

The Windsor-Essex Parkway Project Geotechnical Investigation and Design Report Submerged Culvert S-2

(Cahill Drain, Sta. 11+175 LaSalle)



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1 Introduction

1.1 Preface

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

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

The plans for the Parkway strive to build and strengthen linkages within and between 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

This report presents the geotechnical design of the Submerged Culvert S-2, located on Cahill drain in LaSalle sector of the proposed Windsor-Essex Parkway (WEP) project. The culvert will be located between Highway 401 Sta. 11+150L and Sta. 11+200L (LaSalle).

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.

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).

The Submerged Culvert S-2 involves three submerged concrete pipes which will traverse under Highway 401, Highway 3 (Bridge B-10), East Bound Ramp 7 (EBR7), East Bound Ramp 8 (EBR8) and West Bound Ramp 6 (WBR6). It will be located at depths ranging from 6 m and 14 m below the finished grades of the highways. The general arrangement of Culvert S-2 is shown on Drawing 285380-03-060-WIP1-S4201.

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 provided for the earlier submissions of the design report.

¹ References are listed in Section 9.

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.

The design of the Cahill drain Submerged Culvert S-2 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. 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 $Z_v = 0$ and Zonal Acceleration $Z_a = 0$. 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 meet in general the description for Soil Profile Type III (soft clay and silts greater than 12 m in depth). A limited number of cross-hole tests were completed during the background investigation program (ref. R-17 to R-25) at locations distributed strategically along the project alignment between Howard Road (east end) and Matchette Road (west end). The measured velocities of the shear waves were consistently over 200 m/s, with the bulk of results ranging between 200 and 300 m/s.

2.3 Frost Depths

In accordance with MTO–SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-40) and OPSD 3090.101 the frost depth below the ground surface in Windsor area is estimated to 1.0 m². 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.

2.4 Site Conditions

Cahill Submerged Culvert S-2 site is situated in the LaSalle segment of the Parkway, approximately 1 km east of the border between the Windsor and LaSalle Municipalities in Ontario. The culvert is located beneath Bridge B-10 (Highway 3) and west of the bridge's east abutment. The existing open ditch Cahill drain running roughly east-west crosses the existing Highway 3 through the proposed culvert. As indicated earlier in Section 1.2, Culvert S-2 will traverse under the proposed Highway 401, Highway 3 (Bridge B-10), EBR7, EBR8 and WBR6. The culvert will traverse under Bridge B-10 between bridge Pier 3N and Pier 3S located west of bridge's east abutment. The ground topography around the proposed location of the culvert is essentially flat with elevations ranging between 180.0 and 182.0³. Adjacent land use is typically urban residential, parkland and light commercial.

² Ontario Provisional Standard Drawings are included at the end of the report text.

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

3 Geotechnical Investigations

3.1 Scope and Procedures of Geotechnical Investigations

Geotechnical investigations involving a number of boreholes, cone penetration tests (CPT) and Nilcon vane tests had been carried out in 2007-09 by Golder Associates (ref. R-16 to R-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-2 comprised a total of 6 boreholes (B10-2, B10-3, B10-4, B10-5, B10-6 and B10-7), 3 cone penetration tests (CPT B10-2, CPT B10-3 and CPT 41-RW) and 1 Nilcon vane profile (adjacent to Borehole B10-3). Table 3-1 lists the test holes put down at or in close proximity (within 100 m or so) of the Culvert S-2 site during the previous and the current geotechnical investigations.

Table 3-1: Test Holes at and around Submerged Culvert S-2 Site

Reference	Boreholes	Nilcon Vane Tests	CPTs
Additional Investigation (2011)	B10-2		CPT B10-2
	B10-3	NIL B10-3	CPT B10-3
	B10-4		
	B10-5		
	B10-6		
	B10-7		
	CPT 41-RW(*)		CPT 41-RW
Previous Studies (2007-09)	BH/CPT-317(*)		CPT-317
	BH-318		

(*)Shallow boreholes drilled to facilitate CPT testing.

Drawing 285380-04-090-WIP1-4201 shows the locations of the test holes and an interpreted soil stratigraphic profile along the WEP centreline for the general area around Culvert S-2. Drawings 285380-04-090-WIP1-4202 and 285380-04-090-WIP1-4203 shows the locations of the test holes put down in close proximity to the submerged culvert S-2 site and stratigraphic sections along and across the Culvert S-2.

3.1.1 Fieldwork for Additional Geotechnical Investigation

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 either a 50 mm diameter split spoon sampler or the 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 and transported to AMEC's Tecumseh 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 the Borehole B10-3 location. Table 3-2 summarizes the depths of overburden penetration and elevation ranges where rock coring and Nilcon vane tests were carried out.

Table 3-2: Overburden Thickness, Rock Coring, Nilcon Vane Tests and Instrumentation in Boreholes

Borehole	Location	GS Elevation (m)	Overburden Thickness (m) ^(*)	Test, Instrument & Elevation				
				Rock Coring	Nilcon Vane	S-Piez.	VWP	MHSG
B10-2 (2011)	4679069.6N 332866.8E	182.3	33.2	149.1 to 147.2	-			
B10-3 (2011)	4679083.2N 332836.8E	182.2	32.4	149.8 to 148.0	176.7 to 162.2		172.1 162.1 149.9	
B10-4 (2011)	4679068.8N 332867.5E	182.4	32.7	149.7 to 147.0	-			
B10-5 (2011)	4679023.7N 332901E	182.3	32.6	149.7 to 147.9	-	149.7	170.7 162.1	
B10-5-P32 (2011)	4679021.0N 332901.4E	182.3	32.3	150.0 to 139.5	-		150.3	
B10-6 (2011)	4678990.9N 332930.4N	182.0	32.3	149.7 to 147.3	-			
B10-7 (2011)	4678978.0N 332968.0E	182.2	32.6	149.6 to 146.5	-		173.8 162.4	162.4
BH-318 (Pre-Bid)	4679049.3N 332857.8E	182.3	32.6	149.7 to 144.9	-			

Legend: VWP Vibrating wire piezometer
S-Piez. Screen elevations for Standpipe Piezometers
MHSG Spider magnet heave/settlement gauge
GS Ground Surface
(*) Overburden includes existing fill thickness

Rock cores were examined in the field and transported to AMEC's Tecumseh (Windsor) laboratories for further examination. For each core run, rock core recovery and rock quality designation (RQD) were determined. The core recovery and RQD values are given on the borehole logs. The rock cores were photographed in the laboratory and the photographs are presented in Appendix G. 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 CPT 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). All three CPTs (completed during the 2011 additional investigation) were advanced to refusal encountered near elevations 150 to 152. CPT-317 (completed by Golder Associates during pre-bid investigation) was terminated earlier at around elevation 158. A pore pressure dissipation test was carried out in CPT 41-RW at 14.0 m (elevation 167.9) depth below ground surface.

The locations of boreholes, Nilcon tests and CPTs executed during the 2011 additional investigation and the previous investigations, and the inferred soil profile along the WEP alignment (Sta. 10+900L to Sta. 11+500L), are shown on Drawings 285380-04-090-WIP1-4201. The test hole locations in plan and soil stratigraphic section at the culvert location are shown on Drawings 285380-04-091-WIP1-4202 and 285380-04-091-WIP1-4203. 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.

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

Vibrating Wire Piezometers (VWP): The VWP transducers (RST Model VW2100, 0.35 MPa for shallow to mid-depth and 0.7 MPa for deep installations) were installed at selected depths and their 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. Each magnetic torus was placed 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 obtained from Boreholes B10-2, B10-3, B10-4, B10-5, B10-6 and B10-7 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 geotechnical laboratory tests 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 3-1⁴ 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-34). 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:

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

Where:

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

Q_t is the corrected total cone tip resistance;

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

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-2 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. In CPTs indicating pore pressures higher than cone tip resistance (e.g., upper clay and lower clay stratum in CPT-317, CPT B10-2 and CPT B10-3), the undrained shear strength was estimated from the excess pore pressures (using the N_u method). Figure 3.3a presents the undrained shear strength (S_u) and maximum past pressure (P_c') profiles for WEP segment between Sta. 10+900L and Sta. 11+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:

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

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'_{vo} , of normally consolidated soil,

OCR is the overconsolidation ratio, and

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

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

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

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 CPT's put down between Sta. 11+100L and Sta. 11+300L are presented on Figure 3.3b. The moisture content plot in Figure 3.3b includes data from boreholes located within Sta.10+900L to Sta.11+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 existing ground surface elevation around the location of the Culvert S-2 varies from approximately 182.0 at the inlet structure (north of WBR6) to 180.0 at the outlet structure (south of EBR7).

The general soil stratigraphy encountered at the borehole and CPT locations consists of following successive strata: topsoil and thin discontinuous upper sand deposit (up to 0.7 m thick); an extensive clayey silt to silty clay deposit to approximate elevation 151; and a discontinuous 0 to 1.3 m thick lower granular deposit at about elevation 150; and limestone bedrock below about elevation ranging from 149 to 150. The thickness of the clayey silt to silty clay deposit based on the available nearby boreholes is about 32.5 m.

4.1 Top Soil and Upper Granular Deposit

A layer of top soil was encountered at the ground surface in all boreholes located around Culvert S-2. The thickness of the top soil layer ranges from 200 to 800 mm at borehole locations. A layer of upper granular deposit was encountered beneath the topsoil in Boreholes B10-2 and B10-6. The upper granular deposit consisted of fine sand to silty sand and was encountered approximately between elevations 181.2 and 182.0 (maximum depth of 900 mm from ground surface). The thickness of the upper granular deposit varied from 500 to 700 mm at the borehole locations and is at very loose to loose state of compactness.

4.2 Silty Clay to Clayey Silt Stratum

An extensive deposit of clayey silt to silty clay stratum was encountered directly underlying the top soil (or upper granular layer) at about elevation 181.5 m. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into 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	181.5 to 177.0	177.0 to 175.0	175 to 163	163 to 151
Natural Water Content, w_N , %	9 to 31 (16)*	15 to 23 (20)	11 to 39 (22)	14 to 37 (20)
Liquid Limit, w_L	31 to 44 (37)	30 to 34 (31)	24 to 39 (31)	23 to 34 (28)
Plastic Limit, w_P	14 to 23 (18)	15 to 17 (16)	14 to 20 (16)	13 to 17 (15)
Plasticity Index, PI	17 to 22 (19)	15 to 17 (16)	10 to 21 (15)	9 to 18 (13)
Liquidity Index, LI	0 to 0.1	0.1 to 0.3	0.1 to 1.1	0 to 1.4
Design Unit Weight, kN/m^3	21	21	20	20.5

* The numbers in parentheses represent average values.

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

- Clay Crust layer: $> 80 \pm 20$ kPa
- Clay Transition layer: 80 ± 20 kPa to 55 ± 10 kPa
- Upper silty clay: 55 ± 10 kPa to 45 ± 10 kPa to 50 ± 10 kPa
- Lower clayey silt: 50 ± 10 kPa to 65 ± 10 kPa to $> 75 \pm 10$ 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:

$$C_c = 0.0086w_N - 0.0086$$

$$C_r = 0.11C_c$$

$$C_s = 0.25C_c$$

$$C_\alpha = 0.028C_c$$

Where C_c , C_r , C_s and C_α are coefficients of compression, reloading, swelling and creep, respectively.

The interpreted compressibility parameters used for the silty clay / clayey silt substrata for Submerged Culvert S-2 site are summarized in Table 4-2 based on average water content data. Based on the variation of undrained shear strength and maximum past pressures, the upper silty clay stratum was sub-divided into two strata (namely, Upper Silty Clay-1 and Upper Silty Clay-2) for analyses purposes.

The moduli of elasticity for undrained and drained conditions were estimated using empirical correlation based on published information (ref. R-45) and local experience (ref. R-21). For the unweathered silty clay to clayey silt stratum, the following empirical relationships were used;

$$\text{Elastic modulus (undrained conditions)} \quad E_u = 300 \times S_u$$

$$\text{Elastic modulus (drained conditions)} \quad E' = 0.9 \times E_u$$

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	181.5 to 177	177 to 175	175 to 166	166 to 163	163 to 151
Average Natural Water Content, w_N , %	16	20	23	18	20
Average Total Unit Weight (kN/m ³)	21	21	20	20	20.5
Preconsolidation Pressure	550	550 to 325	325 to 230	230 to 260	260 to 380
Over-Consolidation Ratio	> 4	4 to 2	2.0	1.2	1.2
Virgin Compression Index, C_c	0.129	0.163	0.189	0.146	0.163
Recompression Index, C_r	0.014	0.018	0.021	0.016	0.018
Swelling Index, C_s	0.032	0.041	0.047	0.037	0.041
Secondary Compression Index, C_α	0.0036	0.0046	0.0053	0.0041	0.0046

Note: 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	35	0.49	31	0.35
Transition	20	0.49	18	0.35
Upper Silty Clay-1	15	0.49	13	0.35
Upper Silty Clay-2	14	0.49	12	0.35
Lower Clayey Silt	20	0.49	18	0.35

* - Assumed values (ref R-45)

The effective shear strength properties applicable to the silty clay stratum were determined from triaxial and direct shear tests performed during the pre-bid and additional geotechnical investigations (Figure 4.3) and supported by published PI versus ϕ' relationships (ref. R-21, R-30, Figure 4.4), and 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)

The hydraulic conductivity of the clayey silt to silty clay 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 stratum, dense to very dense lower granular deposit was encountered in Boreholes B10-2, B10-3, B10-4, B10-5, B10-7 and BH-318. This lower granular deposit consisted of sand and gravel to sandy silt. This layer was encountered at about elevation 150 and 151, and varied in thickness from 0.7 to 2.0 m at the borehole locations. Based on the Standard Penetration Test (SPT) “N” value ranging between 23 and 75, the lower granular deposit is considered to be in dense to very dense state of compactness.

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 149.1 to 149.8 at borehole locations in the vicinity of Submerged Culvert S-2. Photographs of rock cores recovered from the additional investigation are provided in Appendix F.

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. R-28). 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 $m_i = 12$ for a crystalline limestone, a disturbance factor of 0.7, and a factor of safety of 3.0, an allowable bearing capacity of the rock has been calculated to range from 5.3 MPa to 13.5 MPa. The mean allowable bearing capacity is determined to be 9.2 MPa using the Hoek and Brown strength criterion for determining the bearing capacity of a fractured rock mass (Wyllie, 1999, ref. R-47).

4.5 Groundwater Conditions

The piezometric water levels within the overburden and the bedrock were measured to be at about elevations 179 to 181 and 177 to 178, respectively. Summary of measured piezometric levels within the overburden and the bedrock are presented in Table 4-5. These observations suggest a downward gradient between the overburden and the bedrock. However, based on experience at other locations along the project area, occurrence of localized 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 182. Groundwater was measured in a shallow test pit near Borehole B10-3 at 0.45 below grade.

Table 4-5: Summary of Measured Water Levels

Borehole	Surface Elevation	Piezometer Type	Screen / Sensor Elevation	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	Elevation
BH B10-3	182.2	VWP	172.1	Clayey Silt	June 25, 2011	179.8
					July 23, 2011	179.1
		VWP	162.1	Clayey Silt	June 25, 2011	179.9
					July 23, 2011	179.4
		VWP	149.9	Silty Sand	June 25, 2011	177.1
					July 23, 2011	176.6
BH B10-5	182.3	VWP	170.7	Clayey Silt	June 4, 2011	180.8
					July 23, 2011	180.1
		VWP	162.1	Clayey Silt	June 4, 2011	179.5
					July 23, 2011	178.4
		VWP	150.3	Clayey Silt/ Limestone	June 4, 2011	178.3
					July 23, 2011	177.0
		S-Piez	148 – 149.4	Clayey Silt/ Limestone	July 10, 2011	178.0
					July 23, 2011	177.3
BH B10-7	182.2	VWP	173.8	Clayey Silt	June 25, 2001	181.0
					July 23, 2011	180.7
		VWP	162.4	Clayey Silt	June 25, 2001	178.3
					July 23, 2011	178.1

Legend: VWP Vibrating Wire Piezometer
S-Piez. Screen elevations for Standpipe Piezometer

4.6 Subsurface Gases

The groundwater in the project area, especially within the lower granular deposit and bedrock, is known to contain dissolved hydrogen sulphide (H_2S) and methane (CH_4) gases that are liberated from the water on exposure to atmospheric pressure. The H_2S gas can frequently be detected by odour at concentrations of about 0.5 mg/L and can be corrosive at concentrations of about 2 mg/L to 3 mg/L 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 investigations around Culvert S-2 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. A summary of measured concentration of H_2S gas is provided in Table 4-6. These results indicate that H_2S gas may be present in the culvert S-2 site as well.

Table 4-6: Summary of Measured Concentration of H_2S Gas

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

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-44). 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 CPT advanced around submerged culvert S-2 site are shown in Figure 3.4. Selected design values obtained from the profiles are summarized in Table 5-1. As noted in Section 4.2, the upper silty clay stratum was sub-divided into two strata (namely, Upper Silty Clay-1 and Upper Silty Clay-2) for analyses purposes. 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 (S_u), kPa	ϕ'_{\max} (degrees)	ϕ'_{cs} (degrees)	Preconsolidation Pressure (σ'_p), kPa
Clay Crust	181.5 – 177	75	30	N/A	550
Transition	177 – 175	75 to 55	30	N/A	550 to 325
Upper Silty Clay-1	175 – 166	55 to 43	30	26	325 to 230
Upper Silty Clay-2	166 – 163	43 to 50	30	26	230 to 260
Lower Clayey Silt	163 - 151	50 to 65	30	26	260 to 380

Note: The ranges of S_u and σ'_p values indicate variation top to bottom with depth.

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	Anisotropy Ratio, k_h/k_v	Initial Void Ratio, e_0
	cm/sec		
Clay Crust	6.8×10^{-7}	1	0.55
Transition	3.9×10^{-7}	2	0.55
Upper Silty Clay - 1	1.1×10^{-7}		0.65
Upper Silty Clay - 2	1.1×10^{-7}		0.50
Lower Clayey Silt	2.0×10^{-7}	1	0.55

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, 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 stress deformation assessments referenced in this report assume that the general excavation for Highway 401 is completed first and then the concrete pipes are installed by trenching to about 0.3 m below 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.
- If used, 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 SSP.
- 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 estimates of temporary cut slopes were performed using the material properties summarized in Table 5.1. These analyses indicated that for conventional open excavation from existing grade the top width of the temporary excavation trench would be in excess of 60 m (assuming overall temporary excavation slope of 2H: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 during excavations 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 171.6), and a clayey silt layer thickness of 17 m below the deepest excavation. The estimated factor of safety against hydrostatic uplift was 1.47 based on the weight of the clay cap only.

For shored excavations, the factors of safety against basal stability were estimated to be 1.1 and 2.0 for excavations from existing ground surface and the proposed Highway 401 grade, respectively. Hence, the shoring system should be developed on the basis of an engineered design. In the case of sheet pile shoring, 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-2 is shown on Drawing 285380-03-060-SEG1-S4201 and Figure 5.1. The Submerged Culvert S-2 will consist of three 3.0 m diameter pipes which will be laid down in parallel array near Sta. 11+175L to Sta. 11+200L. Culvert S-2 will be installed across the large excavation for Highway 401 and will be routed under Highway 3 (Bridge B-10), EBR7, EBR8 and WBR6. Construction of the submerged culvert will involve variable depths of excavation below existing ground surface under these structures as indicated below:

- Highway 401 and EBR8: 10.5 m (6.0 m below highway grade)
- Highway 3 (Bridge B-10): 10.5 m
- EBR7: 10.5 m (14.5 m below EBR7 grade)
- WBR6: 10.5 m

Relevant configuration information for Culvert S-2 and the ground/soil conditions are summarized in Table 5.3.

Table 5-3: Summary of Control Elevations of Submerged Culvert S-2

Approximate Elevation	Subsurface Soil and Groundwater Conditions	Proposed Highway and Submerged Culvert Elements
±186		EBR7 Pavement Surface
±182		WBR6 Pavement Surface
182 to 182.5	Existing ground surface	
180	Average piezometric level in shallow soils	
177	Average piezometric level in bedrock	
179.5 to 175.8		Base of Inlet Structure
178.8 to 175.1		Base of Outlet Structure
181.5 to 177	Clay Crust	
177.8 to 178.0		Highway 401 Pavement Surface
±177.5		EBR8 Pavement Surface
177 to 175	Transitional Clay	
175 to 166	Upper Silty Clay-1	
171.2 to 172.1		Approximate Pipe invert elevation
170.7 to 171.7		Approximate base of excavation
166 to 163	Upper Silty Clay-2	
163 to 151	Lower Silty Clay	
151 to 149	Compact to dense Sand and Gravel	
150 to 149	Bedrock surface	

The large excavation for Highway 401, embankment for EBR7, temporary excavations and backfilling during installation of culvert pipes 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 estimate the ground movements at the base of the submerged culvert below Highway 401 and EBR8, and EBR7 for the short-term and long-term loading conditions.

As shown in Figure 5.1, the southern half of the pipe culvert involves more complex configuration than the northern half of the pipe culvert. Therefore, for the purposes of ground deformation estimates along the culvert alignment, the southern half of the pipe culvert was chosen for the SDA. Bridge B-10 piers are supported on end bearing piles which will be driven to bedrock. Therefore, no loadings from the bridge structure onto the culvert are anticipated. As such, the piles supporting the Bridge B-10 piers (PIER 3S and 3N) are not incorporated in the SDA model. Separate SDA models were examined at sections across Culvert S-2 (along HWY 401 centreline and EBR7 centreline) to evaluate the effect of culvert installation along Hwy 401 corridor.

The configuration of the Sigma/W model along the Culvert S-2 is presented in Figure E.1. The SDA model is simulated based on the following construction stages:

- a) Generation of the initial (in-situ) stress condition for level ground;
- b) Excavation for Highway 401 and EBR8;
- c) Installation of culvert pipes and Outlet structure;
- d) Construction of embankment for EBR7; and
- e) 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.

Stage (a): 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 1 m below Highway 401 grade (Figure E-1). In this regard relief trenches for the pipe bedding and cover should be implemented.

The ground water collected within the granular bedding will be hydraulically connected to the 100 mm Slope Subdrain and 100 mm Wall Subdrain through relief trenches. Estimated transmissivity of the relief trenches is about one order of magnitude higher than the estimated rate of 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 design 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 clay 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 soil layers.

Construction Stage (b): Excavation for Highway 401 and EBR8 was assumed to occur over a period of 3 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.

Construction Stages (c) and (d): Stage (c), installation of culvert pipes and Outlet structure, was assumed to occur over a period of “one week”. The EBR7 embankment (Stage (d)) was then added allowing a period of 1 week.

Stage (e): 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 excavation for Highway 401 and EBR8 (Stage (b)) and at the end of construction (Stage (d)) are 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.

Additional SDA models were examined at two sections across Culvert S-2 (namely, along Highway 401 centreline and EBR7 centerline) to evaluate deformations caused by temporary excavation, culvert installation and backfilling. It should be noted that these SDA were performed assuming conventional open excavations. 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-2, along EBR7 centreline) is presented in Figure E-8. The SDA was performed to simulate the following loading stages:

- Generation of the initial (in-situ) stress condition for level ground;
- Temporary excavation to elevation 171.0m (~0.3m below Pipe base); and
- Installation of Pipes and backfilling to ground surface.

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

SDA model at a section across Culvert S-2 along Highway 401 centreline was also examined to estimate deformations caused by temporary excavation, culvert installation and backfilling. The configuration of

the Sigma/W model is shown in Figure E-11. Calculated undrained (immediate) deformations are presented in Figures E-12 and E-13.

5.4.3 SLS Performance

Estimated magnitude of vertical deformations of the ground surface and culvert determined from the SDA are summarized in Table 5-4 at representative successive stages during and after construction. The deformations reported in the Table 5-4 incorporate deformations caused by main excavations along the depressed Highway 401 and deformation caused by localized trenching and backfilling for the culvert.

Table 5-4: Summary of Interpreted Calculated Deformations

Parameter	End of Pipe Trench Excavation (mm)	End of Construction (Undrained) (mm)	Post-Construction (Long-term) ^(f) (mm)	Remarks
Maximum Heave/Settlements at Pipe Invert				
Below Highway 401	+55	-40	+20	Figures E-2, E-3, E-4, E-9, E-10, E-12, E-13 and E-14
Below EBR8	+55	-40	+15	
Below EBR7 Centreline	+50	-75	-20	
Near Inlet/Outlet Structure	+25	-25	-10	
Maximum Heave Settlement along Highway 401 Pavement^(a)				
0m	N/A	-45 ^(c)	+25 ^(d)	Figure E-12, E-13 and E-14
5m	N/A	-45 ^(c)	+25 ^(d)	
15m	+5	-10	+35 ^(d)	
Beyond 15m	-5	-5	+35 ^(d)	
Maximum Heave Settlement along EBR7 Pavement^(a)	N/A	-35 ^(c)	-10	Figure E-3 and E-4

- (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
- (f) Post construction ground movements are anticipated to stabilize within approximately 10 to 15 years following completion of construction (Figure E-7)

As mentioned earlier, SDA models were developed to examine the effect of trenching and installation of the drain pipes across EBR7 (Figure E-8) and Highway 401 (Figure E-11) centerlines. These models are represented by the cross sections of assumed open cut from existing ground surface and the level of major permanent cut for Highway 401 respectively. These analyses results indicate temporary heave of up to 55 mm (Figure E-9) and 35 mm (Figure E-12) at the base of trench (during trenching) below EBR7 and HWY401 centrelines, respectively. The SDA analyses indicate that pipe invert below EBR7 centreline will settle by about 60 mm (Figure E-9 and E-10) due to pipe installation and backfilling to existing ground surface elevation at EBR7. It is also estimated that pipe invert below Highway 401 will settle by about 45 mm (Figure E-12 and E-13) due to pipe installation and backfilling to Highway 401 elevation.

No tangible ground deformations were obtained (Figures E-10 and E-13) away from the open trench after the completion drain pipes installation.

The estimated ground movements along Highway 401 centreline at pavement elevation are shown on Figure E-14 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 EBR 8 which is at about the same elevation as Highway 401. It is anticipated that EBR 7 will be constructed after the installation of culvert pipes. Therefore, the post-construction settlement along the EBR 7 centreline will be about 15 mm uniform settlement (Figures E-4) at pavement elevation.

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, which for properly compacted backfill should be small. The compaction specifications should be rigorously adhered to during construction in order to minimize these risks.

