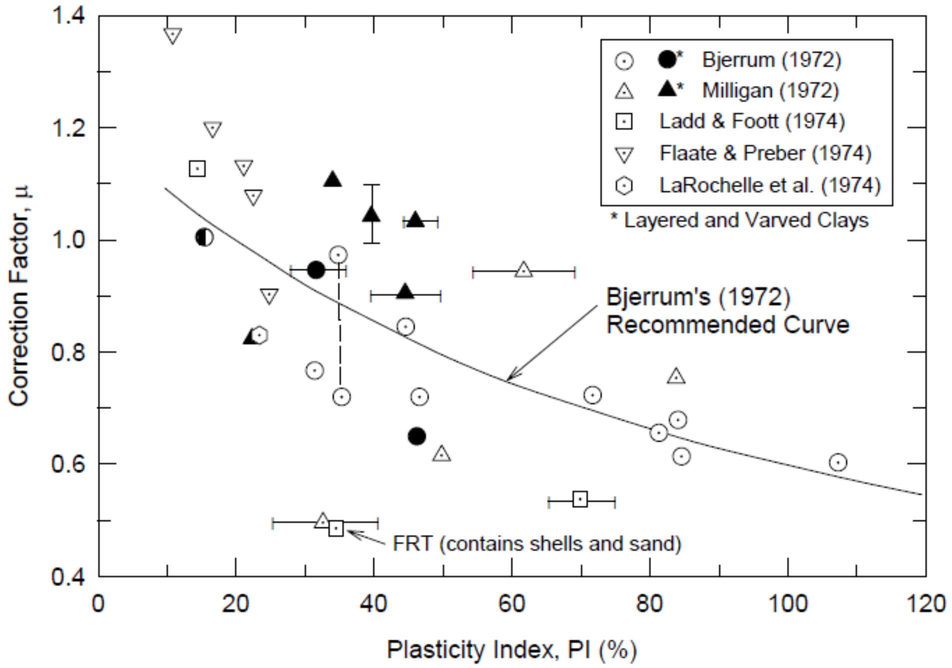


Figure 3-2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays (Ladd & DeGroot, 2004)

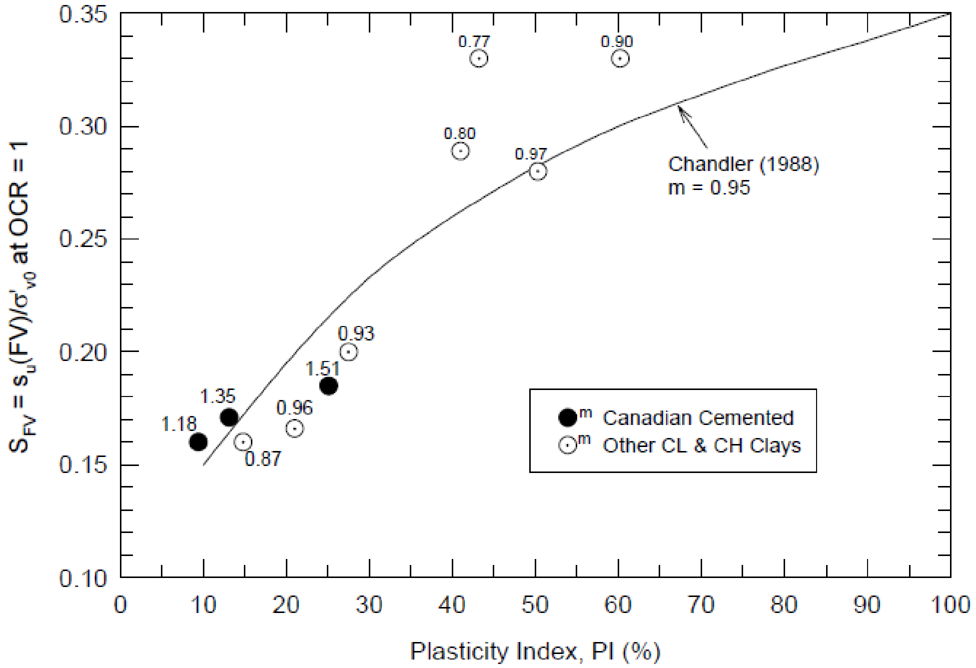
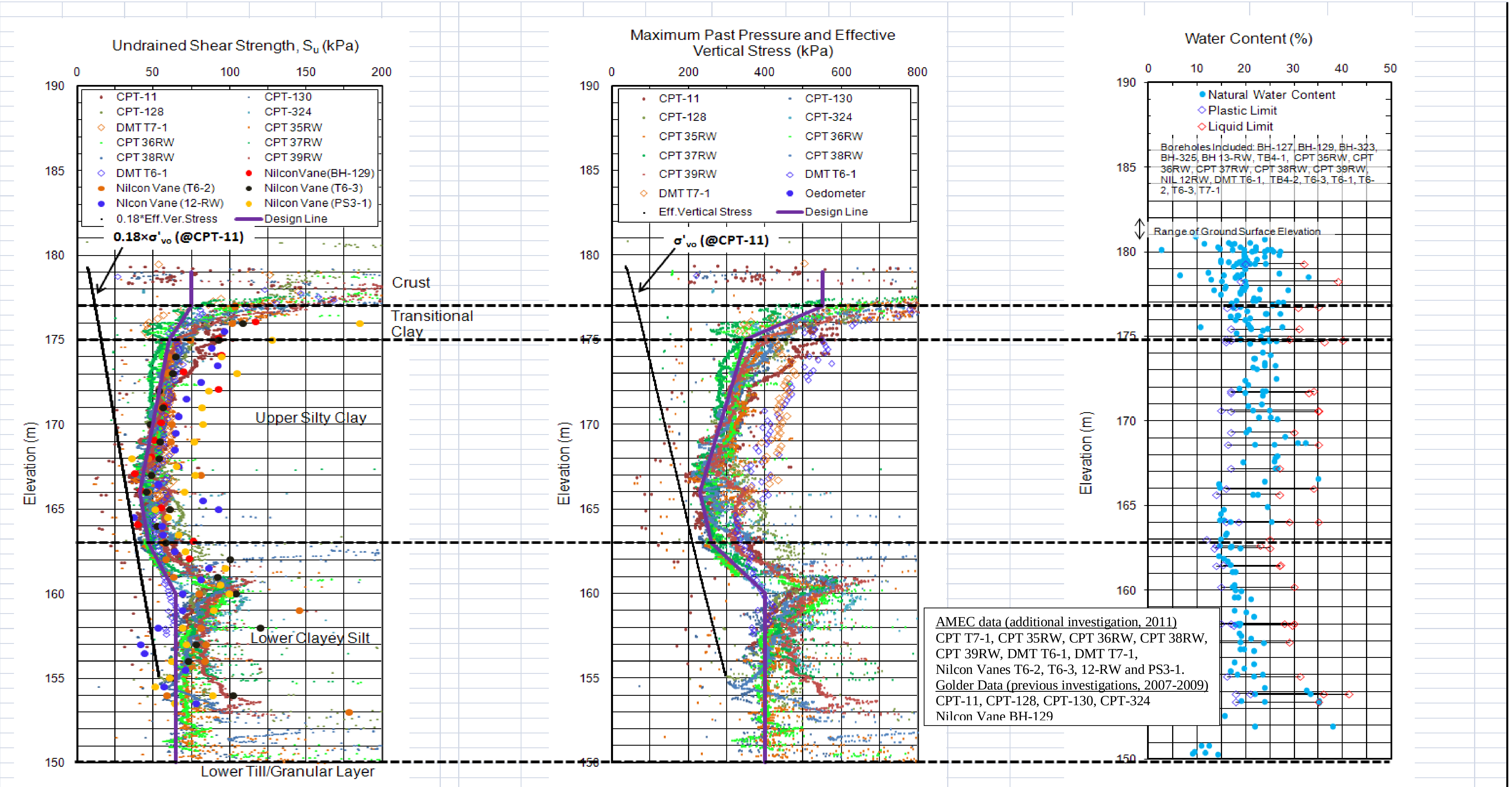


Figure 3-3a: Soil Property Profiles for for Sta. 14+700W to 10+500L



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. $OCR = [(S_u/\sigma'_V)/S]^{1/m}$

amec Earth & Environmental

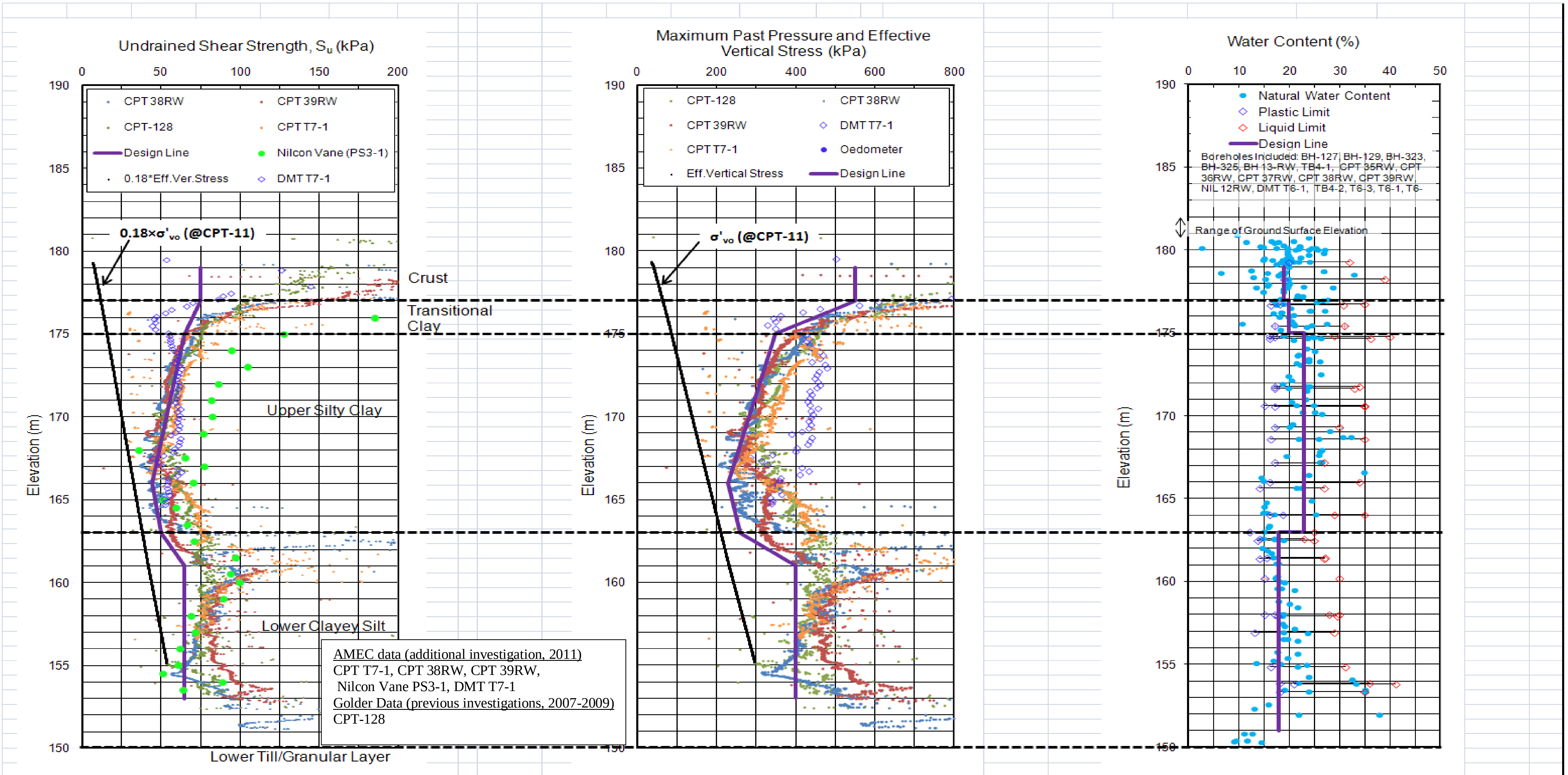
CLIENT :

PROJECT: WINDSOR ESSEX PARKWAY

TITLE: SOIL PROPERTIES PROFILES
STA. 14+700W TO 10+500L

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

Figure 3-3b: Soil Property Profiles for Lennon Submerged Culvert S-1



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. $OCR = [(S_u/\sigma'_V)/S]^{1/m}$

amec Earth & Environmental

CLIENT :

PROJECT: WINDSOR ESSEX PARKWAY

TITLE: SOIL PROPERTIES PROFILES
LENNON DRAIN SUBMERGED CULVERT S-1

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

Figure 4-1: Compressibility Parameters at WEP

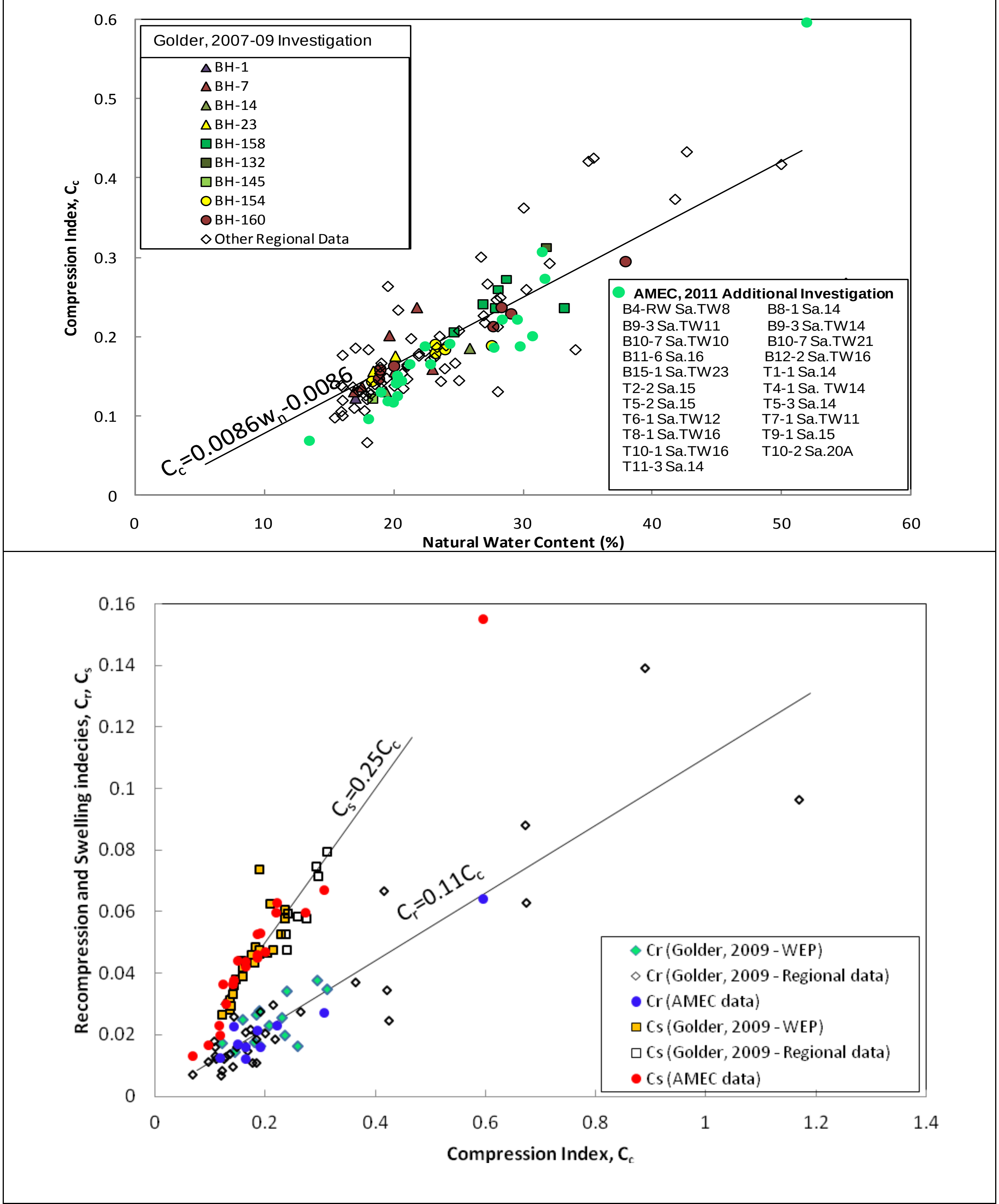


Figure 4-2: Cc versus Cα Relationship at WEP

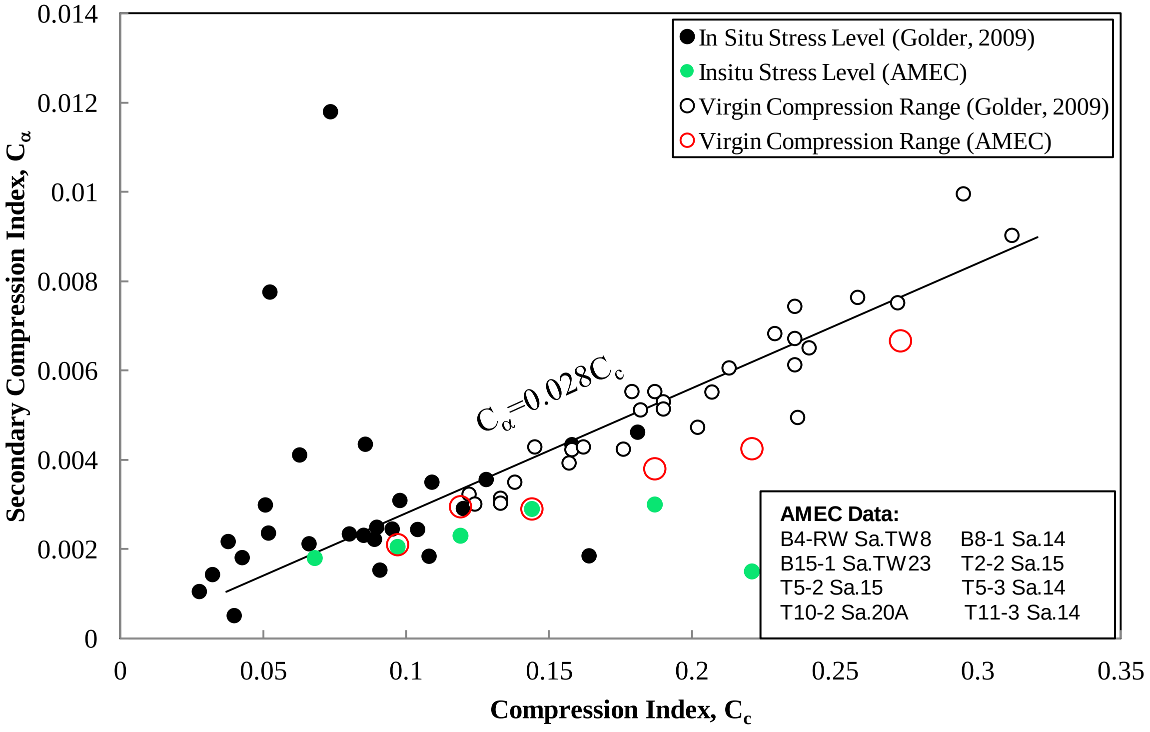


Figure 4-3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP

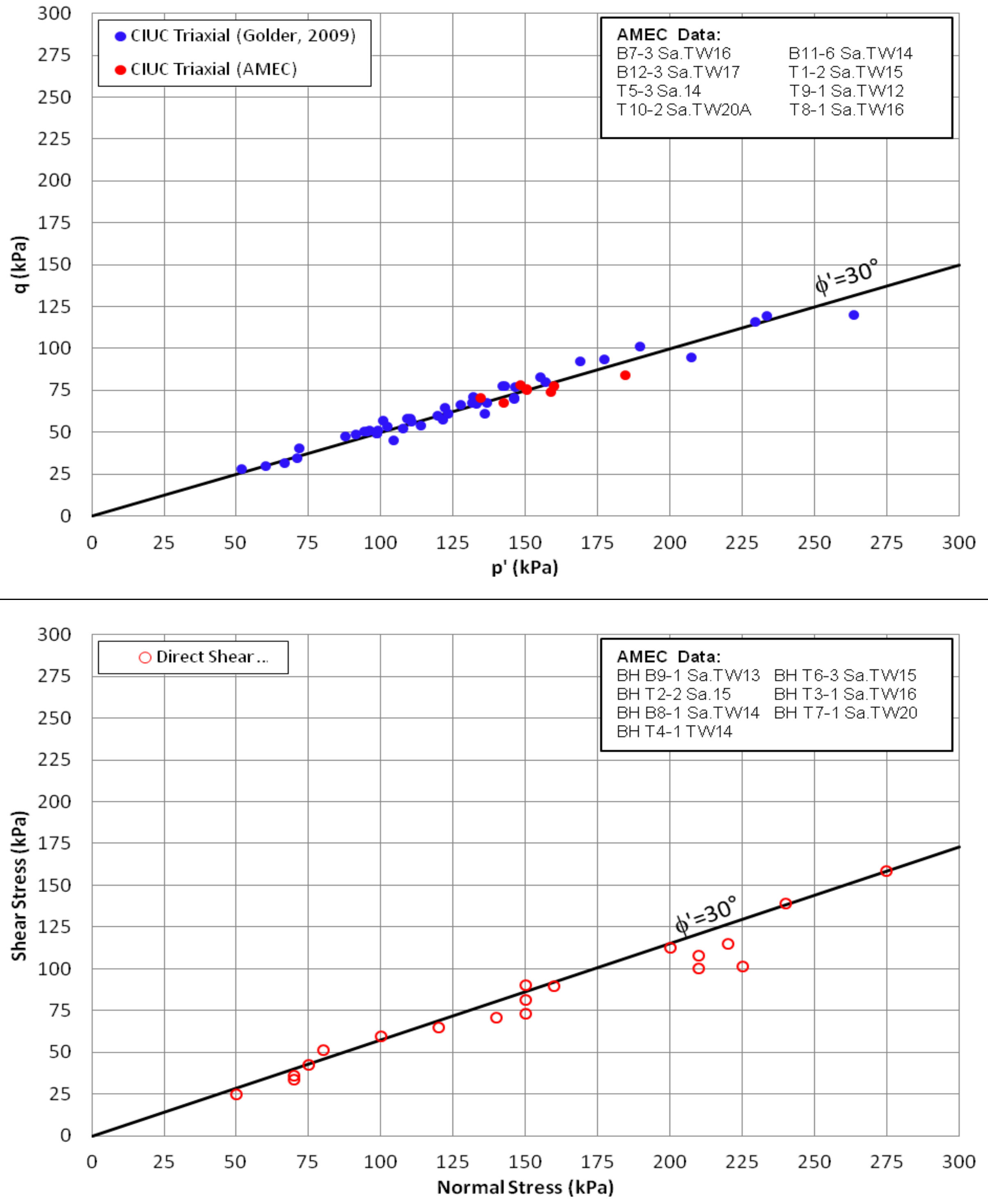


Figure 4-4: Relationship between sin ϕ′ and Plasticity Index for Normally Consolidated Soils (Kenney, 1959)

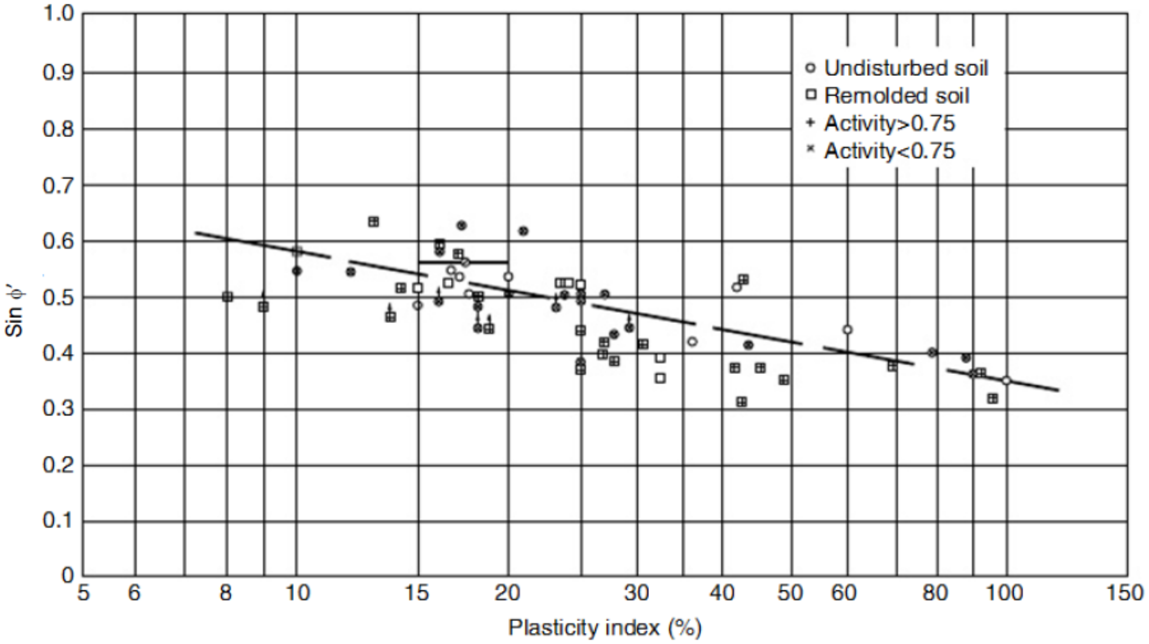
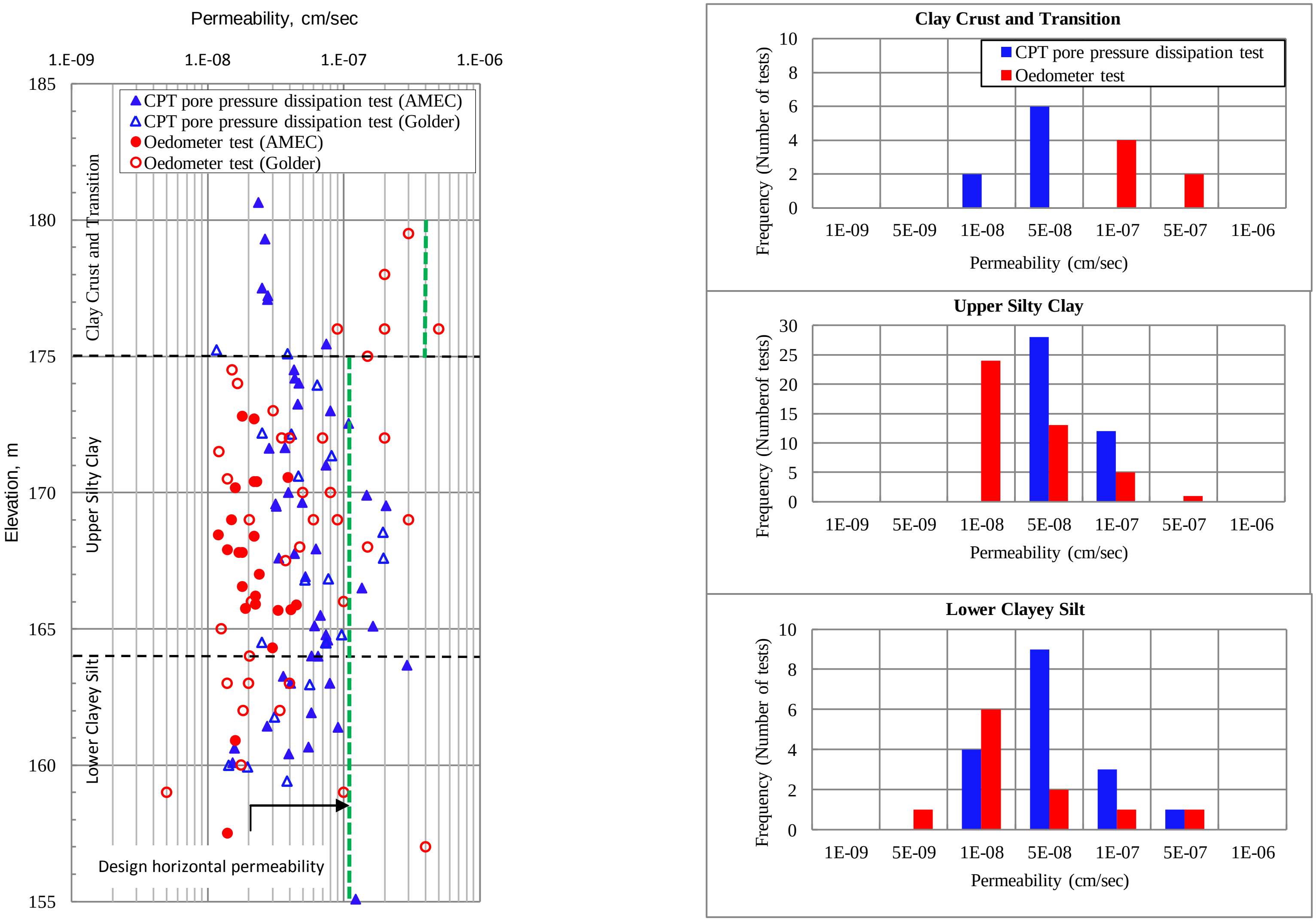
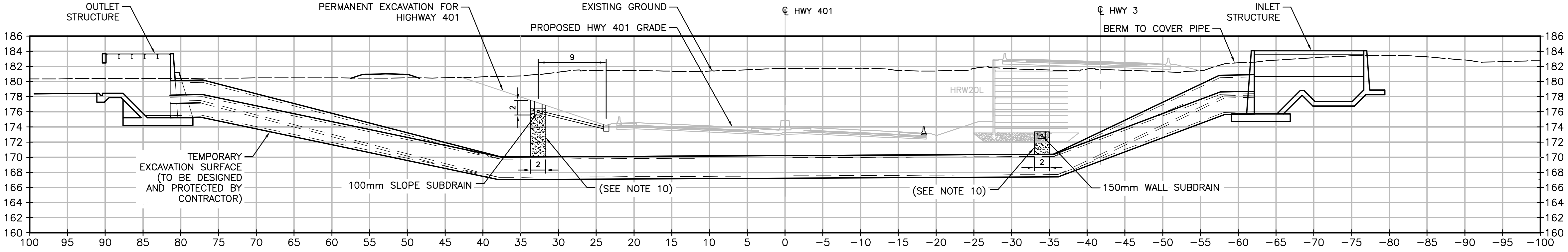


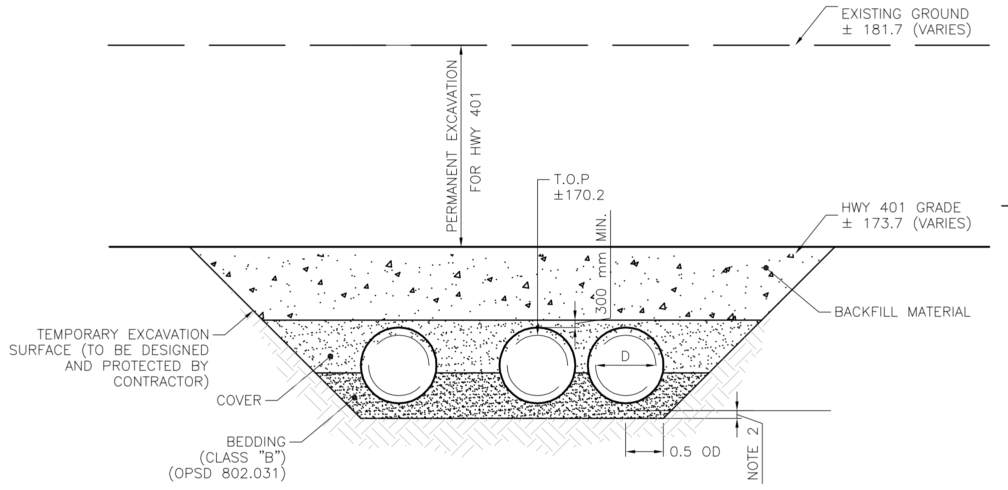
Figure 4-5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests



PLAN
SCALE 1:300

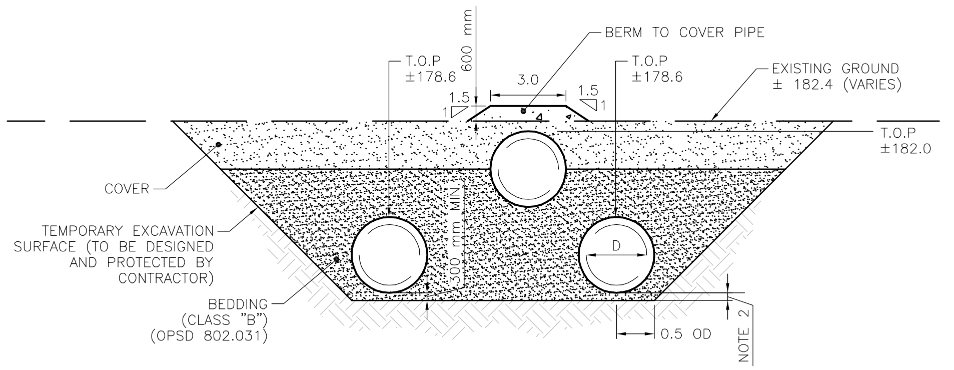


EXCAVATION FOR HIGHWAY 401
PROFILE ALONG LENNON DRAIN SUBMERGED CULVERT S-1
SCALE 1:300



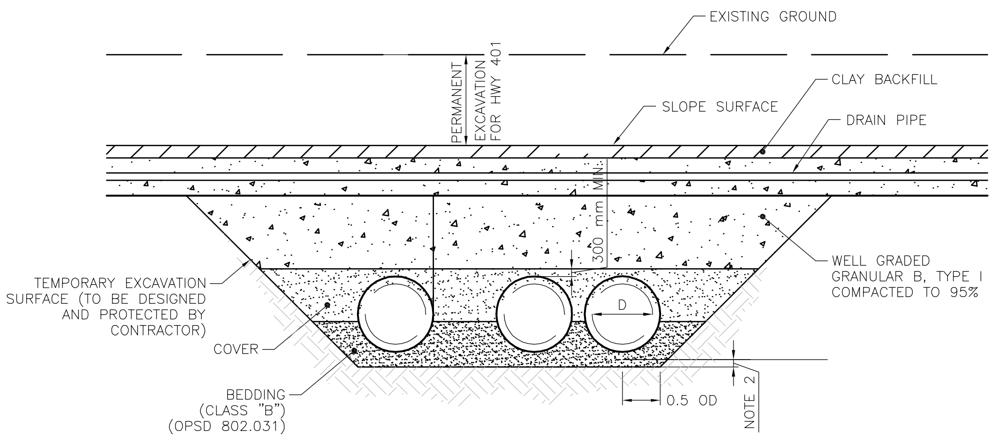
BACKFILL AROUND CULVERT UNDER HIGHWAY

1
SCALE 1:150



BACKFILL AROUND CULVERT AT INLET END

2
SCALE 1:150



BACKFILL AROUND CULVERT AT SUBDRAIN

3
SCALE 1:150

NOTES:

1. PIPE BED SHALL BE SHAPED TO BE ABLE TO RECEIVE BOTTOM OF PIPE.
2. MINIMUM BEDDING DEPTH BELOW PIPE SHALL BE 300mm.
3. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE SHOWN.
4. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
5. THIS DRAWING ILLUSTRATES THE GENERAL ARRANGEMENTS AT SELECTED REPRESENTATIVE LOCATIONS OF THE CULVERT BASED ON GEOTECHNICAL DESIGN ANALYSES.
6. 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.
7. CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS. EXCAVATED CLAY SURFACES ARE SUSCEPTIBLE TO DETERIORATION AND EXPERIENCE DEFORMATIONS AND INSTABILITY; THEY ARE TO BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED AND TREATED AS REQUIRED.
8. FOR DETAILS OF SUBDRAIN REFER TO HIGHWAY DESIGN.
9. APPLICABLE OPSD: OPSD 802.031
10. PIPE BEDDING RELIEF DRAIN TRENCH (GRANULAR "A" OR CLEAN GRANULAR "B" TYPE I – OPSS 1010 WITH 100% PASSING SIEVE 37.5mm) COMPACTED TO 95% SPMDD CONNECTED TO SLOPE/RETAINING WALL RELIEF DRAIN SYSTEM.

DOC: S-1 PLAN W CULVERT SECTIONS_FIG 5.1

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

HIGHWAY 401
LENNON SUBMERGED CULVERTS S-1
CULVERT EXCAVATION AND BACKFILL DETAILS

DWG. BY:	CHK. BY:	FIGURE NO.:
SJL	GN	5.1
DATE: July-12	SHEET: 1 OF 1	

Appendix A Borehole and CPT Logs from Additional Geotechnical Investigation

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 stratums, 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 Cohesionless Soils	SPT N-Value*
Very loose	0 to 5
Loose	5 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

Consistency of Cohesive Soils	Undrained Shear Strength kPa
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.

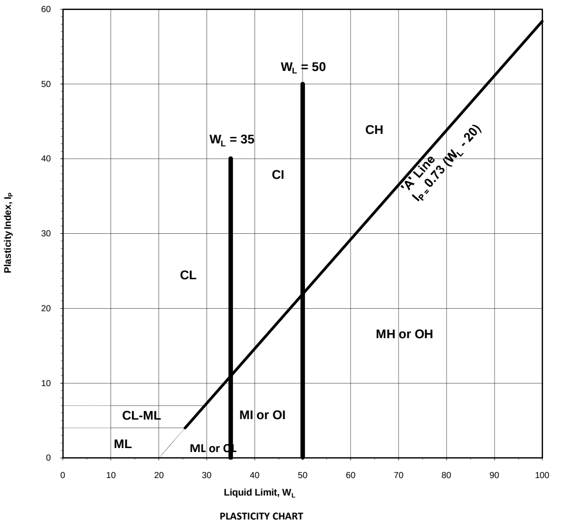
Comments

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 PARTICAL 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, & HARDNESSOF THE COARSE GRAINS, LOCAL OR GEOLOGICAL NAME & OTHER PERTINENT DESCRICTIVE INFORMATION, & SYMBOL IN PARENTHESIS.	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 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				ATTERBERG LIMITS BELOW A-LINE OR Ip LESS THAN 4	ABOVE A-LINE WITH Ip BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			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										ATTERBERG LIMITS ABOVE A- LINE WITH Ip GREATER THAN 7	
	SILT AND CLAYS	LIQUID LIMIT LESS THAN 35	DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)							
			NONE	QUICK	NONE	ML	INORGANIC SILTS & SANDY SILTS OR SLIGHTLY PLASTICITY, ROCK FLOUR	GIVE TYPE, NAME, IF NECESSARY, INDICATE DEGREE AND CHARACTER OF PLASTICITY, AMOUNT AND MAXIMUM SIZE OF COURSE GRAINS, COLOUR IN WET CONDITION, ODOUR, IF ANY, LOCAL OR GEOLOGIC NAME & OTHER PERTINENT DESCRICTIVE INFORMATION & SYMBOL IN PARENTHESIS.			$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN 1 AND 3	
			MEDIUM TO HIGH	NONE TO VERY SLOW	MEDIUM	CL	SILTY CLAYS (INORGANIC), GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS					
			SLIGHT TO MEDIUM	SLOW	SLIGHT	OL	ORGANIC SILT OF LOW PLASTICITY, ORGANIC SANDY SILTS					
		LIQUID LIMIT BETWEEN 35 AND 50	NONE TO SLIGHT	SLOW TO QUICK	SLIGHT	MI	INORGANIC COMPRESSIBLE FINE SANDY SILT WITH CLAY OF MEDIUM PLASTICIT, CLAYEY SILTS					
			HIGH	NONE	MEDIUM TO HIGH	CI	SILTY CLAYS (INORGANIC) OF MEDIUM PLASTICITY				NOT MEETING ALL GRADATION FOR SW	
			SLIGHT TO MEDIUM	VERY SLOW	SLIGHT	OI	ORGANIC SILTY CLAYS OF MEDIUM PLASTICITY	FOR UNDISTURBED SOILS AND INFORMATION ON STRUCTURE, STRATIFICATION, CONSISTANCY IN UNDISTURBED AND REMOLDED STATES, MOISTURE & DRAINAGE CONDITION.				
		LIQUID LIMIT GREATER THAN 50	SLIGHT TO MEDIUM	SLOW TO NONE	MEDIUM	MH	INORGANIC SILTS, HIGHLY COMPRESSIBLE MICACEOUS OR DIATOMECACOUS FINE SANDY SILTS, ELASTIC SILTS				ATTERBERG LIMITS BELOW A- LINE OR Ip LESS THAN 4	ABOVE A-LINE WITH Ip BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			HIGH TO VERY HIGH	NONE	HIGH	CH	CLAYS (INORGANIC) OF HIGH PLASTICITY, FAT CLAYS					
			MEDIUM TO HIGH	NONE TO VERY SLOW	SLIGHT TO MEDIUM	OH	ORGANIC CLAYS OF HIGH PLASTICITY				ATTERBERG LIMITS ABOVE A- LINE WITH Ip GREATER THAN 7	
HIGH ORGANIC SOILS	READILY IDENTIFIED BY COLOUR, ODOUR, SPONGY FEEL & FREQUENTLY BY FIBROUS TEXTURE					Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS					

FRACTION	U.S STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS		
		PASSING	RETAINED	PERCENT	DESCRIPTOR
GRAVEL	COARSE	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	



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

<u>BOUNDARY CLASSIFICATION</u>: BOUNDARY CLASSIFICATION: SOILS POSSESSING CHARACTERISTICS OF TWO GROUPS ARE DESIGNATED BY COMBINATIONS OF GROUP SYMBOLS FOE EXAMPLE GW-GC WELL GRADED GRAVEL-SAND MIXTURE WITH CLAY BINDER

AMEC Earth & Environmental,
a Division of AMEC American

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	SUSCEPTIBI LIIY TO SURFICIAL EROSION	SUSCEPTIBILI TY TO FROST ACTION	DRAINAGE CHARACTERISTICS
WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	MEDIUM	NEGLIGIBLE	NEGLIGIBLE	EXCELLENT
POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GP	VERY PERVI0US	GOOD	NEGLIGIBLE	GOOD	MEDIUM	NEGLIGIBLE	NEGLIGIBLE	EXCELLENT
SILTY GRAVELS, POORLY GRADED GRAVEL- SAND-SILT MIXTURES	GM	SEMI-PERVIOUS TO IMPERVIOUS	GOOD	NEGLIGIBLE	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	NEGLIGIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	LOW TO MEDIUM	SLIGHT	NEGLIGIBLE	EXCELLENT
POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SP	PERVIOUS	GOOD	VERY LOW	FAIR TO GOOD	LOW TO MEDIUM	MODERATE	NEGLIGIBLE 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	NEGLIGIBLE	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 PRACTICALL Y
ORGANIC SILTY CLAY OF MEDIUM PLASTICITY	OI	SEMI-PERVI0US 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 NEGLIGIBLE	NEGLIGIBLE	PRACT I.CALL Y IMPERVIOUS
ORGANIC CLAYS OF HIGH PLASTICITY	OH	IMPERVIOUS	POOR	HIGH	POOR	LOW	MODERATE	NEGLIGIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
PEAT AND OTHER HIGHLY ORGANIC SOILS	Pt	-	-	-	-	LOW	SEVERE	-	FAIR TO GOOD

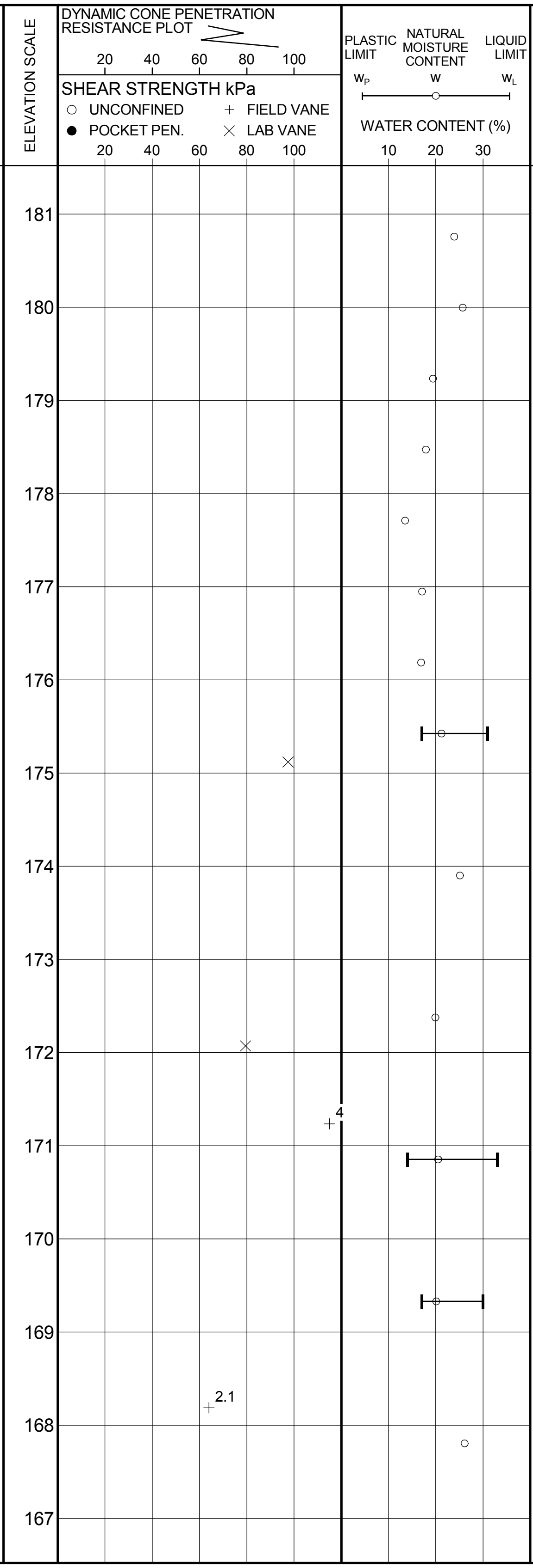
RECORD OF BOREHOLE No T7-1

1 OF 3

METRIC

W.P. RFP No. 09-54-1007
LOCATION 4679413.6N, 332295.2E
ORIGINATED BY DG
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE 7 Jul 11 - 7 Jul 11
CHECKED BY MSO

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
181.5	Pavement Surface										-Vibrating Wire Piezometers (VWP) installed in adjacent boring at (coordinates) Spider Magnets (MG) installed in adjacent boring at (coordinates) -Slight hydrocarbon odour
0.0 181.3 0.2	50mm **ASPHALT** Over 200mm Crushed Limestone Sand and Gravel **FILL**										
180.0	**FILL** Silty Clay and Topsoil Green and Black	1	SS	3							
1.5 179.4	**SAND** Poorly Graded (Fine) Trace organics, saturated Green Grey to Brown	2	SS	3							
2.1	**CLAYEY SILT** Some sand, trace gravel Grey -Trace medium-coarse gravel Very soft to very stiff Trace fine-medium gravel, pink clay nodules	3	SS	10							
		4	SS	15							
		5	SS	16							
	-Trace fissures	6	SS	14							
	-Trace pink clay nodules	7	SS	6							
	Fine sand nodules Trace fine gravel, pink clay nodules	8	TW	PH		20.6	2	17	49	32	
		9	TW	PH							
		10	TW	PH							-VWP T7-1-P9 installed at 9.14m below ground surface (EL. 172.4m) -MG installed at 9.44m below ground surface (EL. 172.1m)
			VT								
	-Trace fine-coarse gravel	11	TW	PH			4	22	38	36	
		12	TW	PH		20.6	3	22	40	35	
			VT								
	-Trace fine-medium gravel	13	SS	2							



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

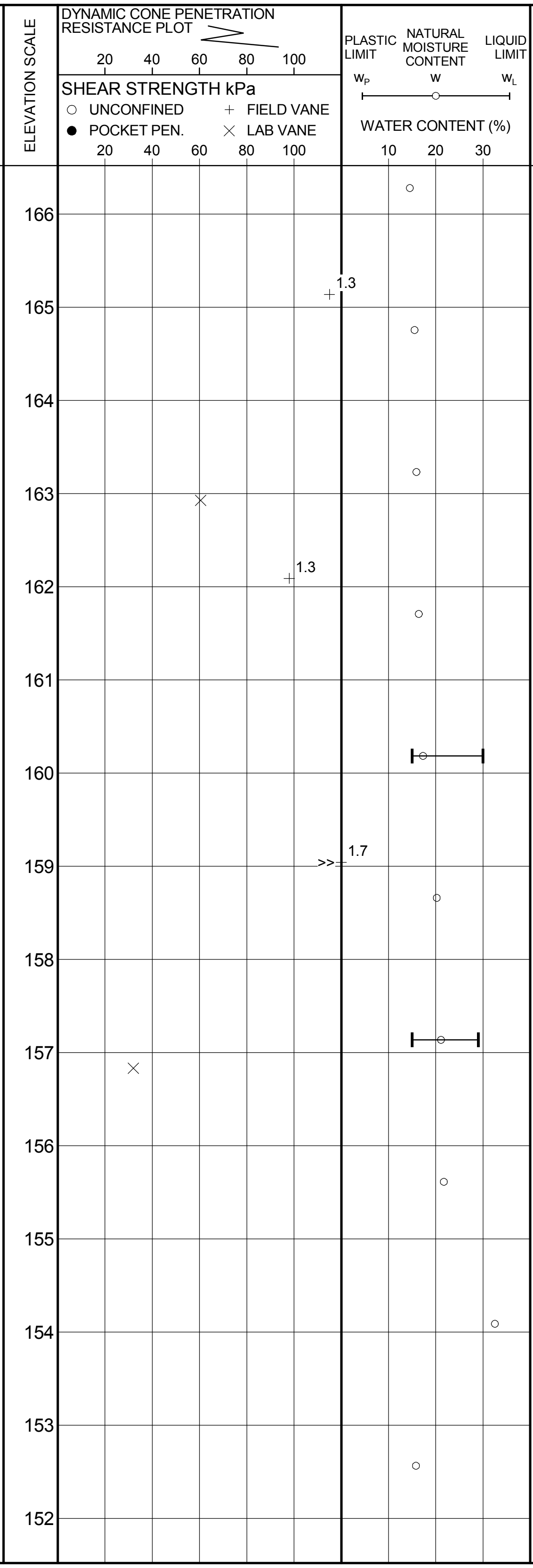
○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No T7-1

2 OF 3 **METRIC**

W.P.	RFP No. 09-54-1007	LOCATION	4679413.6N, 332295.2E	ORIGINATED BY	DG
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	7 Jul 11 - 7 Jul 11	CHECKED BY	MSO

SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
CLAYEY SILT Some sand, trace gravel Grey *(continued)*	14	TW	PH						
		VT							
-Trace sand and fine-coarse gravel	15	SS	6						
	16	TW	PH						
		VT							
-Trace fine-medium gravel	17	TW	PH						-VWP T7-1-P20 and MG T7-1-SM20 installed at 19.81m below ground surface (EL. 161.7m)
	18	TW	PH	21.5	5	26	44	25	
		VT							
-Trace coarse sand	19	SS	PH						-no recovery in shelby tube; sample retrieved by pushing split spoon
	20	TW	PH		5	20	47	28	
-Trace fine-coarse sand Wet Silt seams	21	SS	PH						-no recovery in shelby tube; sample retrieved by pushing split spoon
	22	SS	3						-pulled up shelby, end bent, no recovery in shelby tube; sample retrieved by pushing split spoon
Sandy -Red clay seams Wet	23	SS	7						



Continued Next Page

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

RECORD OF BOREHOLE No T7-1

3 OF 3 **METRIC**

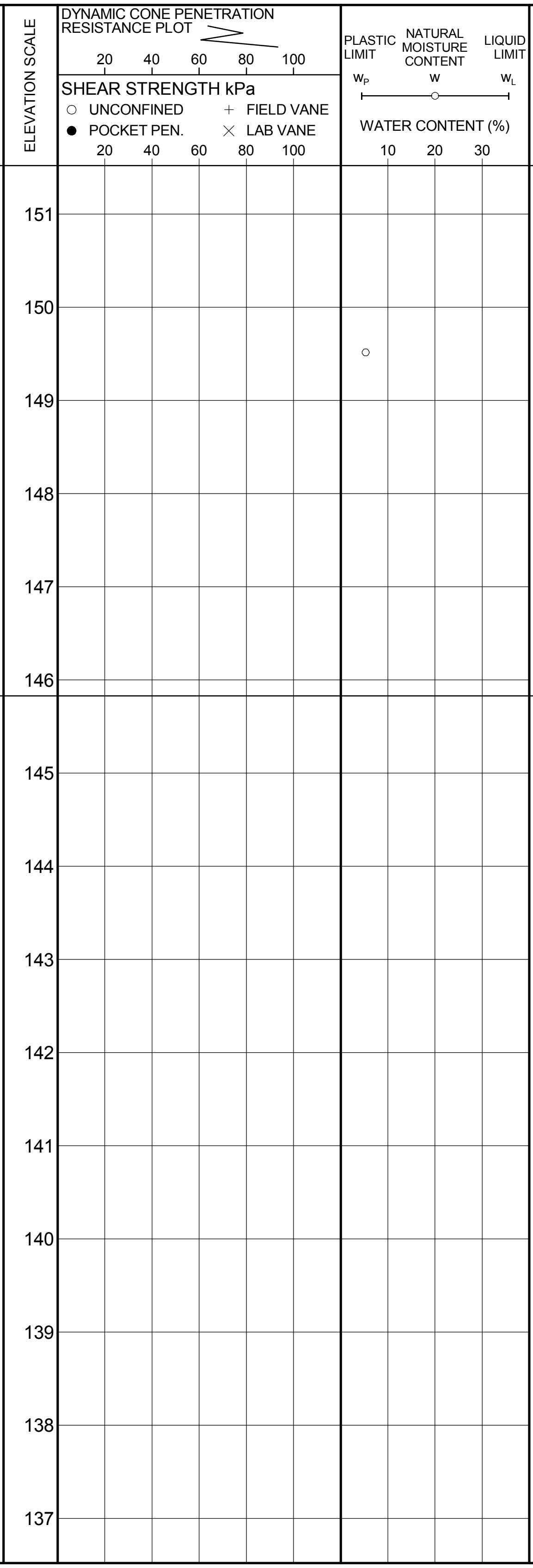
W.P. RFP No. 09-54-1007
LOCATION 4679413.6N, 332295.2E
ORIGINATED BY DG
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE 7 Jul 11 - 7 Jul 11
CHECKED BY MSO

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
151.3					
30.2	**SAND** And weathered **LIMESTONE** Cobbles and boulders (inferred) Very dense	24	SS	50/ 115mm	-no recovery, spoon bouncing continued to drill to 32m
		25	SS	50/ 150mm	
148.9 32.6	**LIMESTONE** Medium to coarse grained Porous, vuggy, fractured at location between 33.07m and 33.22m Clacite crystallization is visible Brown	1	RC		RQD = 100%
		2	RC		RQD = 100%
146.7 34.8					
146.4 35.1	**LIMESTONE** Laminated, medium to fine grained, porous Pitted at location between 34.78m and 35.14m Brown to Grey				
146.0 35.5	**LIMESTONE** Fine Grained Vuggy, calcite crystals visible Grey				
145.8 35.7	**LIMESTONE** Fine Grained Laminated, porous and dense Grey				

END OF BOREHOLE
No groundwater observed prior to starting wash boring below approx. 9.6 m on July 7, 2011

Piezometric levels in VWP T7-1-P9 (Shallow):
July 24, 2011: EL. 180.4m
August 6, 2011: EL. 180.5m

Piezometric levels in VWP T7-1-P20 (Mid-depth):
July 24, 2011: EL. 180.4m
August 6, 2011: EL. 180.4m



UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No PS3-1

1 OF 3 **METRIC**

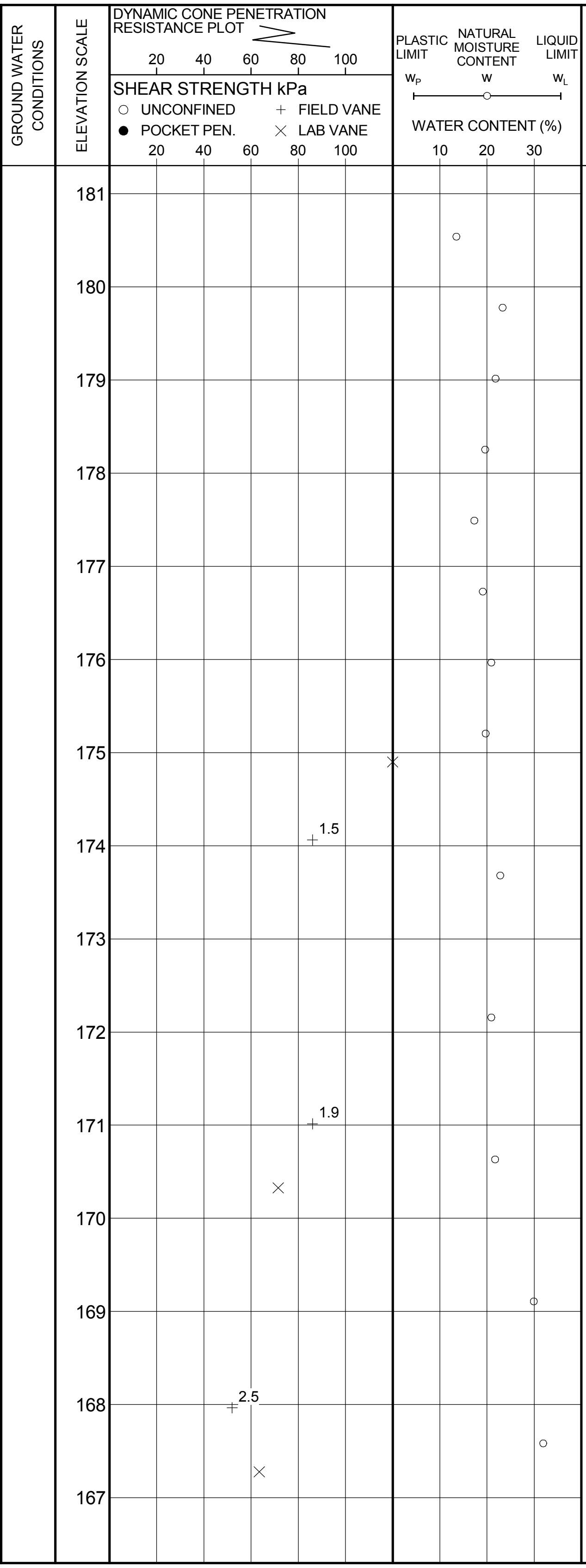
W.P.	RFP No. 09-54-1007	LOCATION	N4679421.9, E332245.3	ORIGINATED BY	RL
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	9 Aug 11 - 11 Aug 11	CHECKED BY	MSO

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
181.3	Ground Surface
181.9 0.0	**ASPHALT**
180.5 0.8	**FILL** Crushed limestone sand and gravel Grey
180.2 1.1	**FILL** Sand and gravel Brown
179.5 1.8	**SAND** Trace silt Loose Brown
	CLAYEY SILT Trace sand Firm to very stiff Grey, trace pink nodules Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
1	SS	10	
2	SS	9	
3	SS	14	
4	SS	16	
5	SS	17	
6	SS	16	
7	SS	9	
8	TW	PH	21.4
	VT		
9	TW	PH	
10	SS	5	
	VT		
11	TWtw	PH	20.6
12	TW	PH	
	VT		
13	TW	PH	21.4



REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

-Vibrating Wire Piezometers (VWP) and Spider Magents (MG) installed in borehole
-Nilcon vane advanced adjacent to sampled borehole from 5 m to 27.5 m (El. 176.3 m to El. 153.8 m)

-corrosivity sample

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

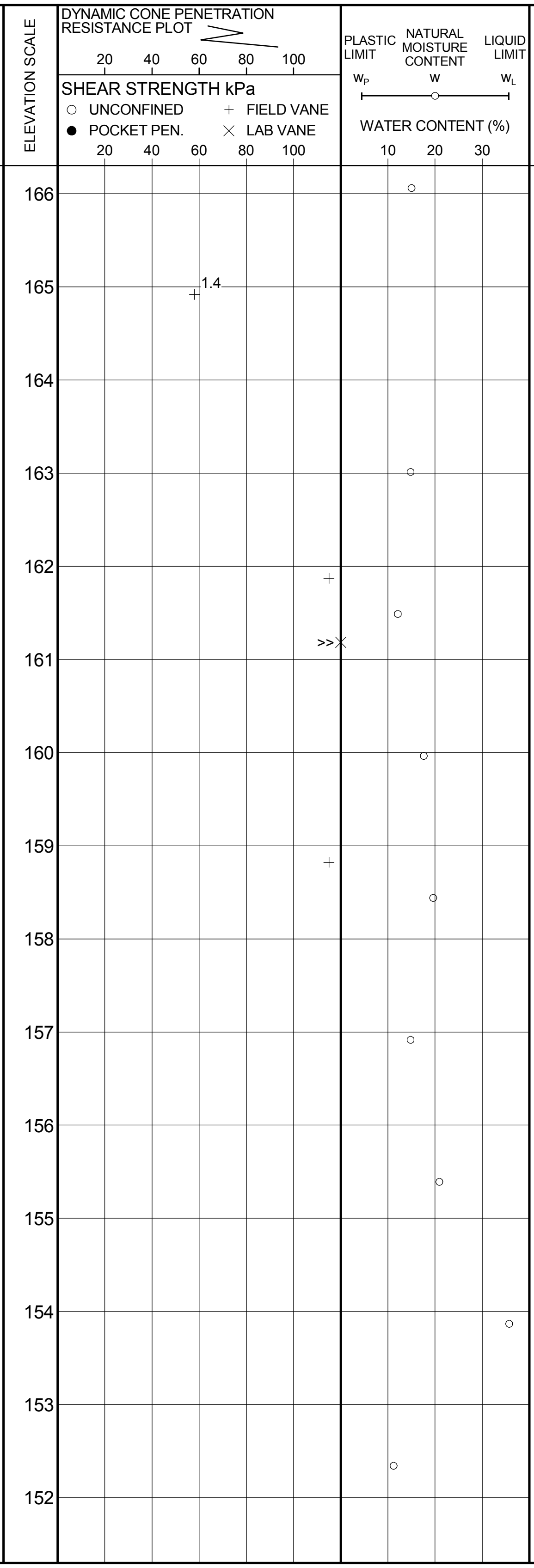
○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No PS3-1

2 OF 3 **METRIC**

W.P. RFP No. 09-54-1007
LOCATION N4679421.9, E332245.3
ORIGINATED BY RL
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE 9 Aug 11 - 11 Aug 11
CHECKED BY MSO

SOIL PROFILE		SAMPLES			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		
	CLAYEY SILT Trace sand Firm to very stiff *(continued)*	14	TW	PH		-VWP PS3-1-P15 and MG PS3-1-SM15 installed at 15.2m below ground surface (El. 166.1 m)
			VT			
						-No sample No.15
		16	TW	PH		
			VT			
		17	TW	PH	22.2	
		18	TW	PH		
			VT			
		19	TW	PH		-VWP PS3-1-P23 and MG PS3-1-SM23 installed at 22.9m below ground surface (El. 158.3 m)
		20	SS	14		
	-Sand seams	21	SS	10		
		22	SS	10		
		23	SS	22		



Continued Next Page

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

RECORD OF BOREHOLE No PS3-1

3 OF 3

METRIC

W.P. RFP No. 09-54-1007
LOCATION N4679421.9, E332245.3
ORIGINATED BY RL
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE 9 Aug 11 - 11 Aug 11
CHECKED BY MSO

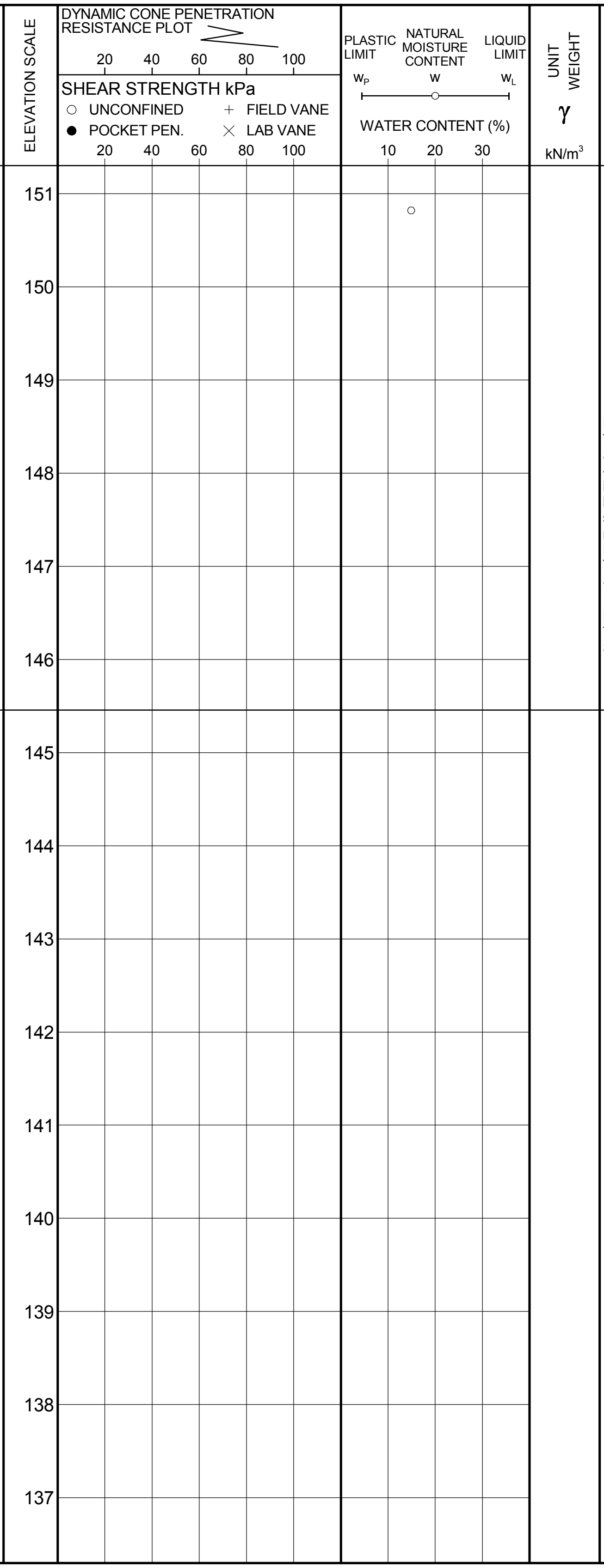
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
150.8 30.5	**SILT** Some clay, trace fine sand, some gravel, rock pieces Very dense Grey Wet	24	SS	77	
		25	SS		
148.8 32.5	**COBBLES AND BOULDERS**	26	RC		RQD = 0% TCR = 20% SCR = 1% -VWP PS3-1-P33 installed at 32.9m below ground surface (El. 148.3 m)
147.8 33.5	**LIMESTONE** Fine Grained, well crystallized, brown, vuggs throughout filled with calcite mineralization	27	RC		RQD = 90% TCR = 100% SCR = 97%
		28	RC		RQD = 90% TCR = 100% SCR = 100%
145.9 35.4	**LIMESTONE** Fine Grained, well crystallized, grey to white, black inclusions, dense				
145.5 35.8	**END OF BOREHOLE**				

No groundwater observed during drilling due to wash boring

Water level measured in Piezometer VWP PS3-1-P15 at elevation 179.5m on November 3, 2011
Water level measured in Piezometer VWP PS3-1-P15 at elevation 179.5m on November 11, 2011

Water level measured in Piezometer VWP PS3-1-P23 at elevation 176.8m on November 3, 2011
Water level measured in Piezometer VWP PS3-1-P23 at elevation 176.7m on November 11, 2011

Water level measured in Piezometer VWP PS3-1-P33 at elevation 176.8m on Nov 3, 2011
Water level measured in Piezometer VWP PS3-1-P33 at elevation 176.8m on Nov. 11, 2011



+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No PS4-1

1 OF 3 **METRIC**

W.P.	RFP No. 09-54-1007	LOCATION	N4679483.2, E332301.5	ORIGINATED BY	NB
DIST		BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
HWY	WEP				
DATUM	Geodetic	DATE	13 Aug 11 - 15 Aug 11	CHECKED BY	MSO

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
182.9	Ground Surface
0.0	**TOPSOIL**
182.6 0.3	Brown-Grey **SILTY SAND** Fine, loose Light Brown Loose Compact
179.1 3.8	Grey **CLAYEY SILT** Trace sand, trace gravel Stiff -Trace sand Very stiff Hard Some sand, trace gravel Very stiff Firm
169.5 13.4	Grey **CLAYEY SILT** Soft to firm

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	3					
2	SS	8					
3	SS	8					
4	SS	8		-corrosivity sample			
5	SS	16					
6	SS	13					
7	SS	17					
8	SS	34					
9	SS	31					
10	SS	17					
11	TW	PH	20.5	2	19	39	40
	VT						
12	TW	PH					
13	TW	PH	19.5				
	VT						
14	TW	PH		-end of drilling Aug 13; continue			

GROUND WATER CONDITIONS

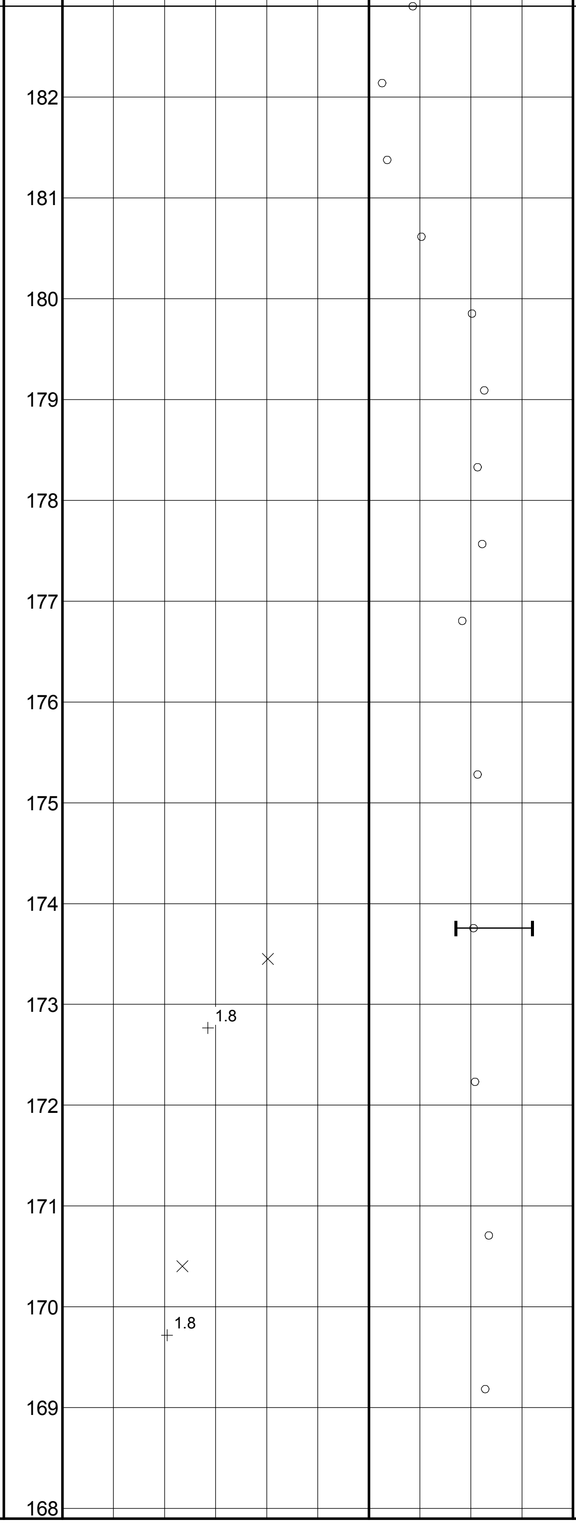
ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● POCKET PEN. × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30



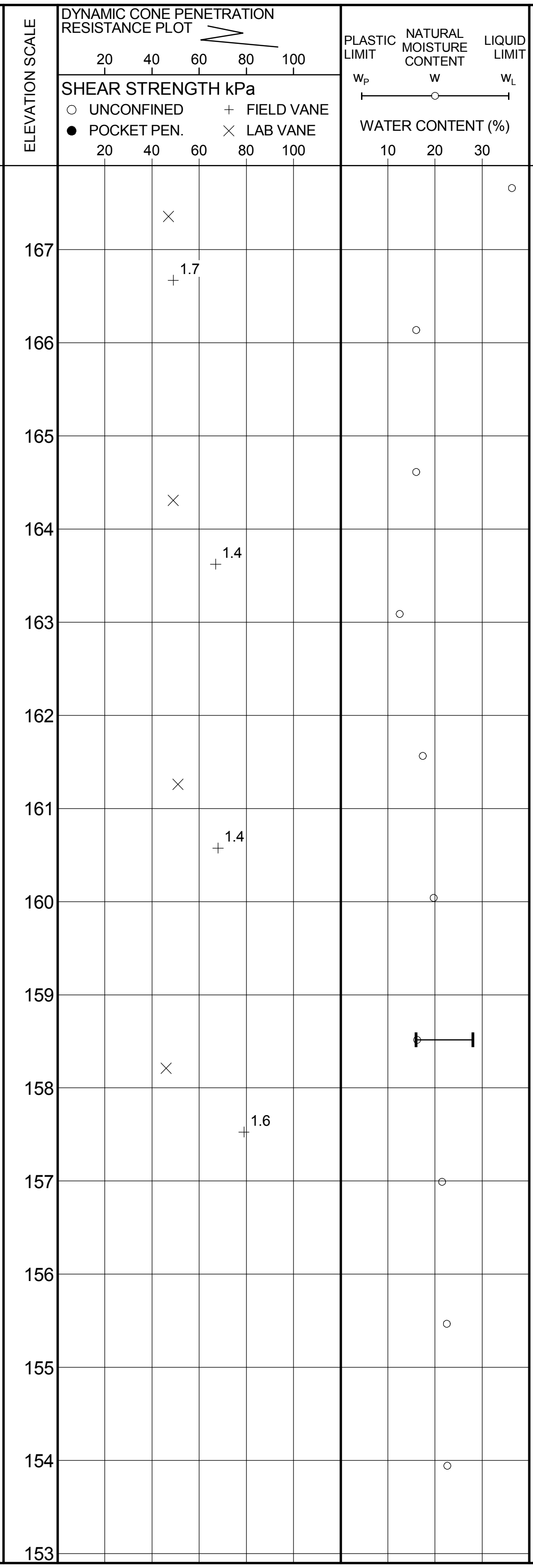
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No PS4-1

2 OF 3 **METRIC**

W.P. RFP No. 09-54-1007	LOCATION N4679483.2, E332301.5	ORIGINATED BY NB
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 13 Aug 11 - 15 Aug 11	CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Grey **CLAYEY SILT** Soft to firm *(continued)* -Some silt, trace sand	15	TW	PH	20.2	Aug 15
			VT			
166.1 16.8	Grey **CLAYEY SILT** Some sand, trace gravel	16	TW	PH		
		17	TW	PH	21.8	
			VT			
		18	TW	PH		
		19	TW	PH	21.2	
			VT			
		20	TW	PH		
		21	TW	PH	21.4	8 25 38 30
			VT			
	-Trace sand and gravel	22	TW	PH		
	-Trace to some sand Stiff	23	SS	12		
	Very stiff	24	SS	17		



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

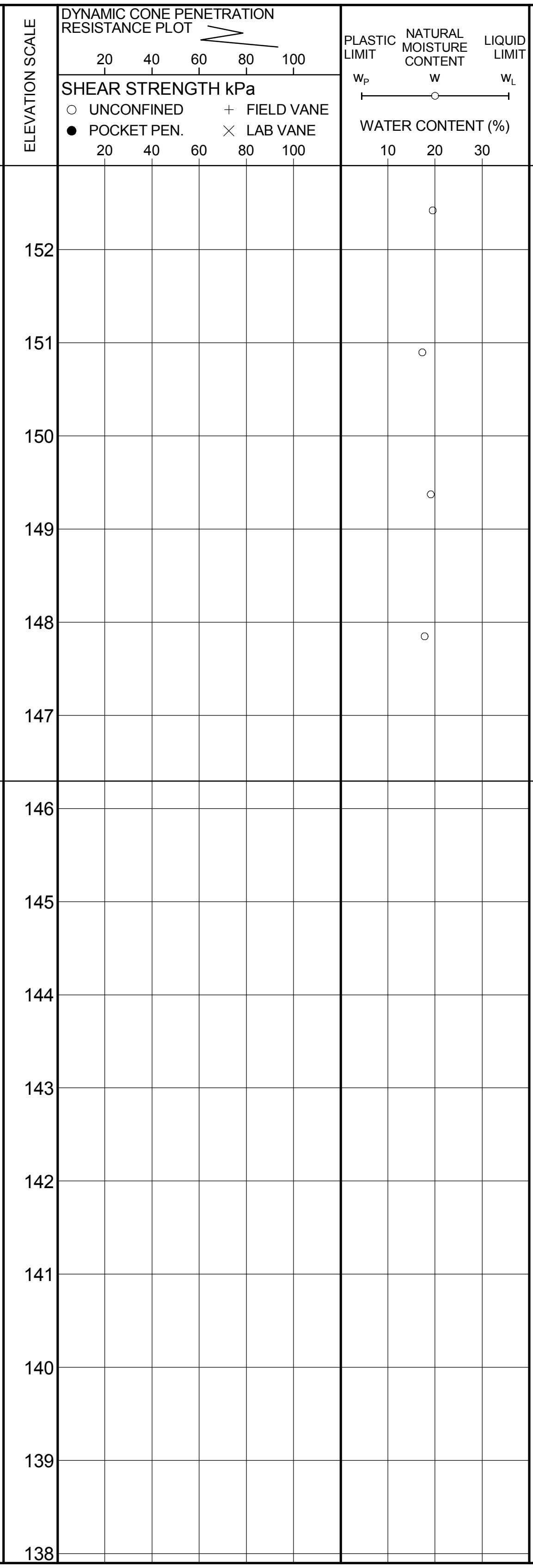
RECORD OF BOREHOLE No PS4-1

3 OF 3

METRIC

W.P. RFP No. 09-54-1007
LOCATION N4679483.2, E332301.5
ORIGINATED BY NB
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE 13 Aug 11 - 15 Aug 11
CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Grey **SILTY CLAY** Grey **CLAYEY SILT** Some sand, trace gravel *(continued)*	25	SS	20			
	-Trace sand and gravel Hard	26	SS	31			
149.7 33.2	**COBBLES AND BOULDERS**	27	SS				-SPT refusal at 30.7m
		28	SS				-SPT refusal at 35.4m
147.3 35.6	Grey **LIMESTONE** Fine Grained, fractured, porous Fracture is running parallel to the core length all along.	29	RC				RQD = 64% TCR = 98%
146.3 36.6	**END OF BOREHOLE**						



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

RECORD OF BOREHOLE No HG-MW-1

1 OF 1 **METRIC**

W.P. RFP No. 09-54-1007
LOCATION N4679501.0, E332278.2
ORIGINATED BY SD
DIST
HWY WEP
BOREHOLE TYPE Truck Mounted Drill - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE 16 Jul 11 - 16 Jul 11
CHECKED BY MSO

SOIL PROFILE		SAMPLES					
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.0	Ground Surface						
182.8 0.0	Black **TOPSOIL**						-Observation Well installed in sampled borehole
0.2	Light Brown **SILTY SAND** Loose	1	SS	6		182	0 86 14
	Reddish Brown	2	SS	6		181	
	Brown	3	SS	6			
180.0 3.0	Grey **CLAYEY SILT** Stiff	4	SS	7		180	
179.5 3.5	**END OF BOREHOLE** (no refusal)					179	
						178	
						177	
						176	
						175	
						174	
						173	
						172	
						171	
						170	
						169	

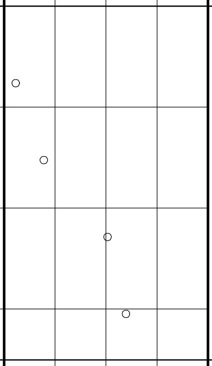
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● POCKET PEN. × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³



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

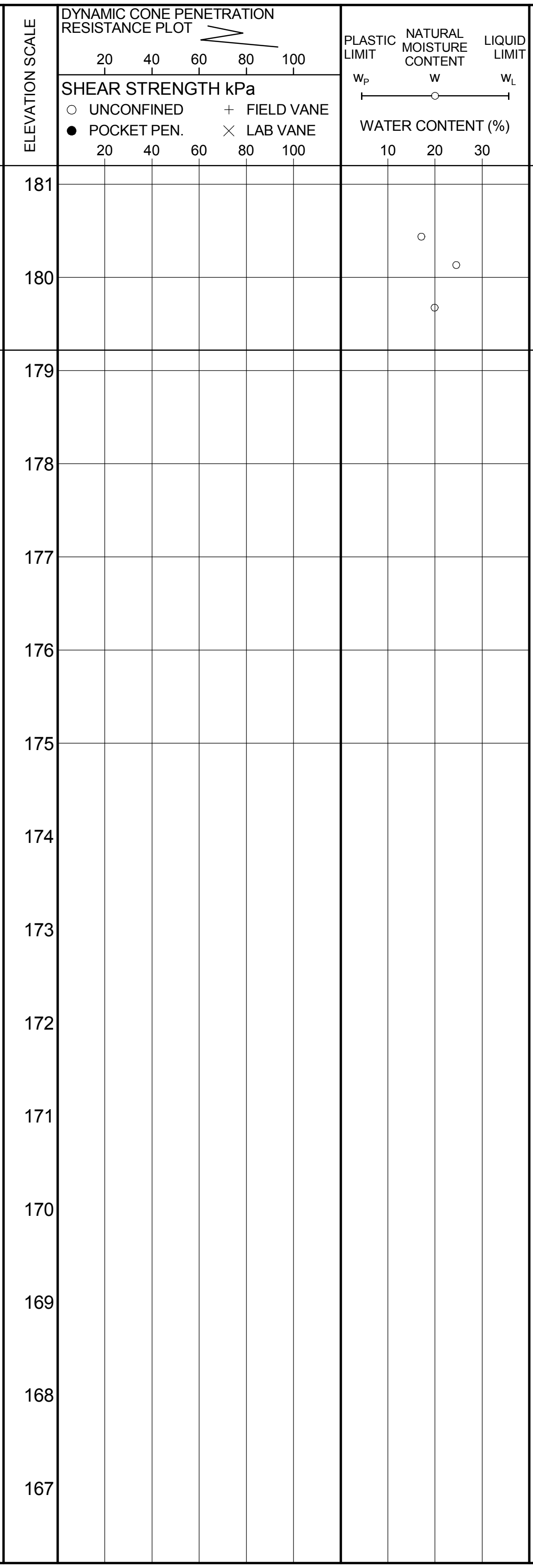
RECORD OF BOREHOLE No CPT38-RW

1 OF 1

METRIC

W.P. RFP No. 09-54-1007
LOCATION N4679485.0, E332131.9
ORIGINATED BY
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE 27 Jul 11 - 27 Jul 11
CHECKED BY MSO

SOIL PROFILE			SAMPLES					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.2	Ground Surface							
0.0 180.6	**FILL** Crushed Limestone, grey							
0.6 180.4	**TOPSOIL**							
0.8 180.1	**FINE SAND** Poorly graded Brown		1A, B	SS	11			
1.1 179.2	**SILTY CLAY** Some sand, trace gravel Mottled brown and grey to brown		2	SS	13			
2.0	**END OF SAMPLED BOREHOLE** Continued with CPT to refusal Borehole dry on completion							