Outlet and Inlet Structures:

Net soil stress increases at the base of Inlet and Outlet structures are expected to be nominal. However, the phreatic surface around Inlet and Outlet structures is expected to be lowered from elevation 180 (existing condition) to about elevation 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 heave/settlement at Inlet and Outlet structures. Based on these calculations and SDA discussed in Section 5.4.2, it is anticipated that the Inlet and Outlet structures would experience long-term settlement (post-construction) of less than 25 mm.

5.4.4 ULS Bearing Resistance

A net 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 171.5 m. For the concrete pipe installed along the sloped excavation (to connect to Inlet or Outlet structures), a net factored geotechnical resistance of 120 kPa (elevation 171.5) to 200 kPa (elevation 177) was determined. A net factored geotechnical resistance of 200 kPa was estimated for the Inlet and Outlet structures.

Retaining walls WCRW1, WCRW2, WCRW3 and WCRW4 extend from the Inlet structure on either sides of the Cahill drain and are structurally connected to the Inlet structure. The height of the retaining walls WCRW1, WCRW2, WCRW3 and WCRW4 are 2.7 m, 3.6 m, 3.7 m and 4.5 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 impacted by temporary excavations and construction. The performance of the temporary excavations and shoring should be continuously monitored.

Permanent Retaining Structures

Long term earth pressures of compacted backfill against buried structures may be calculated on the basis of the parameters listed in Table 5-5.

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 to 35	29 to 32	22 to 30
Coefficients of Static Lateral Earth Pressure:			
• 'Active' or Unrestrained, $K_a^{(*)}$	0.27 to 0.30	0.31 to 0.35	0.33 to 0.45
• 'At Rest' or Restrained, $K_o^{(*)}$	0.43 to 0.46	0.47 to 0.52	0.50 to 0.62
• 'Passive', $K_p^{(*)}$	3.3 to 3.7	2.9 to 3.2	2.2 to 3.0

^(*)Values are given for level backfill and ground surface behind the wall 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:

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

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

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

Where: ϕ = Friction angle of backfill material, and

β = Slope of the backfill surface.

The long term earth pressures against the buried structures should consider the earth pressure coefficients listed above 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 on Appendix I (Figure I.1) and Appendix E 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.

Seepage from runoff infiltrations or perched water within the fill and upper granular 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 that the duration of the slope exposure is limited to 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 in the required excavation segments of WEP 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 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. 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 200 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 400 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 OPSS 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 m² 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 Construction 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 B10-2, B10-3, B10-4, B10-5, B10-6 and B10-7 located nearby the culvert S-2. 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 B10-2 Sample 10	173.2	8.19	44	6540	< 0.2	< 20
Borehole B10-3 Sample 25	150.2	7.95	130	3470	< 0.2	267
Borehole B10-4 Sample 24	151.7	7.90	123	1910	< 0.2	567
Borehole B10-5 Sample 11	171.6	7.89	188	3420	< 0.2	186
Borehole B10-6 Sample 23	152.8	7.84	218	1680	< 0.2	657
Borehole B10-7 Sample 26	151.7	7.94	102	3220	< 0.2	172

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 monitoring of the response (ground movements) 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-2 and Bridge B-10 Piers

The Culvert S-2 will traverse under Bridge B-10 between bridge Pier 3N and Pier 3S located west of bridge's east abutment. These piers are supported on end bearing piles which will be driven to bedrock. Therefore, no loads from the bridge structure onto the culvert are anticipated. The geotechnical aspects of Bridge B-10 are being addressed under a separate cover.

It is expected that the bridge's piles will be driven before the culvert installation so than no concerns about the dynamic effects should arise with respect to the stability of temporary works, or with the performance of the backfill to the culvert and culvert structure itself.

Temporary excavations for culvert installation should not be carried out below the line at 45 degree from the closest edge of the pile cap. If this is not possible, the structural pile design should be reviewed with respect to the potential pile overstressing due to temporary culvert excavations.

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.

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Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
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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 Cahill Submerged Culvert S-2 was developed by Mr. Ganan Nadarajah, P.Eng. under the direction of Dr. Dan Dimitriu, P.Eng. (Project Technical Lead). Dr. Narendra S. Verma, P.Eng. (Project Technical Director) provided the senior review of the report. Mr. Matt Oldewening, P.Eng. managed the geotechnical investigation and Mr. Brian Lapos, P.Eng., is the Project Manager.

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Yours truly,

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Drawings

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
Doc No.: 285380-04-119-0009 (Geocres No. 40J3-18)

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

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

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UNLESS OTHERWISE SHOWN

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

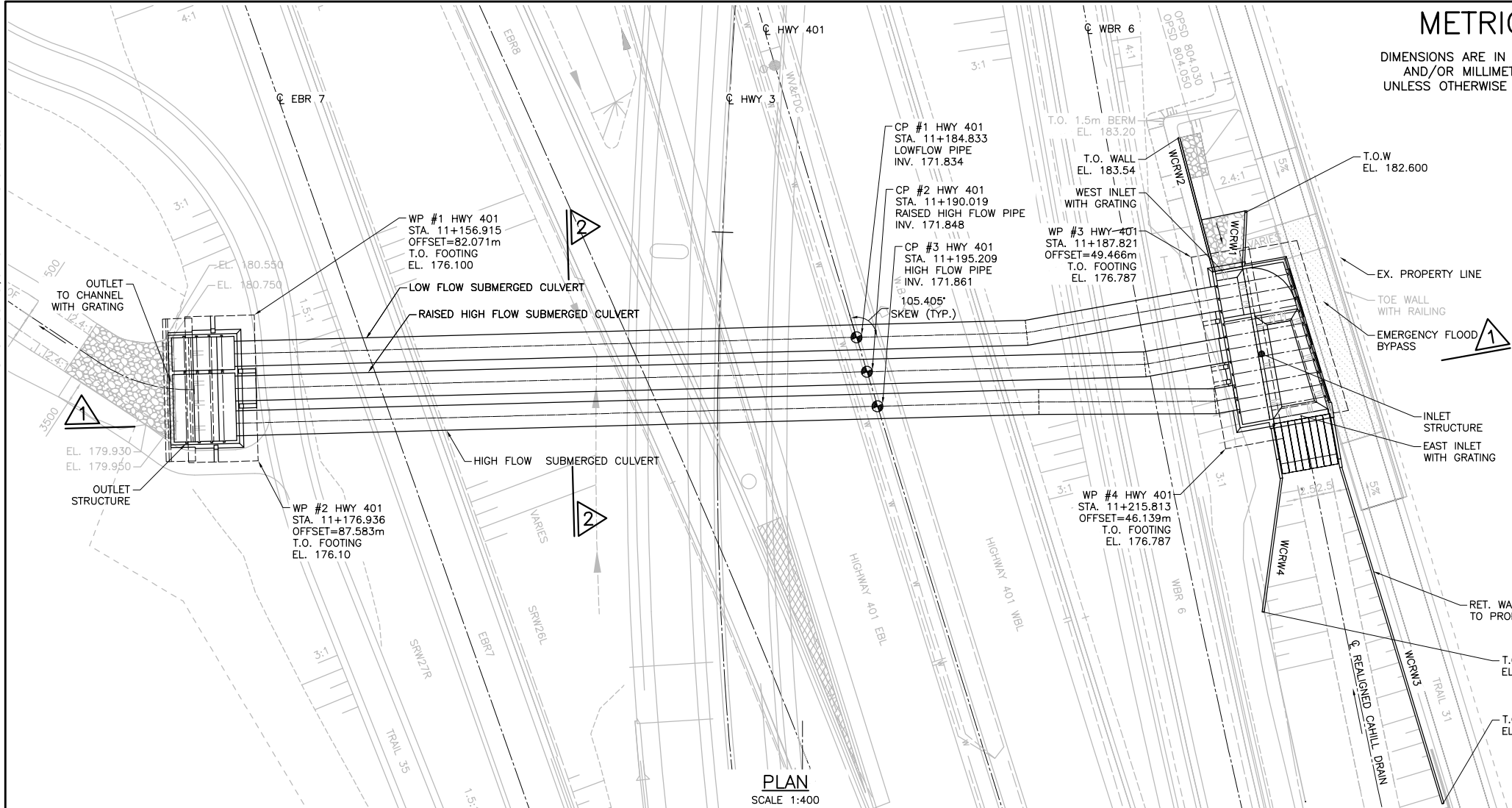
NEW CONSTRUCTION
HWY 401
CAHILL SUBMERGED CULVERTS S-2
GENERAL ARRANGEMENT

SHEET
S4201

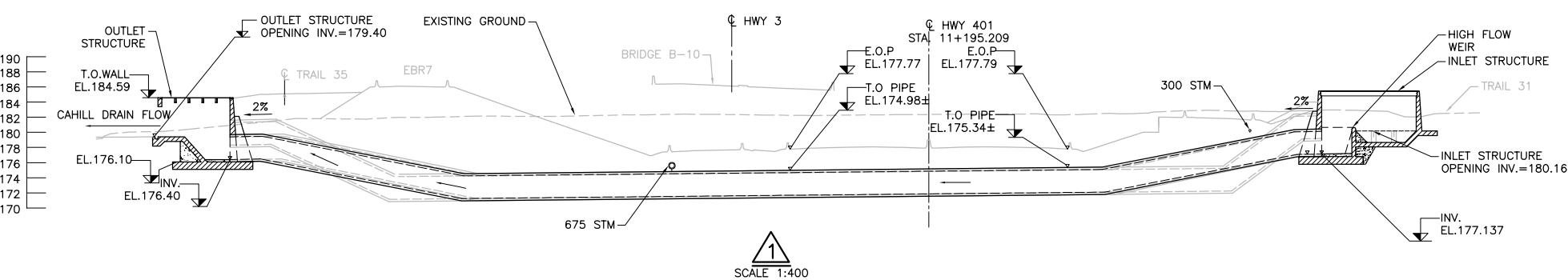
Phase 1
IFC

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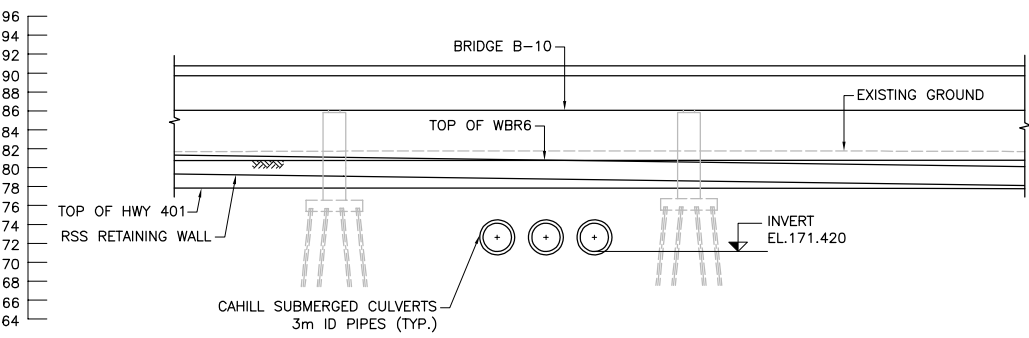
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 2. MAX LIVE LOAD ON ROOF GRATINGS 4kPa.



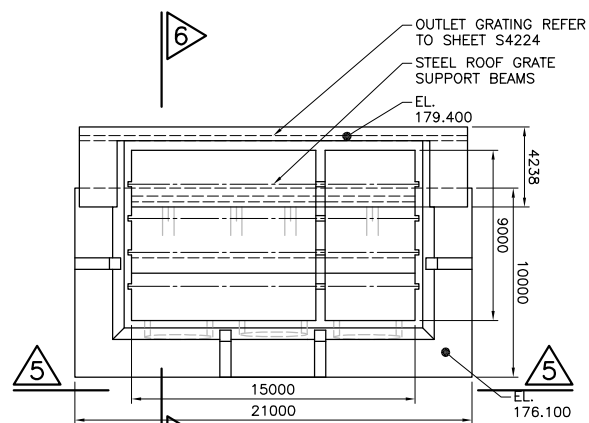
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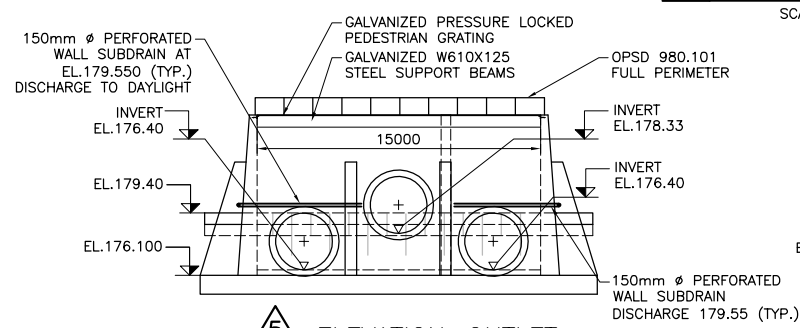
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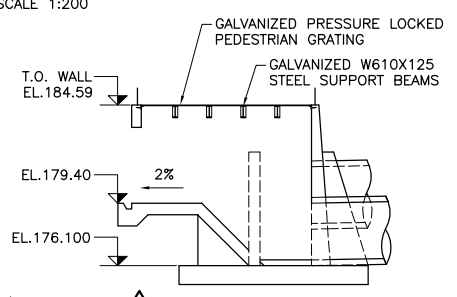
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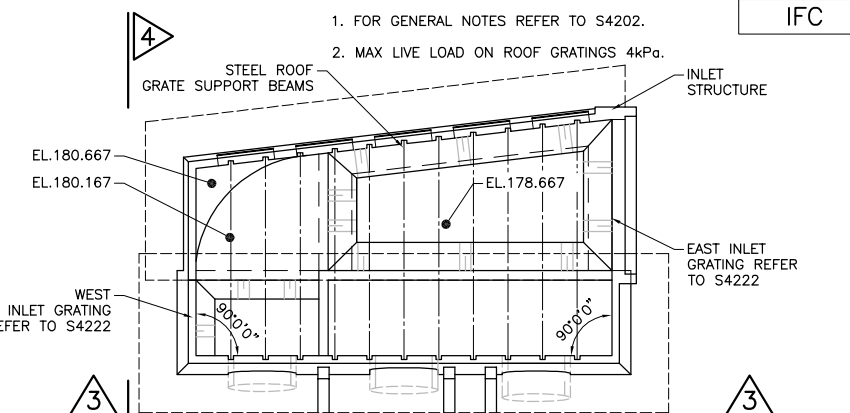
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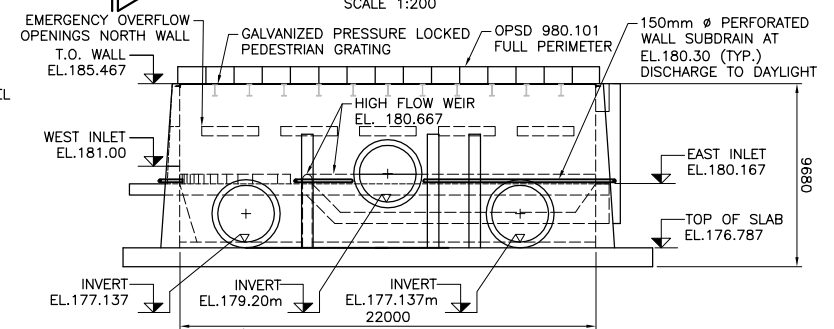
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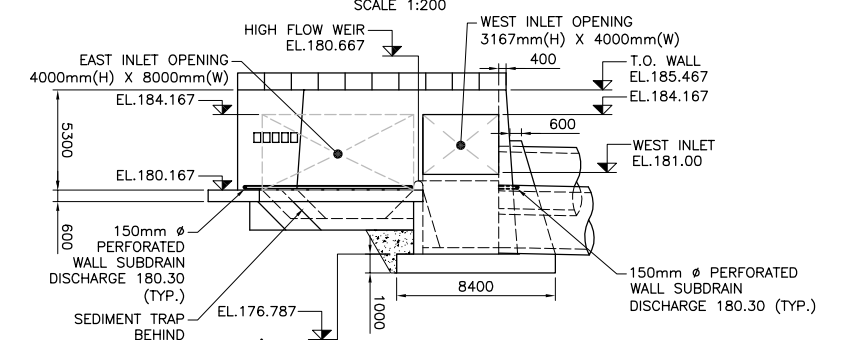
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SCALE 1:200



PLAN-INLET
SCALE 1:200



3 ELEVATION-INLET
SCALE 1:200



4 ELEVATION-INLET
SCALE 1:200

NOT FOR CONSTRUCTION

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DESIGN	CRD	CHK	JF	CODE CAN/CSA S6-06 LOAD CL 625-ONT
DRAWN	BW	CHK	BR	SITE 6-635 DATE 21-AUG-11

METRIC



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

REVISIONS			
10-AUG-12	0	GN	ISSUED FOR CONSTRUCTION
DATE	REV.	BY	DESCRIPTION
DESIGN	WH	APR DD	DATE 15-JUL-11

LOCATION PLAN & INTERPRETED
STRATIGRAPHIC PROFILE
STA 10+900L TO STA 11+500L

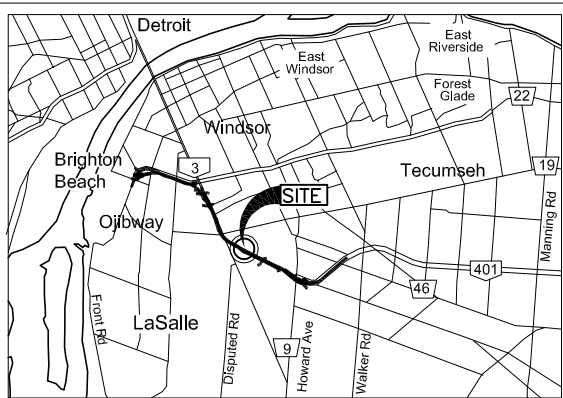


SHEET

G4202

Phase 1

IFC



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

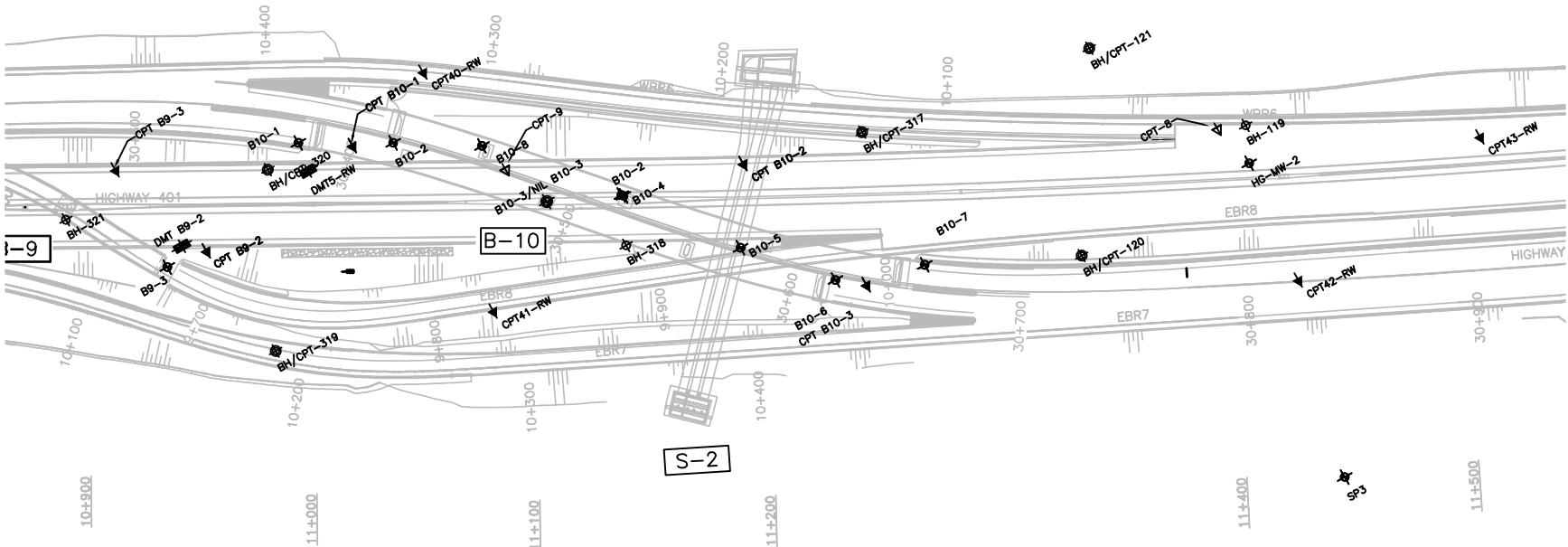
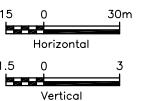
LEGEND

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

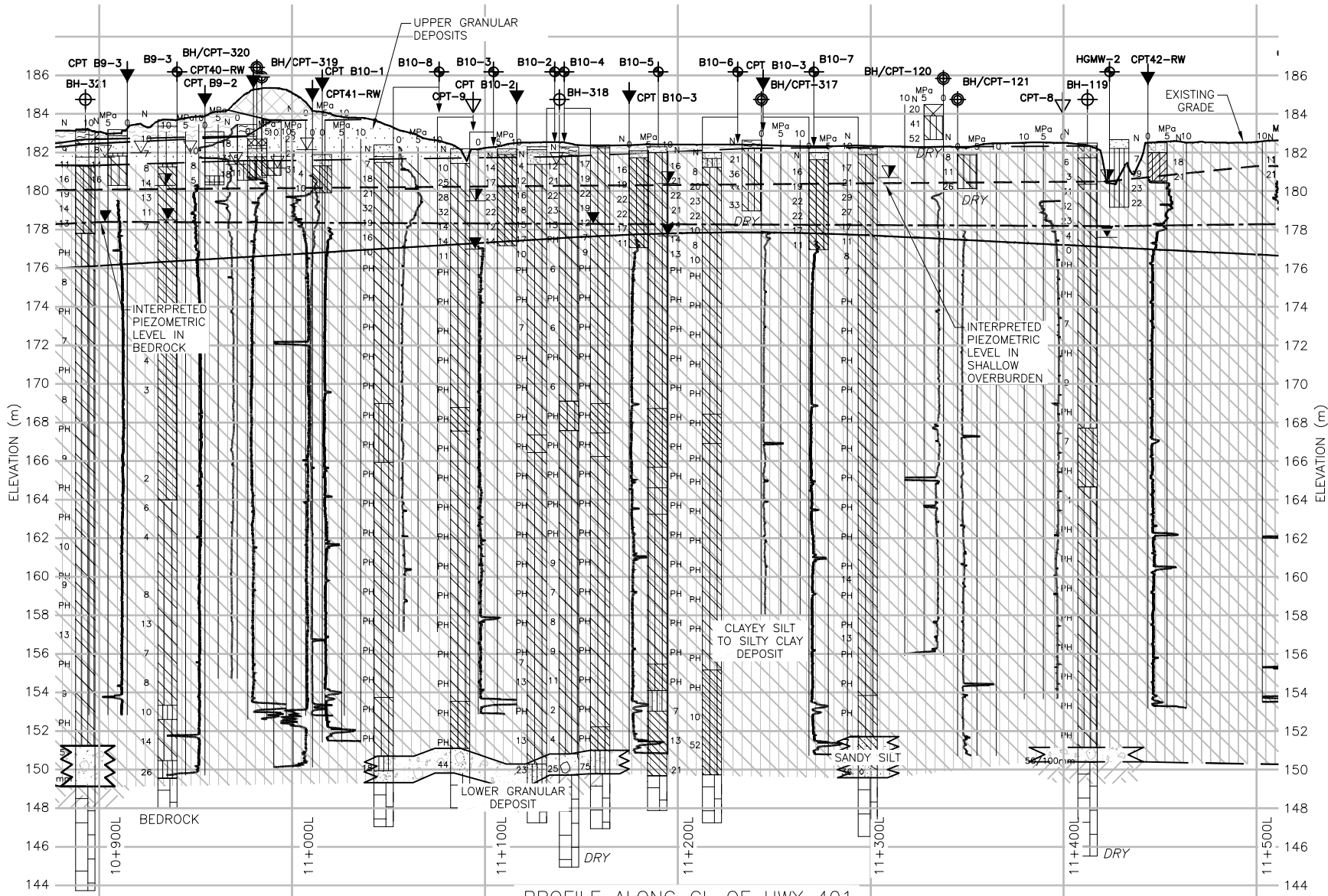
NOTES

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

SCALES

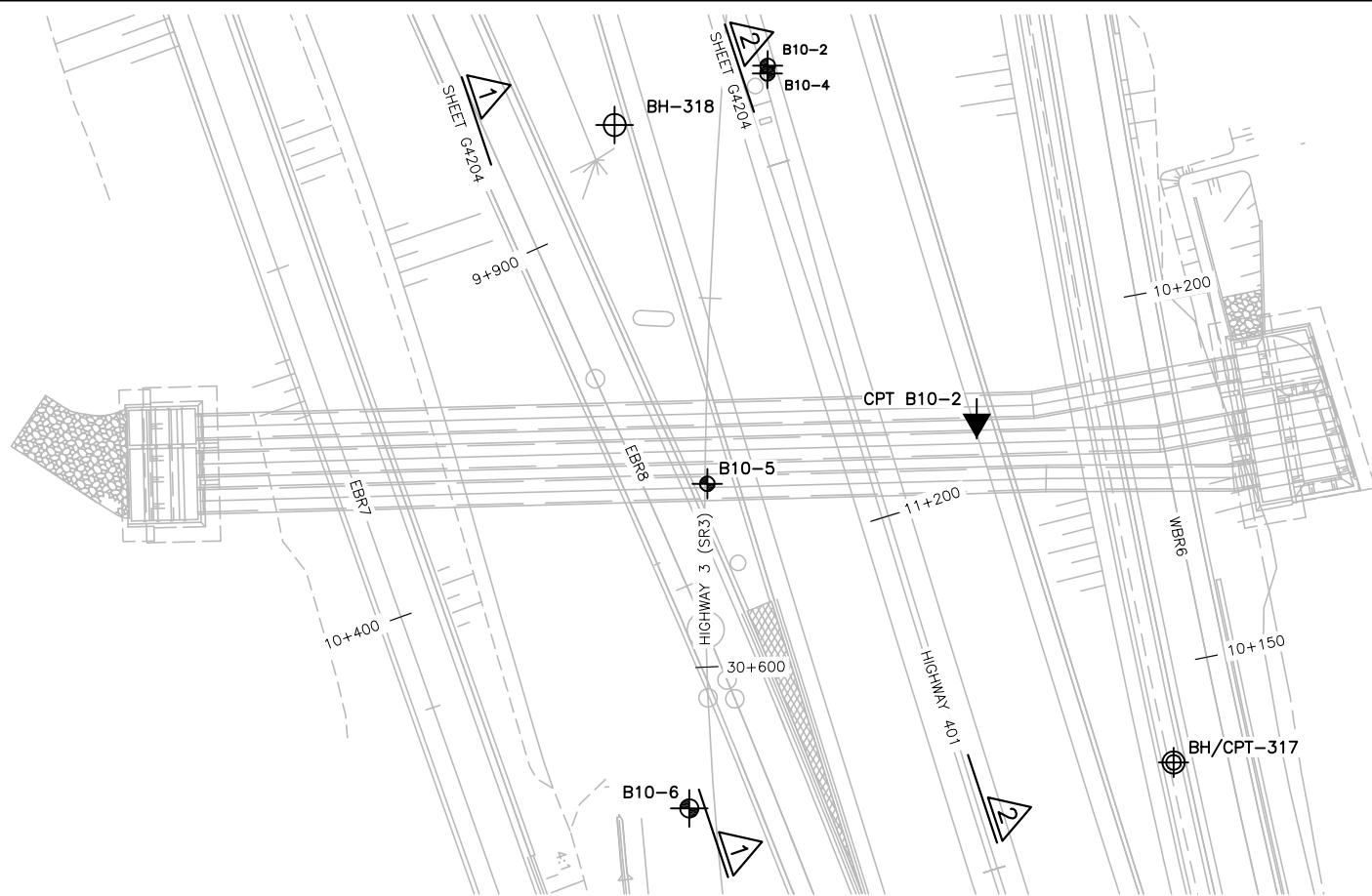


PLAN
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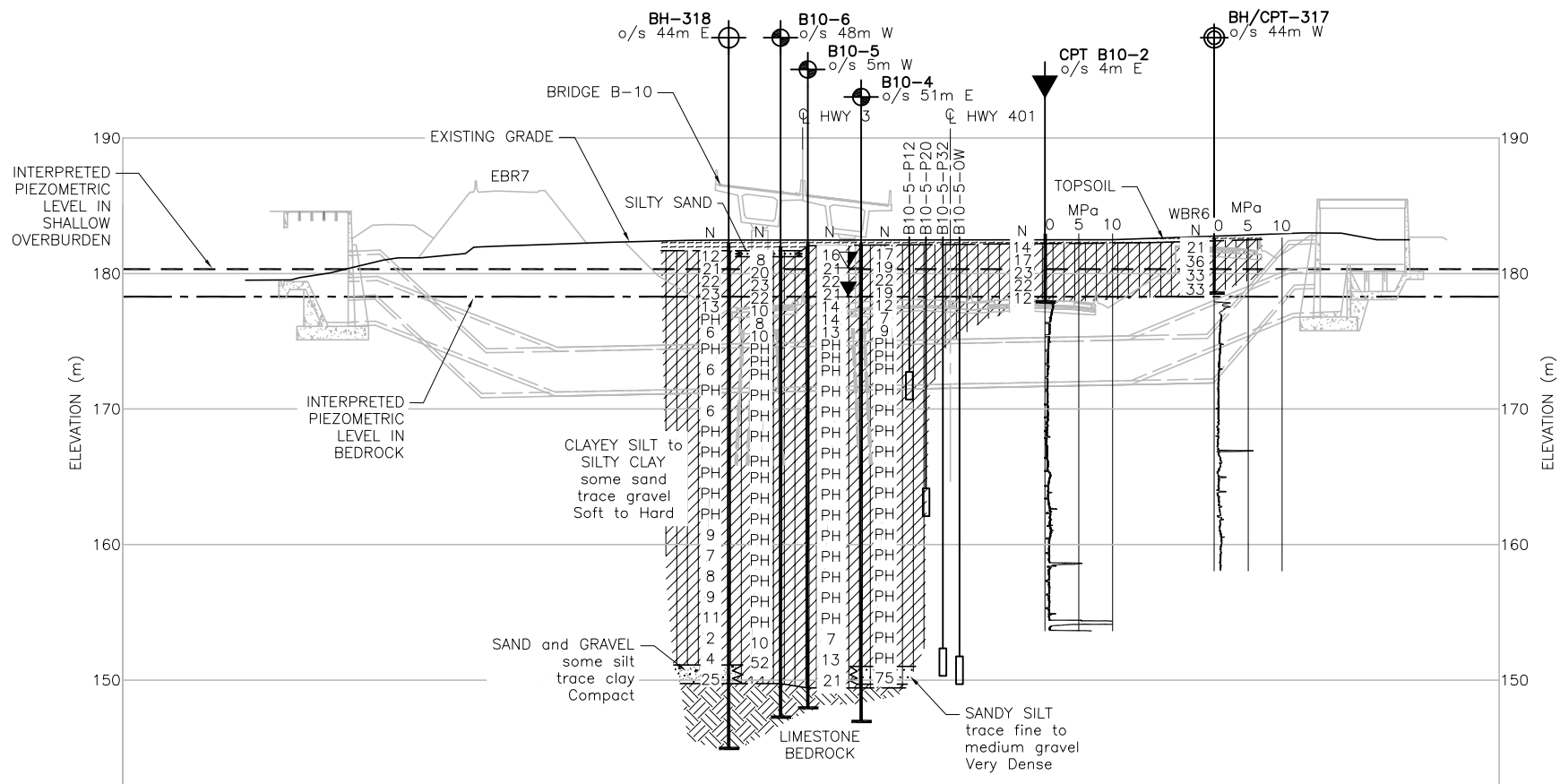
PROFILE ALONG CL OF HWY 401

HORIZONTAL SCALE 1:1500
VERTICAL SCALE 1:150



PLAN

HORIZONTAL SCALE 1:500



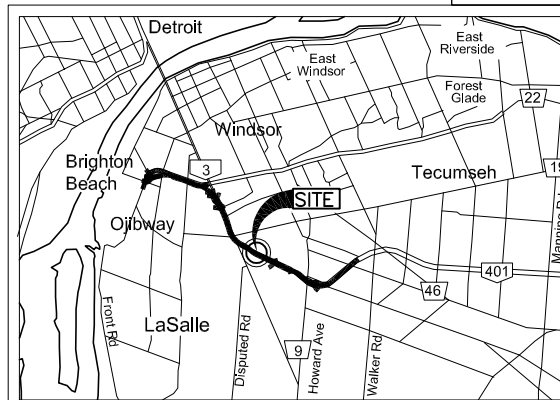
PROFILE ALONG SUBMERGED CULVERT

HORIZONTAL SCALE 1:500
VERTICAL SCALE 1:250

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWNWindsor-Essex
Parkway Project
RFP No. 09-54-1007NEW CONSTRUCTION
HWY 401
CAHILL SUBMERGED CULVERTS S-2
BOREHOLE LOCATIONS & SOIL STRATASHEET
G4203Phase 1
IFC

No.	ELEVATION	CO-ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC BOREHOLES			
B10-2	182.3	4679069.6	332866.8
B10-4	182.4	4679068.8	332867.5
B10-5	182.3	4679023.7	332901.0
B10-6	182.0	4678990.9	332930.4
CPT B10-2	182.2	4679056.3	332920.1
PREVIOUS BOREHOLES			
BH/CPT-317	182.6	4679041.7	332972.4
BH-318	182.3	4679049.3	332857.8



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
- BLOWS/0.3m UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
- MHS - MAGNETIC HEAVE/SETTLEMENT GAUGE (SM)
- P - VIBRATING WIRE PIEZOMETER (VWP)
- OW - OBSERVATION WELL
- DRY BOREHOLE DRY DURING DRILLING
- WATER LEVEL DURING DRILLING
- WATER LEVEL (SHALLOW PIEZO)
- WATER LEVEL (DEEP PIEZO)
- 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 /BEDROCK
- DOLOSTONE

NOTES

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- ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

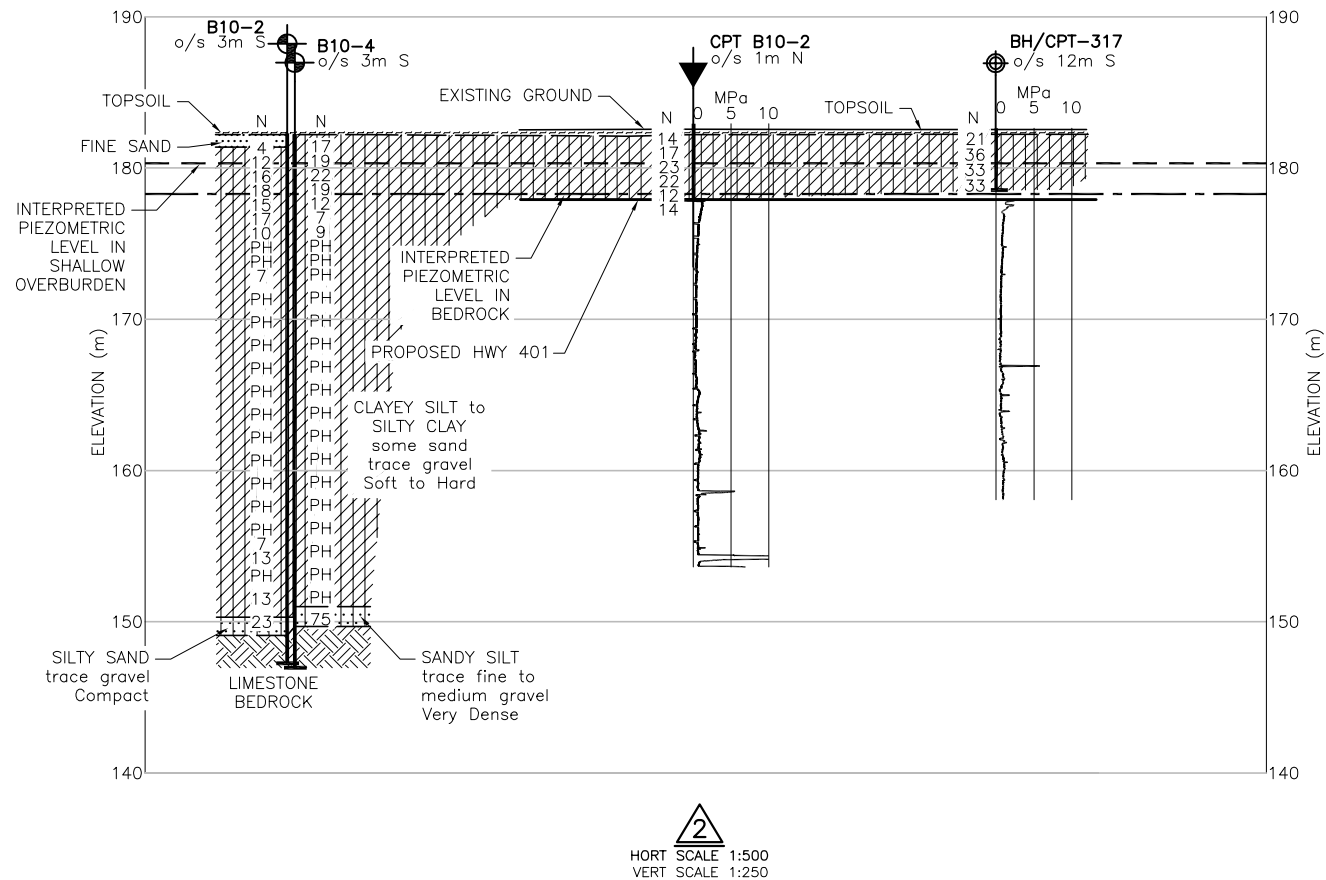
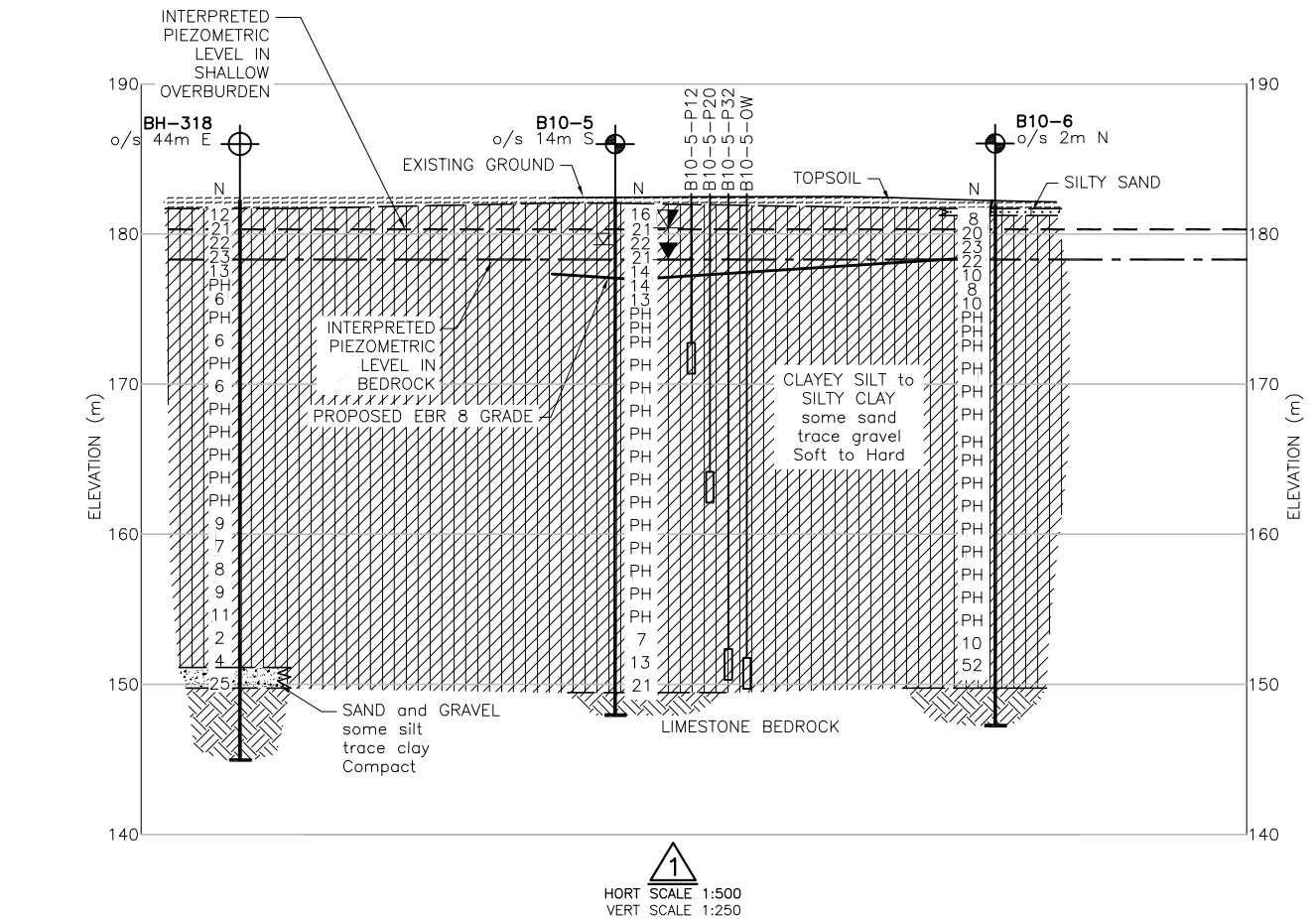
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DOC: 285380-04-090-WP1-4203

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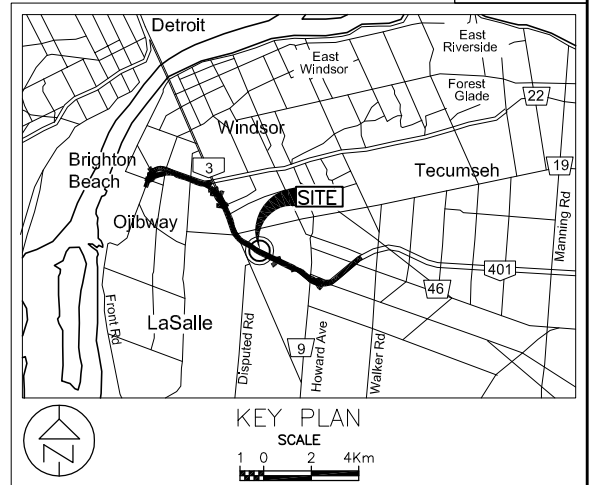
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



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

NEW CONSTRUCTION
HWY 401
CAHILL SUBMERGED CULVERTS S-2
SOIL STRATIGRAPHY

SHEET
G4204
Phase 1
IFC



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

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

NOTES

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- ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

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DRAWN	MM	CHK	GN	SITE	6-635	DATE	02-AUG-11	

DOC: 285380-04-091-WIP1-4204

Figures

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
Doc No.: 285380-04-119-0009 (Geocres No. 40J3-18)

Date: August/2012
Rev: 0
Page No.: Figures

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

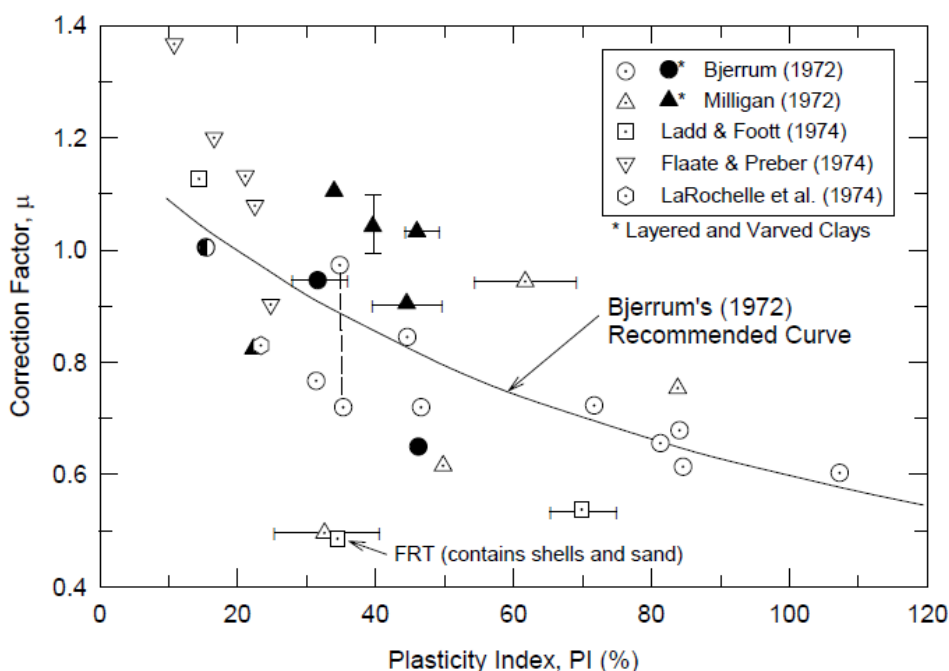


Figure 3-2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays
(Ladd & DeGroot, 2004)

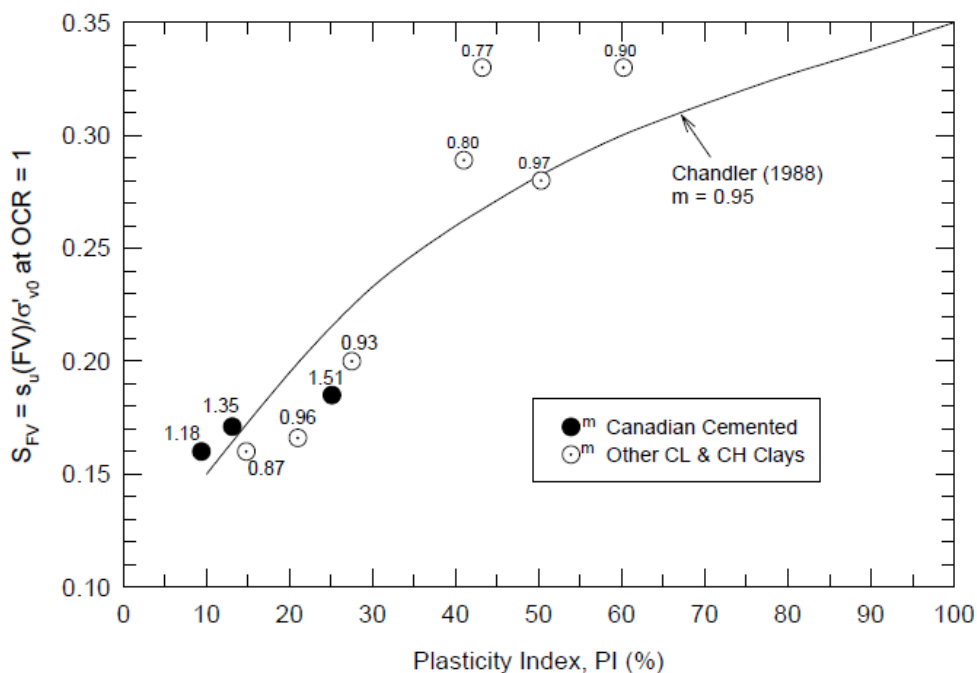


Figure 3-3: Soil Property Profiles for Sta. 10+900L to 11+500L

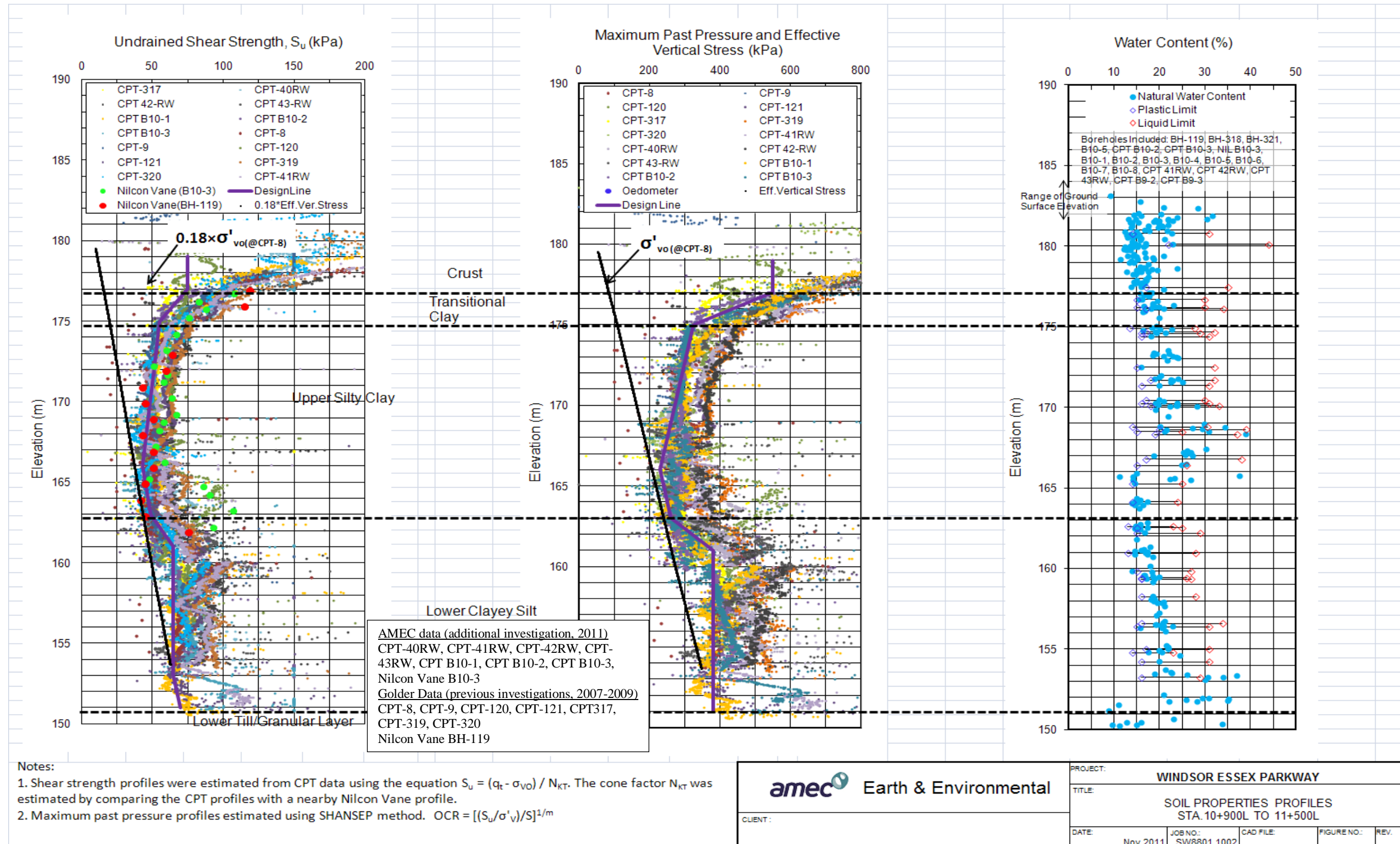
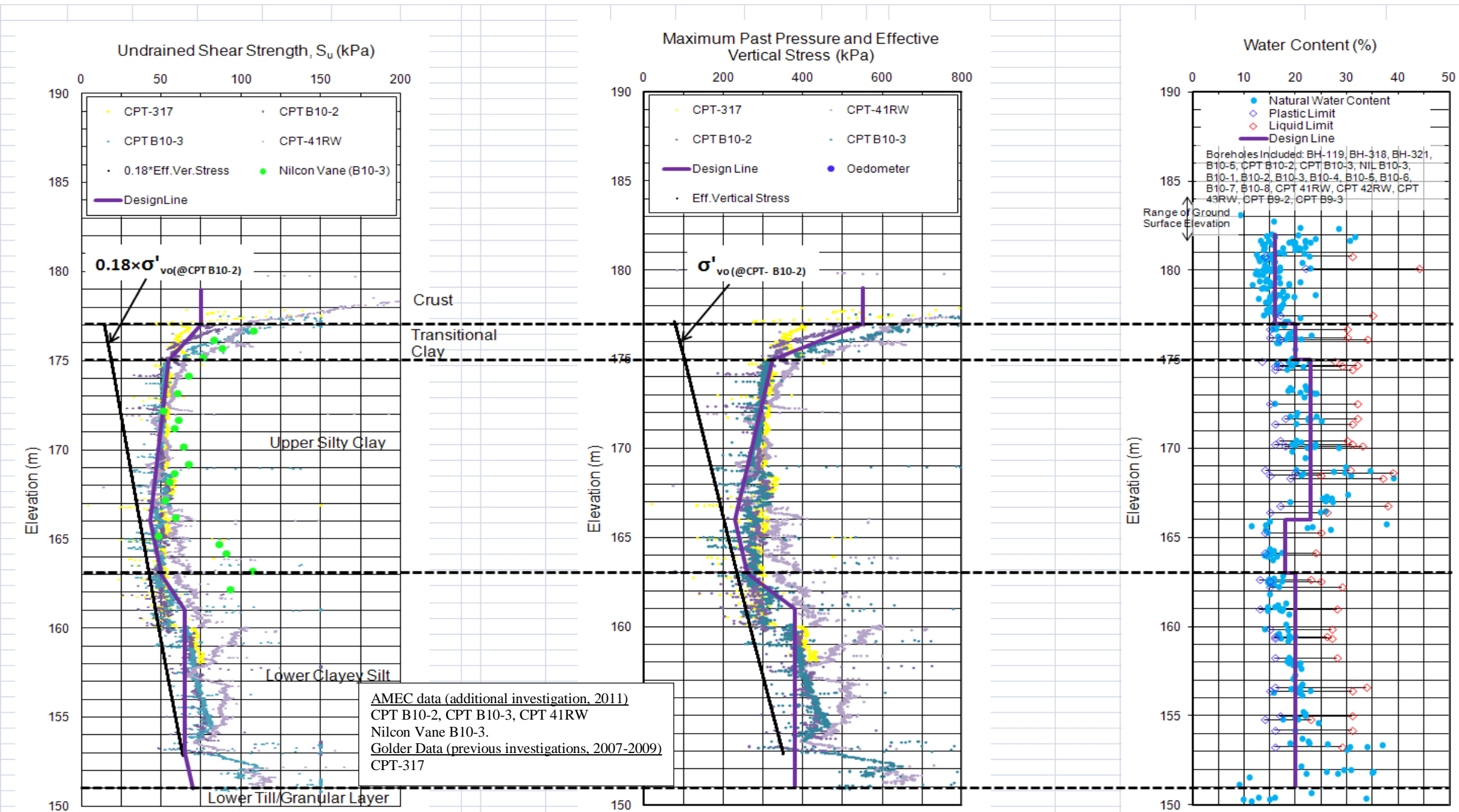


Figure 3-4: Soil Property Profiles for Cahill Submerged Culvert S-2



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

amec Earth & Environmental

CLIENT:

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: SOIL PROPERTIES PROFILES STA. 11+100L TO 11+300L				
DATE: Nov 2011	JOB NO.: SW8801.1002	CAD FILE:	FIGURE NO.:	REV.