+3, ×3: Numbers refer to Sensitivity

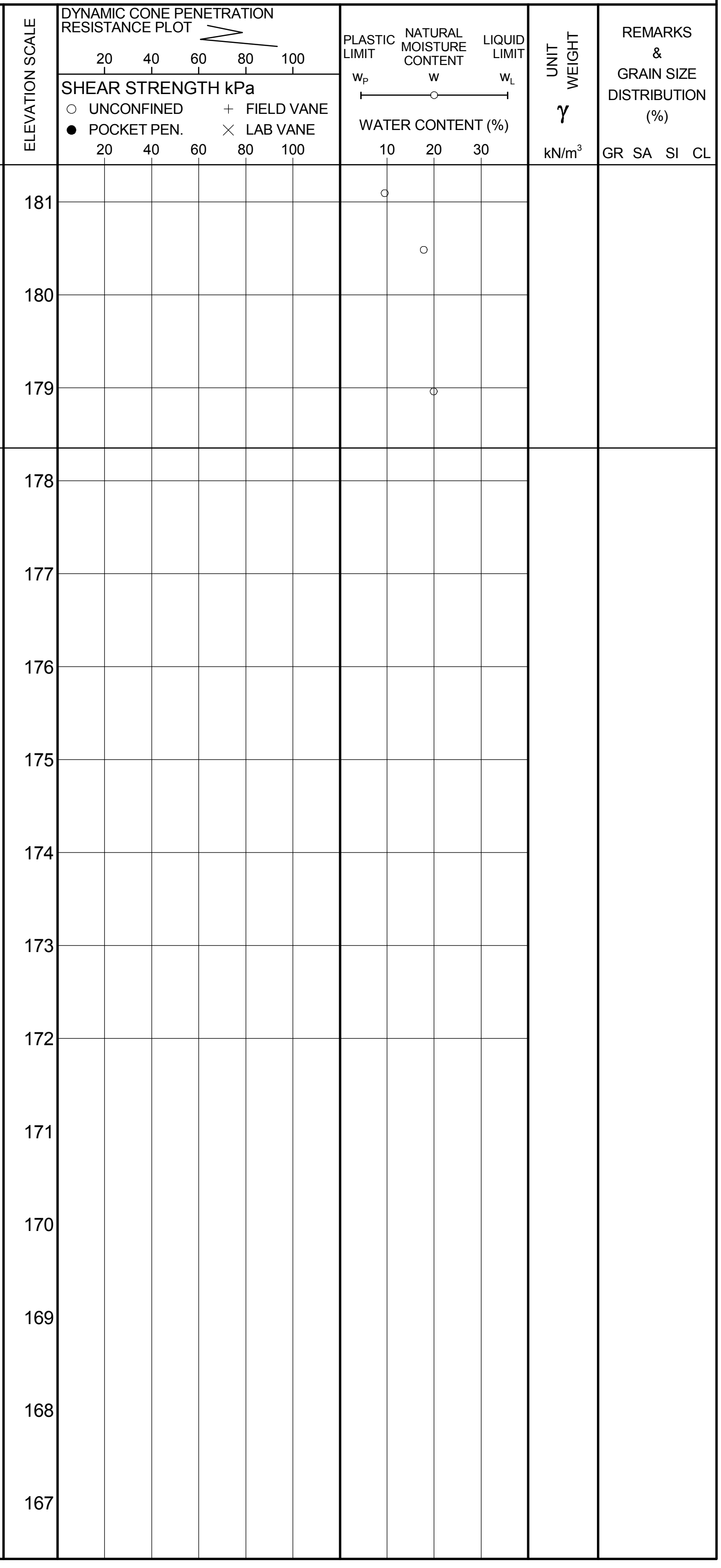
○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CPT39-RW

1 OF 1 **METRIC**

W.P. RFP No. 09-54-1007
LOCATION N4679460.1, E332253.2
ORIGINATED BY TA
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE 28 Jul 11 - 28 Jul 11
CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	
181.4	Pavement Surface					
0.0	**ASPHALT**					
181.3 / 0.1	**CONCRETE**					
181.0 / 0.4	**FINE SAND** Poorly graded Brown		1	SS	11	
			2	SS	6	∇
179.9 / 1.5	**CLAYEY SILT** Some sand, trace gravel Grey					
			3	SS	14	
178.4 / 3.0	**END OF SAMPLED BOREHOLE** Continued with CPT to refusal Groundwater encountered at elevation 179.9m during drilling					



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

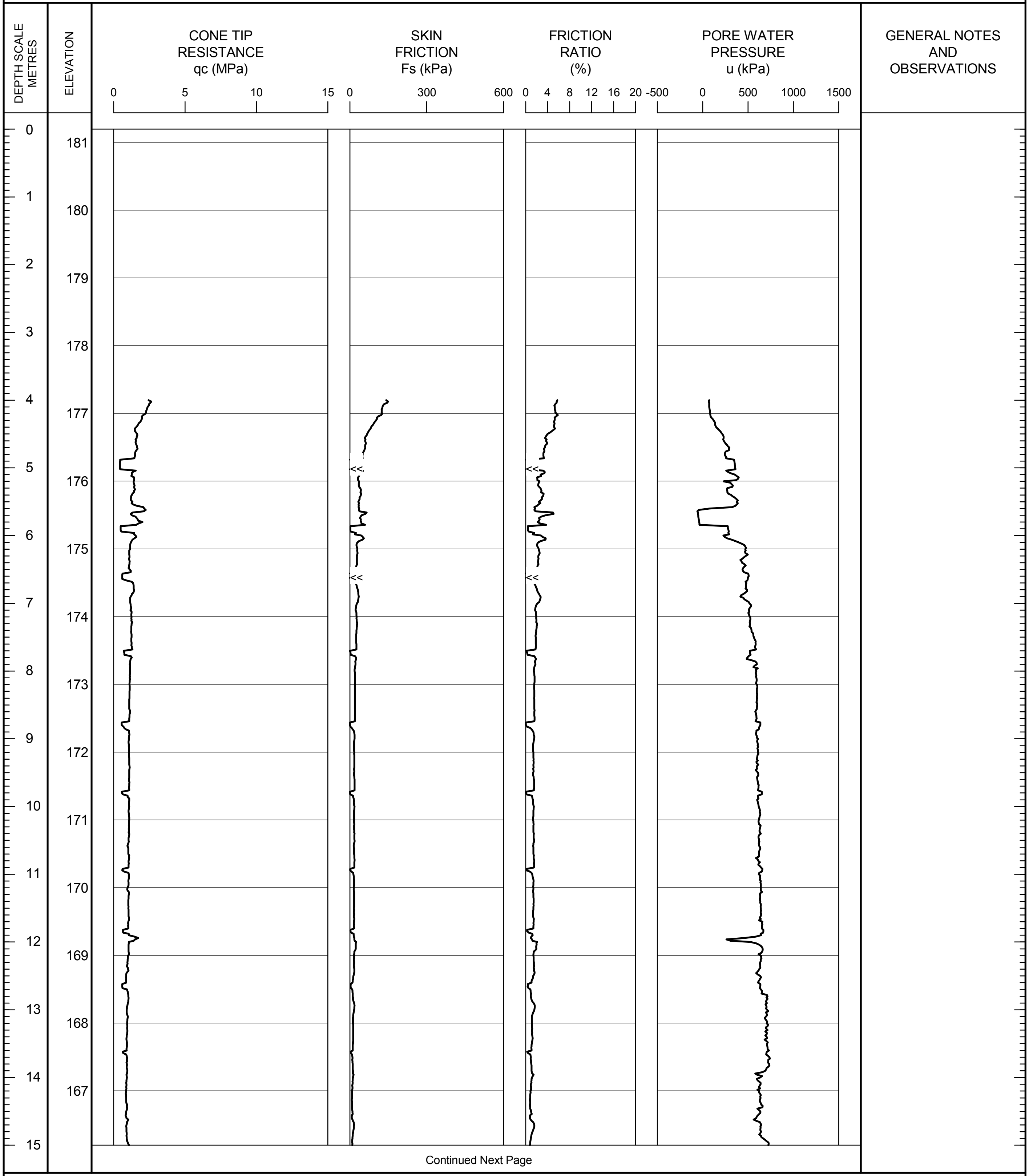
$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF CONE PENETRATION TEST CPT T7-1

METRIC

PROJECT Windsor-Essex Parkway
TEST DATE 7/22/2011 - 7/22/2011
SHEET 1 OF 2
LOCATION N4679345.0; E332316.9
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2 PREDRILL DEPTH: 3.37 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

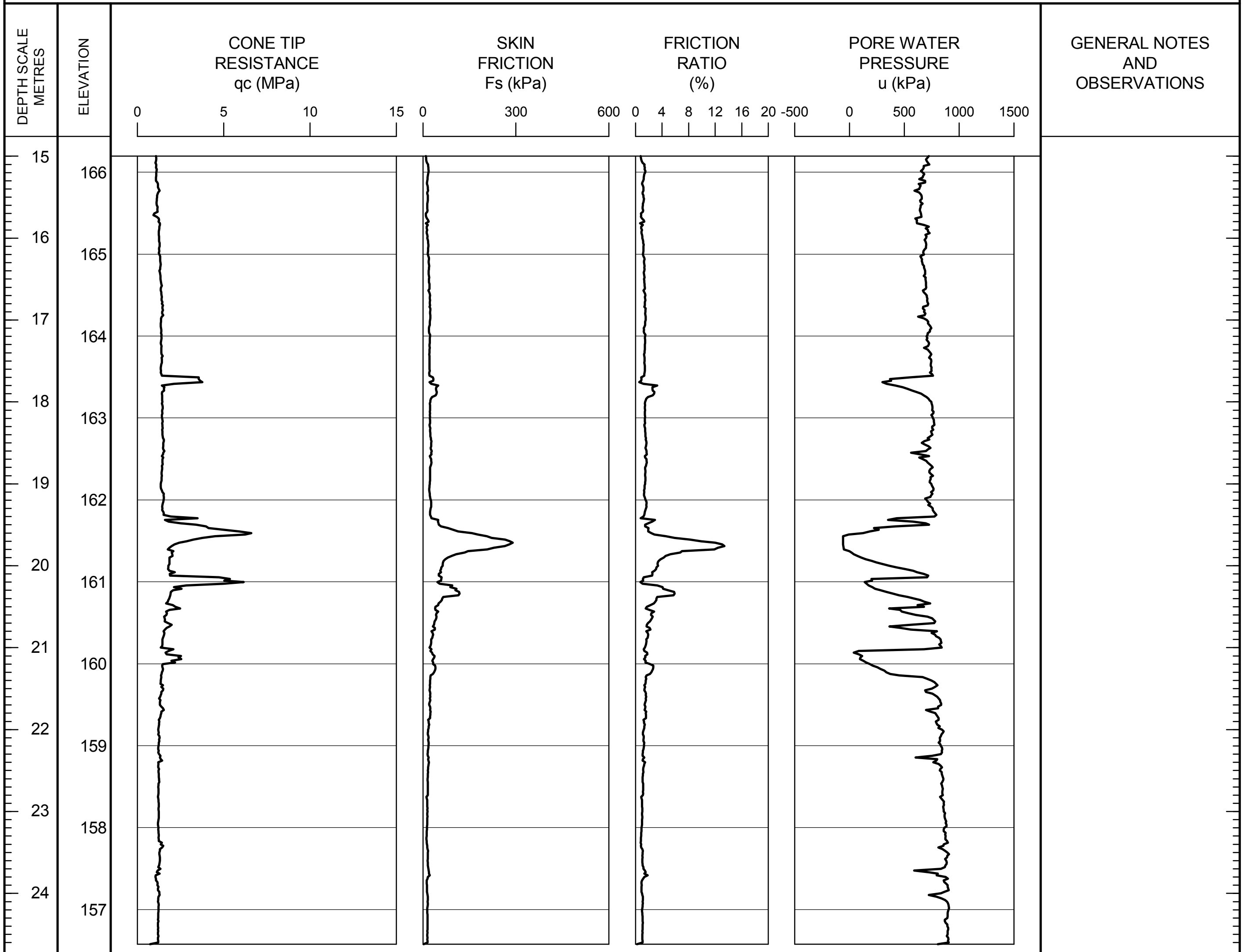
CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T7-1

METRIC

PROJECT	Windsor-Essex Parkway	TEST DATE	7/22/2011 - 7/22/2011	SHEET	2 OF 2
LOCATION	N4679345.0; E332316.9			DATUM	Geodetic

GROUND SURFACE ELEVATION: 181.2 PREDRILL DEPTH: 3.37 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

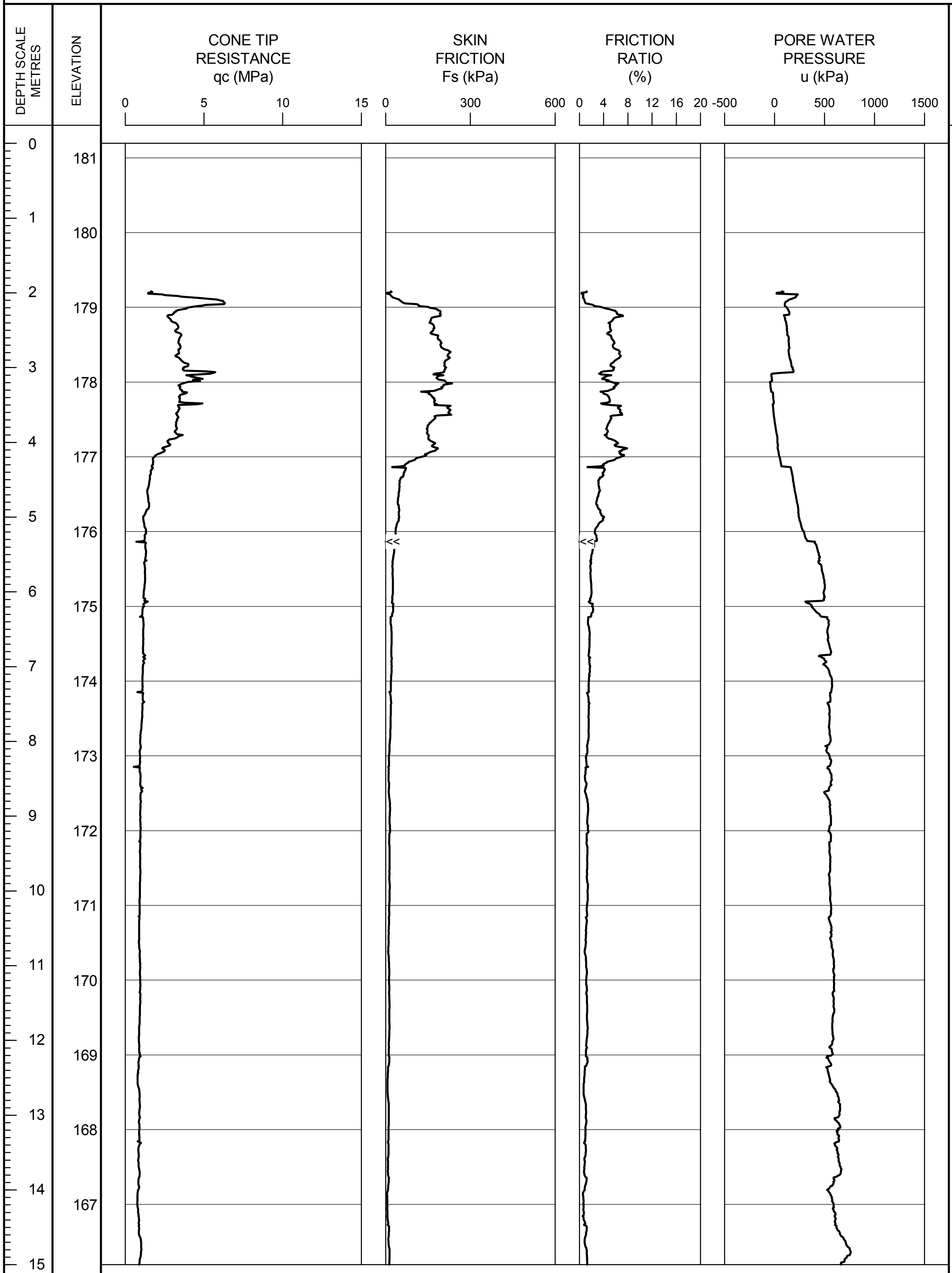
WEP CPT LOG CPT T7-1.GPJ ONTARIO MOT.GDT 21/12/11

RECORD OF CONE PENETRATION TEST CPT 38-RW

METRIC

PROJECT Windsor-Essex Parkway
TEST DATE 7/27/2011 - 7/27/2011
SHEET 1 OF 3
LOCATION N4679485.0; E332131.9
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2
PREDRILL DEPTH: 1.88
CORRECTION FACTOR A: 0.8
CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

Continued Next Page

OPERATOR: TA
CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 38-RW

METRIC

PROJECT Windsor-Essex Parkway

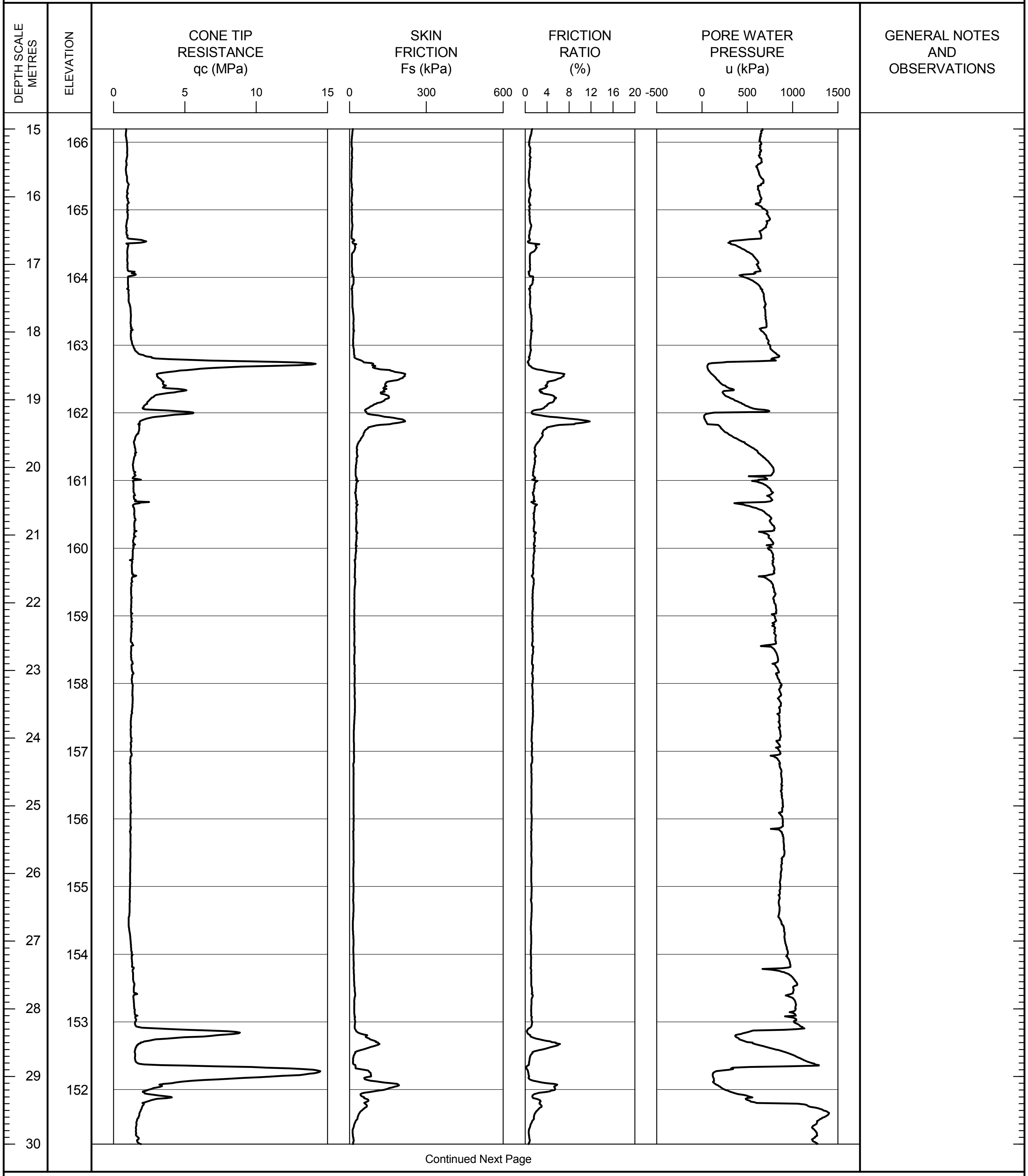
TEST DATE 7/27/2011 - 7/27/2011

SHEET 2 OF 3

LOCATION N4679485.0; E332131.9

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2 PREDRILL DEPTH: 1.88 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

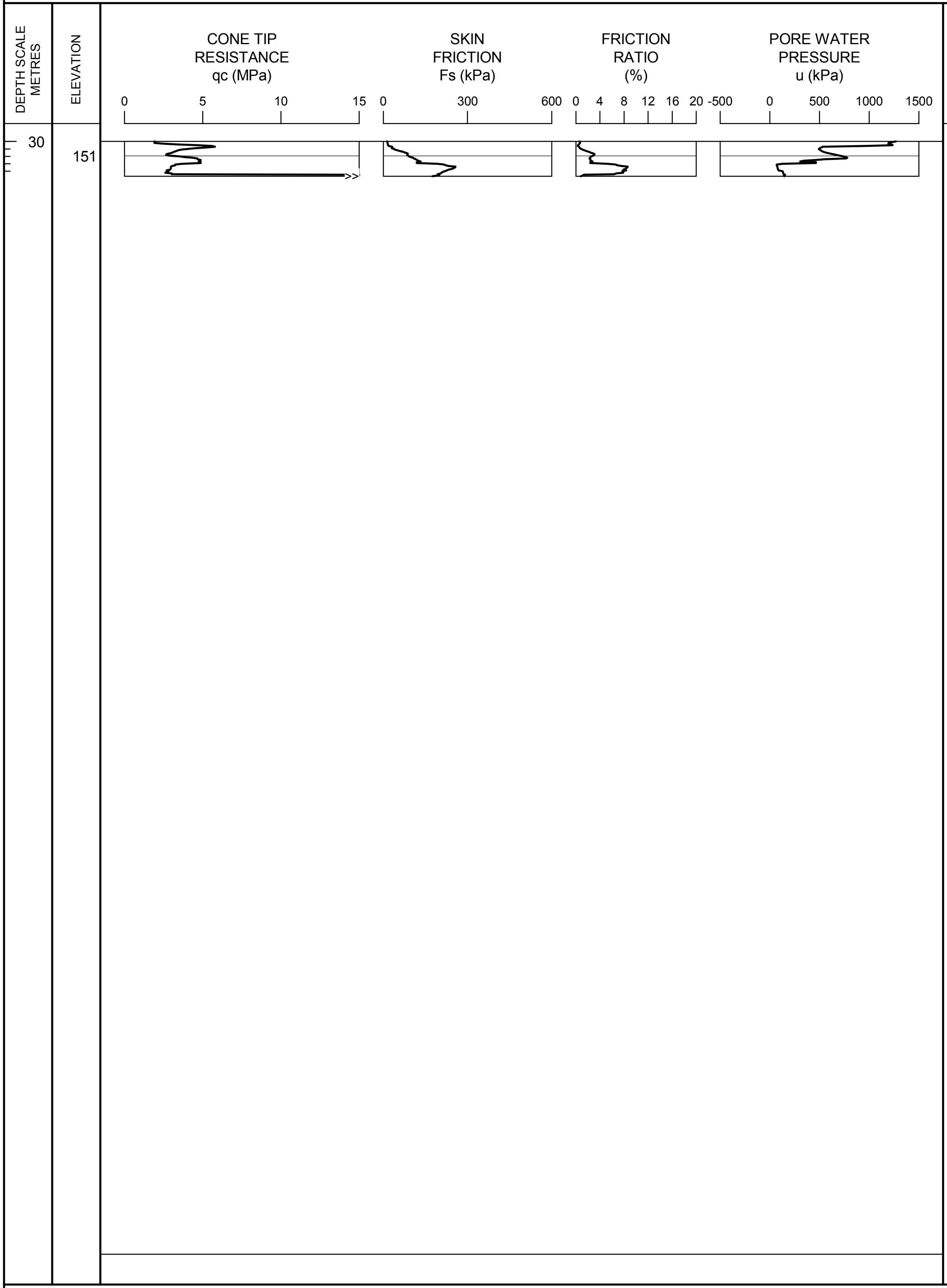
CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 38-RW

METRIC

PROJECT	Windsor-Essex Parkway	TEST DATE	7/27/2011 - 7/27/2011
SHEET	3 OF 3		
LOCATION	N4679485.0; E332131.9	DATUM	Geodetic

GROUND SURFACE ELEVATION: 181.2 PREDRILL DEPTH: 1.88 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 39-RW

METRIC

PROJECT Windsor-Essex Parkway

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

SHEET 1 OF 2

LOCATION N4679460.1; E332253.2

DATUM Geodetic

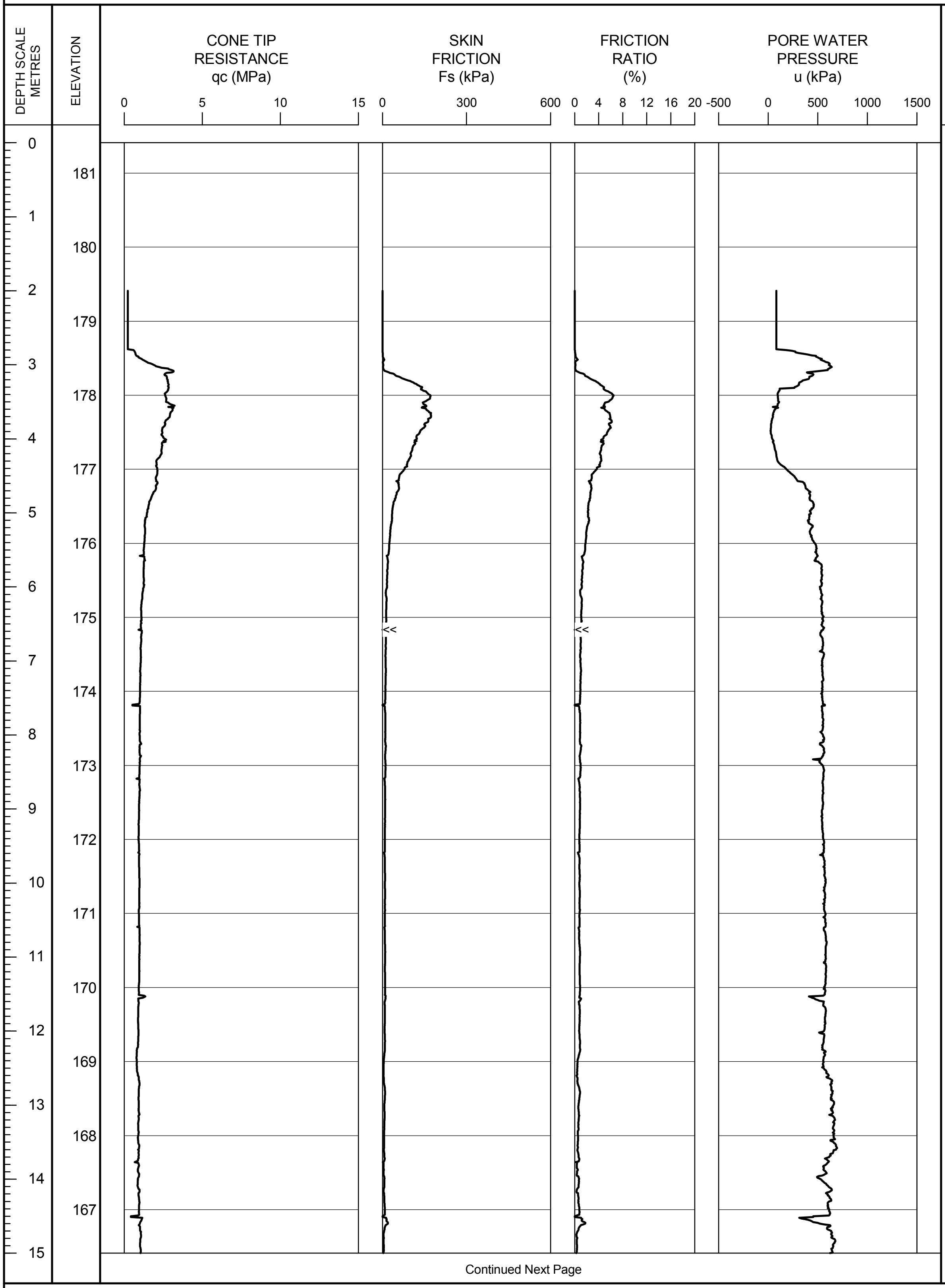
GROUND SURFACE ELEVATION: 181.4

PREDRILL DEPTH: 2

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0

GENERAL NOTES AND OBSERVATIONS



Continued Next Page

OPERATOR: TA

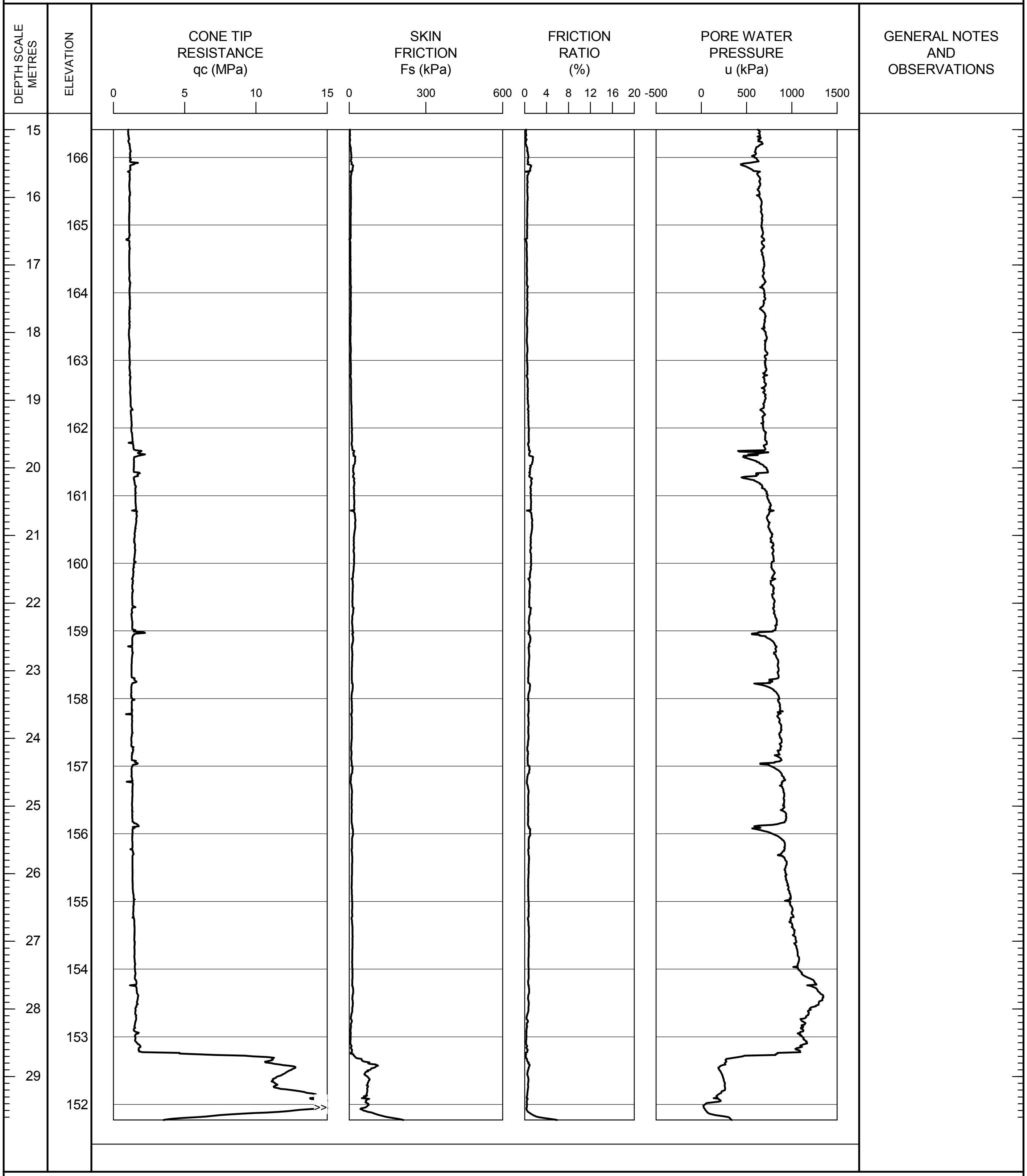
CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 39-RW