Figure 4-1: Compressibility Parameters at WEP

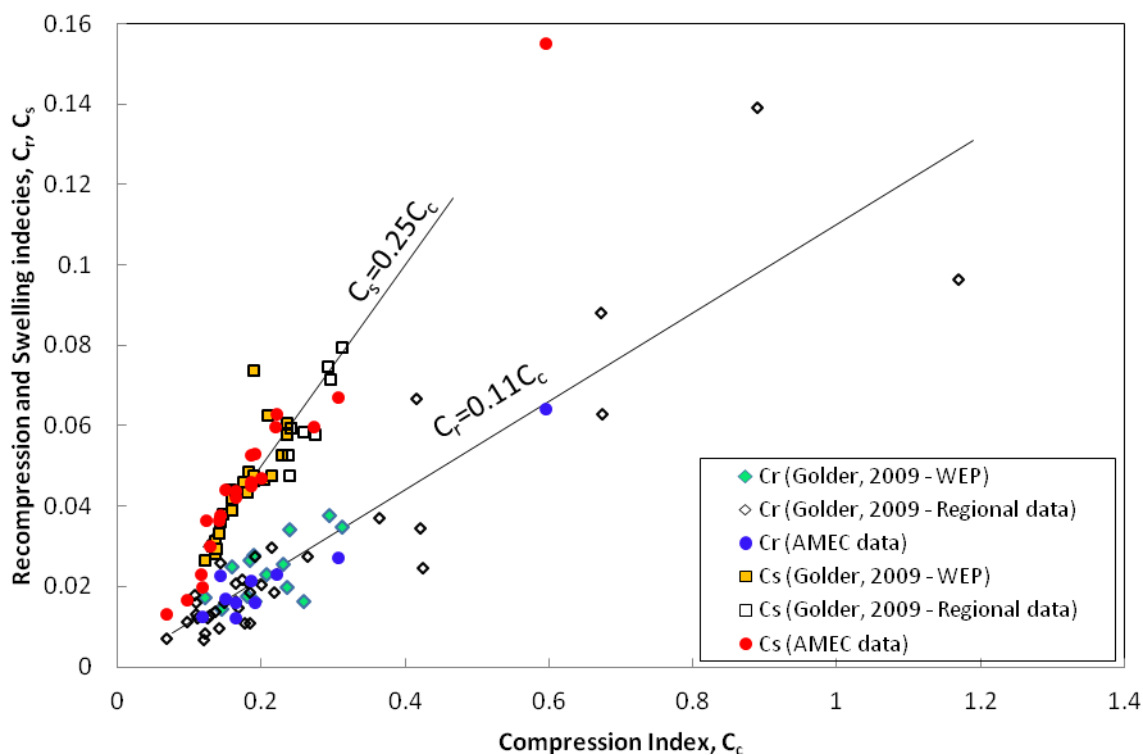
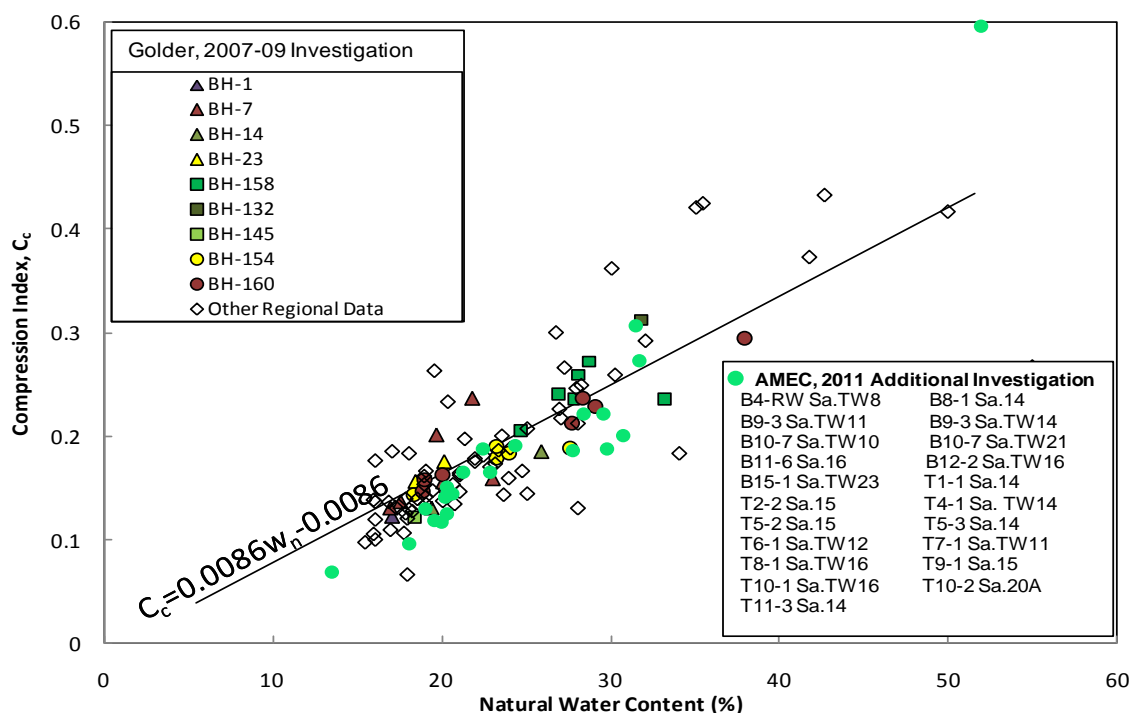


Figure 4-2: C_c versus C_α Relationship at WEP

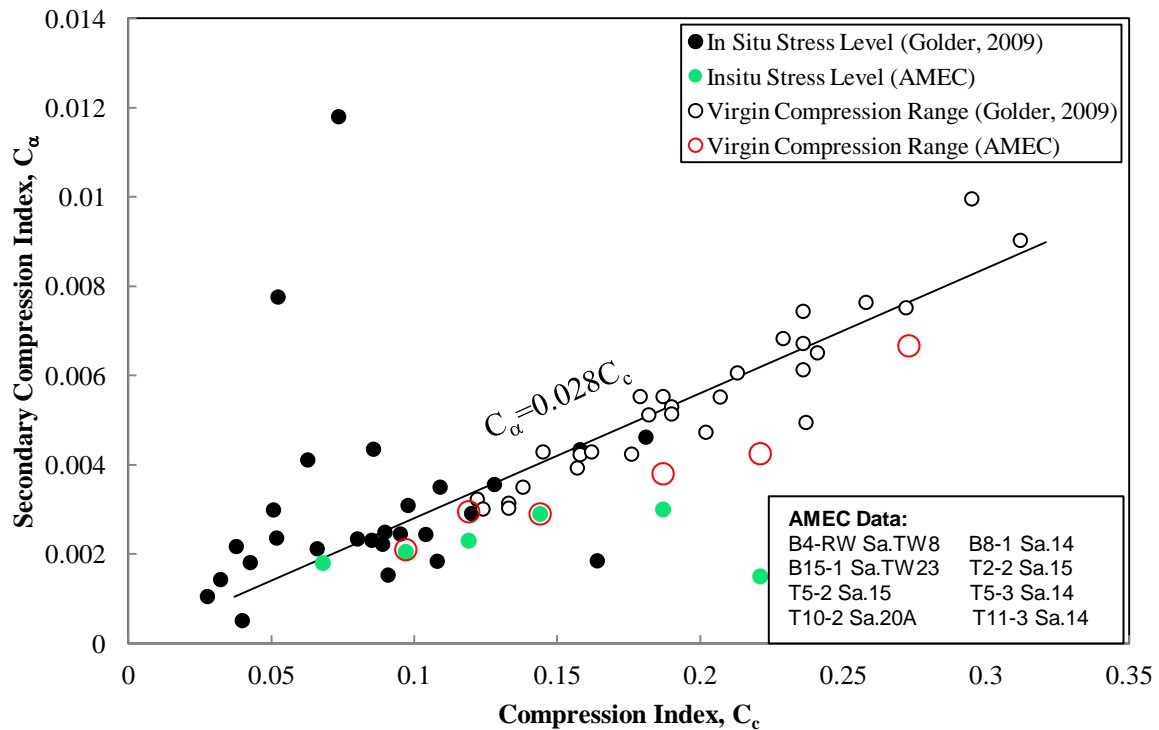


Figure 4-3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP

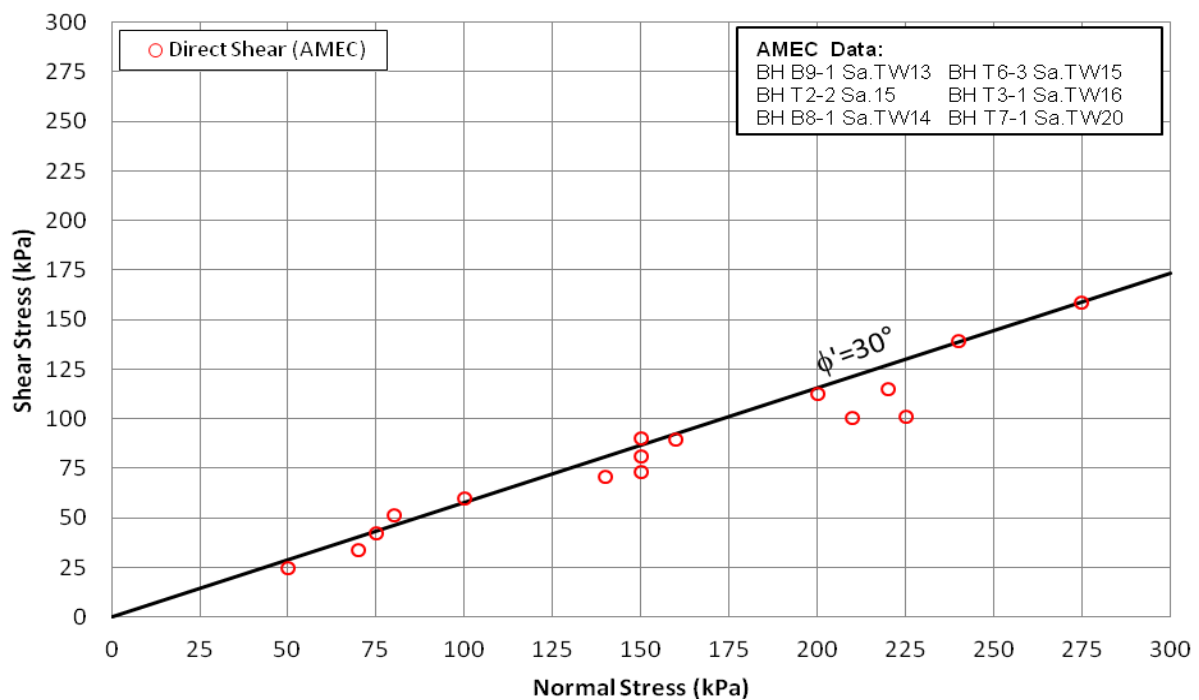
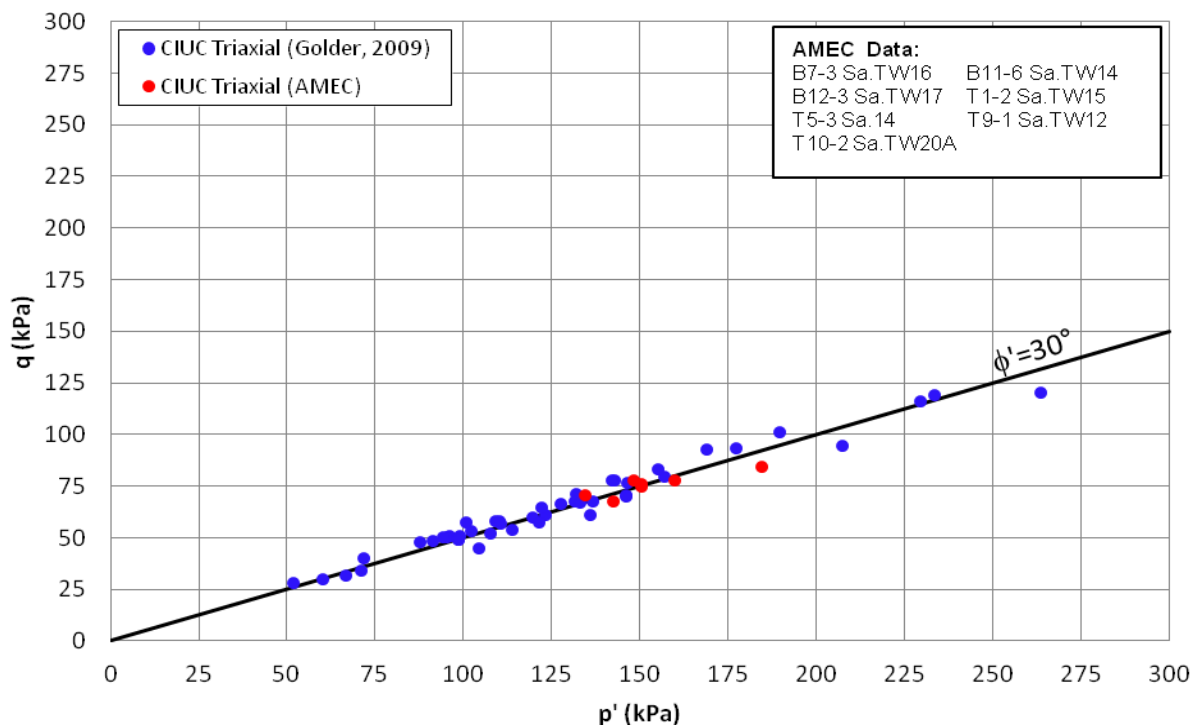


Figure 4-4: Relationship between $\sin \phi'$ and Plasticity Index for Normally Consolidated Soils

(Kenney, 1959)

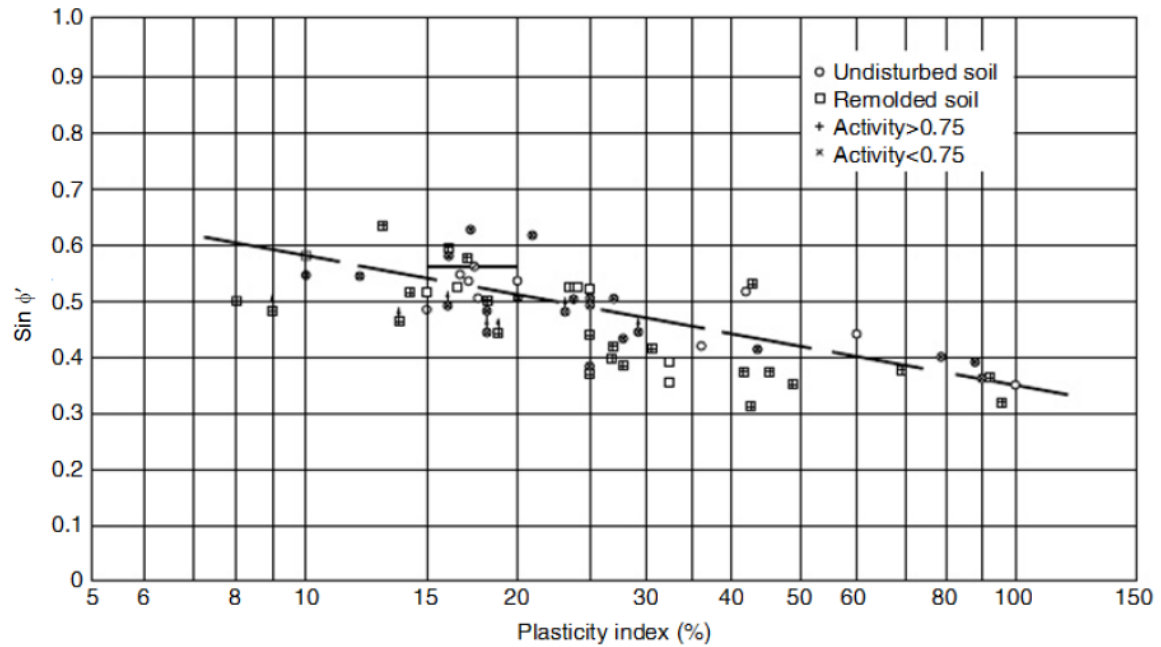
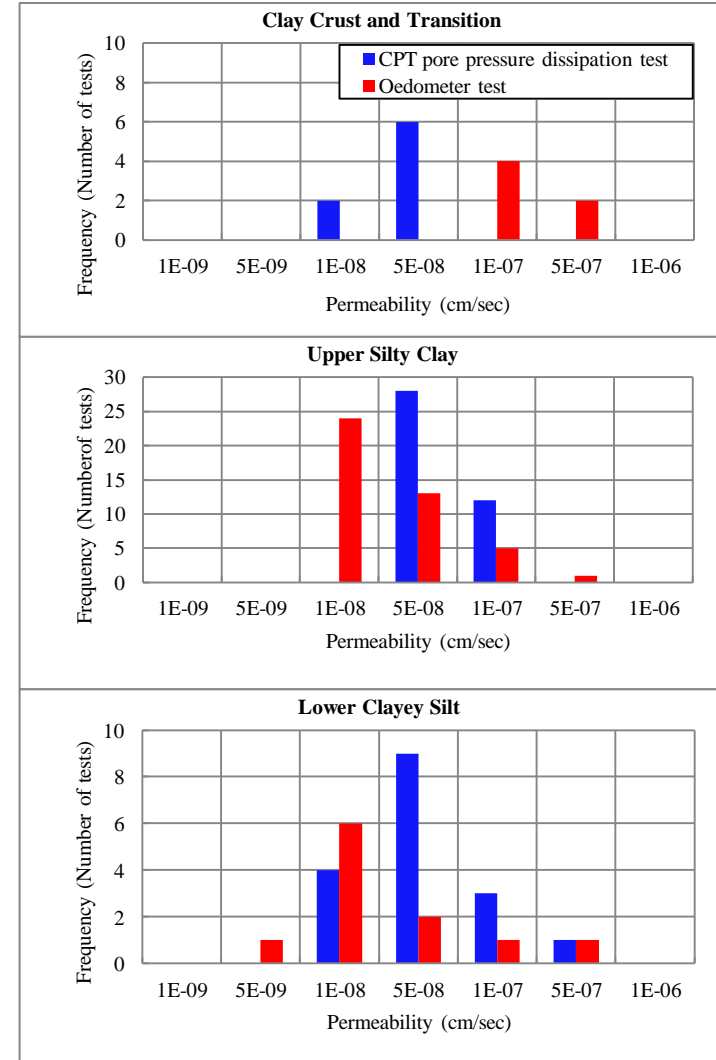
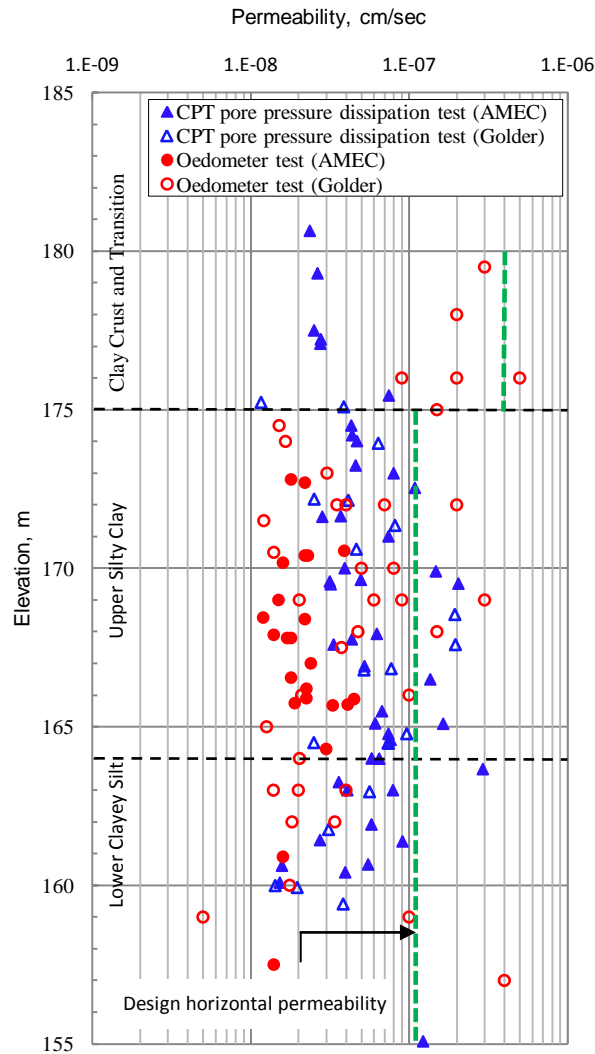
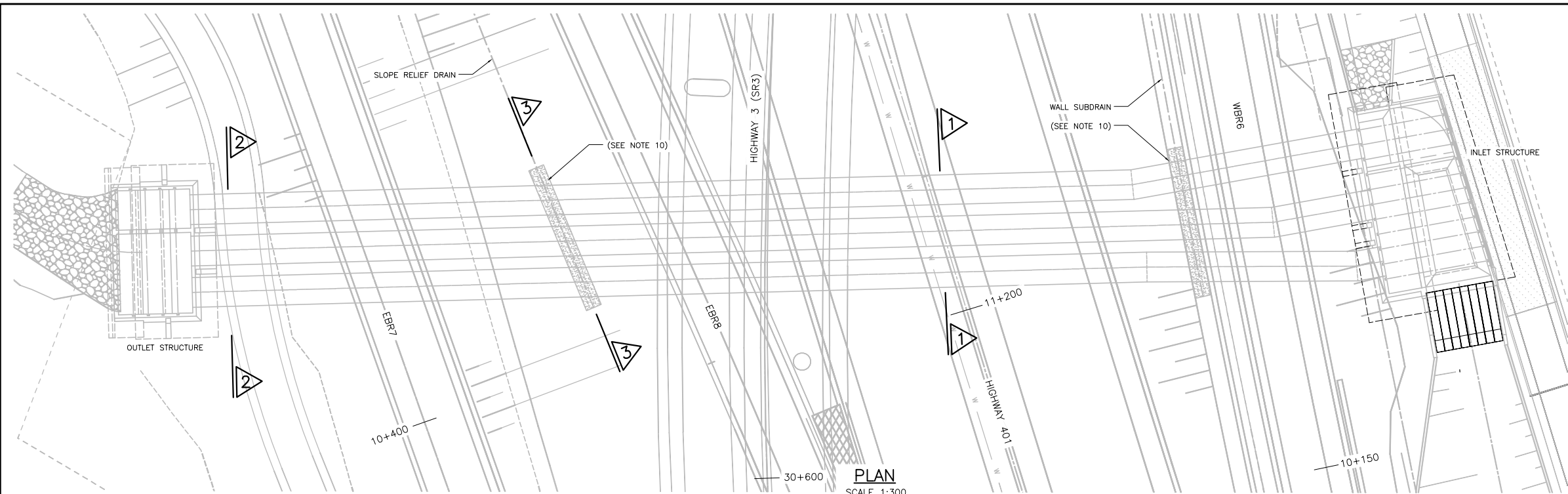