METRIC

PROJECT	Windsor-Essex Parkway	TEST DATE	7/28/2011 - 7/28/2011	SHEET	2 OF 2
LOCATION	N4679460.1; E332253.2			DATUM	Geodetic

GROUND SURFACE ELEVATION: 181.4 PREDRILL DEPTH: 2 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

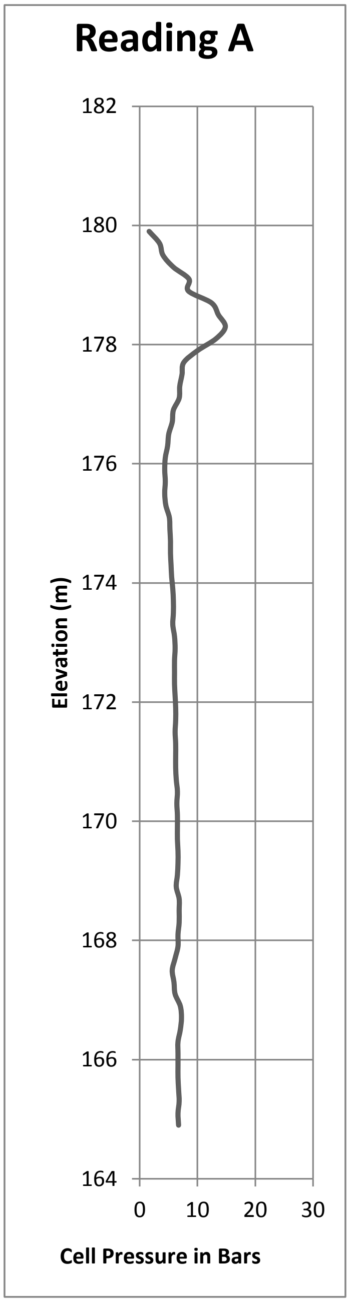
WEP CPT LOG CPT-RW.GPJ ONTARIO MOT.GDT 06/01/12

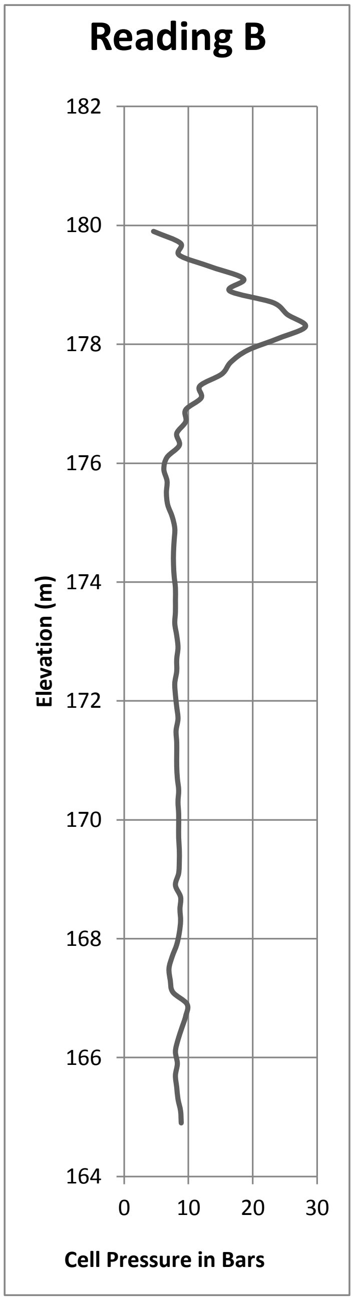
RECORD OF DILATOMETER TEST DMT T7-1

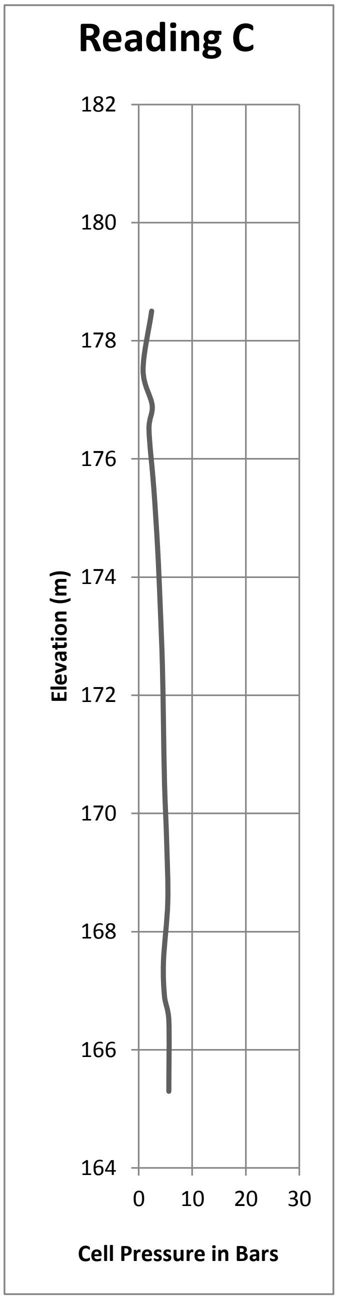
Project : Windsor-Essex Parkway
Location: N 4679368.7; E 332355.7
Ground Surface Elevation : 181.5

Test Date: 7/15/2011
Predrill Depth : 1.5 m
Delta A: 0.14 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.22 Bar





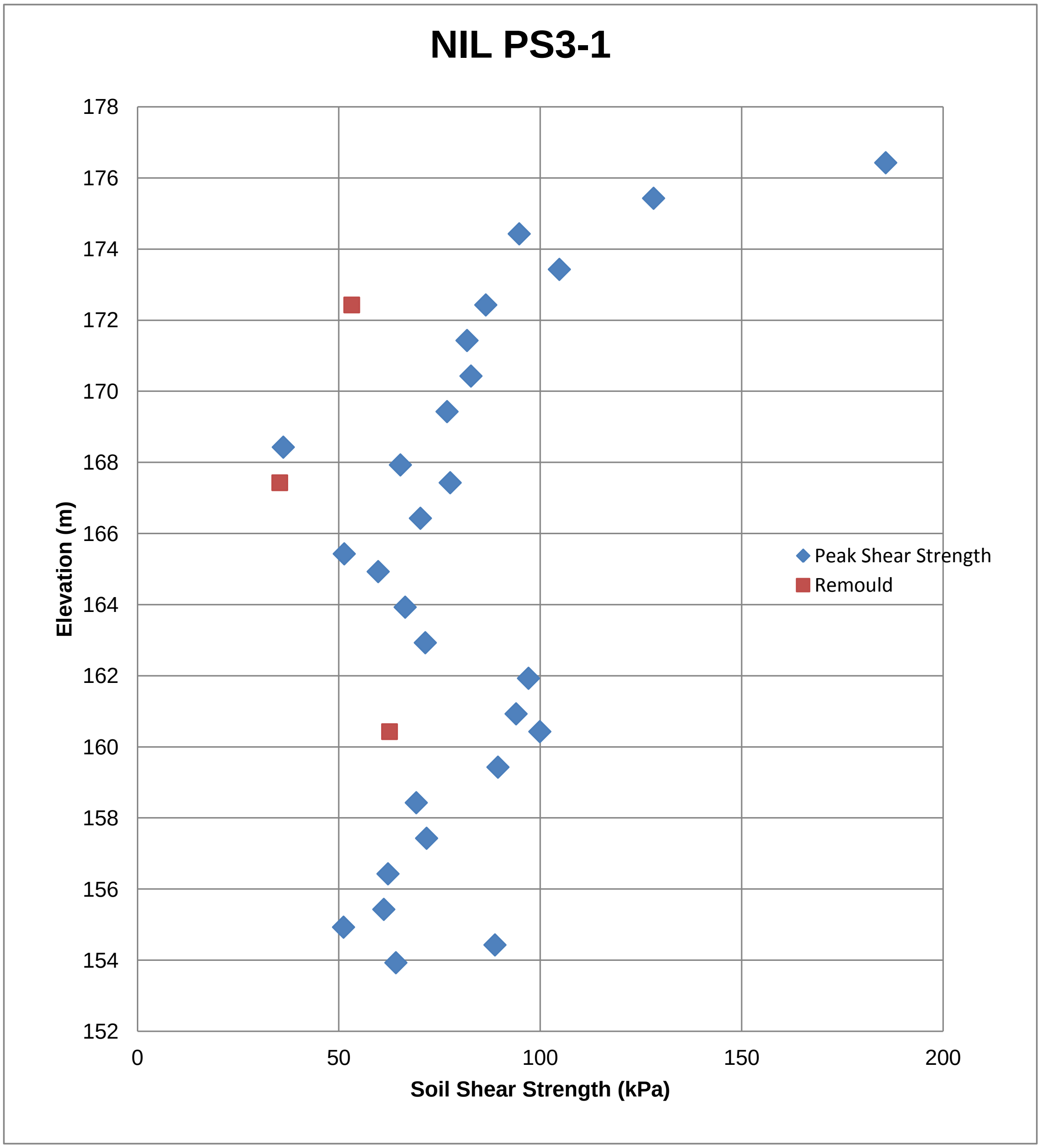


Operator: LC

Checked: DD

RECORD OF NILCON VANE TEST NIL PS3-1

Project : Windsor-Essex Parkway	Test Date: 8/26/2011	Sheet 1 of 1
Location: N4679419.4; E332251.2	Predrill Depth : 5.0 m	Datum Geodetic
Ground Surface Elevation: 181.4 m		



Operator: SD

Checked: DD

Appendix B Borehole and CPT Logs from Previous Investigations

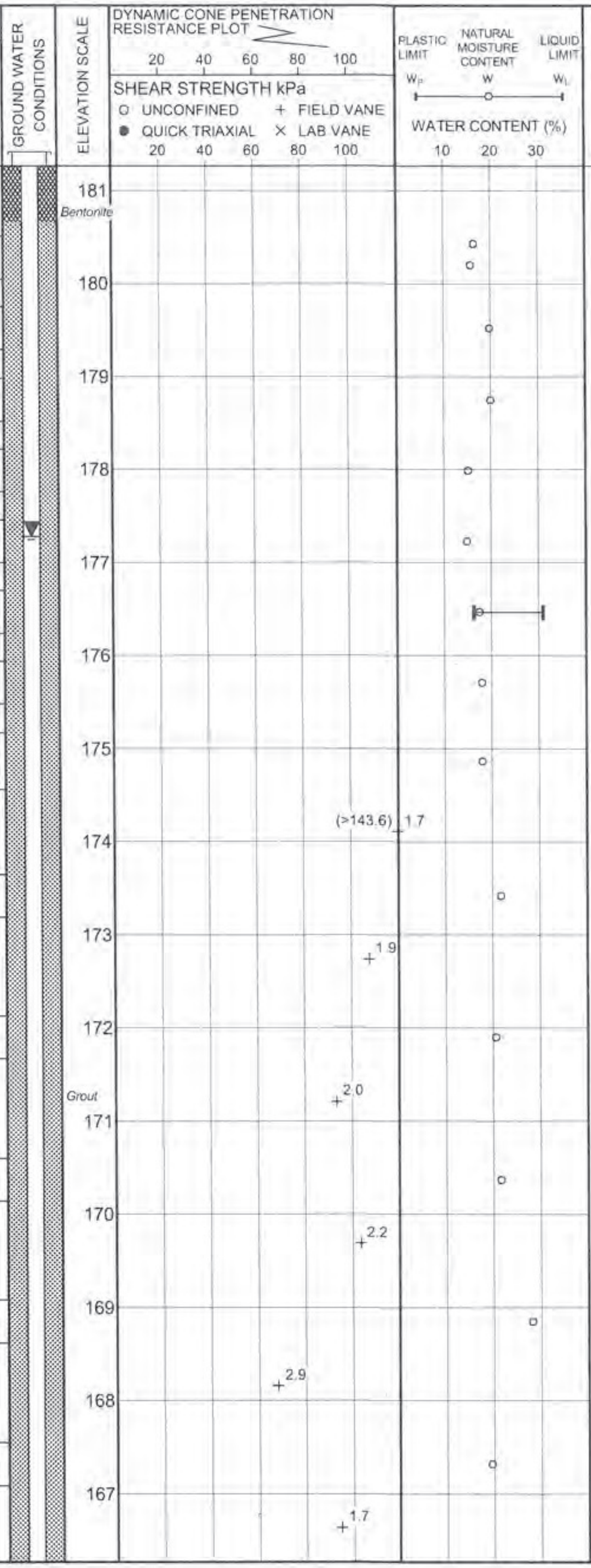
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 127

1 OF 4 **METRIC**

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4679370.9 ;E 332251.6
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE March 11, 2008 - March 13, 2008
ORIGINATED BY SM
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
181.27	GROUND SURFACE								
0.00	FILL, sand and gravel Grey								
0.15	FILL, sand, trace silt, trace clay Loose Dark brown								
180.36 0.91	SAND, trace silt Loose Brown	1	SS	5					
179.90 1.37	SILTY CLAY, some sand, trace gravel Stiff to very stiff Grey	2	SS	10					
		3	SS	16					
178.37 2.90	CLAYEY SILT, some sand, trace gravel Stiff to very stiff Grey	4	SS	27					
		5	SS	16					
		6	SS	15		3	21	42	35
		7	SS	10					
		8	TO	PH					
		9	SS	10					
		10	SS	6					
		11	SS	7					
		12	TO	PH					
		13	SS	7					



Continued Next Page

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

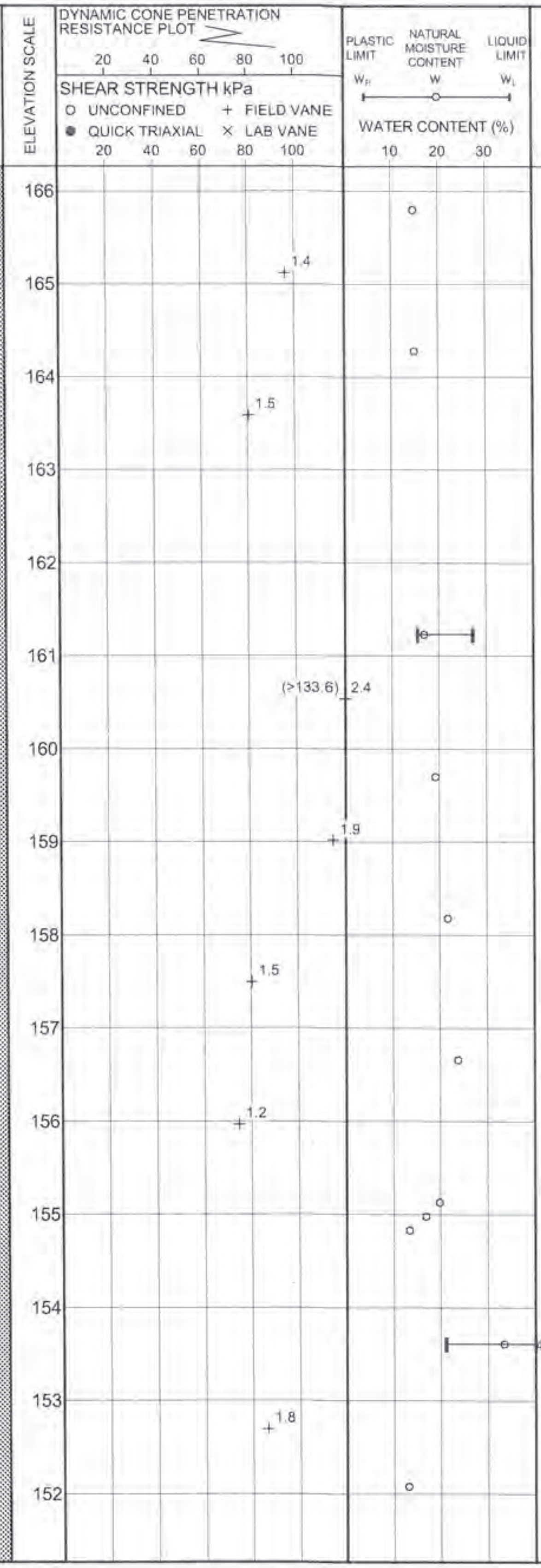
RECORD OF BOREHOLE No 127

2 OF 4 **METRIC**

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4679370.9 ;E 332251.6
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE March 11, 2008 - March 13, 2008
ORIGINATED BY SM
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION
	CLAYEY SILT, some sand, trace gravel Stiff to very stiff Grey
158.94 22.33	SILTY CLAY, some sand, trace gravel Stiff to very stiff Grey
156.12 25.15	SAND, some clay, trace silt, trace gravel Compact Grey
155.06 26.21	CLAYEY SILT, some sand, trace gravel Very stiff Grey
26.37	SILT, some clay, some sand, trace gravel Compact Grey
154.60 26.67	SILTY CLAY, some sand, trace gravel Stiff Grey
153.08 28.19	CLAYEY SILT, some sand, trace gravel Very stiff Grey

SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14	SS	8	
15	TO	PH	
16	SS	8	
17	SS	11	1 22 45 32
18	TO	PH	
19	TO	PH	
20	SS	5	
21	SS	25	(19)
22	SS	5	0 2 45 53
23	TO	PH	



Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 127

3 OF 4 **METRIC**

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4679370.9 ;E 332251.6
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE March 11, 2008 - March 13, 2008
ORIGINATED BY SM
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, some sand, trace gravel Very stiff Grey					151				
150.54 30.73	SANDY SILT, trace clay, trace gravel, with cobbles Very dense Grey	24	SS	163						(39)
						150				
		25	SS	100/ 50mm		149				
148.47 32.80	DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous Brown to grey	26	NQ RC		Bentonite		36	0	0	
	(FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	27	NQ RC		Sand	148 147	100	100	100	
		28	NQ RC		Screen	146	100	100	100	UC
145.16 36.11	END OF BOREHOLE									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

Borehole dry during drilling between March 11 and 13, 2008.

Water level measured in deep piezometer at elev. 177.74m on March 20, 2008.

Water level measured in deep piezometer at elev. 178.27m on July 22, 2008.

Water level measured in deep piezometer at elev. 178.12m on August 11, 2008.

Water level measured in deep piezometer at elev. 177.87m on September 19, 2008.

Water level measured in deep piezometer at elev. 177.74m on November 11, 2008.

Water level measured in deep piezometer at elev. 177.28m on January 28, 2009.

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

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

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 127

SHEET 4 OF 4

LOCATION: N 4679370.9 ;E 332251.6

INCLINATION: -90°   AZIMUTH: —

DRILLING DATE: March 11, 2008 - March 13, 2008

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	ELEVATION	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE	148.47													
33	MUD ROTARY NO ROCK CORE	LIMESTONE/BASALT - broken core, likely cobbles from till	32.80 148.20 33.07	1												
		DOLOSTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous with localized vugs, dark brown		2			148									
34			146.89 34.38				147									
		LIMESTONE, fresh, medium strong, thinly laminated, medium grained, faintly porous, grey to brown	146.47 34.80													
35		DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated to bedded, very fine grained to fine grained, faintly porous, light grey to grey		3			146						JN,PL,SM Cl JN,C,Ro Ca JN,PL,SM Ca JN,PL,SM Ca JN,PL,SM Cl			
			145.55 35.72													
36		DOLOSTONE/LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, brown	145.16 36.11													
		END OF DRILLHOLE														
37																
38																
39																
40																
41																
42																
43																
44																
45																
46																
47																

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: SG
CHECKED: SJB

Golder Associates
London, Ontario

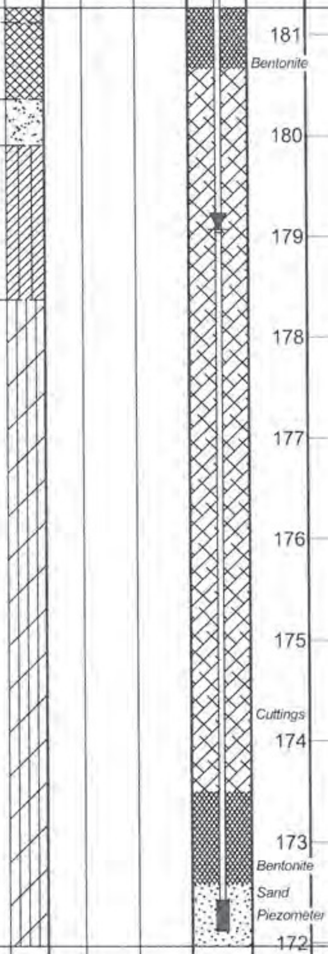
RECORD OF BOREHOLE No 127A

1 OF 1 **METRIC**

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4679370.9 ;E 332251.6
BOREHOLE TYPE POWER AUGER, SOLID STEM
DATE March 13, 2008
ORIGINATED BY SM
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION
181.27	SOIL CONDITIONS INFERRED FROM BOREHOLE No. 127 GROUND SURFACE
0.00	FILL, sand and gravel Grey
0.15	FILL, sand, trace silt, trace clay Loose Dark brown
180.36 0.91	SAND, trace silt Loose Brown
179.90 1.37	SILTY CLAY, some sand, trace gravel Stiff to very stiff Grey
178.37 2.90	CLAYEY SILT, some sand, trace gravel Stiff to very stiff Grey
171.97 9.30	END OF BOREHOLE Water level measured in shallow piezometer at elev. 172.35m on March 20, 2008. Water level measured in shallow piezometer at elev. 179.06m on July 22, 2008. Water level measured in shallow piezometer at elev. 179.12m on August 11, 2008. Water level measured in shallow piezometer at elev. 179.11m on September 19, 2008. Water level measured in shallow piezometer at elev. 179.07m on January 28, 2009.

STRAT PLOT
SAMPLES: NUMBER, TYPE, "N" VALUES
GROUND WATER CONDITIONS
ELEVATION SCALE



DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%)
10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-128 1 OF 1 METRIC

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4679490.6 ;E 332200.8
BOREHOLE TYPE POWER AUGER, SOLID STEM
DATE September 5, 2008
ORIGINATED BY CC
COMPILED BY SJL
CHECKED BY SJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
180.87	GROUND SURFACE										
0.00	FILL, silty sand, trace gravel, trace organics with clayey silt Compact Brown	▩	1	SS	10				○		
180.26 0.61	CLAYEY SILT, trace sand, trace gravel Firm to very stiff Mottled brown and grey	▧	2	SS	7	∇	180		○		
			3	SS	17				○		
179.04 1.83	END OF BOREHOLE Water level in borehole at about elev. 179.7m during drilling on September 5, 2008.										