Figure 4-5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests

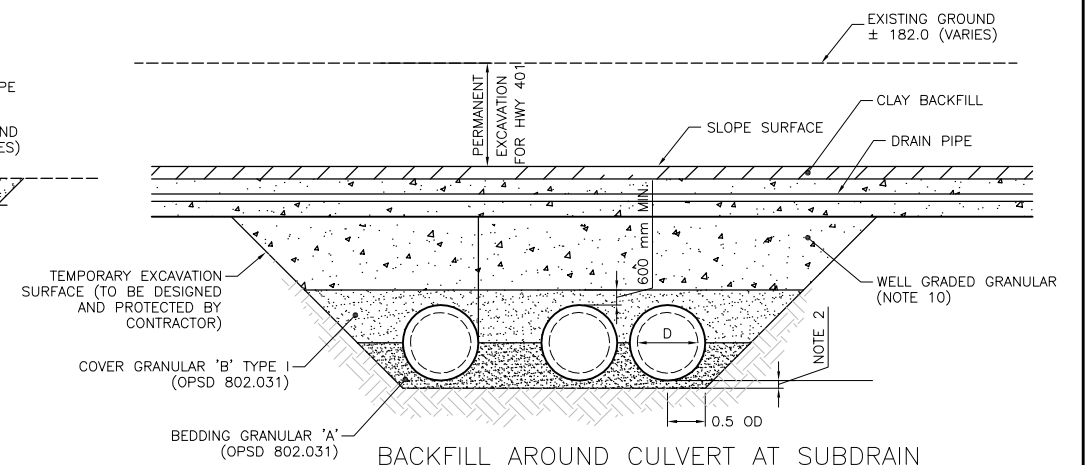
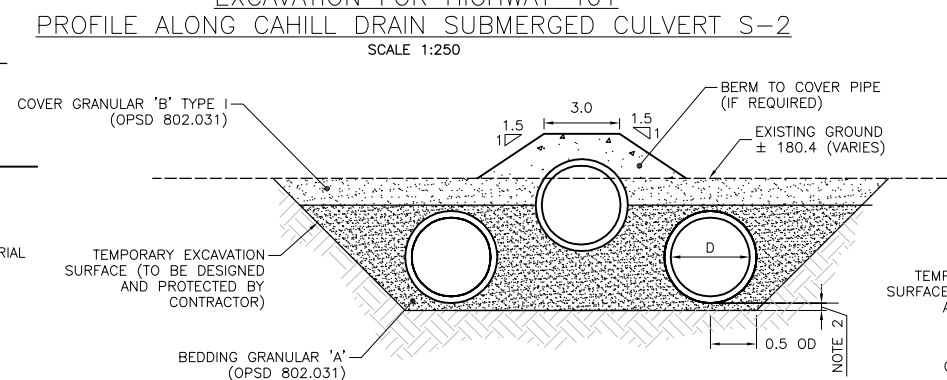
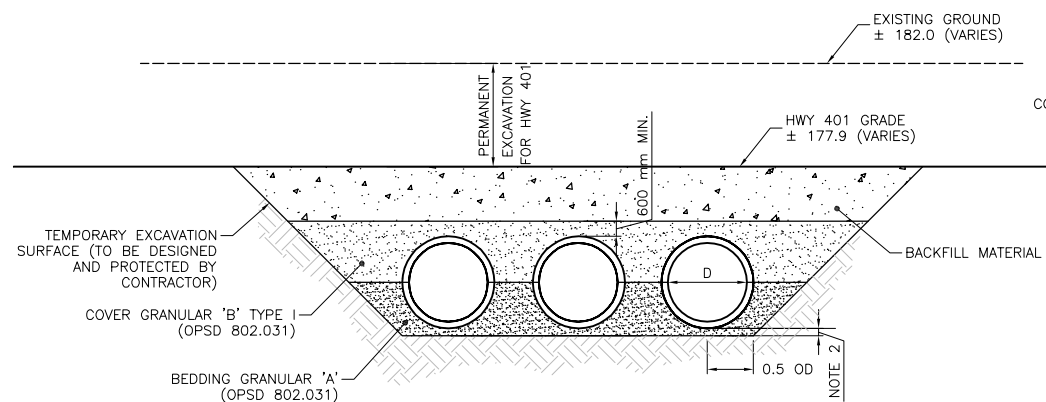
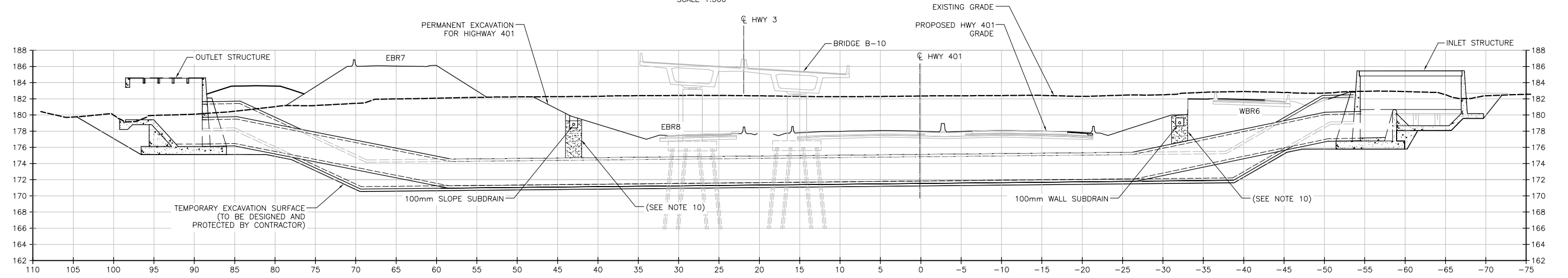


DOC: S-2 PLAN W CULVERT SECTIONS_FIG 5.1



NOTES:

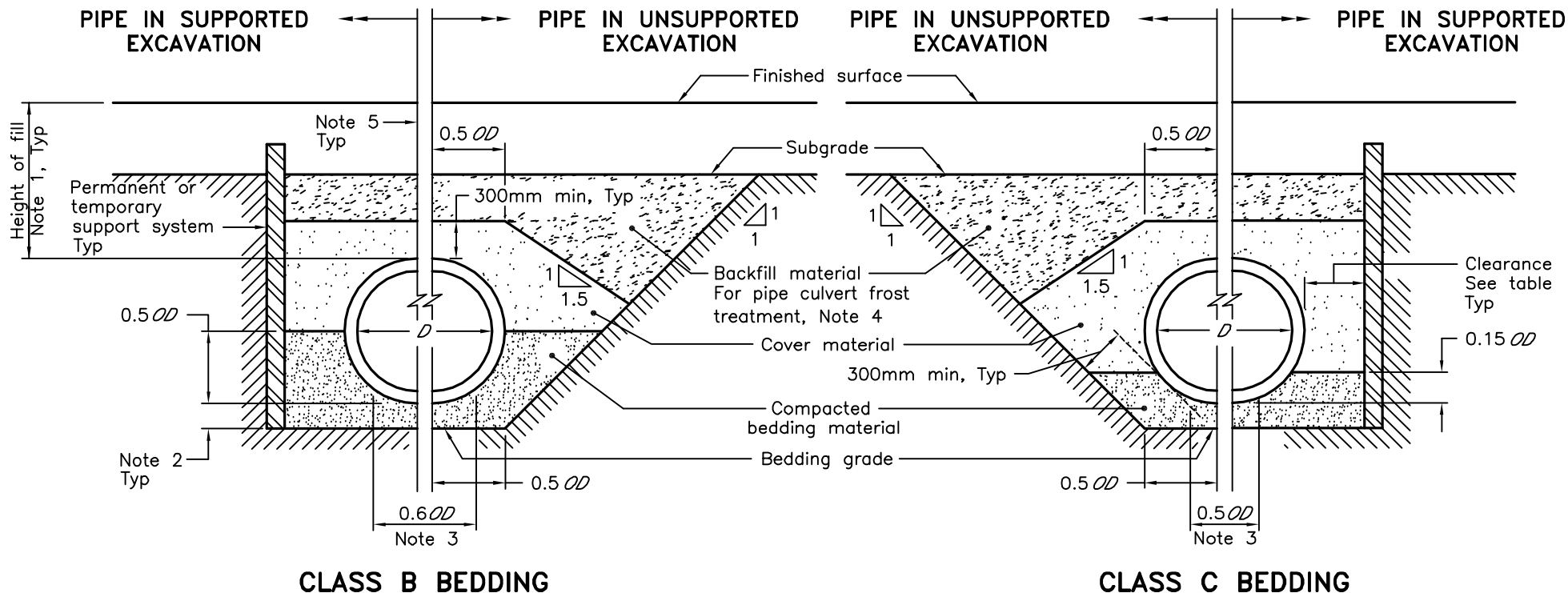
1. PIPE BED SHALL BE SHAPED TO BE ABLE TO RECEIVE BOTTOM OF PIPE.
2. MINIMUM BEDDING DEPTH BELOW PIPE SHALL BE 300mm.
3. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE SHOWN.
4. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
5. THIS DRAWING ILLUSTRATES THE 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 - OPSD 1010 WITH 100% PASSING SIEVE 37.5mm) COMPACTED TO 95% SPMD CONNECTED TO SLOPE/RETAINING WALL RELIEF DRAIN SYSTEM.



Applicable OPSDs

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
Doc No.: 285380-04-119-0009 (Geocres No. 40J3-18)

Date: August/2012
Rev: 0
Page No.: Applicable OPSDs



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 $0.15D$. 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

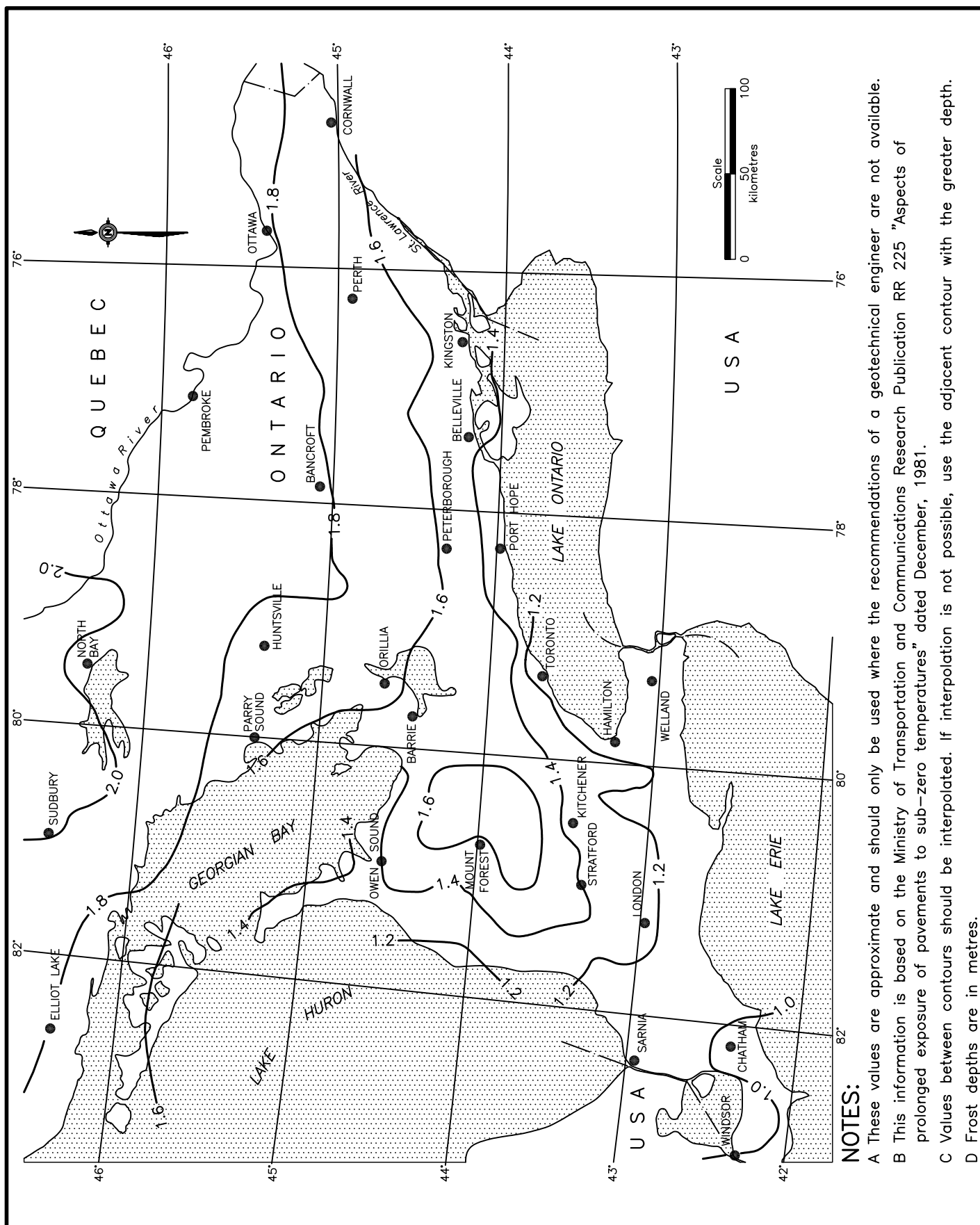
Nov 2010

Rev 2

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

OPSD 802.031





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.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005

Rev 0

FOUNDATION
FROST DEPTHS
FOR SOUTHERN ONTARIO

OPSD - 3090.101



Appendix A Borehole and CPT Logs from Additional Geotechnical Investigation

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
Doc No.: 285380-04-119-0009 (Geocres No. 40J3-18)

Date: August/2012
Rev: 0
Page No.: Appendix A

METRIC

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

METRIC

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

RECORD OF BOREHOLE No B10-2

3 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)			
								○ UNCONFINED	+	FIELD VANE											
								● POCKET PEN.	×	LAB VANE											
							20	40	60	80	100										
										</											

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

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

RECORD OF BOREHOLE No B10-3

1 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)			
								○ UNCONFINED	+ FIELD VANE										
								● POCKET PEN.	× LAB VANE										
182.2	Ground Surface														GR SA SI CL				
0.0	TOPSOIL Clayey														-vibrating wire piezometers (VWP) installed in borehole -hand dug test hole for confirmation of topsoil thickness				
181.4															-hand dug test hole for confirmation of topsoil thickness filled with groundwater at 0.45m below ground surface after 30 min.				
0.8	CLAYEY SILT Some sand, trace gravel Firm to hard Mottled brown and grey Trace pink nodules Trace organics Brown		1	SS	10														
			2	SS	25														
			3	SS	28														
			4	SS	32														
	Grey		5	SS	14														
			6	SS	14														
			7	SS	11														
			8	TW	PH														

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

METRIC

Continued Next Page

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

METRIC

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

RECORD OF BOREHOLE No B10-4

1 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE						
182.4	Ground Surface																
180.0	TOPSOIL																
0.2	CLAYEY SILT Some sand, trace gravel Firm to very stiff Mottled brown and grey																
			1	SS	17								○				
			2	SS	19									○			
	Brown		3	SS	22								○				
			4	SS	19								○				
	Brown-grey		5	SS	12								○				
			6	SS	7								○				
	Grey		7	SS	9								○				
			8	TW	PH									○			
			9	TW	PH									○			
			10	TW	PH									○			
				VT													
			11	TW	PH									○			
				VT													
			12	TW	PH									○			
169.0	SILTY CLAY Grey, trace black silt and pink clay nodules Trace angular gravel																
13.4			13	TW	PH									○			
167.5																	

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

RECORD OF BOREHOLE No B10-4

2 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			
								○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE									
								20 40 60 80 100												
14.9	CLAYEY SILT Some sand, trace gravel Soft to stiff, Grey (continued)						167							○	20.4	-end of drilling june 7; continue June 8				
			14	TW	PH		166						○							
							165													
				VT																
			16	TW	PH		164													
							163													
			17	TW	PH		162							○						
				VT																
			18	TW	PH		161							○	21.5					
							160													
			19	TW	PH		159													
							158							○	21.0					
			20	TW	PH		157													
							156													
			21	TW	PH		155							○						
							154													
			22	TW	PH		153													
			23	TW	PH									○						

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

RECORD OF BOREHOLE No B10-4

3 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100	○ UNCONFINED	+ FIELD VANE						● POCKET PEN.	× LAB VANE	20 40 60 80 100
152.2							152											
30.2	SILTY CLAY Soft Grey		24	TW	PH													
151.0							151											
31.4	FINE SANDY SILT Trace fine-med gravel Very dense Grey		25	SS	75		150											
149.7			26	RC														
32.7	LIMESTONE Medium to fine grained Laminated, stylolites present White to grey		27	RC			149											
			28	RC			148											
147.0							147											
35.4	END OF BOREHOLE No groundwater observed during drilling due to wash boring																	
							146											
							145											
							144											
							143											
							142											
							141											
							140											
							139											
							138											

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

RECORD OF BOREHOLE No B10-5

2 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED ● POCKET PEN.	+ FIELD VANE × LAB VANE							
							20 40 60 80 100	20 40 60 80 100	10 20 30							
	SILTY CLAY Grey, some pink and black nodules, sandy clay lenses (continued)		14	TW	PH											
					VT											
				15	TW	PH										
				16	SS	PH										
					VT											
				17	TW	PH										
165.7 16.6	CLAYEY SILT Some sand, trace gravel Firm to stiff Grey															

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

METRIC

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

1 OF 4

METRIC

Continued Next Page

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

RECORD OF BOREHOLE No B10-5-P32

2 OF 4

METRIC

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

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W		
						20	40	60	80	100						
	Augered to 3.05m, then NW casing to bedrock. No sampling overburden vibrating wire piezometer (VWP) installed in borehole (continued)					167										
						166										
						165										
						164										
						163										
						162										
						161										
						160										
						159										
						158										
						157										
						156										
						155										
						154										
						153										

Continued Next Page

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

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

METRIC

[illegible]

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

Continued Next Page

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

4 OF 4

METRIC

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

RECORD OF BOREHOLE No B10-6

1 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
182.0	Ground Surface							20	40	60	80	100					
0.0	TOPSOIL							○ UNCONFINED	+	FIELD VANE							
181.7	300mm thick, organic sandy silt, black							● POCKET PEN.	×	LAB VANE							
0.3								20	40	60	80	100					
181.2	SILTY SAND							WATER CONTENT (%)									
0.8	Orange-brown																
	CLAYEY SILT																
	Some sand, trace gravel		1	SS	8		181										
	Stiff to very stiff																
	Brown																
	Some small pink clay nodules below 4.5m		2	SS	20		180										-bulk auger sample from 1.5m to 3.8m
	Grey																
	-Layers of fine to medium sand, clayey sand in upper 2 m		3	SS	23		179										-cobble/boulder encountered augers chattering
			4	SS	22												
			5	SS	10		178										
			6	SS	8		177										
			7	SS	10		176										
			8	TW	PH		175										
			9	TW	PH		174								21.3	2 24 38 36	
			10	TW	PH		173										
			VT				172										
			11	TW	PH		171								20.3		
			VT				170										
			12	TW	PH		169										
	Some fine sand seams at approx 12 m below ground surface (El. 170 m)		VT														
168.4	SILTY CLAY						168										
13.6	Trace silt, trace gravel		13	TW	PH										19.4	1 10 41 48	
	Grey, some pink and black clay nodules,																
			VT														

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

RECORD OF BOREHOLE No B10-6

2 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)			
								○ UNCONFINED	+ FIELD VANE										
								● POCKET PEN.	× LAB VANE										
166.9 15.1	CLAYEY SILT Some sand, trace gravel Grey													21.3	-end of drilling May 14; restarted May 16 -switched to wash boring				
			14	TW	PH		166												
				VT															
				15	TW	PH		165											
				16	TW	PH		164											
			17	TW	PH		162												
			18	TW	PH		161												
			19	TW	PH		159												
			20	SS	PH		157												
		21	TW	PH		156													
155.2 26.8	SILTY CLAY with SILT/FINE SAND Laminated Firm Grey					155													
			22	TW	PH		154												
										</									

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

METRIC

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

RECORD OF BOREHOLE No B10-7

1 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE		● POCKET PEN.						× LAB VANE		
182.2	Ground Surface						20	40	60	80	100									
0.0	TOPSOIL																			
181.9	Black																			
0.3	CLAYEY SILT																			
	Some sand, trace gravel		1	SS	17								○			-inclinometer casing installed in borehole				
	Soft to very stiff																			
	Brown		2	SS	21								○			-Vibrating Wire Piezometers (VWP) installed in adjacent boring at 4678976.3N, 332969E;				
	Some pink nodules below approx. 12m (El. 170 m)															Spider Magnets (MG) installed in adjacent boring at 4678979.5N, 332966E				
			3	SS	29								○							
			4	SS	27								○							
			5	SS	17								○							
			6	SS	11								○							
	Grey		7	SS	8								○							
			8	SS	7								○							
			9	TW	PH															
			10	TW	PH															
				VT																
			11	TW	PH															
			12	TW	PH															
				VT																
			13	TW	PH															
				</																

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

METRIC

Continued Next Page

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

RECORD OF BOREHOLE No B10-7

3 OF 3

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED		+ FIELD VANE			w _p w w _L				
								● POCKET PEN.		× LAB VANE							
							20 40 60 80 100										
151.7							152										
30.5			26	SS	7		151										
150.3																	
31.9			27	SS	36		150										
149.6																	
32.6			28	RC			149										
							148										
			29	RC			147										
146.5																	
35.7																	
							146										
							145										
							144										
							143										
							142										
							141										
							140										
							139										
							138										

-end of drilling
May 27; continue
May 31
ROD = 35%
TCR = 60%
SCR = 46%

ROD = 88%
TCR = 99%
SCR = 93%

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 17/05/12

RECORD OF BOREHOLE No CPT B10-2

1 OF 1

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)				
								○ UNCONFINED	+	FIELD VANE	×						LAB VANE				
								● POCKET PEN.													
182.2	Ground Surface																				
0.0	TOPSOIL																				
181.9	275mm thick, organic clay, black																				
0.3	CLAYEY SILT																				
	Some sand, trace gravel		1	SS	14																
	Stiff to very stiff																				
	Mottled brown and grey		2	SS	17																
	Brown																				
			3	SS	23																
			4	SS	22																
	Grey		5	SS	12																
	-Vertical silt seams		6	SS	14																
177.2	END OF SAMPLED BOREHOLE																				
5.0	Continued with CPT from 5.0 m to refusal at 29.3 m (El. 177.2 m to El. 152.9 m)																				

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

RECORD OF BOREHOLE No CPT B10-3

1 OF 1

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			
								○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE									
								20	40	60	80						100	10	20	30
182.1	Ground Surface						182													
0.0	300mm Black organic clay																			
181.7	TOPSOIL																			
0.4	CLAYEY SILT Some sand, trace gravel Stiff to very stiff Mottled brown and grey Brown		1	SS	16		181													
			2	SS	19		180													
			3	SS	22		179													
			4	SS	22		178													
			5	SS	17															
	Grey																			
			6	SS	11															
177.1							177													
5.0	END OF SAMPLED BOREHOLE Continue with CPT from 4.6 m to refusal at 31.3 m (El. 177.5 m to El. 150.8 m) Borehole dry upon completion						176													
							175													
							174													
							173													
							172													
							171													
							170													
							169													
							168													

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

METRIC

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

RECORD OF BOREHOLE No NIL B10-3

1 OF 1

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED	● POCKET PEN.	+ FIELD VANE	× LAB VANE								
								20	40	60	80						100	10	20
182.2	Ground Surface																		
0.0	TOPSOIL																		
181.9	Black																		
0.3	CLAYEY SILT																		
	Some sand, trace gravel		1	SS	17								○						
	Very stiff to hard																		
	Mottled brown and grey																		
	Brown		2	SS	19								○						
			3	SS	36								○						
			4	SS	32								○						
	Grey		5	SS	16								○						
													○						
			6	SS	15														
177.0	END OF SAMPLED BOREHOLE																		
5.2	Continued with Nilcon Vane from 5.3 m to refusal at 20.0 m (El. 176.9 m to El. 162.2 m)																		
	Borehole dry on completion																		

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

RECORD OF CONE PENETRATION TEST CPT B10-2

METRIC

PROJECT Windsor-Essex Parkway

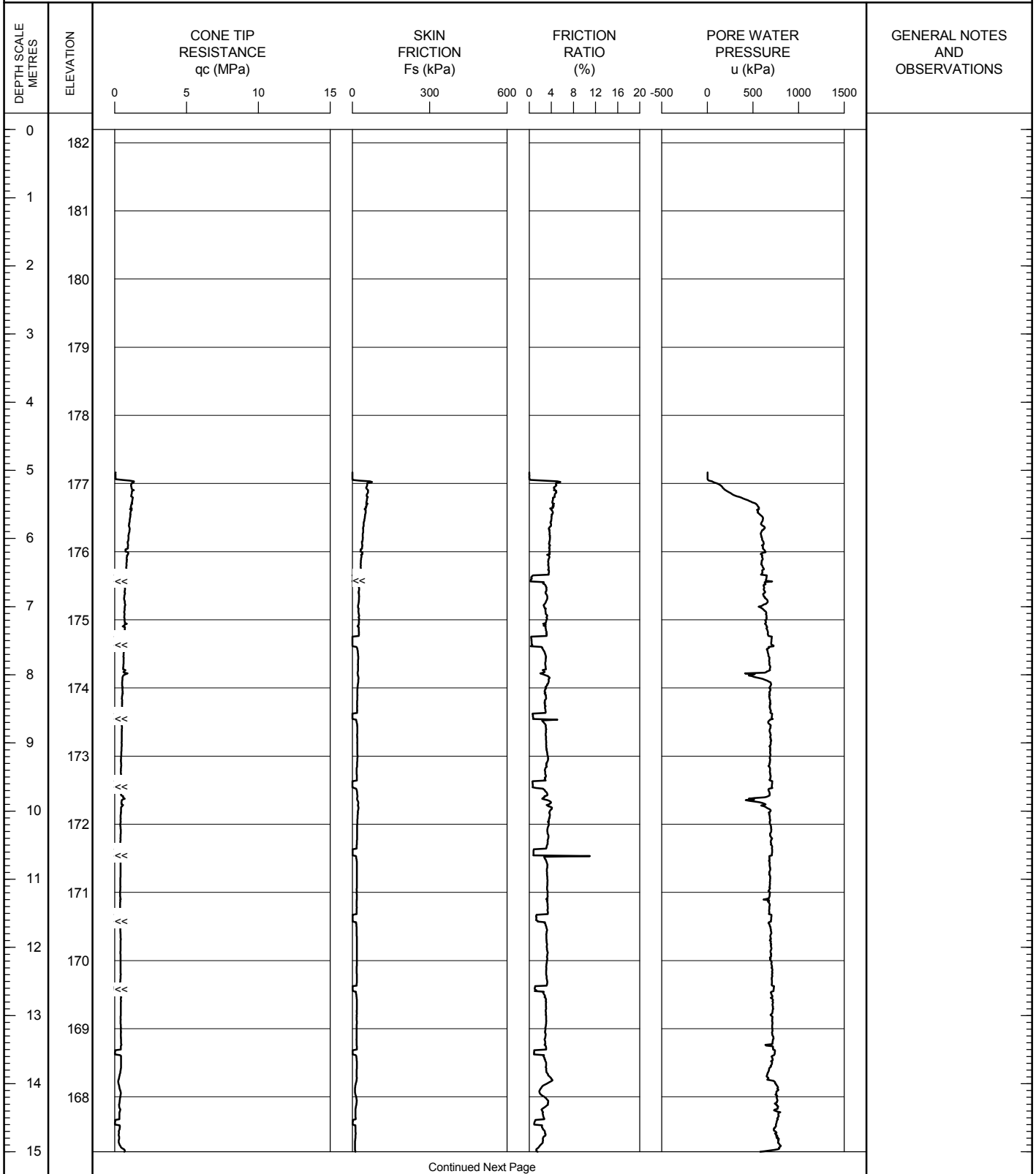
TEST DATE 5/13/2011 - 5/13/2011

SHEET 1 OF 2

LOCATION N4679056.3; E332920.1

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.2 PREDRILL DEPTH: 4.92 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT B10-2

METRIC

PROJECT Windsor-Essex Parkway

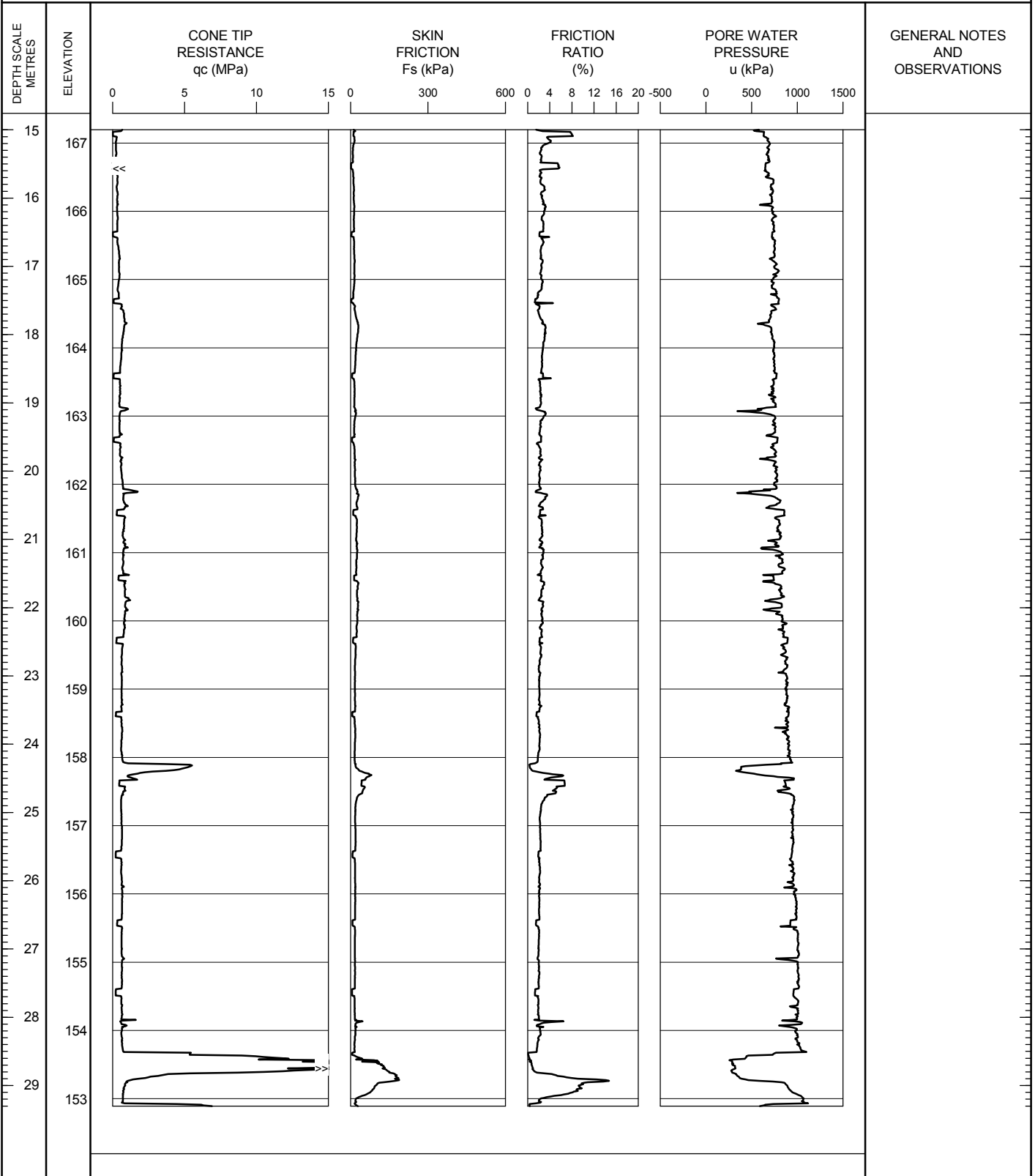
TEST DATE 5/13/2011 - 5/13/2011

SHEET 2 OF 2

LOCATION N4679056.3; E332920.1

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.2 PREDRILL DEPTH: 4.92 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