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

Golder Associates
London, Ontario

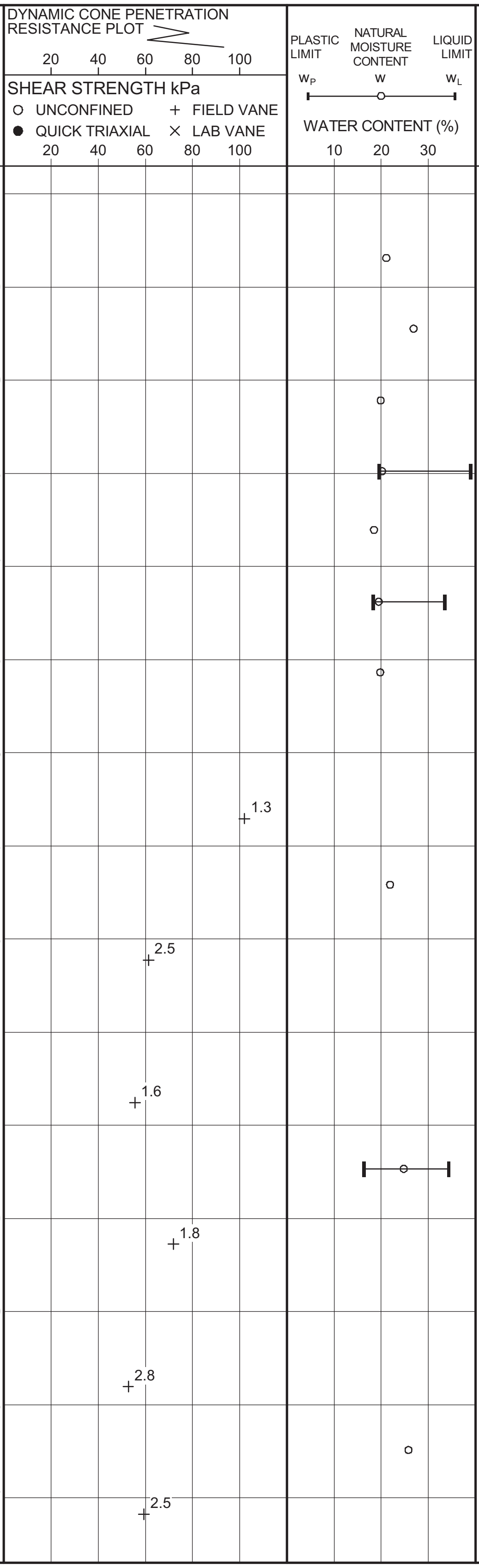
RECORD OF BOREHOLE No 323

1 OF 4 **METRIC**

PROJECT 09-1132-0080
W.P.
LOCATION N 4679521.4 ;E 332167.6
ORIGINATED BY MK/MR
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY LMK/DMB
DATUM GEODETIC
DATE December 15, 2009 - December 17, 2009
CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.30	ROAD SURFACE					
0.02	ASPHALT PAVEMENT					
0.30	FILL, sand and gravel, crushed Brown				181	
	FILL, clayey silt, some topsoil, some sand Firm to stiff Dark brown and grey	1	SS	8		
179.93 1.37	CLAYEY SILT, some sand Stiff to very stiff Brown				180	
		2	SS	10		
		3	SS	17	179	
178.40 2.90	SILTY CLAY, some sand Very stiff Brown	4	SS	23	178	0 17 43 40
177.70 3.60	CLAYEY SILT, some sand, trace gravel Stiff to very stiff Grey	5	SS	20		
		6	SS	18	177	2 20 38 40
		7	SS	12	176	
		8	TO	PH	175	
					174	
		9	SS	6		
					173	
		10	TO	PH	172	
					171	
		11	SS	8		
					170	
		12	TO	PH	169	
					168	
		13	SS	5		
					167	

GROUND WATER CONDITIONS: Grout



UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates
London, Ontario

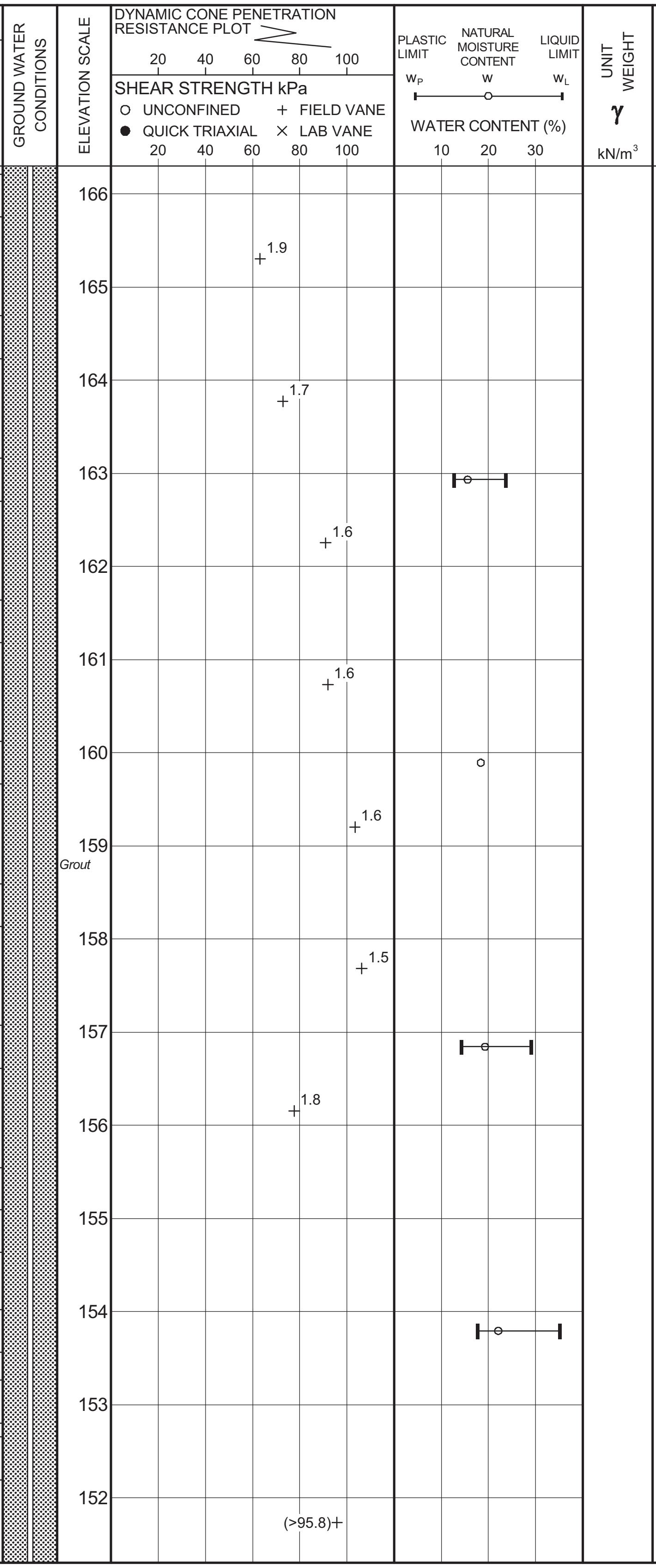
RECORD OF BOREHOLE No 323

2 OF 4 **METRIC**

PROJECT	09-1132-0080		
W.P.		LOCATION	N 4679521.4 ;E 332167.6
DIST	WEST HWY 401 / 3	BOREHOLE TYPE	POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATUM	GEODETIC	DATE	December 15, 2009 - December 17, 2009
ORIGINATED BY	MK/MR	COMPILED BY	LMK/DMB
CHECKED BY			

ELEV DEPTH	SOIL PROFILE DESCRIPTION
	CLAYEY SILT, some sand, trace gravel Stiff to very stiff Grey
155.84 25.46	SILTY CLAY, some sand, trace gravel Firm to stiff Grey
153.40 27.90	CLAYEY SILT, some sand, trace gravel Stiff Grey
151.65 29.65	

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14	TO	PH	
15	TO	PH	
16	SS	10	2 28 47 23
17	TO	PH	
18	SS	10	
19	TO	PH	
20	SS	7	
21	TO	PH	
22	SS	5	
23	SS	8	1 17 43 39
24	TO	PH	
25	TO	PH	



Continued Next Page

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

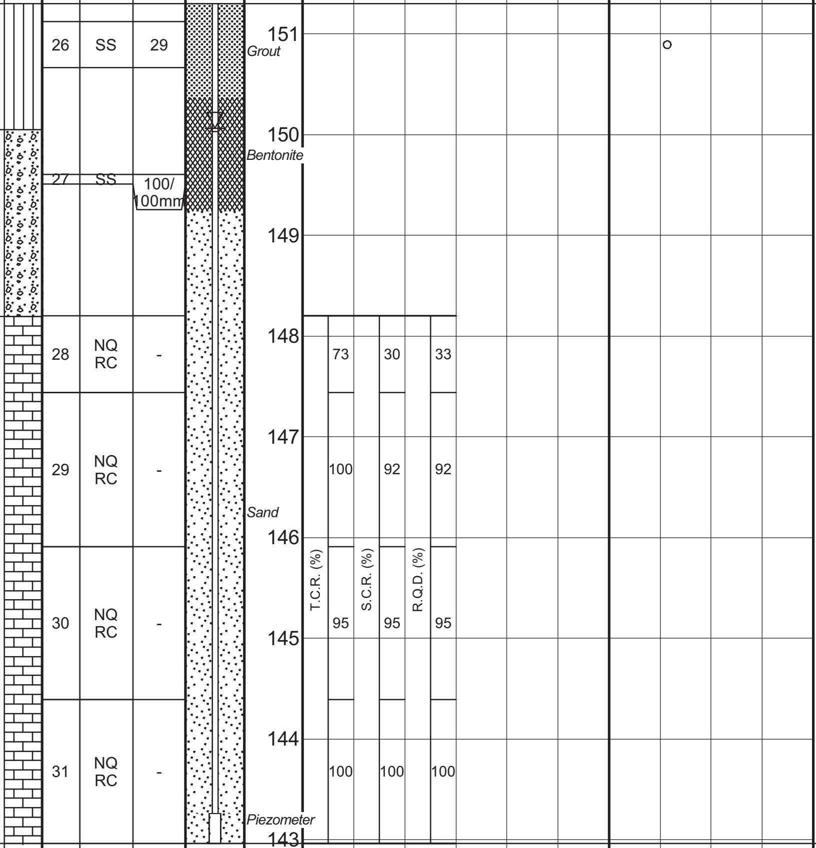
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 323

3 OF 4

METRIC

PROJECT	09-1132-0080		
W.P.		LOCATION	N 4679521.4 ;E 332167.6
DIST	WEST	HWY	401 / 3
BOREHOLE TYPE	POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		
DATUM	GEODETIC	DATE	December 15, 2009 - December 17, 2009
ORIGINATED BY	MK/MR	COMPILED BY	LMK/DMB
CHECKED BY			



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)
	SILT, some clay, some sand, trace gravel Compact Grey	26	SS	29			
150.05 31.25	SAND AND GRAVEL, trace silt Very dense Brown	27	SS	100/ 100mm			
148.19 33.11	LIMESTONE, fresh, medium strong, weakly laminated, very fine to fine grained, faintly porous Light brown to grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	28	NQ RC	-	73	30	33
		29	NQ RC	-	100	92	92
		30	NQ RC	-	95	95	95
		31	NQ RC	-	100	100	100
142.96 38.34	END OF BOREHOLE						

Groundwater encountered at about elev. 150.1m during drilling between December 15 and 17, 2009.

Water level measured at elev. 179.12 on February 24, 2010.

Water level measured at elev. 178.94 on January 6, 2010.

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

PROJECT: 09-1132-0080

RECORD OF DRILLHOLE: 323

SHEET 4 OF 4

LOCATION: N 4679521.4 ;E 332167.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 15, 2009 - December 17, 2009

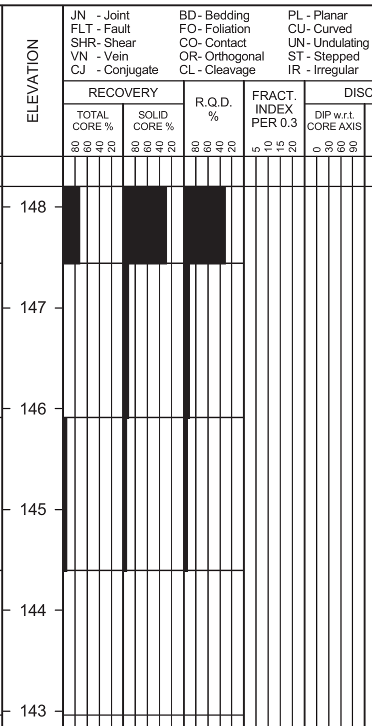
DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: LANTECH

DATUM: GEODETIC

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR / % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE		148.20							
	MUD ROTARY NQ ROCK CORE	LIMESTONE, fresh, medium strong, bedded, brown		33.10	1			Broken core from 33.10m to 33.59m			
34		LIMESTONE, fresh, medium strong, weakly laminated, very fine to fine grained, faintly porous, light brown to grey		147.71 33.59							
		LIMESTONE, fresh, medium strong, weakly laminated to laminated, fine grained, faintly porous with occasional vugs, brown		147.41 33.89	2						
35											
36		LIMESTONE, fresh, medium strong, weakly laminated, very fine grained, faintly porous, grey		145.73 35.57	3						
37		LIMESTONE, fresh, medium strong, weakly laminated, very fine to fine grained, faintly porous, brown, occasional stylolites		145.03 36.27	4						
38		LIMESTONE, fresh, medium strong, weakly laminated, very fine grained, faintly porous with occasional vugs, grey		143.23 38.07							
		END OF DRILLHOLE		142.96 38.34							
39											
40											
41											
42											
43											
44											
45											
46											
47											
48											



DEPTH SCALE

1 : 75

Golder Associates

LOGGED: SG

CHECKED:

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-128

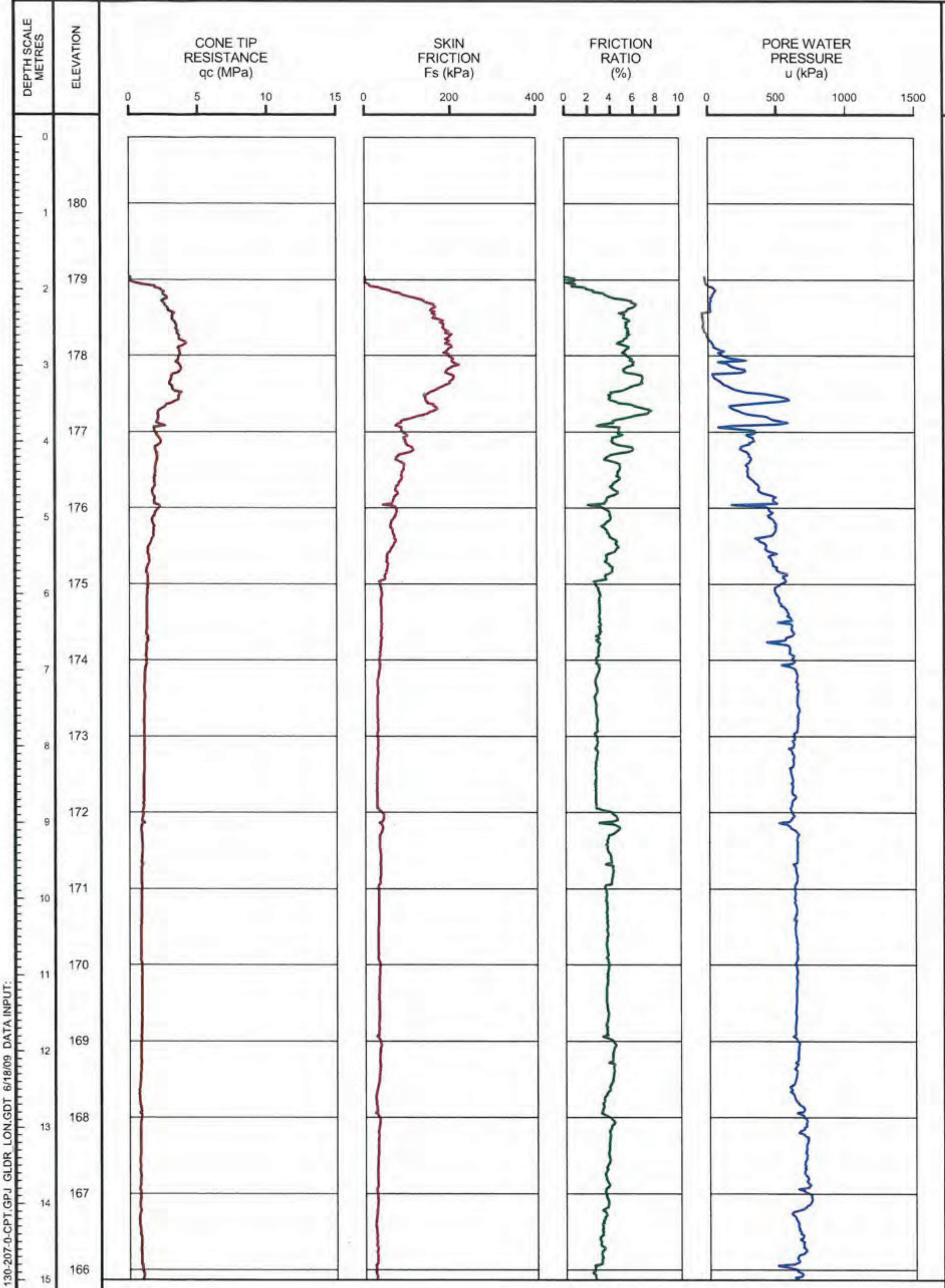
SHEET 1 OF 3

LOCATION: N 4679490.6 ;E 332200.8

TEST DATE: September 5, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 1.83m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



GENERAL NOTES AND OBSERVATIONS

— CONTINUED NEXT PAGE —

DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-128

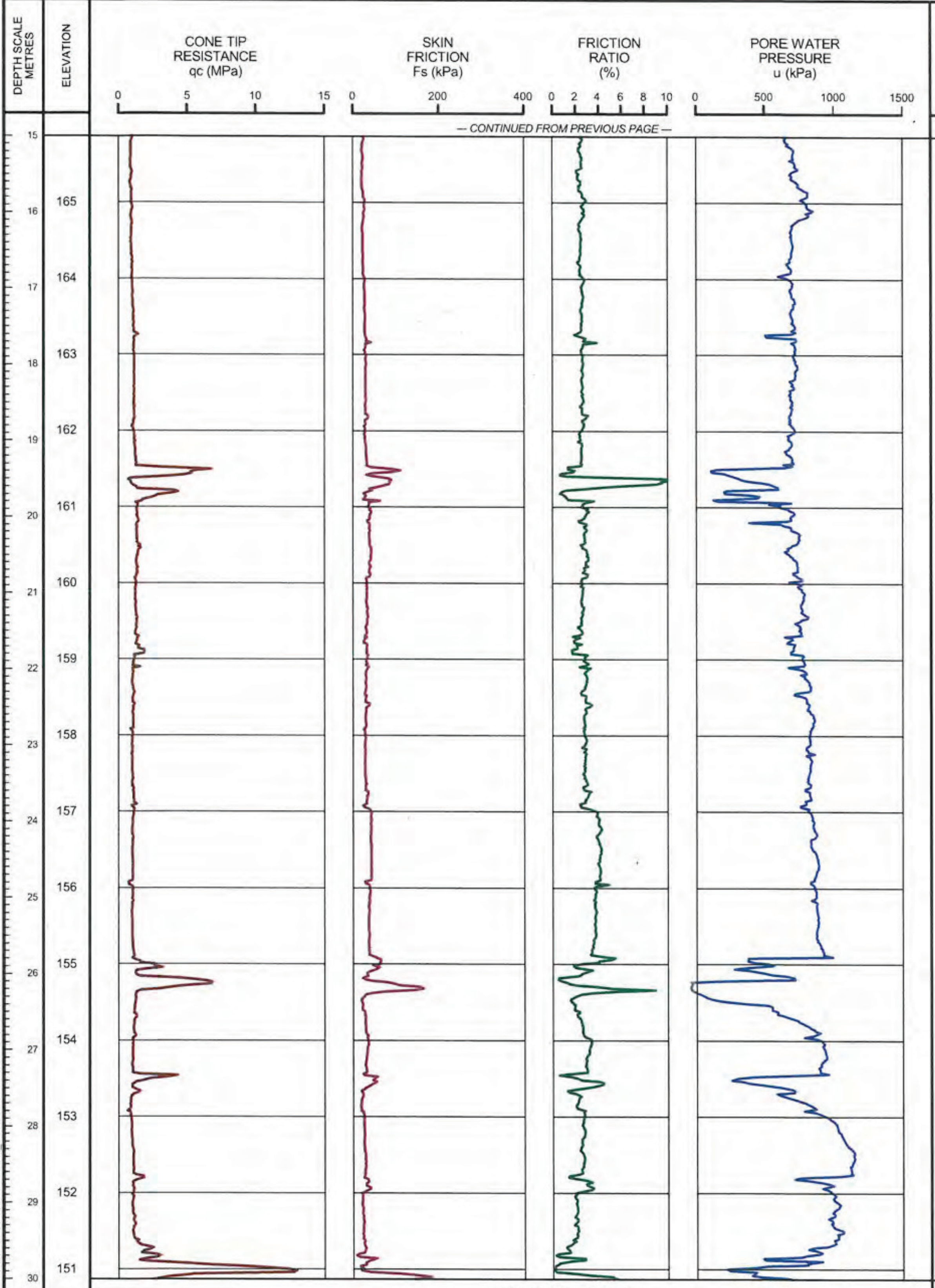
SHEET 2 OF 3

LOCATION: N 4679490.6 ;E 332200.8

TEST DATE: September 5, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 1.83m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



GENERAL NOTES AND OBSERVATIONS

— CONTINUED FROM PREVIOUS PAGE —

— CONTINUED NEXT PAGE —

DEPTH SCALE
1 : 75

Golder Associates

OPERATOR: CC
CHECKED: SJB

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-128

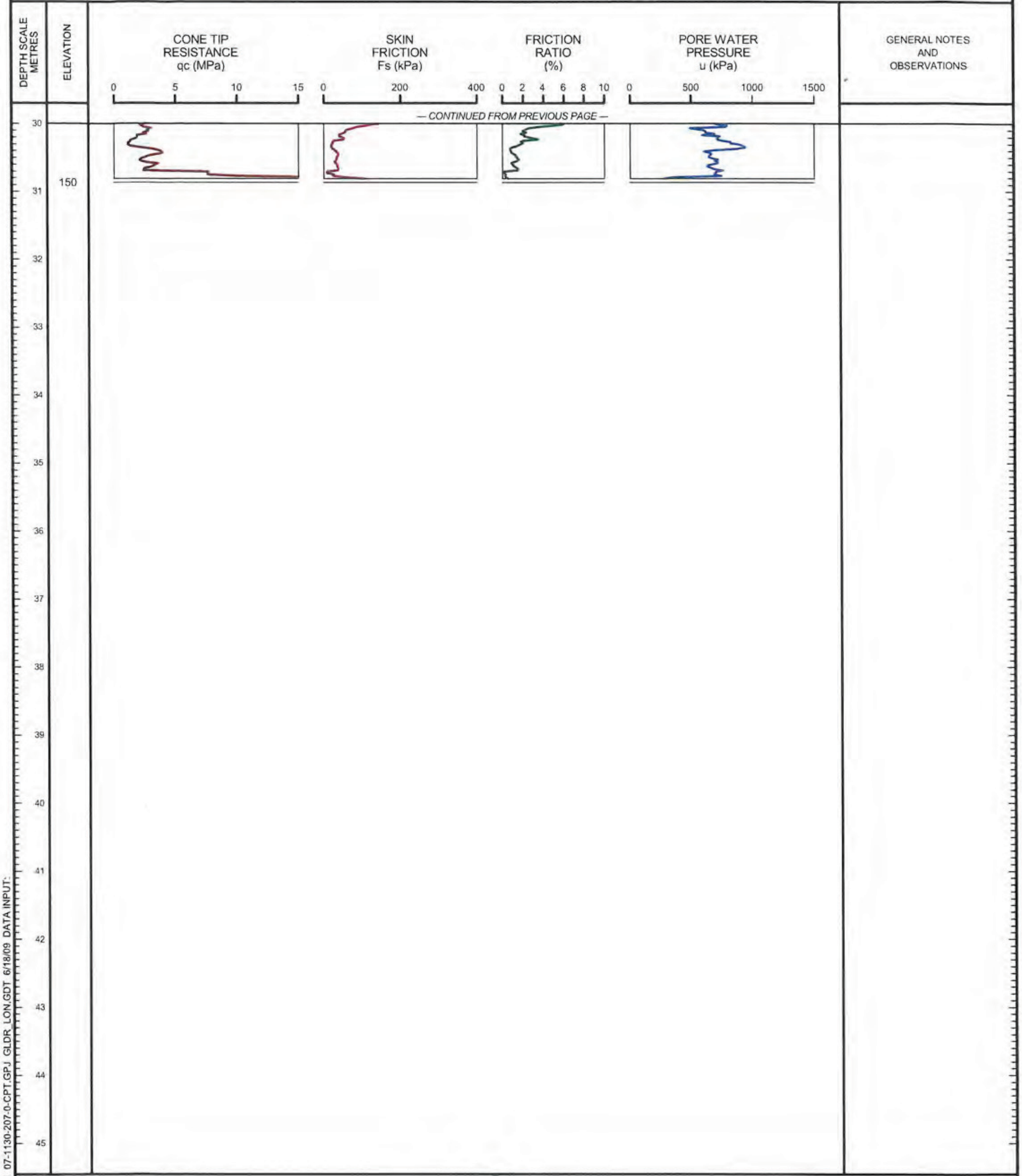
SHEET 3 OF 3

LOCATION: N 4679490.6 ;E 332200.8

TEST DATE: September 5, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 1.83m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



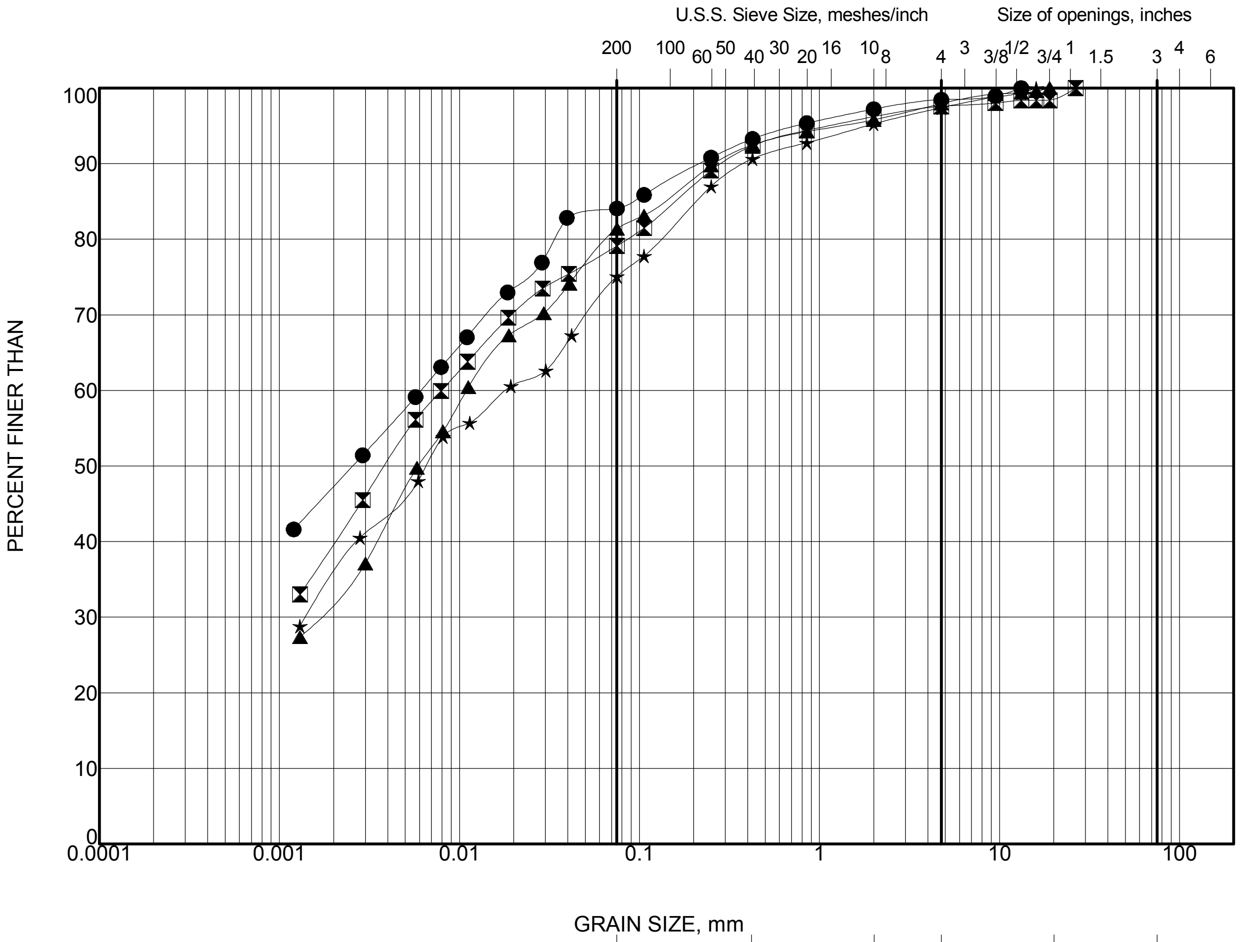
LDN_CPT_01 07-1130-207-0-CPT.GPJ GLDR_LON.GDT 6/18/09 DATA INPUT:

DEPTH SCALE
1 : 75

Golder Associates

OPERATOR: CC
CHECKED: SSB

Appendix C Laboratory Test Results



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

LEGEND:

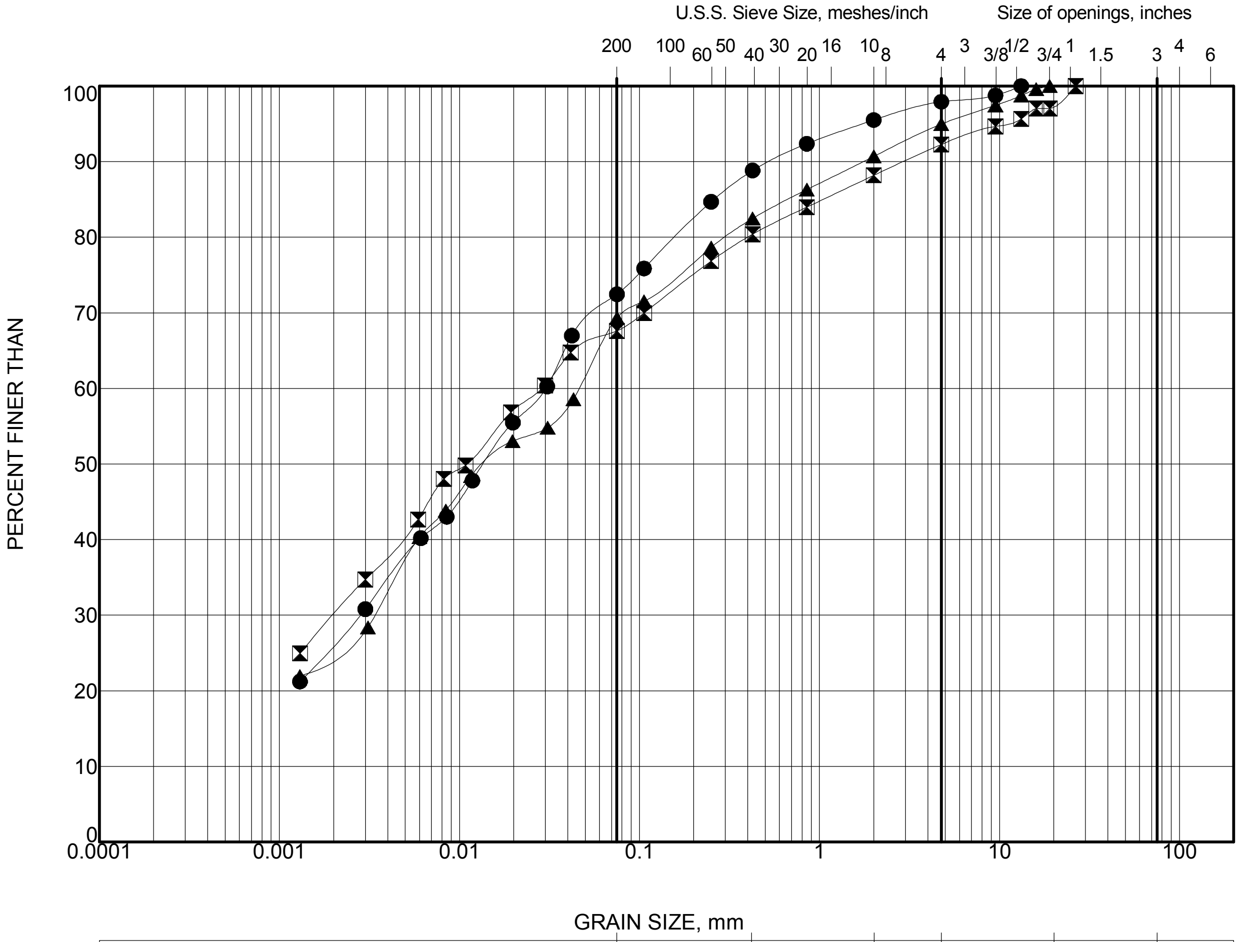
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	PS3-1	13	13.7
⧗	PS4-1	11	9.1
▲	T7-1	8	6.1
★	T7-1	12	12.2

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION
Upper Clay Layer

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.	
		SCALE	REV.
DRAWN	SS	Appendix **C**	
CHECK	GN		



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

LEGEND:

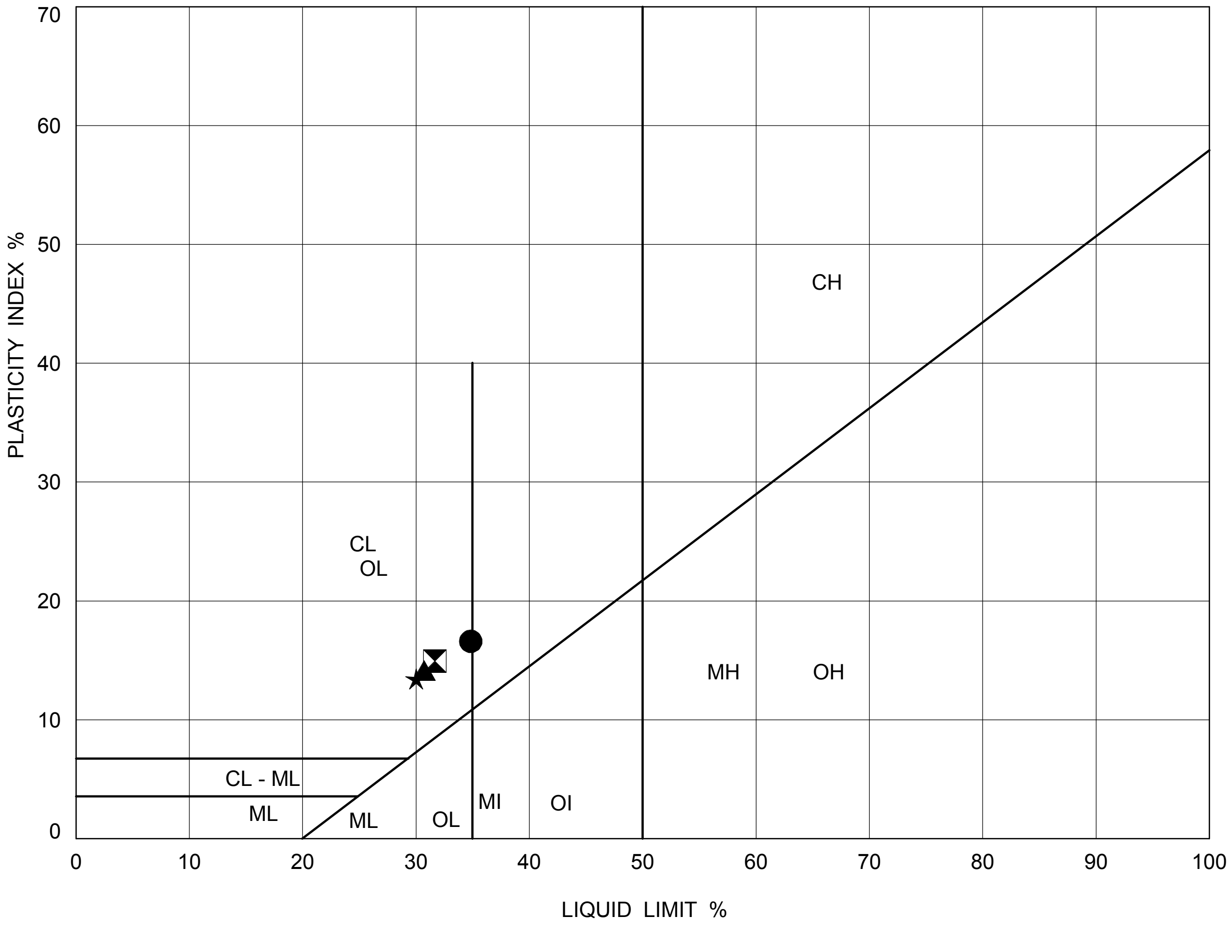
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	PS3-1	17	19.8
⧗	PS4-1	21	24.4
▲	T7-1	18	21.3

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION
Lower Clay Layer

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.
		SCALE / REV.
DRAWN	SS	
CHECK	GN	Appendix **C**



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

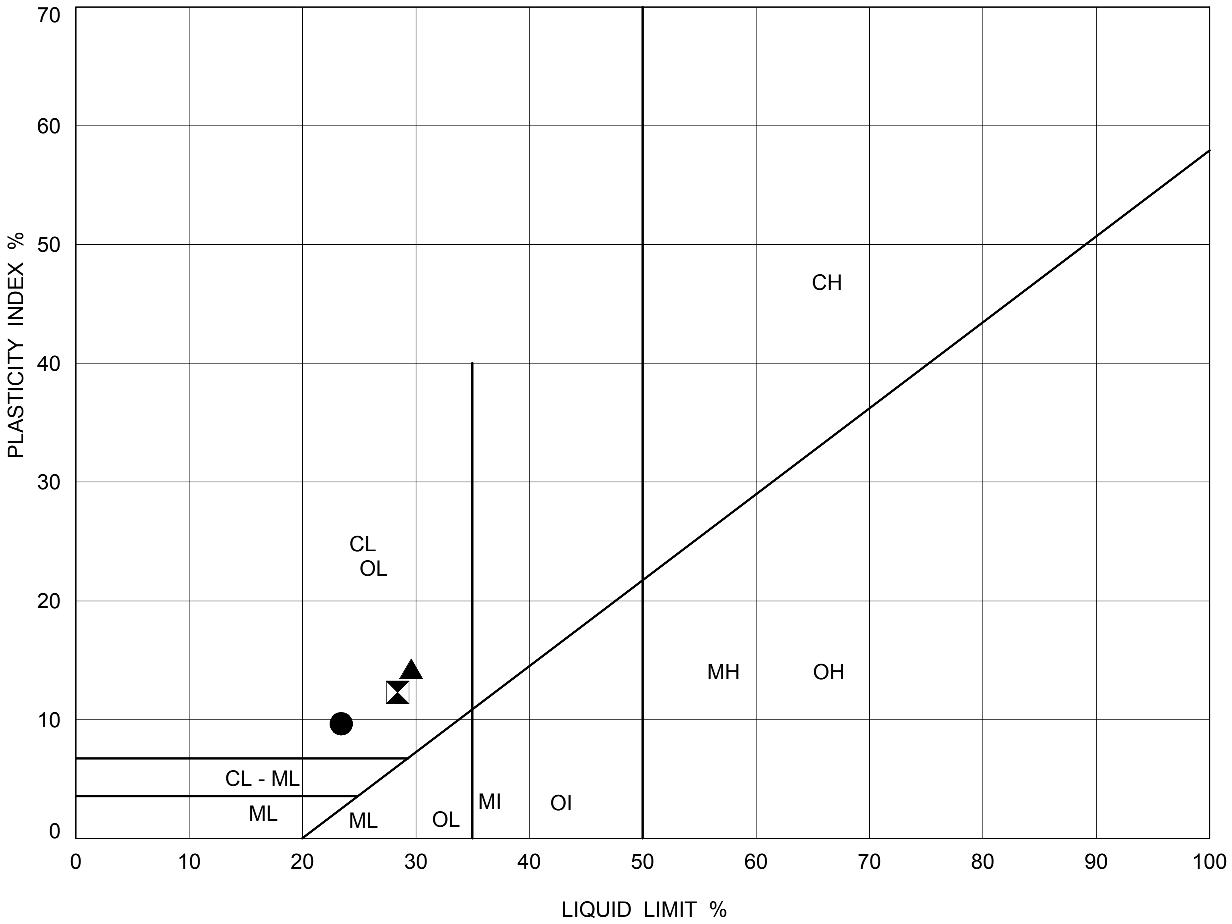
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	PS3-1	13	13.7	**35**	**18**	**17**
⧗	PS4-1	11	9.1	**32**	**17**	**15**
▲	T7-1	8	6.1	**31**	**17**	**14**
★	T7-1	12	12.2	**30**	**17**	**13**

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
PLASTICITY CHART
Upper Clay Layer

PROJECT No. SW8801.1004.101
FILE No.
SCALE
REV.
DRAWN SS
CHECK GN
Appendix **C**

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/04/12



SOIL TYPE	PLASTICITY
C = Clay	L = Low
M = Silt	I = Intermediate
O = Organic	H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	PS3-1	17	19.8	**23**	**14**	**9**
⧗	PS4-1	21	24.4	**28**	**16**	**12**
▲	T7-1	18	21.3	**30**	**15**	**15**

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
PLASTICITY CHART
Lower Clay Layer

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.
		SCALE / REV.
DRAWN	SS	
CHECK	GN	Appendix **C**

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/04/12

Appendix D Analytical Laboratory Test Results

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

Date Received:	15-AUG-11
Report Date:	22-AUG-11 08:24 (MT)
Version:	FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #:	**L1044501**
Project P.O. #:	NOT SUBMITTED
Job Reference:	SW8801.1004.101
C of C Numbers:	112850
Legal Site Desc:	

Gayle Braun

Gayle Braun
Senior Account Manager

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

Environmental www.alsglobal.com
RIGHT SOLUTIONS RIGHT PARTNER

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1044501-1				
	Description	SOIL				
	Sampled Date	12-AUG-11				
	Sampled Time	12:00				
	Client ID	PS3-1,SS10@30', GREY SILTY CLAY				
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	16.9				
	pH (pH units)	8.12				
	Redox Potential (mV)	160				
	Resistivity (ohm cm)	3280				
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20				
Anions and Nutrients	Sulphate (mg/kg)	236				

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:

112850

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: L1044501 Report Date: 22-AUG-11

Client: AMEC EARTH & ENVIRONMENTAL-WINDSOR
11865 County Road 42
TECUMSEH ON N8N 2M1
Contact: SHANE MACLEOD

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT	**Soil**							
Batch R2234818								
WG1330422-2 LCS								
% Moisture			90		%		70-130	15-AUG-11
WG1330422-1 MB								
% Moisture			<0.10		%		0.1	15-AUG-11
PH-WT	**Soil**							
Batch R2235381								
WG1331381-1 CVS								
pH			100		%		80-120	16-AUG-11
WG1331381-2 DUP		**L1044501-1**						
pH		8.12	8.11		pH units	0.12	20	16-AUG-11
RESISTIVITY-WT	**Soil**							
Batch R2238001								
WG1334286-1 CVS								
Resistivity			99		%		70-130	19-AUG-11
SO4-WT	**Soil**							
Batch R2236478								
WG1332024-3 LCS								
Sulphate			101		%		60-140	17-AUG-11
WG1332024-1 MB								
Sulphate			<20		mg/kg		20	17-AUG-11
SULPHIDE-WT	**Soil**							
Batch R2236612								
WG1332830-1 CVS								
Sulphide			106		%		50-120	18-AUG-11
WG1332826-1 MB								
Sulphide			<0.20		mg/kg		0.2	18-AUG-11

Quality Control Report

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

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential							
	1	12-AUG-11 12:00	19-AUG-11 20:29	24	176	hours	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 L1044501 were received on 15-AUG-11 09:30.

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-
WINDSOR
ATTN: SHANE MACLEOD
11865 County Road 42
TECUMSEH ON N8N 2M1

Date Received:	02-SEP-11
Report Date:	09-SEP-11 14:38 (MT)
Version:	FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #:	**L1053293**
Project P.O. #:	NOT SUBMITTED
Job Reference:	SW8801.1004.101
C of C Numbers:	113004
Legal Site Desc:	

Gayle Braun

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1053293-1				
	Description	SOIL				
	Sampled Date	01-SEP-11				
	Sampled Time					
	Client ID	PS4-1,SS4@7.5' SILTY SAND				
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	8.34				
	pH (pH units)	7.52				
	Redox Potential (mV)	338				
	Resistivity (ohm cm)	10000				
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20				
Anions and Nutrients	Sulphate (mg/kg)	<20				

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.

ALS Test Code	Matrix	Test Description	Method Reference**
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:

113004

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: L1053293 Report Date: 09-SEP-11

Client: AMEC EARTH & ENVIRONMENTAL-WINDSOR
11865 County Road 42
TECUMSEH ON N8N 2M1
Contact: SHANE MACLEOD

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT	**Soil**							
Batch R2246130								
WG1342214-2 LCS								
% Moisture			94		%		70-130	02-SEP-11
WG1342214-1 MB								
% Moisture			<0.10		%		0.1	02-SEP-11
PH-WT	**Soil**							
Batch R2246414								
WG1343256-5 CVS								
pH			99		%		80-120	06-SEP-11
REDOX-POTENTIAL-WT	**Soil**							
Batch R2248614								
WG1345733-1 DUP		**L1053293-1**						
Redox Potential		338	333		mV	1.5	25	09-SEP-11
RESISTIVITY-WT	**Soil**							
Batch R2248587								
WG1345734-1 CVS								
Resistivity			100		%		70-130	09-SEP-11
WG1345734-2 DUP		**L1053293-1**						
Resistivity		10000	9710		ohm cm	3.0	25	09-SEP-11
SO4-WT	**Soil**							
Batch R2247979								
WG1343886-2 DUP		**L1053293-1**						
Sulphate		<20	<20	RPD-NA	mg/kg	N/A	30	07-SEP-11
WG1343886-3 LCS								
Sulphate			100		%		60-140	07-SEP-11
WG1343886-1 MB								
Sulphate			<20		mg/kg		20	07-SEP-11
SULPHIDE-WT	**Soil**							
Batch R2247559								
WG1344742-1 CVS								
Sulphide			101		%		50-120	07-SEP-11
WG1344739-1 MB								
Sulphide			<0.20		mg/kg		0.2	07-SEP-11

Quality Control Report

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

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential							
	1	01-SEP-11	09-SEP-11	24	194	hours	EHTL
Resistivity							
	1	01-SEP-11	09-SEP-11	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 L1053293 were received on 02-SEP-11 09:15.

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.

Appendix E Stress-Deformation Analyses

Figure E-1: Sigma/W Model (Along Culvert)

Submerged Culvert S-1
Heave/Settlement
Last Solved Date: 7/24/2012

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m^3 Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m^3 Dilation Angle: 0 °
Name: LowerGranularLayer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 33 ° Unit Weight: 22 kN/m^3 Dilation Angle: 0 °
Name: Clay Transition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 19000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m^3 Dilation Angle: 0 °
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.6 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m^3 Phi': 26 °
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20.5 kN/m^3 Phi': 26 °
Name: Clay Backfill (Drained) Model: Elastic-Plastic Young's Modulus (E): 25000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m^3 Dilation Angle: 0 °
Name: UpperGranularFill Model: Elastic-Plastic Effective Young's Modulus (E'): 30000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m^3 Dilation Angle: 0 °
Name: Pipe/Inlet Model: Linear Elastic Young's Modulus (E): 100000 kPa Unit Weight: 15 kN/m^3 Poisson's Ratio: 0.15
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m^3 Phi': 26 °
Name: GranularBackfill Model: Elastic-Plastic Young's Modulus (E): 22000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 22 kN/m^3 Dilation Angle: 0 °
Name: Light Weight Fill / HRW-20L Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m^3 Poisson's Ratio: 0.35
Name: RGM Model: Linear Elastic Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m^3 Poisson's Ratio: 0.35

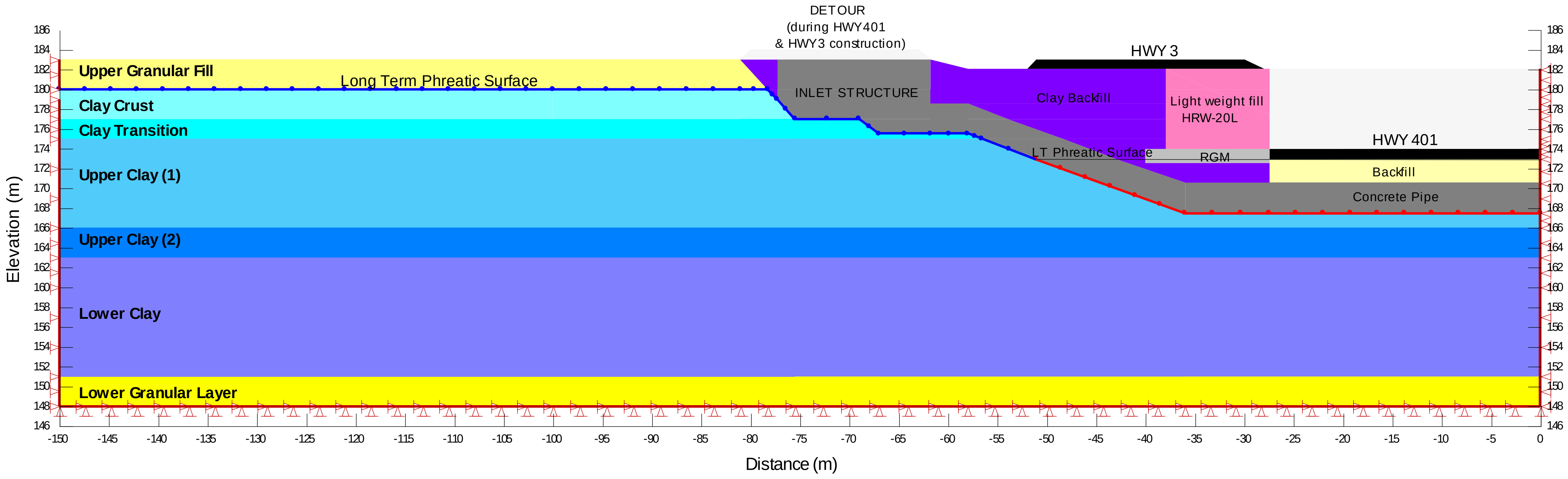


Figure E-2: Cumulative Heave/Settlement – End of Excavation for Highway 401 and HRW-20L

Submerged Culvert S-1
Excavation HWY401
Last Solved Date: 7/23/2012

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: LowerGranularLayer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 33 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Clay Transition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 19000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.6 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: UpperGranularFill Model: Elastic-Plastic Effective Young's Modulus (E'): 30000 kPa Poisson's Ratio: 0.35 Cohesion': 20 kPa Phi': 32 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 °

Note:
Positive (+) sign indicates heave movement
Negative (-) sign indicates settlement.

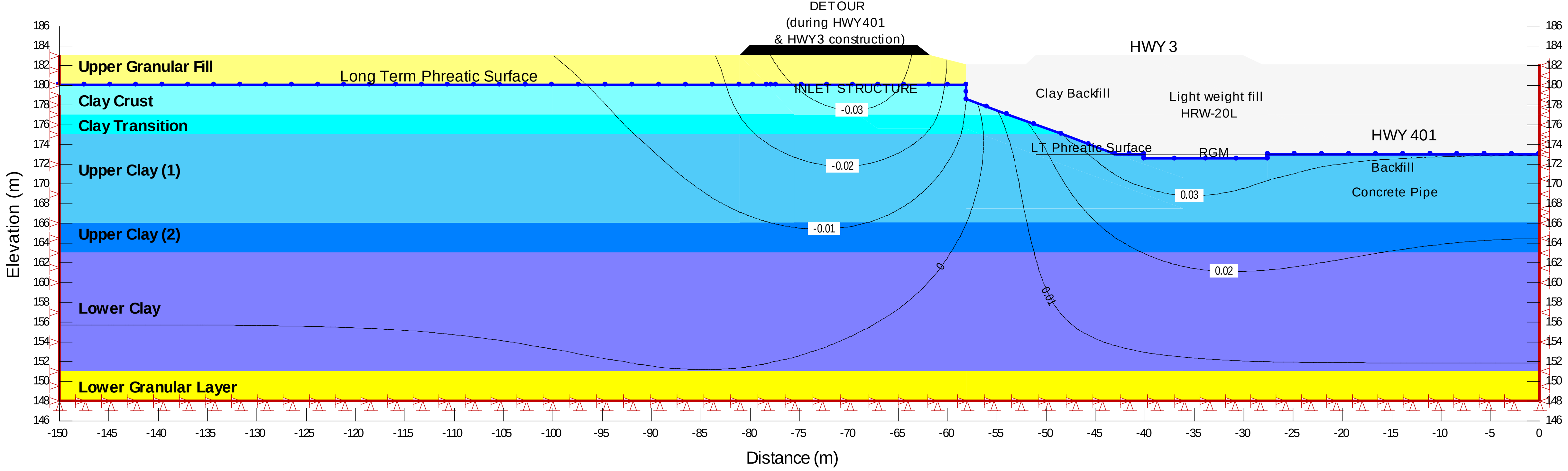


Figure E-3: Cumulative Heave/Settlement - End of Construction

Submerged Culvert S-1
Culvert&HRW-20L
Last Solved Date: 7/25/2012

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: LowerGranularLayer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 33 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Clay Transition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 19000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.6 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Clay Backfill (Drained) Model: Elastic-Plastic Young's Modulus (E): 25000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: UpperGranularFill Model: Elastic-Plastic Effective Young's Modulus (E'): 30000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Pipe/Inlet Model: Linear Elastic Young's Modulus (E): 100000 kPa Unit Weight: 15 kN/m³ Poisson's Ratio: 0.15
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 °
Name: GranularBackfill Model: Elastic-Plastic Young's Modulus (E): 22000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Light Weight Fill / HRW-20L Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m³ Poisson's Ratio: 0.35
Name: RGM Model: Linear Elastic Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35

Note:
Positive (+) sign indicates heave movement
Negative (-) sign indicates settlement.

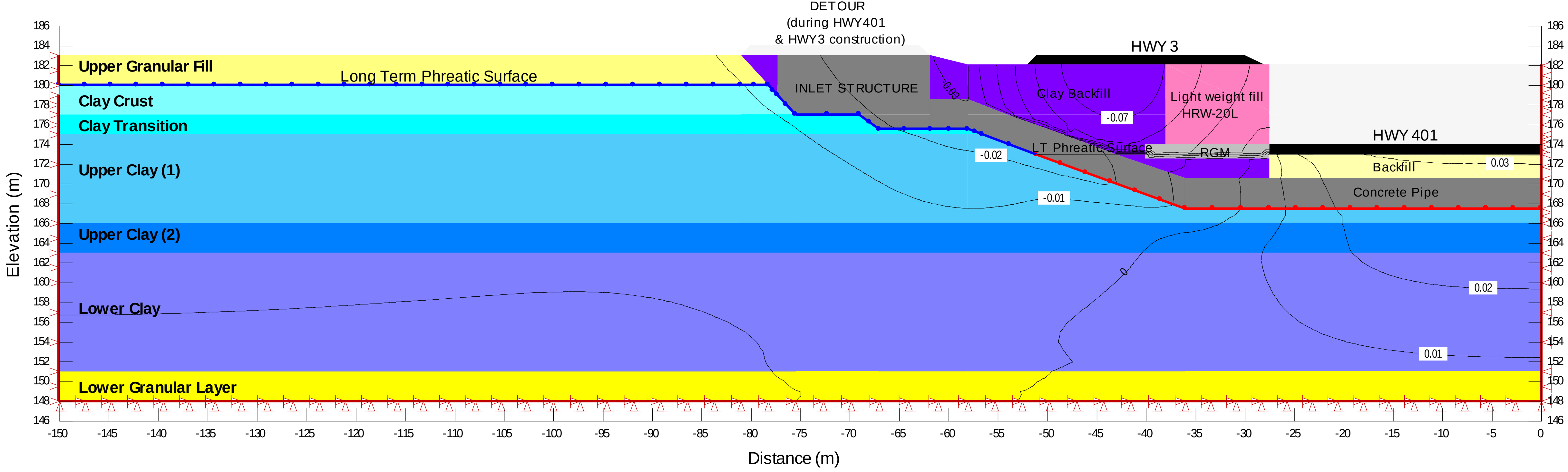


Figure E-4: Cumulative Heave/Settlement - Long-term (Drained)

Submerged Culvert S-1
Heave/Settlement
Last Solved Date: 7/25/2012

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: LowerGranularLayer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 33 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Clay Transition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 19000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.6 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Clay Backfill (Drained) Model: Elastic-Plastic Young's Modulus (E): 25000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: UpperGranularFill Model: Elastic-Plastic Effective Young's Modulus (E'): 30000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Pipe/Inlet Model: Linear Elastic Young's Modulus (E): 100000 kPa Unit Weight: 15 kN/m³ Poisson's Ratio: 0.15
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 °
Name: GranularBackfill Model: Elastic-Plastic Young's Modulus (E): 22000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Light Weight Fill / HRW-20L Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m³ Poisson's Ratio: 0.35
Name: RGM Model: Linear Elastic Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35

Note:
Positive (+) sign indicates heave movement
Negative (-) sign indicates settlement.

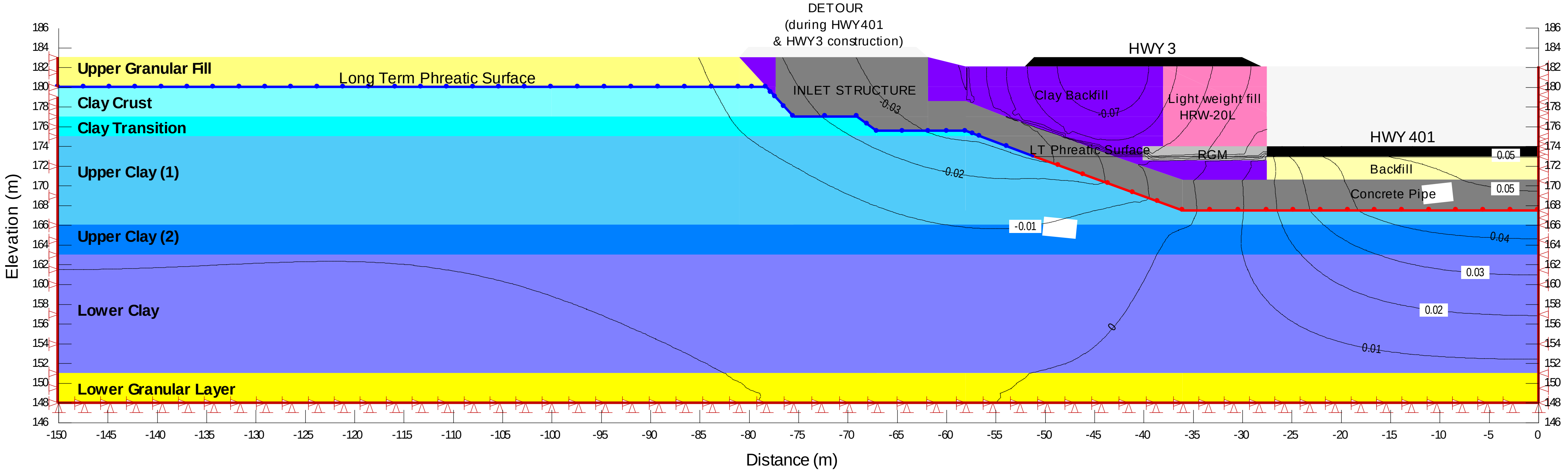


Figure E-5: Stabilized Pore Water Pressure Contours – Long-term (Drained)

Submerged Culvert S-1
Heave/Settlement
Last Solved Date: 7/25/2012

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m^3 Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m^3 Dilation Angle: 0 °
Name: LowerGranularLayer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 33 ° Unit Weight: 22 kN/m^3 Dilation Angle: 0 °
Name: Clay Transition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 19000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m^3 Dilation Angle: 0 °
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.6 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m^3 Phi': 26 °
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20.5 kN/m^3 Phi': 26 °
Name: Clay Backfill (Drained) Model: Elastic-Plastic Young's Modulus (E): 25000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m^3 Dilation Angle: 0 °
Name: UpperGranularFill Model: Elastic-Plastic Effective Young's Modulus (E'): 30000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m^3 Dilation Angle: 0 °
Name: Pipe/Inlet Model: Linear Elastic Young's Modulus (E): 100000 kPa Unit Weight: 15 kN/m^3 Poisson's Ratio: 0.15
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m^3 Phi': 26 °
Name: GranularBackfill Model: Elastic-Plastic Young's Modulus (E): 22000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 22 kN/m^3 Dilation Angle: 0 °
Name: Light Weight Fill / HRW-20L Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m^3 Poisson's Ratio: 0.35
Name: RGM Model: Linear Elastic Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m^3 Poisson's Ratio: 0.35

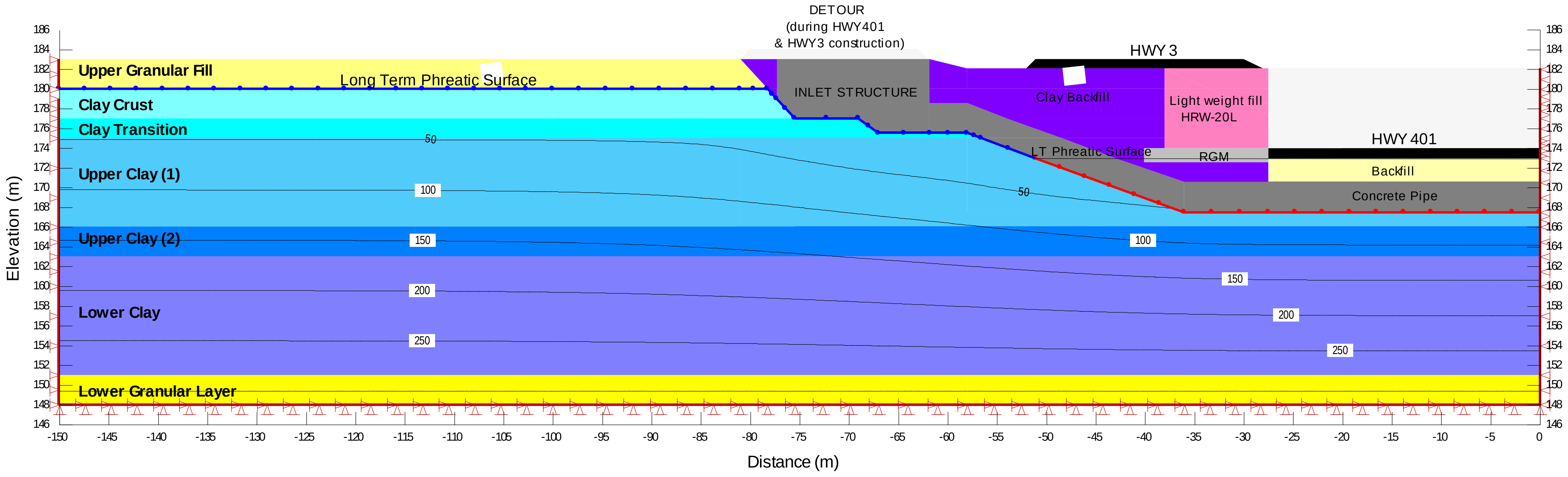


Figure E-6: Cumulative Heave/Settlement at Pipe Invert below Highway 401

21 days = End of Excavation
28 days = End of Construction
14600 days = Long-term Condition
Positive (+) Y-Displacement indicates heave movement and negative (-) Y-Displacement indicates settlement.

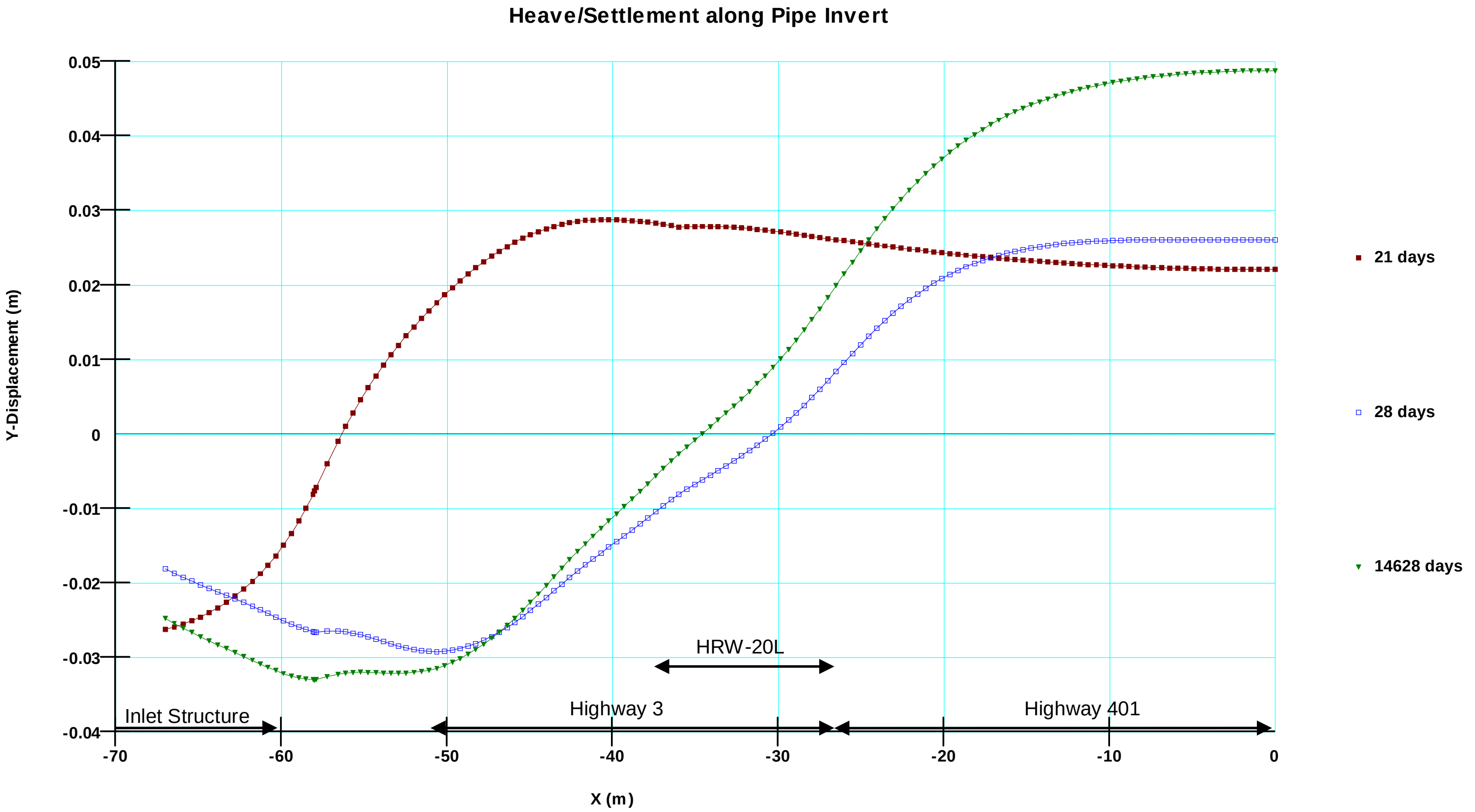


Figure E-7: Cumulative Heave/Settlement at Pipe Invert below Highway 401

28 days = End of Construction
14600 days = Long-term Condition
Positive (+) Y-Displacement indicates heave movement and negative (-) Y-Displacement indicates settlement.

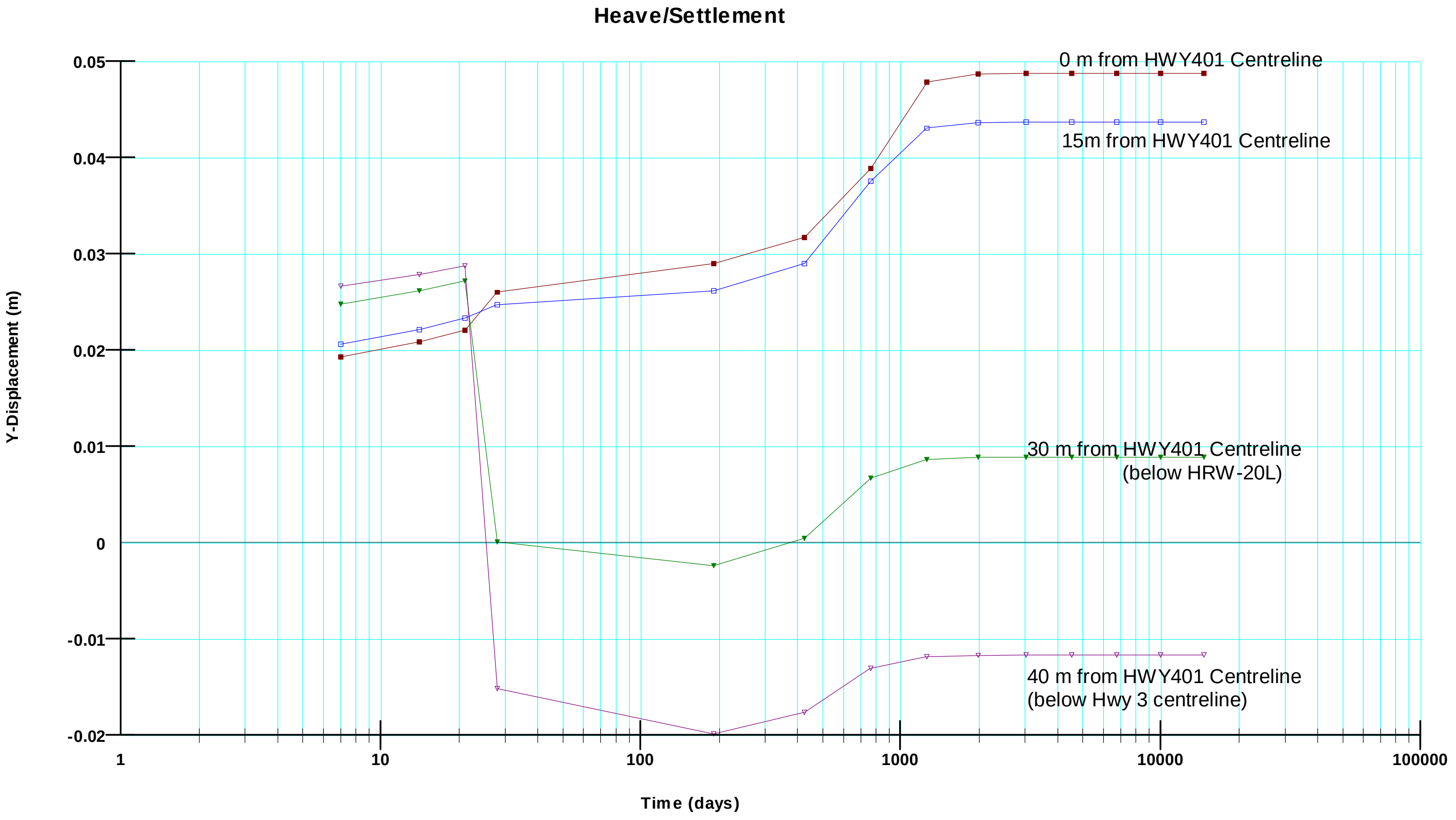


Figure E-8: Sigma/W Model for Culvert Construction (short-term conditions) at Highway 401 Centreline (after Excavation for Highway 401)

Submerged Culvert S-1
BackFill
Last Solved Date: 4/3/2012

Name: Clay Backfill Model: Elastic-Plastic Young's Modulus (E): 25000 kPa Poisson's Ratio: 0.49 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: LowerGranularLayer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 33 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.6 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Pipe/Inlet Model: Linear Elastic Young's Modulus (E): 100000 kPa Unit Weight: 15 kN/m³ Poisson's Ratio: 0.15
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: GranularBackfill Model: Elastic-Plastic Young's Modulus (E): 22000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °

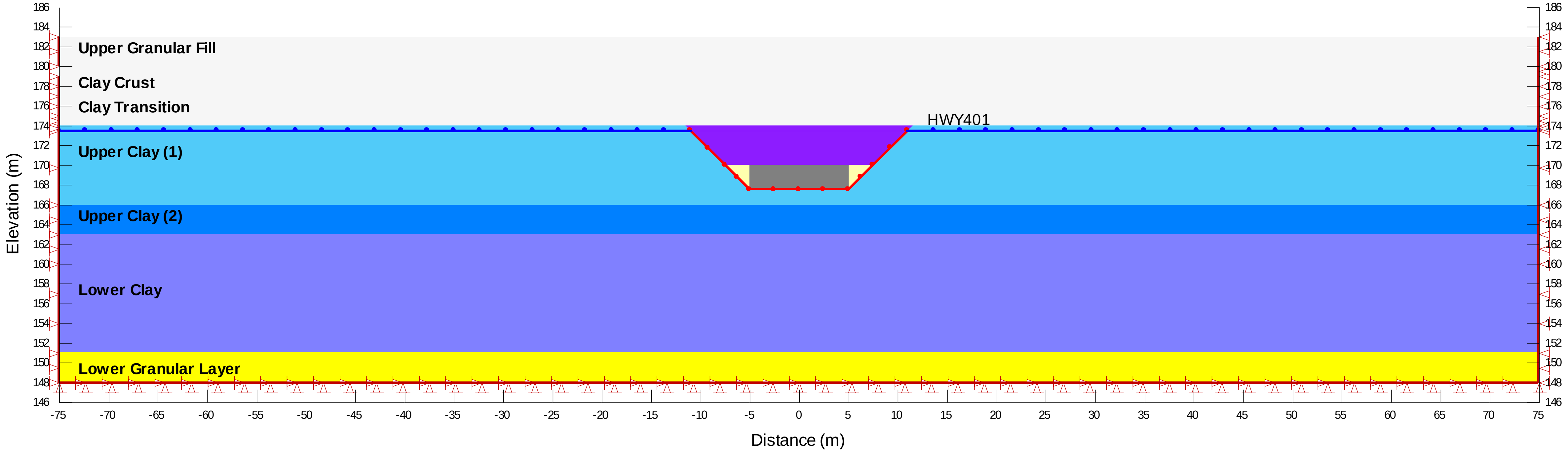


Figure E-9: Incremental short-term Heave/Settlement at End of Temporary Excavation for Culvert Construction at Highway 401 Centreline (after Excavation for Highway 401)

Submerged Culvert S-1
Temp.Excv.
Last Solved Date: 4/3/2012

Name: LowerGranularLayer   Model: Elastic-Plastic   Effective Young's Modulus (E'): 40000 kPa   Poisson's Ratio: 0.35   Cohesion': 0 kPa   Phi': 33 °   Unit Weight: 22 kN/m³   Dilation Angle: 0 °
Name: Upper Clay (1) (Drained)   Model: Soft Clay (MCC)   O.C. Ratio: 1.6   Poisson's Ratio: 0.35   Lambda: 0.082   Kappa: 0.009   Initial Void Ratio: 0.6   Unit Weight: 21 kN/m³   Phi': 26 °   K-Function: Conductivity_Unweathered   Vol. WC. Function: Volumetric WC_Unweathered   Load Response Ratio: 1
Name: Lower Clay (Drained)   Model: Soft Clay (MCC)   O.C. Ratio: 1.5   Poisson's Ratio: 0.35   Lambda: 0.063   Kappa: 0.007   Initial Void Ratio: 0.5   Unit Weight: 20.5 kN/m³   Phi': 26 °   K-Function: Conductivity_Unweathered   Vol. WC. Function: Volumetric WC_Unweathered   Load Response Ratio: 1
Name: Upper Clay (2) (Drained)   Model: Soft Clay (MCC)   O.C. Ratio: 1.4   Poisson's Ratio: 0.35   Lambda: 0.082   Kappa: 0.009   Initial Void Ratio: 0.6   Unit Weight: 20 kN/m³   Phi': 26 °   K-Function: Conductivity_Unweathered   Vol. WC. Function: Volumetric WC_Unweathered   Load Response Ratio: 1

Note:
Positive (+) sign indicates heave movement
Negative (-) sign indicates settlement.

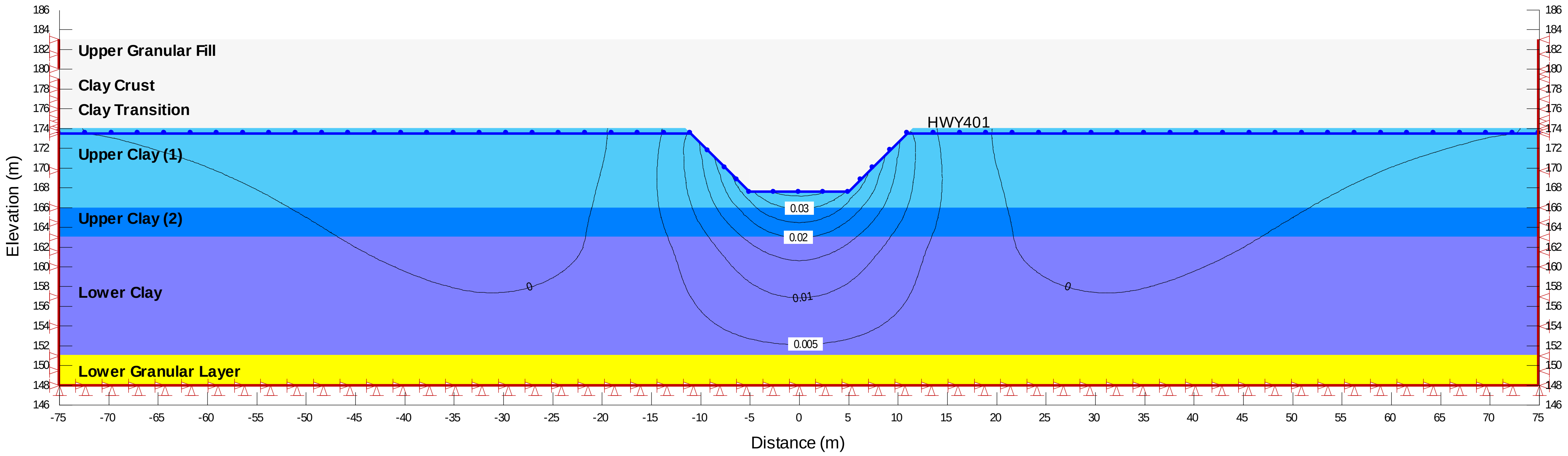


Figure E-10: Incremental short-term Heave/Settlement at End of Culvert Construction at Highway 401 Centreline (after Excavation for Highway 401)

Submerged Culvert S-1
BackFill
Last Solved Date: 4/3/2012

Name: Clay Backfill Model: Elastic-Plastic Young's Modulus (E): 25000 kPa Poisson's Ratio: 0.49 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: LowerGranularLayer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 33 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.6 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Pipe/Inlet Model: Linear Elastic Young's Modulus (E): 100000 kPa Unit Weight: 15 kN/m³ Poisson's Ratio: 0.15
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: GranularBackfill Model: Elastic-Plastic Young's Modulus (E): 22000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 °

Note:
Positive (+) sign indicates heave movement
Negative (-) sign indicates settlement.

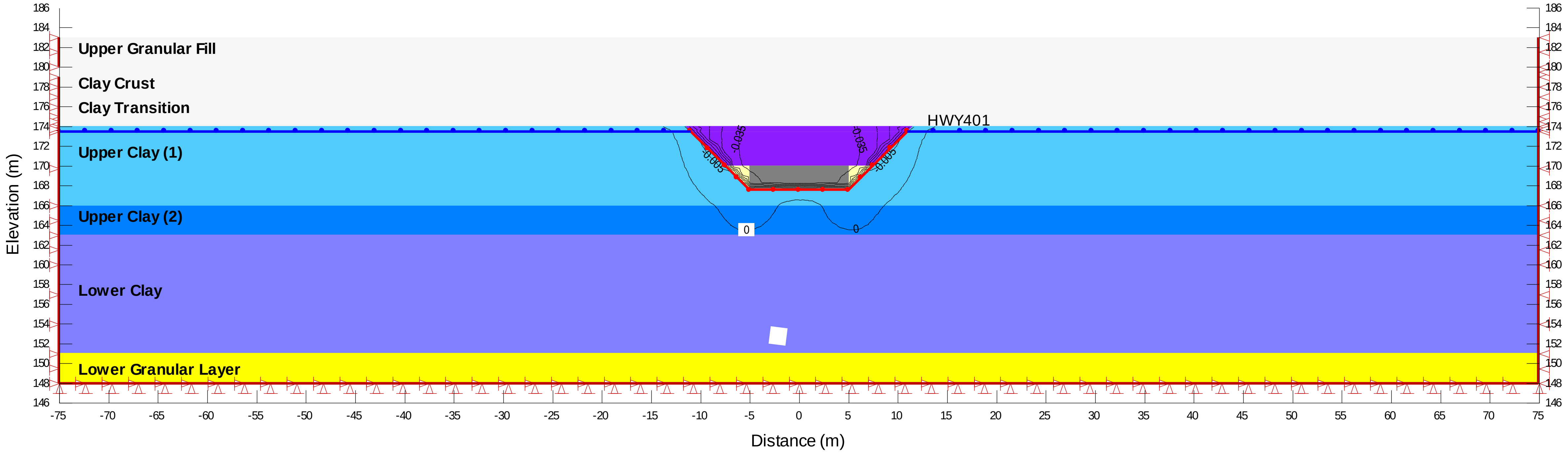
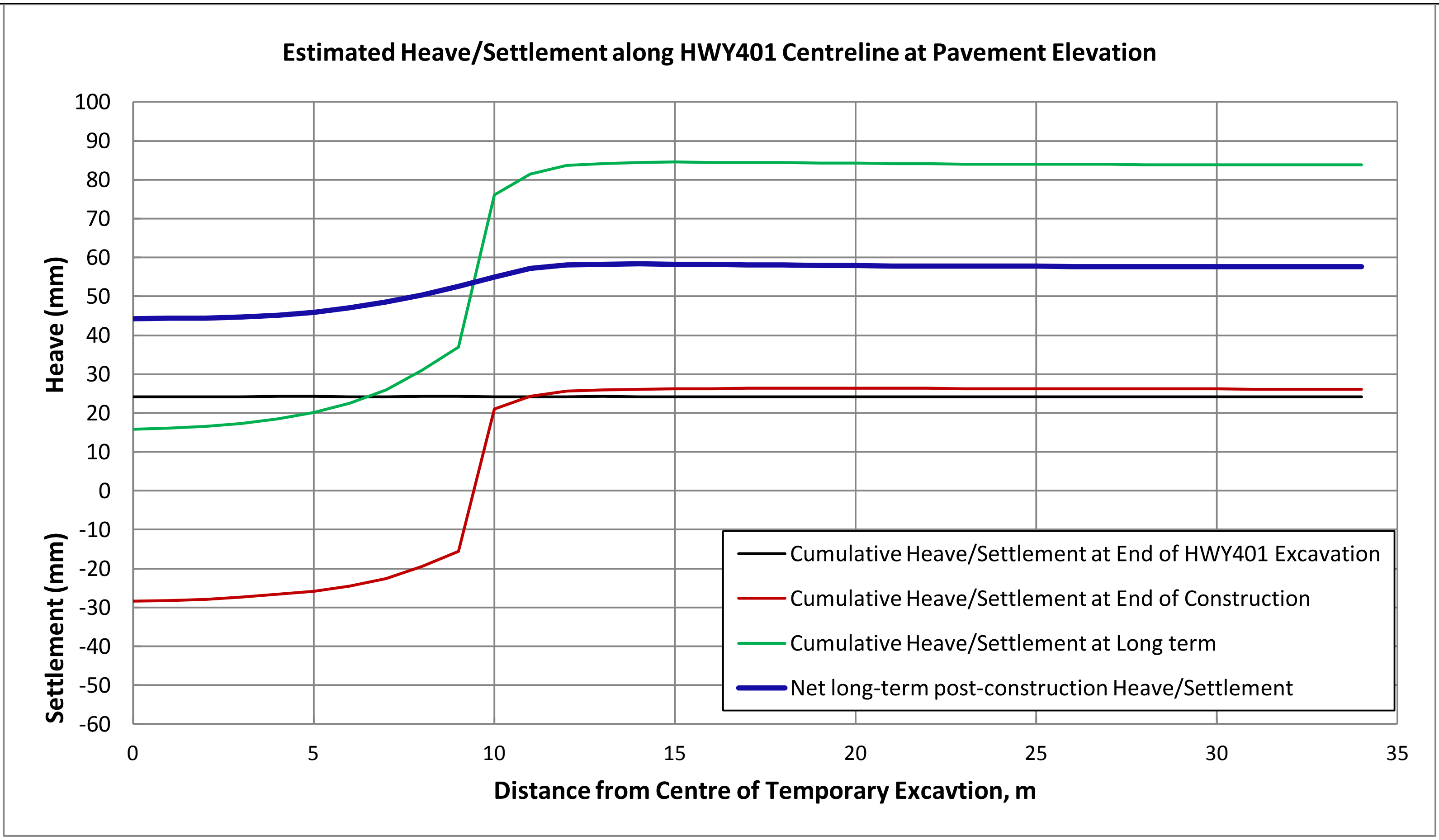


Figure E-11: Estimated Heave/Settlement along Highway 401 Pavement Elevation



Appendix F Rock Core Photographs

Photograph F-1: Borehole T7-1 – Rock Core Elevation 148.9 to 145.8 m

Photograph F-2: Borehole PS3-1 – Rock Core Elevation 149.4 to 145.7 m

Photograph F-3: Borehole PS4-1 – Rock Core Elevation 146.0 to 144.9 m