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

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT B10-3

METRIC

PROJECT Windsor-Essex Parkway

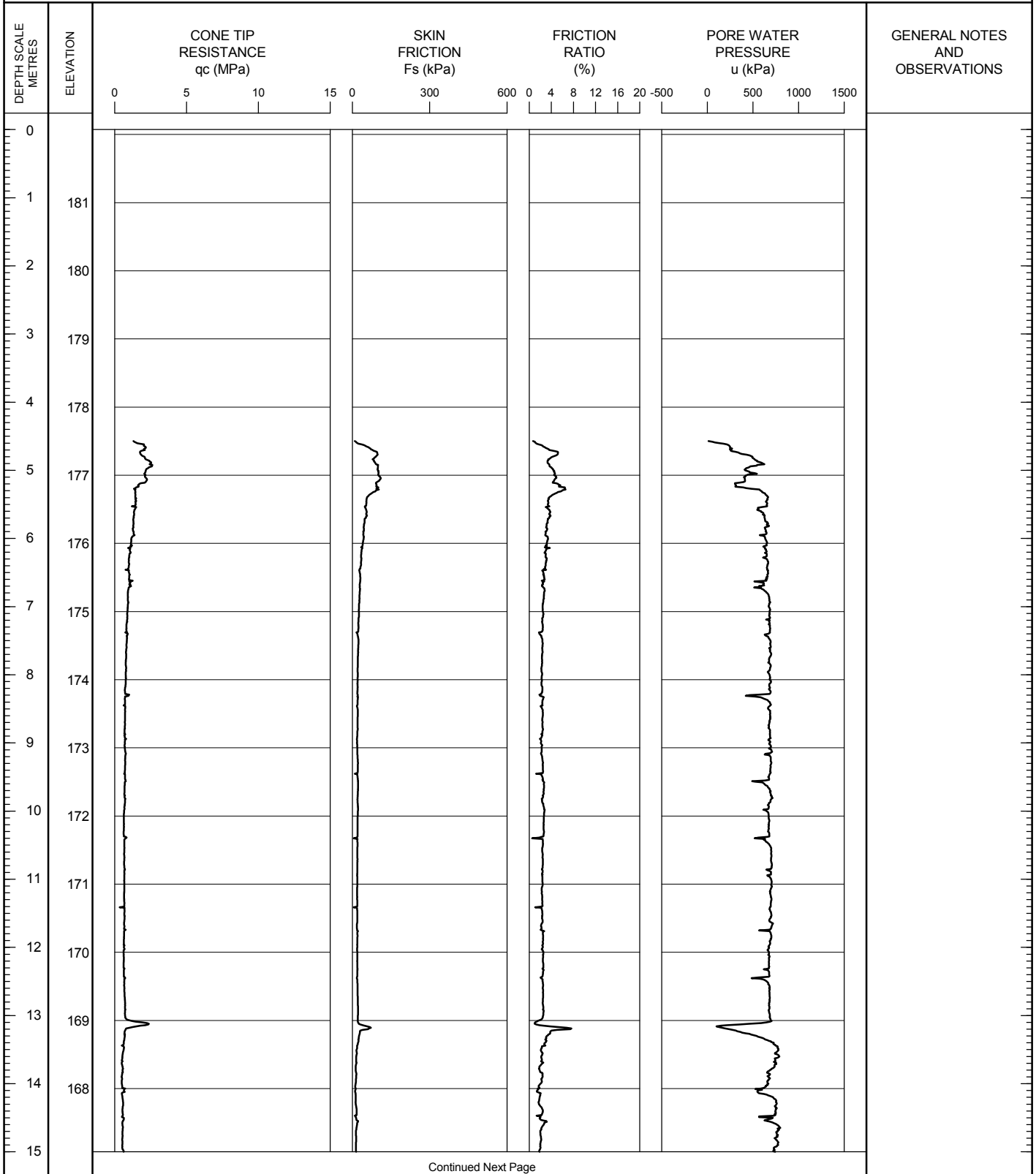
TEST DATE 5/13/2011 - 5/13/2011

SHEET 1 OF 3

LOCATION N4678983.4; E332941.1

DATUM Geodetic

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



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT B10-3

METRIC

PROJECT Windsor-Essex Parkway

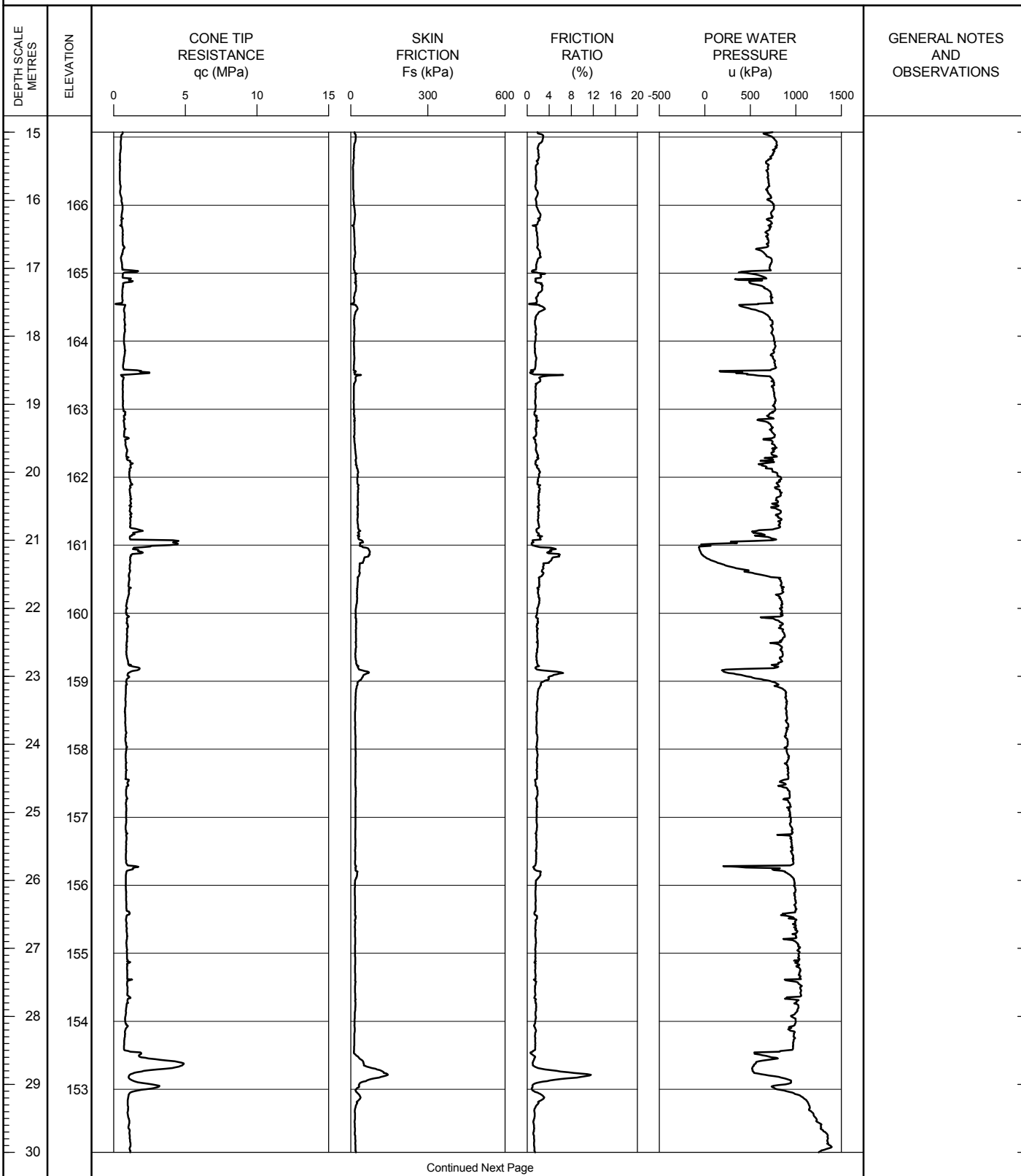
TEST DATE 5/13/2011 - 5/13/2011

SHEET 2 OF 3

LOCATION N4678983.4; E332941.1

DATUM Geodetic

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



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

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT B10-3

METRIC

PROJECT Windsor-Essex Parkway

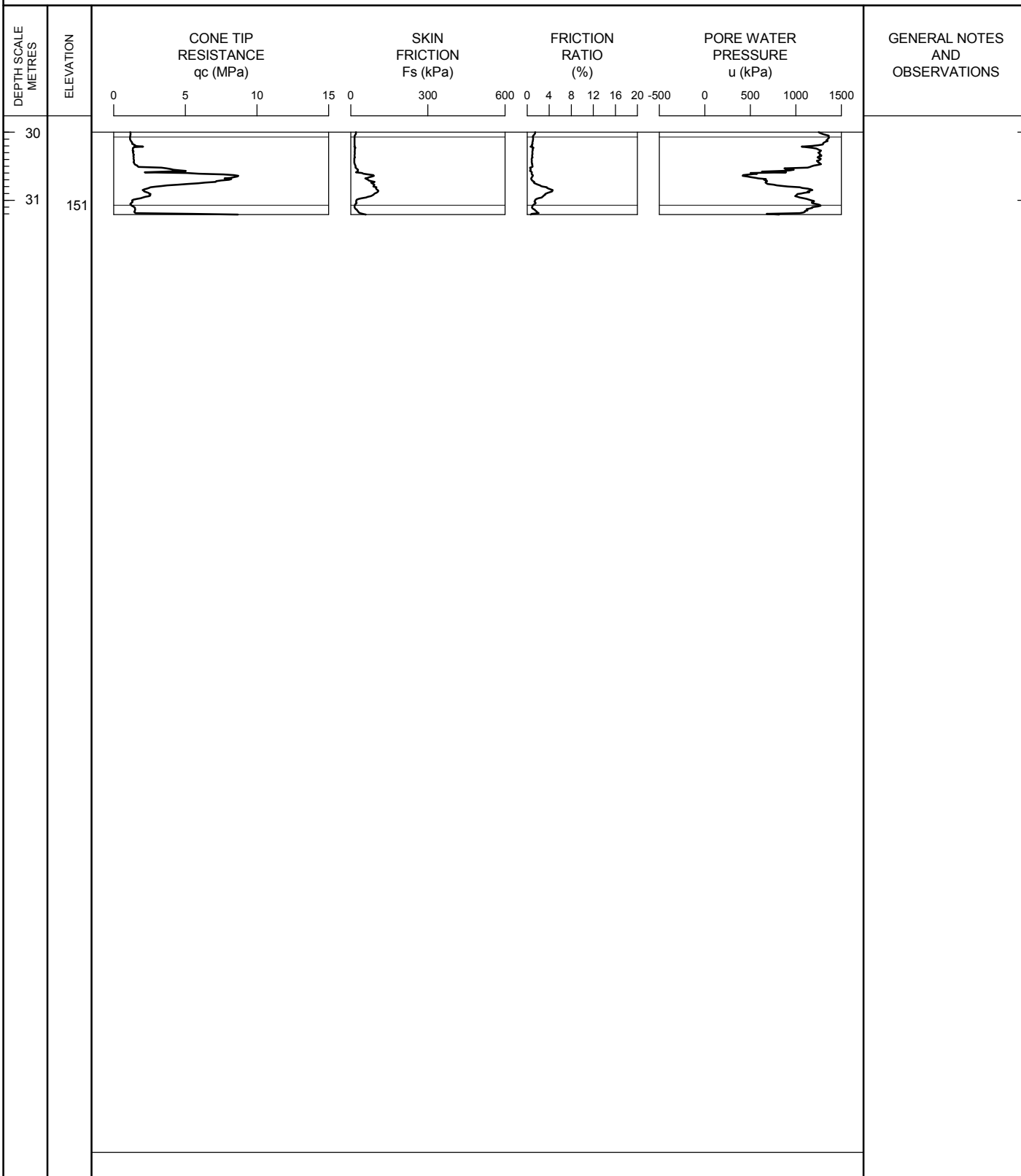
TEST DATE 5/13/2011 - 5/13/2011

SHEET 3 OF 3

LOCATION N4678983.4; E332941.1

DATUM Geodetic

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



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

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 41-RW

METRIC

PROJECT Windsor-Essex Parkway

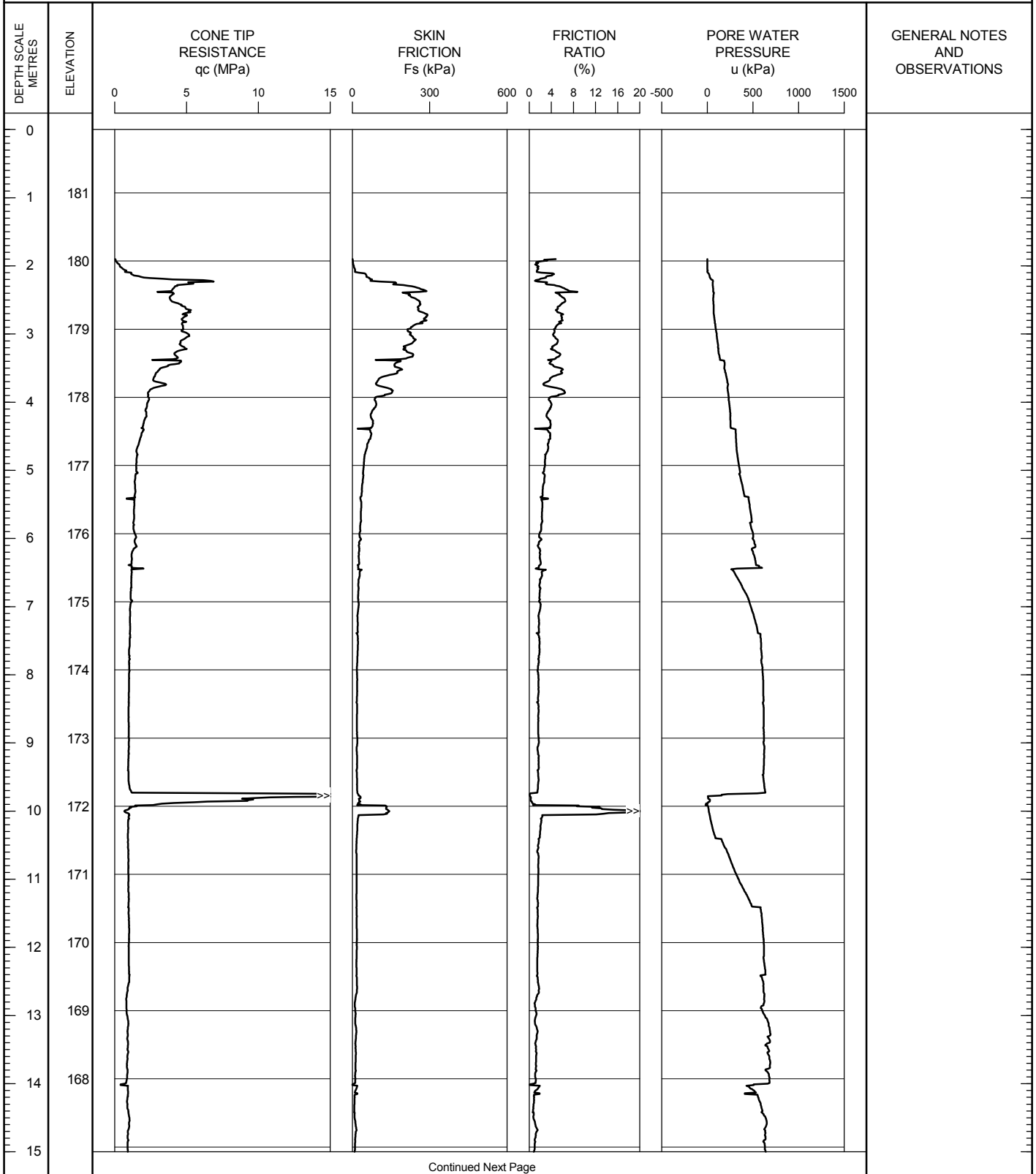
TEST DATE 7/28/2011 - 7/28/2011

SHEET 1 OF 3

LOCATION N4679053.8; E332792.8

DATUM Geodetic

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



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

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 41-RW

METRIC

PROJECT Windsor-Essex Parkway

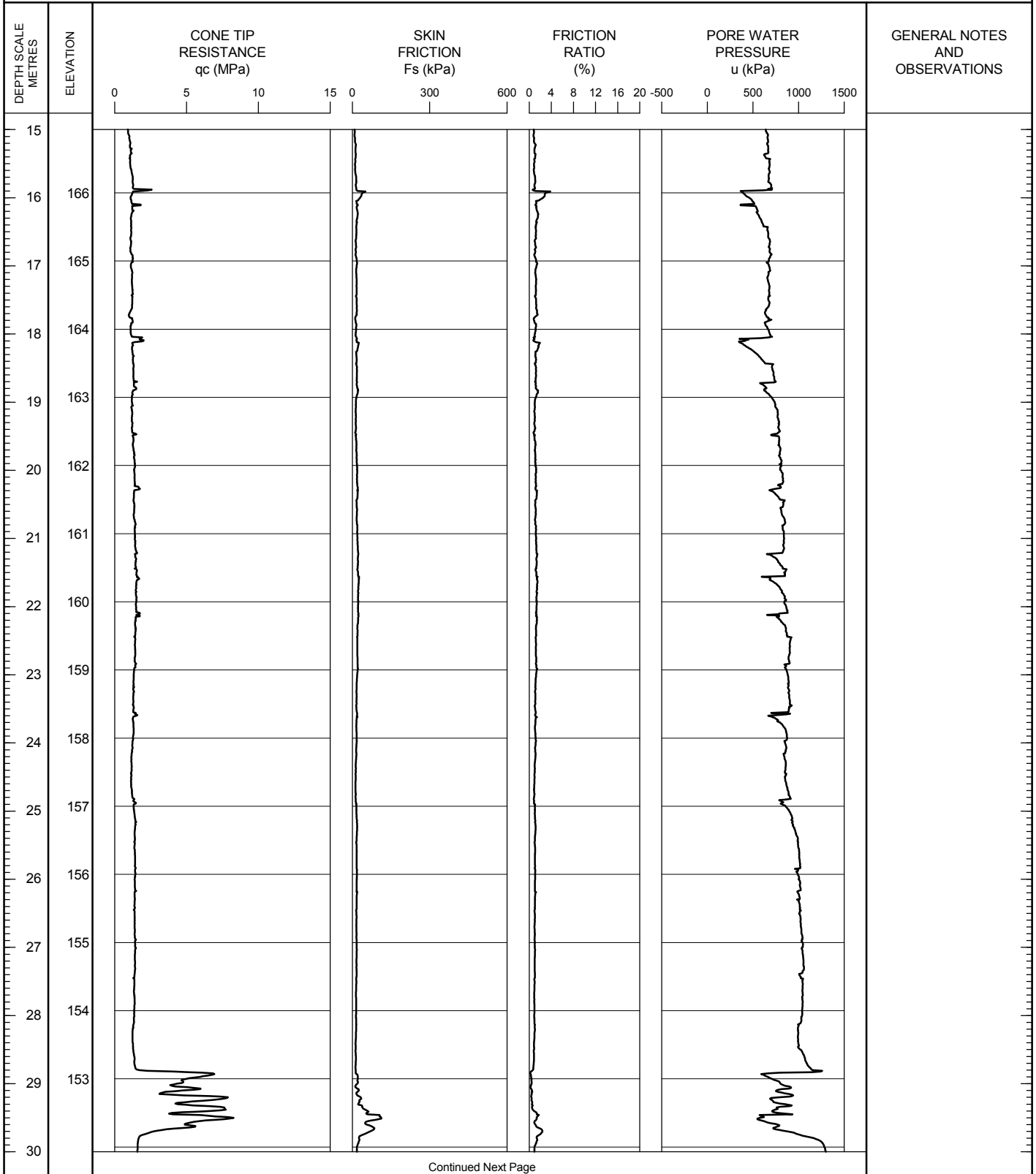
TEST DATE 7/28/2011 - 7/28/2011

SHEET 2 OF 3

LOCATION N4679053.8; E332792.8

DATUM Geodetic

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



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 41-RW

METRIC

PROJECT Windsor-Essex Parkway

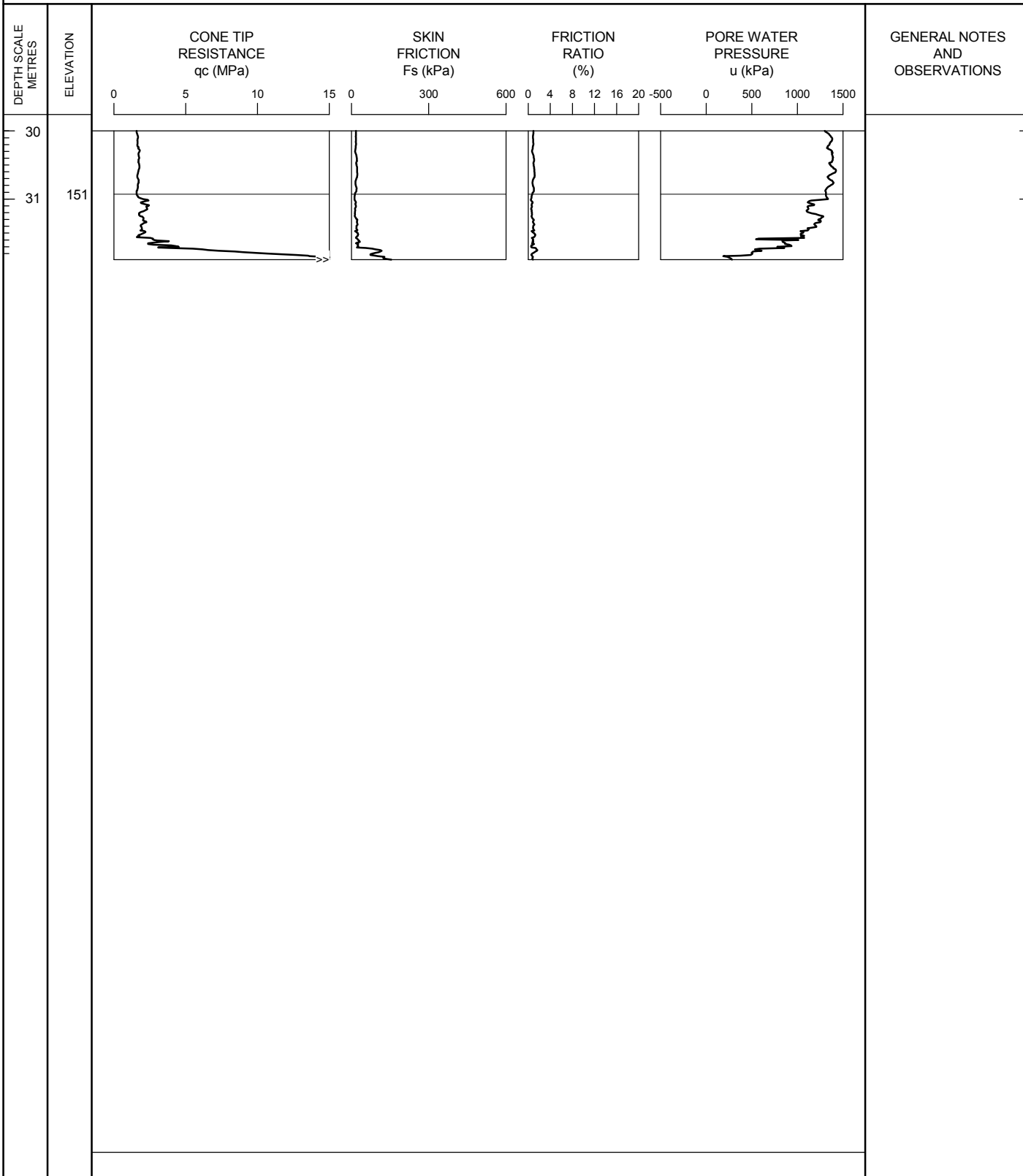
TEST DATE 7/28/2011 - 7/28/2011

SHEET 3 OF 3

LOCATION N4679053.8; E332792.8

DATUM Geodetic

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



OPERATOR: TA

CHECKED: DD

RECORD OF NILCON VANE TEST NIL B10-3

Project : Windsor-Essex Parkway

Test Date: 6/8/2011

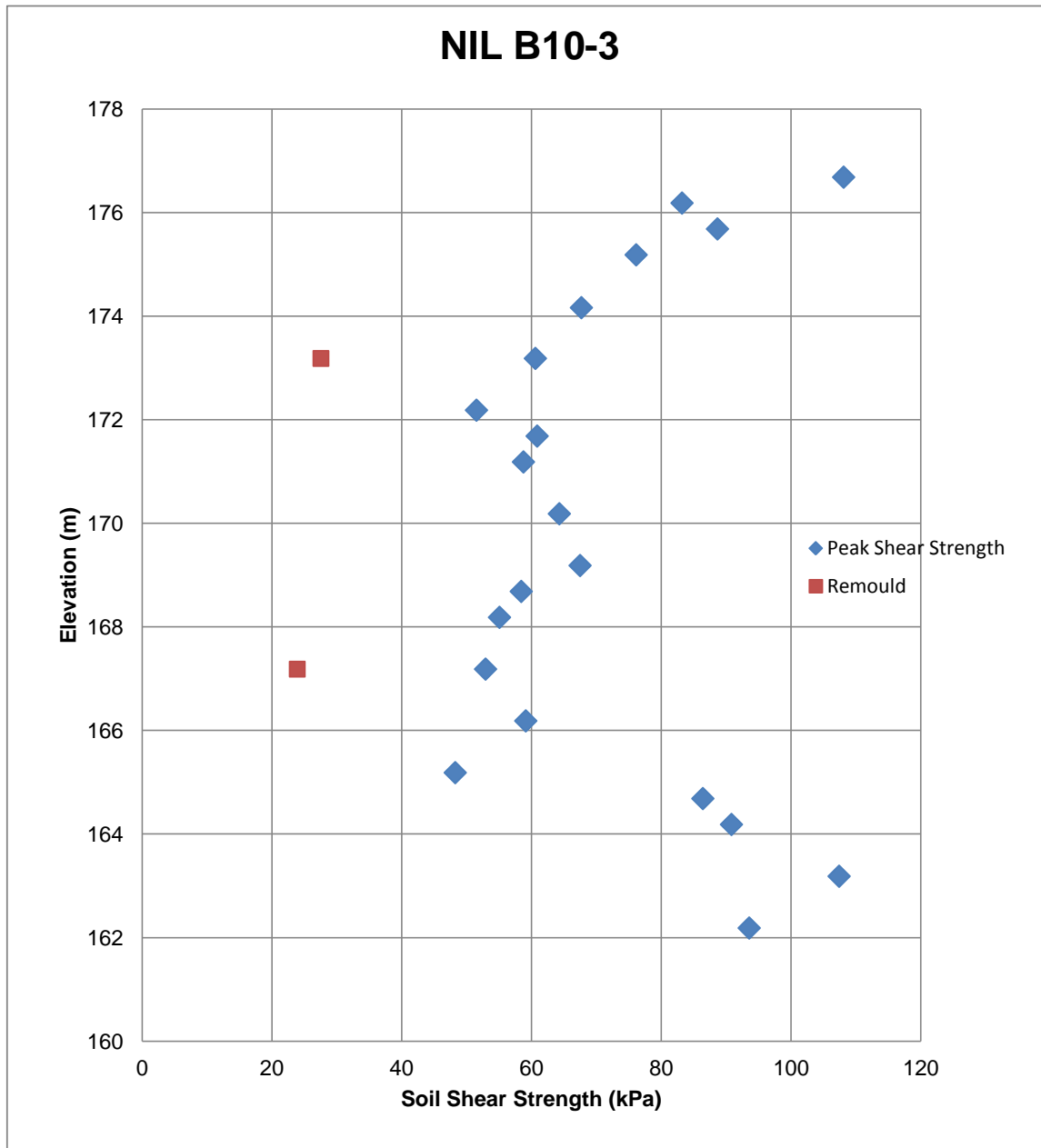
Sheet 1 of 1

Location: N4679084.3; E332841.7

Predrill Depth : 5.0 m

Datum Geodetic

Ground Surface Elevation: 182.2 m



Operator: SD



Checked: DD

Appendix B Borehole and CPT Logs from Previous Investigations

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
Doc No.: 285380-04-119-0009 (Geocres No. 40J3-18)

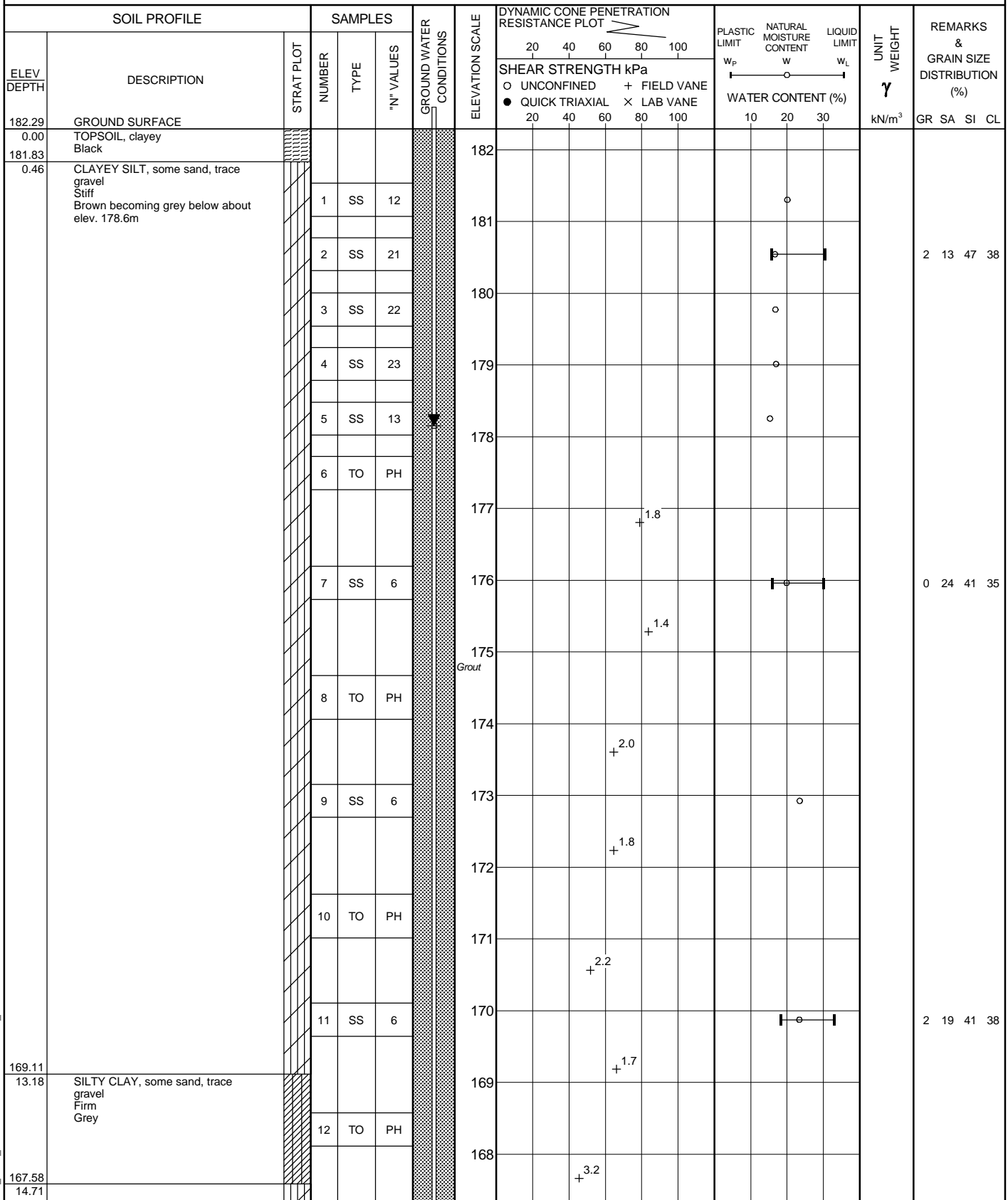
Date: August/2012
Rev: 0
Page No.: Appendix B

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

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

LDN_MTO_06 09-1132-0080.GPJ LDN_MTO.GDT 11/03/10

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No 318		1 OF 4	METRIC
W.P. _____		LOCATION <u>N 4679049.3 ; E 332857.8</u>		ORIGINATED BY <u>SM</u>	
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>LMK/DMB</u>	
DATUM <u>GEODETIC</u>		DATE <u>December 10, 2009 - December 14, 2009</u>		CHECKED BY _____	

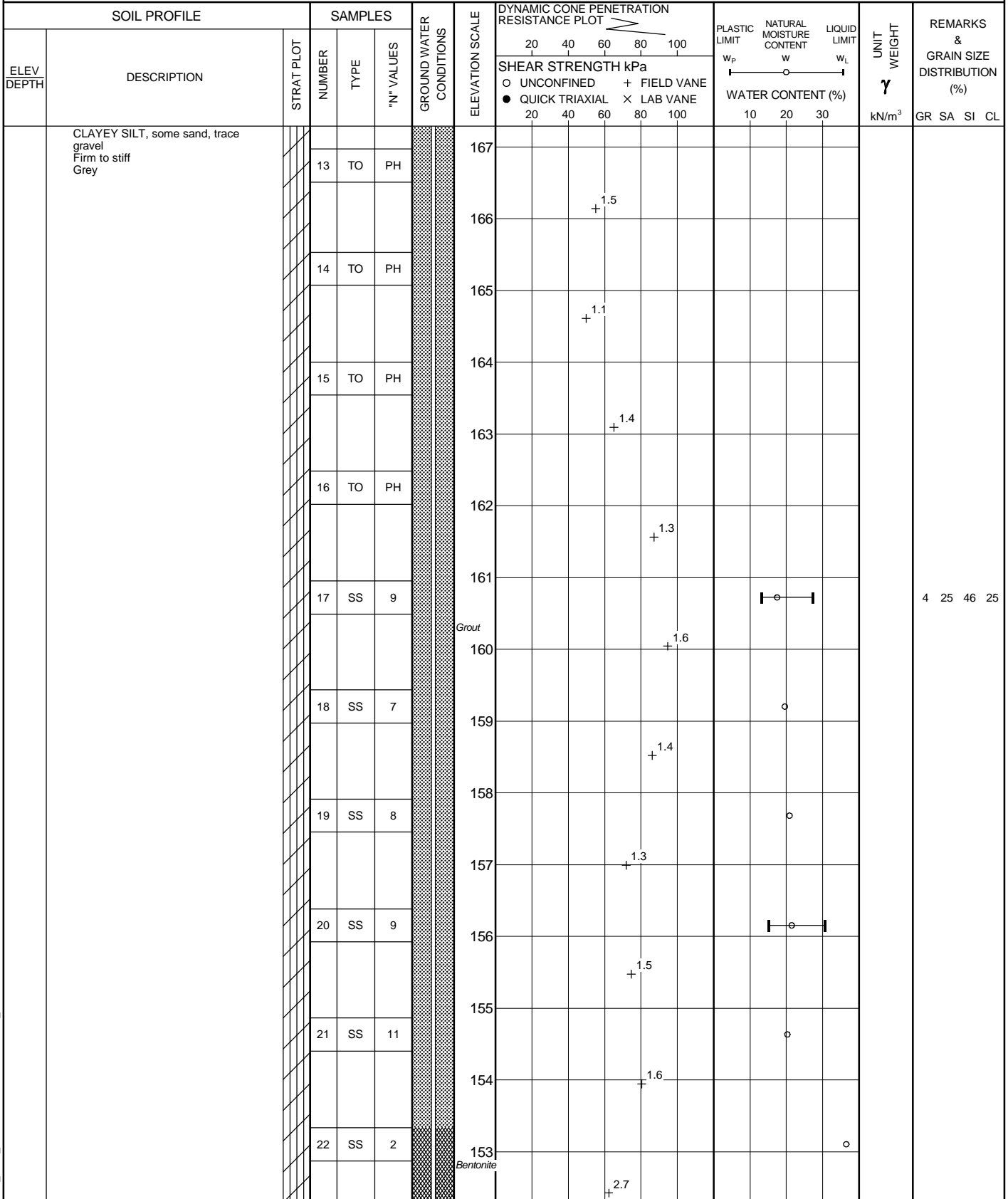


LDN_MTO_06 09-1132-0080 GPJ LDN_MTO.GDT 11/03/10

Continued Next Page

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

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No 318		2 OF 4		METRIC	
W.P. _____		LOCATION <u>N 4679049.3 ; E 332857.8</u>		ORIGINATED BY <u>SM</u>			
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>LMK/DMB</u>			
DATUM <u>GEODETIC</u>		DATE <u>December 10, 2009 - December 14, 2009</u>		CHECKED BY _____			



LDN_MTO_06 09-1132-0080.GPJ LDN_MTO.GDT 11/03/10

Continued Next Page

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

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No 318		3 OF 4	METRIC
W.P. _____		LOCATION <u>N 4679049.3 ; E 332857.8</u>		ORIGINATED BY <u>SM</u>	
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>LMK/DMB</u>	
DATUM <u>GEODETIC</u>		DATE <u>December 10, 2009 - December 14, 2009</u>		CHECKED BY _____	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									WATER CONTENT (%)	
							20	40	60	80	100							
150.81	CLAYEY SILT, some sand, trace gravel Firm to stiff Grey		23	SS	4													
31.48	SAND AND GRAVEL, some silt, trace clay Compact Grey																	
149.72	LIMESTONE, fresh, medium strong, weakly laminated, fine grained, faintly porous Light grey to brown (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		24	SS	25													
32.57			25	NQ RC	-													
			26	NQ RC	-													
			27	NQ RC	-													
			28	NQ RC	-													
144.93	END OF BOREHOLE																	
37.36	Borehole dry during drilling between December 10 and 14, 2009. Water level measured at elev. 178.35 on February 24, 2010. Water level measured at elev. 178.15 on January 6, 2010.																	

PROJECT: 09-1132-0080

RECORD OF DRILLHOLE: 318

SHEET 4 OF 4

LOCATION: N 4679049.3 ;E 332857.8

DRILLING DATE: December 10, 2009 - December 15, 2009

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (mm/min)	COLOUR % RETURN	FLUSH	ELEVATION	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 <small>NOTE: For additional abbreviations refer to list of abbreviations & symbols.</small>										HYDRAULIC CONDUCTIVITY k, cm/sec			DIAMETRAL POINT LOAD INDEX (MPa)		NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
				DEPTH (m)						RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		TYPE AND SURFACE DESCRIPTION	DIP w.r.t. CORE AXIS	10 ⁻⁸	10 ⁻⁶	10 ⁻⁴	10 ⁻²	2	4	6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
				TOTAL CORE %						SOLID CORE %	0			30	60											90																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
				80 60 40 20						80 60 40 20	5 10 15 20	0 30 60 90	10 ⁻⁸	10 ⁻⁶	10 ⁻⁴	10 ⁻²																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
		ROCK SURFACE AT ELEV. 149.72m		149.83 32.46 32.57	1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									

DEPTH SCALE

1 : 75



LOGGED: SG

CHECKED:

LDN_ROCK_03 09-1132-0080-ROCK.GPJ GLDR LDN.GDT 12/03/10 DATA INPUT: LMK

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-317

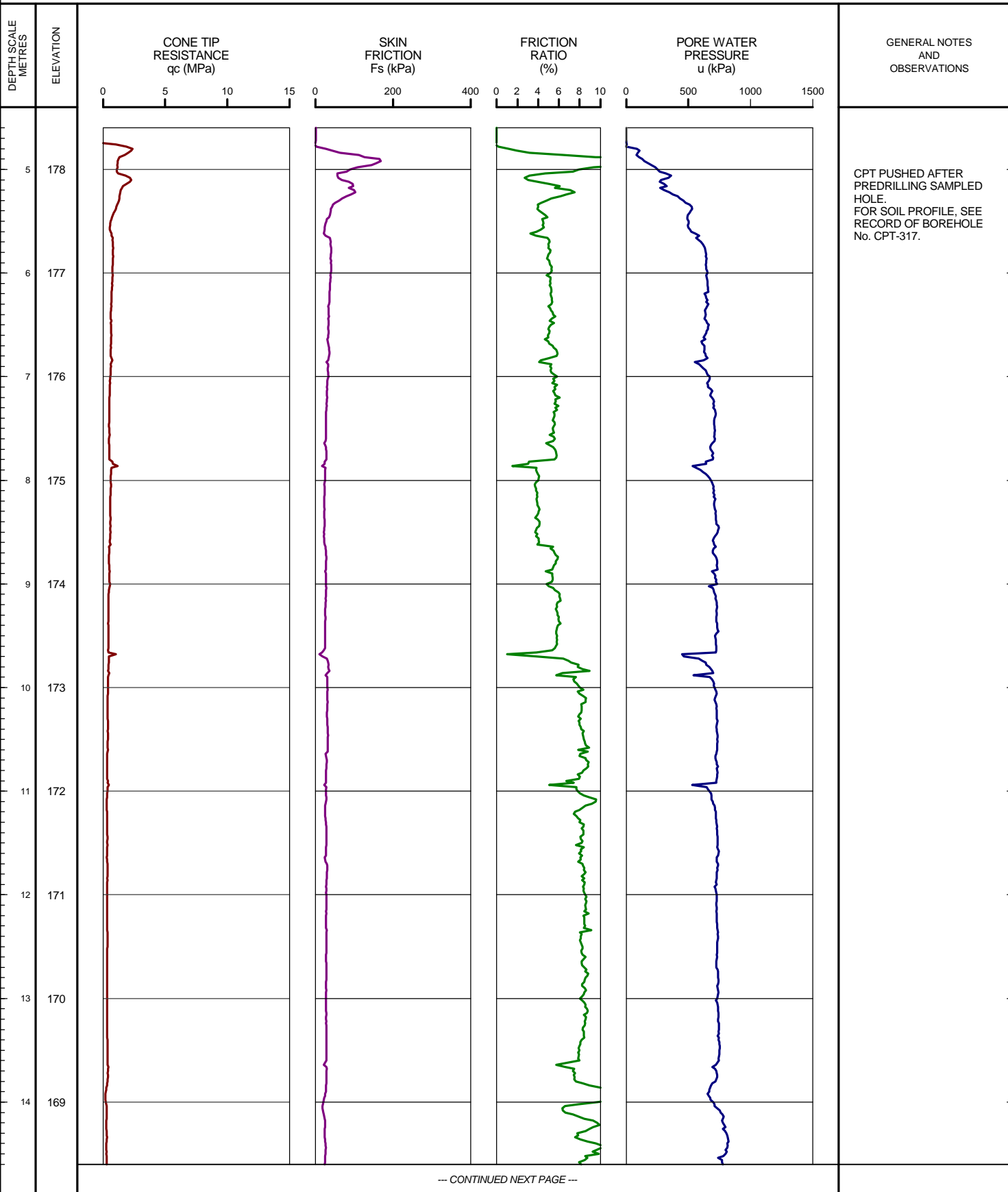
SHEET 1 OF 2

LOCATION: N 4679041.7 ;E 332972.4

TEST DATE: January 26, 2010

DATUM: GEODETIC

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



LDN_CPT_01 09-1132-0080-CPT.GPJ GLDR_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-317

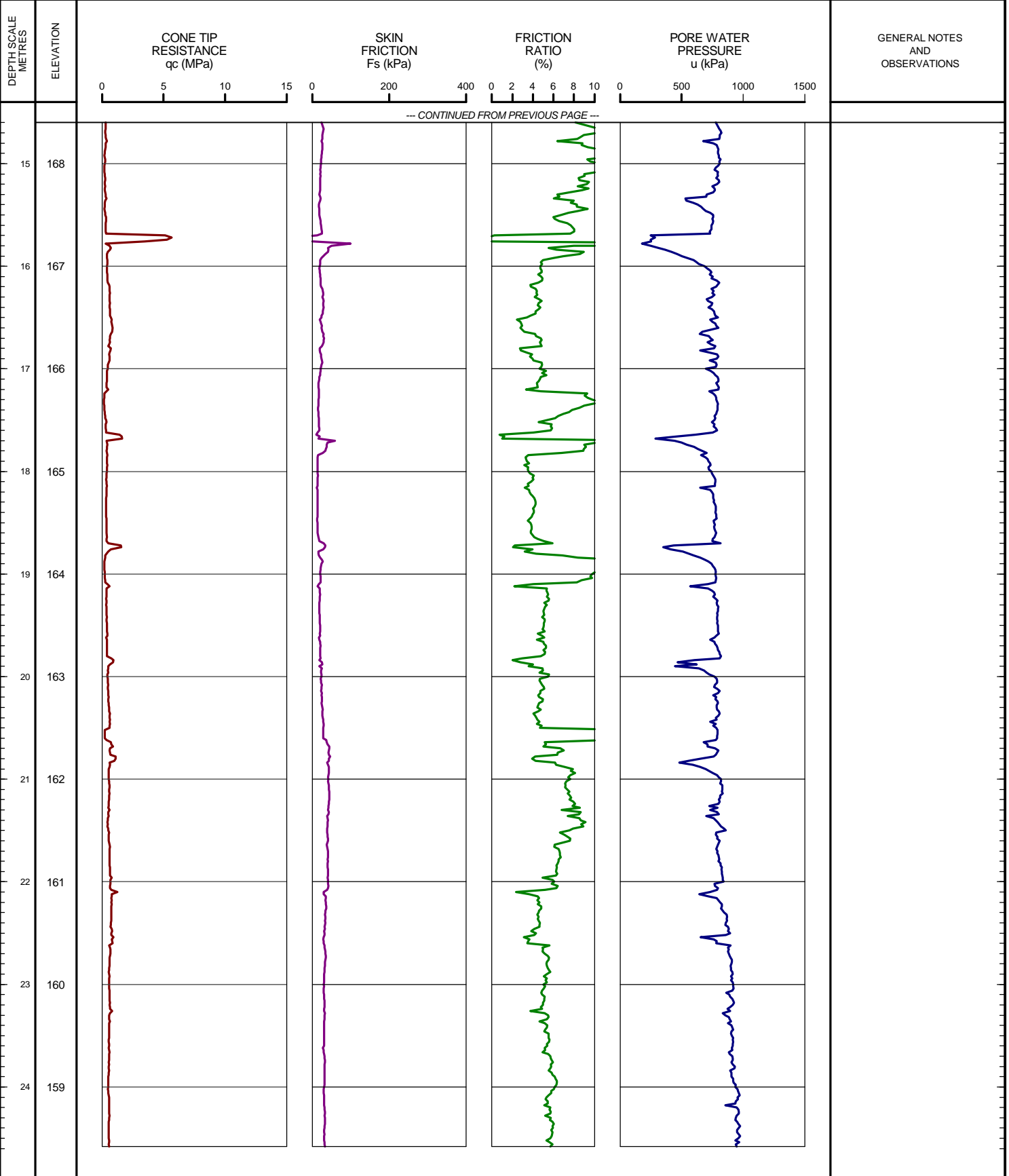
SHEET 2 OF 2

LOCATION: N 4679041.7 ;E 332972.4

TEST DATE: January 26, 2010

DATUM: GEODETIC

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



LON_CPT_01 09-1132-0080-CPT.GPJ GLDR_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



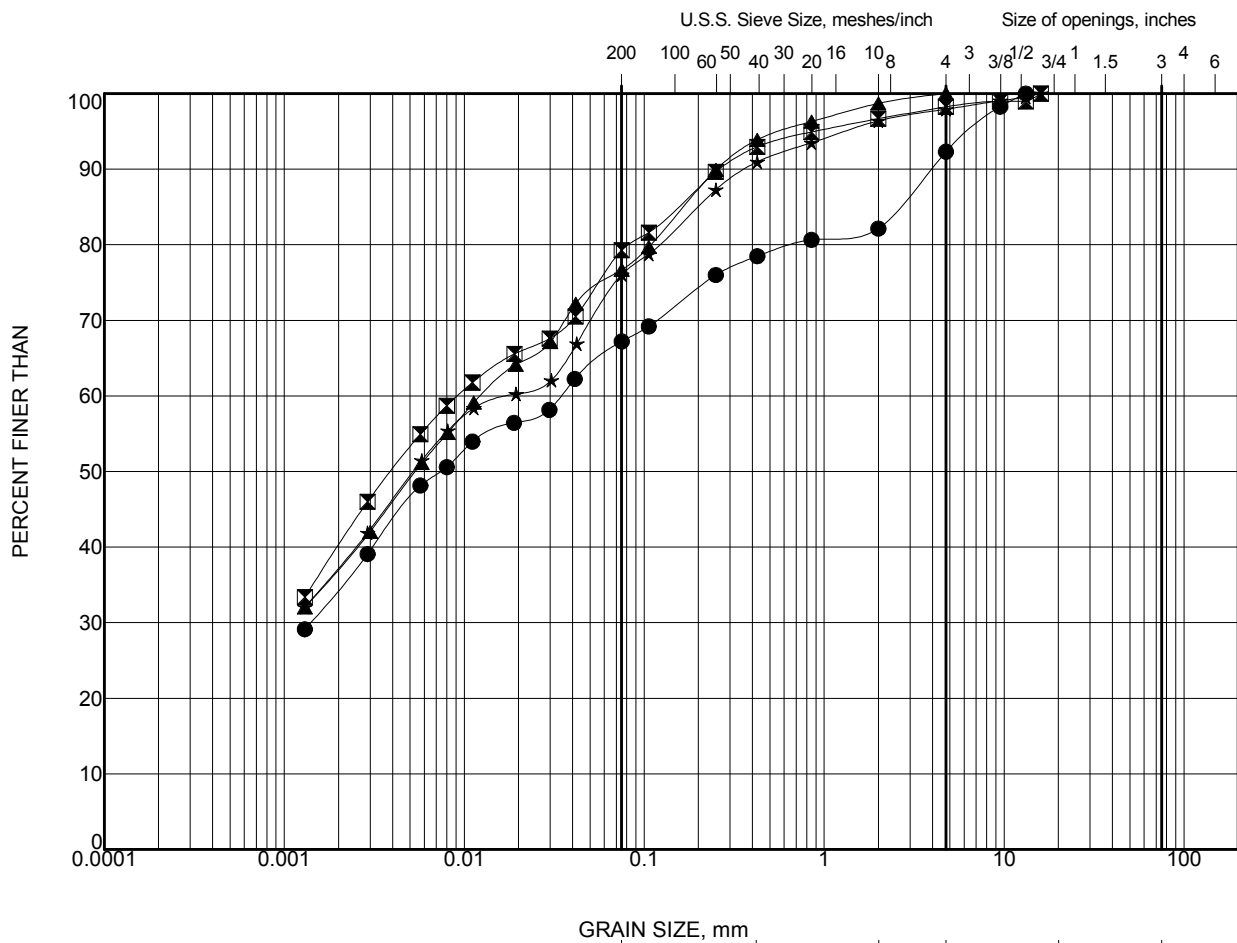
OPERATOR: TA

CHECKED:

Appendix C Laboratory Test Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
Doc No.: 285380-04-119-0009 (Geocres No. 40J3-18)

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

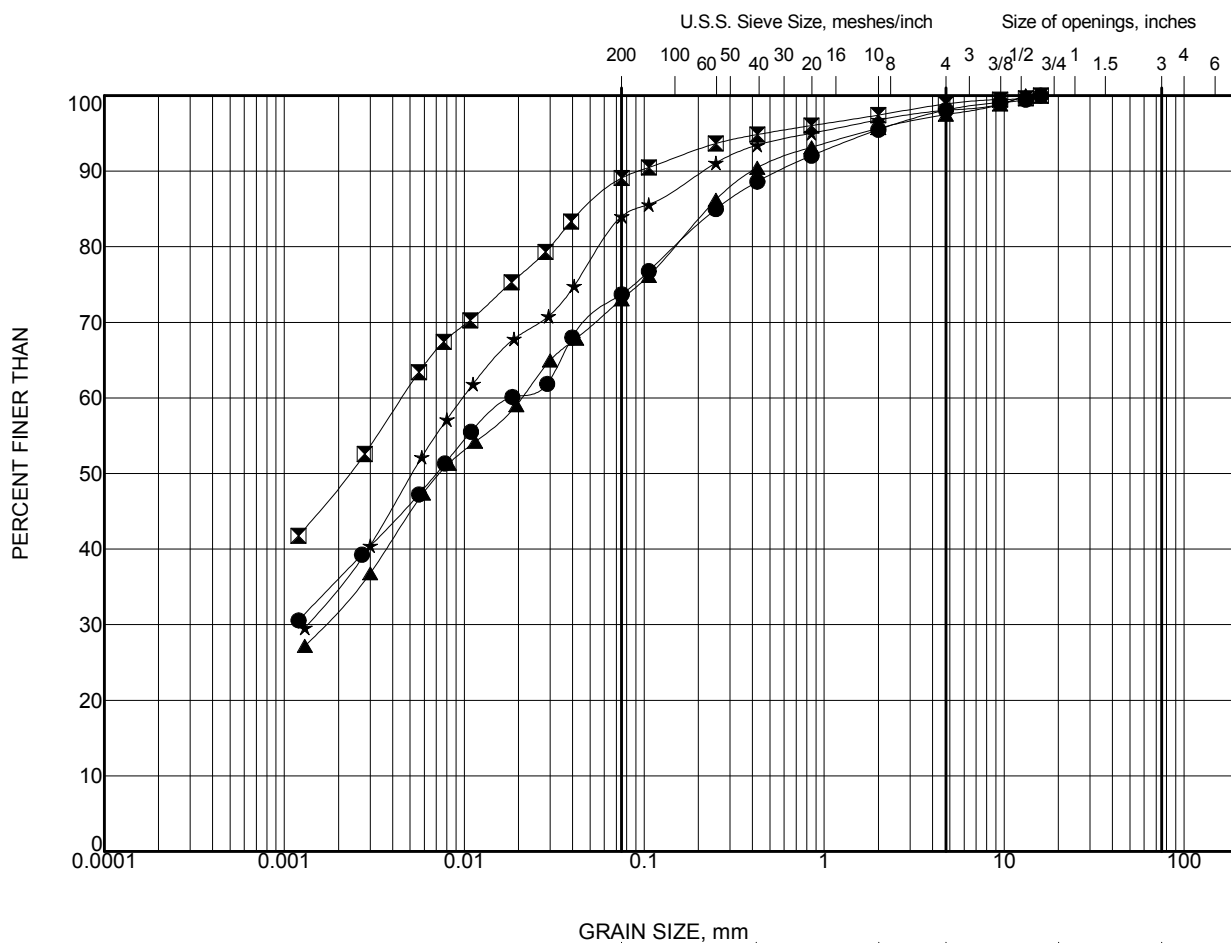


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B10-2	9	7.6
▣	B10-3	8	6.1
▲	B10-4	8	6.1
★	B10-5	11	10.7

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		GRAIN SIZE DISTRIBUTION Upper Clay Layer	
	PROJECT No.	SW8801.1004.101	FILE No.
	DRAWN	SS	SCALE
	CHECK	GN	REV.
FIGURE 1			

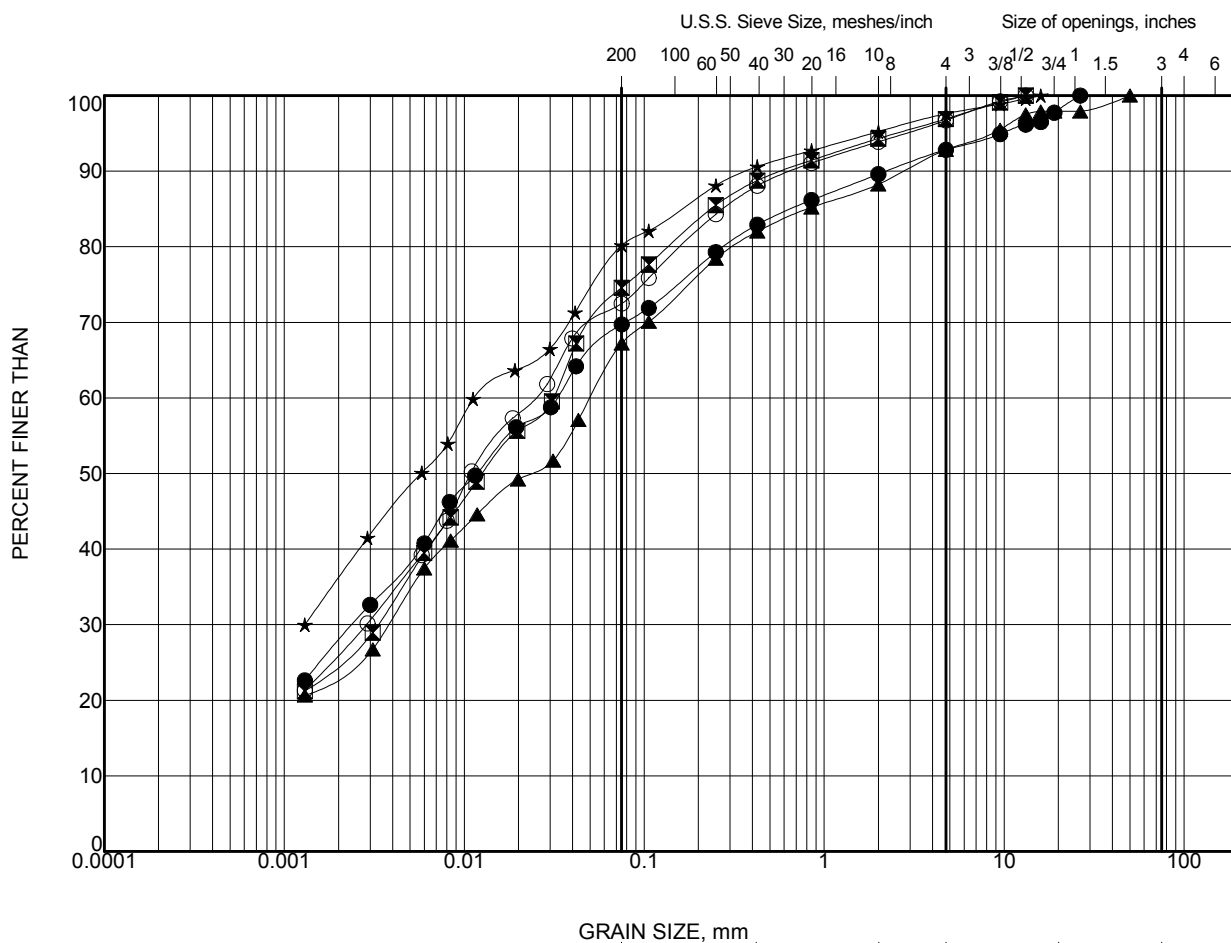


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B10-6	9	7.6
■	B10-6	13	13.7
▲	B10-7	9	7.6
★	B10-7	13	13.7

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		GRAIN SIZE DISTRIBUTION Upper Clay Layer	
	PROJECT No. SW8801.1004.101		FILE No.
	DRAWN	SS	SCALE
	CHECK	GN	REV.
FIGURE 2			



LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B10-2	19	22.9
⊠	B10-3	22	27.4
▲	B10-4	16	18.3
★	B10-4	22	27.4
○	B10-5	17	19.8

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.

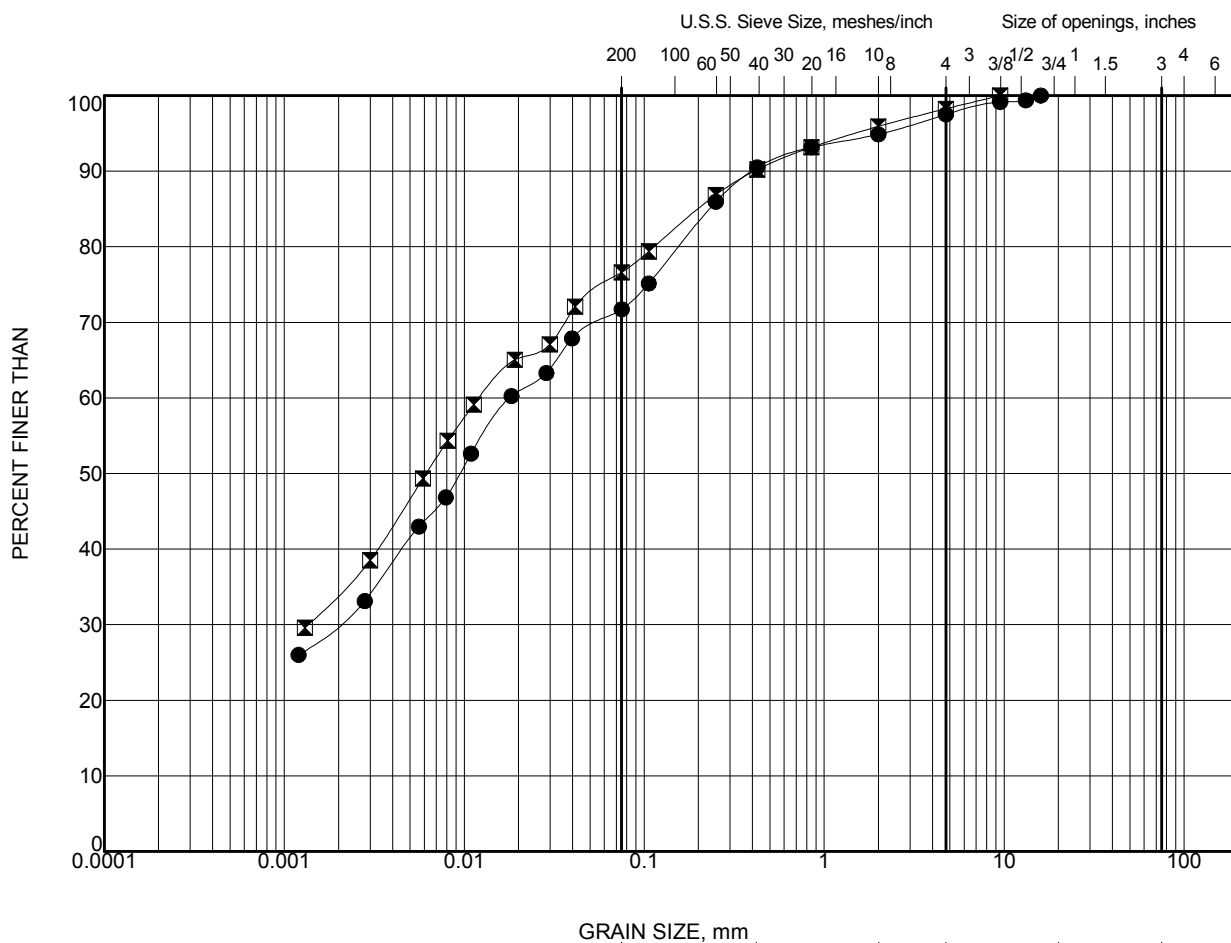
DRAWN SS

SCALE

REV.

CHECK GN

FIGURE 3

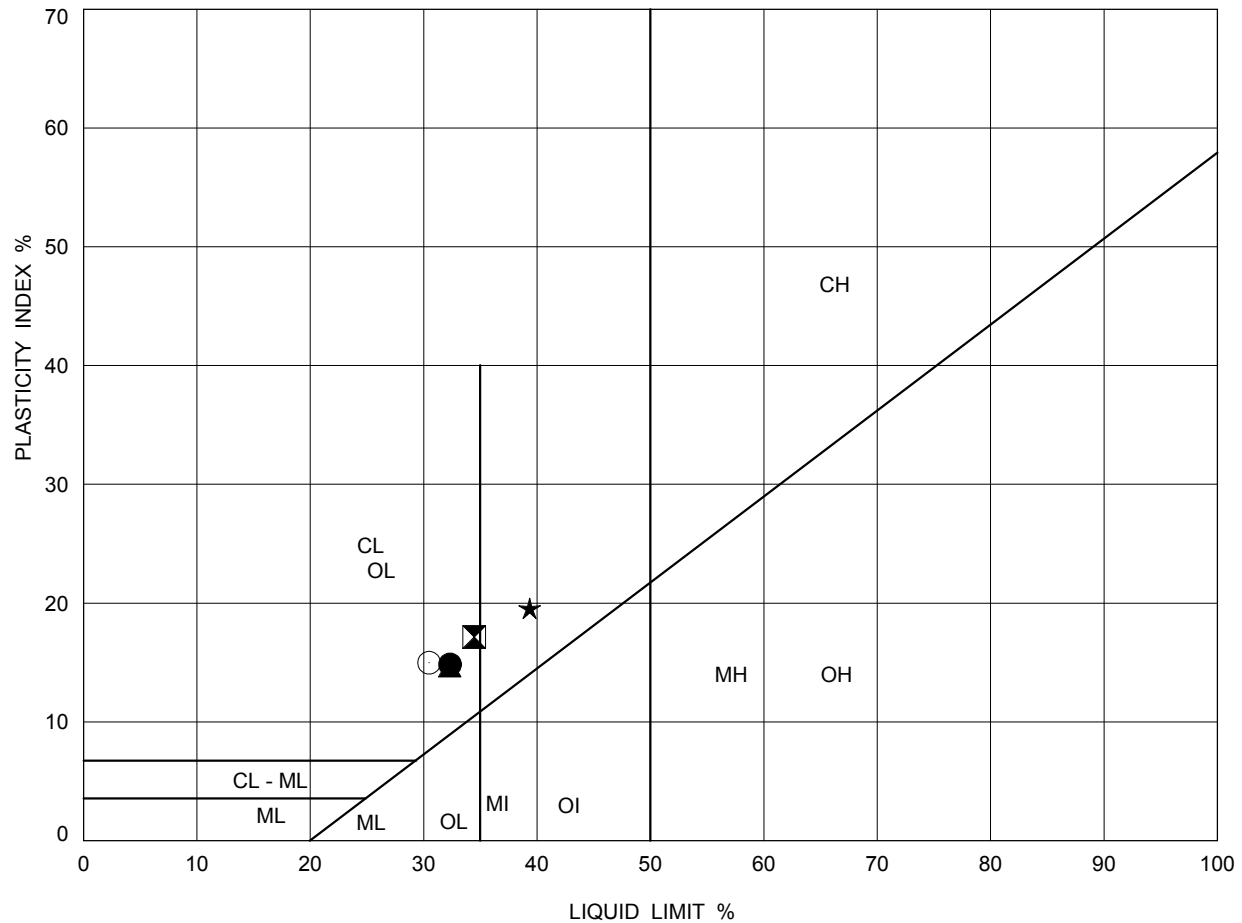


CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B10-6	17	19.8
×	B10-7	20	22.9

PROJECT		Windsor Essex Parkway (WEP)	
		Windsor, Ontario	
TITLE		GRAIN SIZE DISTRIBUTION	
		Lower Clay Layer	
	PROJECT No.	SW8801.1004.101	FILE No.
	DRAWN	SS	SCALE
	CHECK	GN	REV.
FIGURE			4



SOIL TYPE

C = Clay
M = Silt
O = Organic

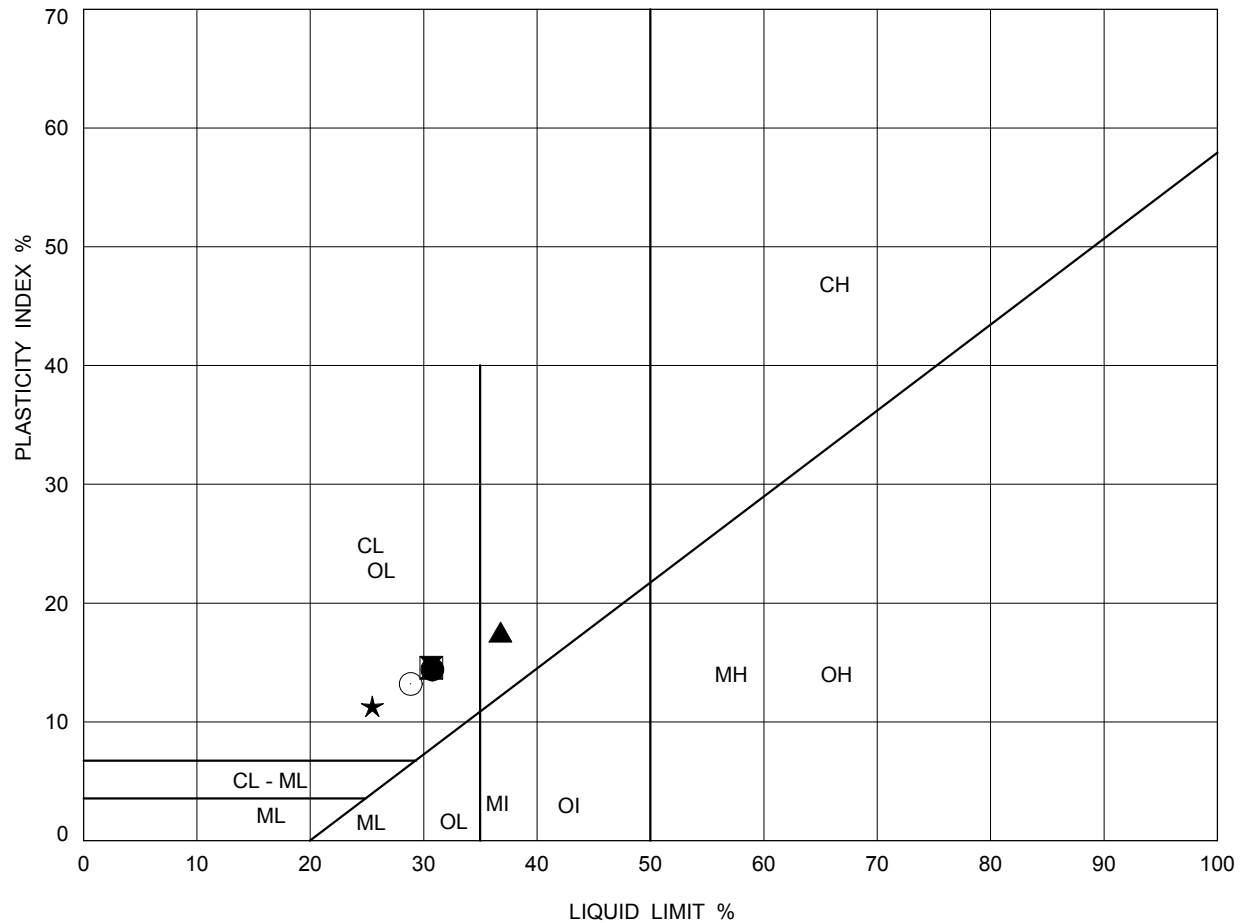
PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B10-2	9	7.6	32	17	15
⊠	B10-3	8	6.1	34	17	17
▲	B10-5	11	10.7	32	18	14
★	B10-5	13	13.7	39	20	19
○	B10-6	7	5.3	30	15	15

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART Upper Clay Layer	
	PROJECT No. SW8801.1004.101		FILE No.
	DRAWN	SS	SCALE
CHECK	GN		REV.
FIGURE 5			





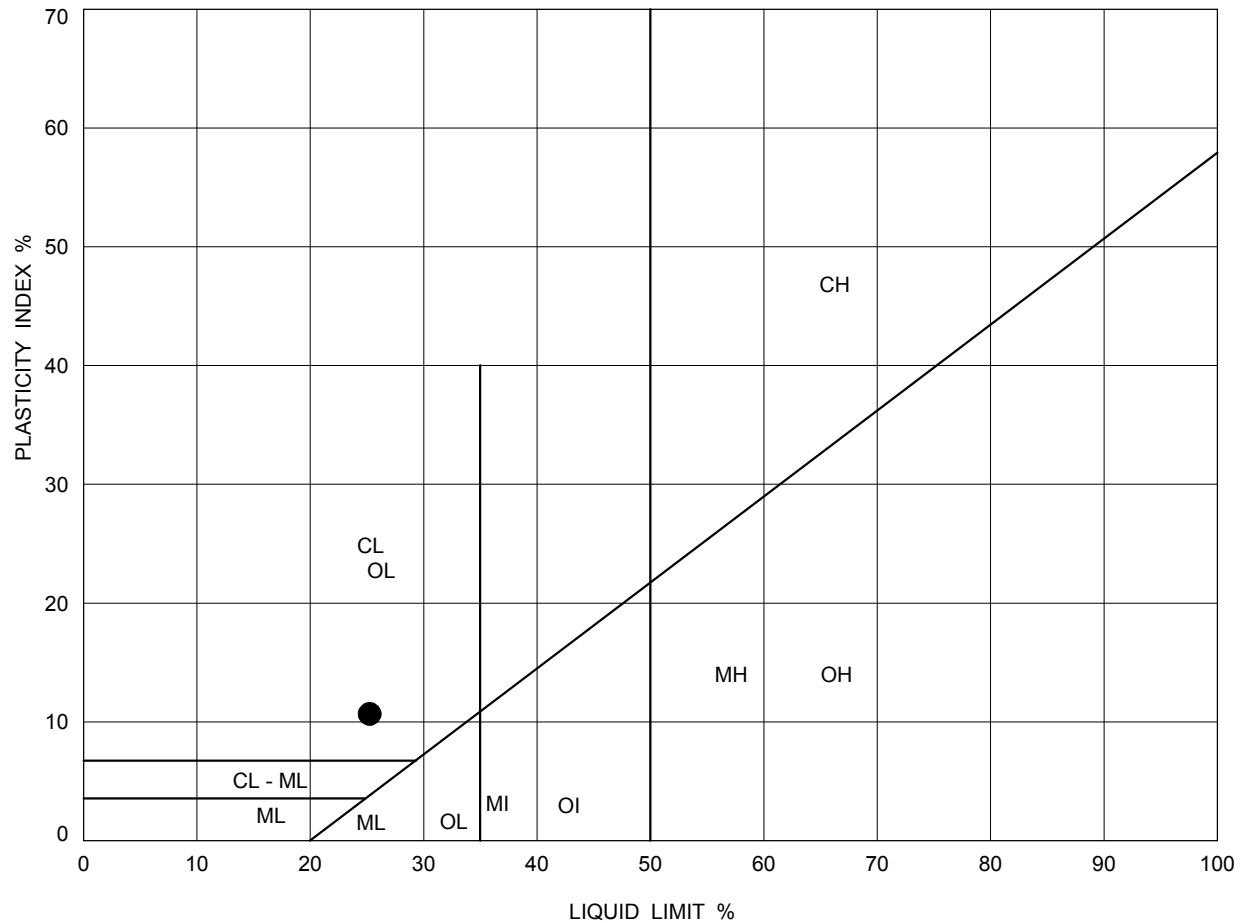
SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B10-6	9	7.6	31	16	15
⊠	B10-6	11	10.7	31	16	15
▲	B10-6	13	13.7	37	19	18
★	B10-6	15	16.8	25	14	11
○	B10-7	9	7.6	29	16	13

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART Upper Clay Layer	
 		PROJECT No. SW8801.1004.101	FILE No.
DRAWN	SS	SCALE	REV.
CHECK	GN	FIGURE 6	





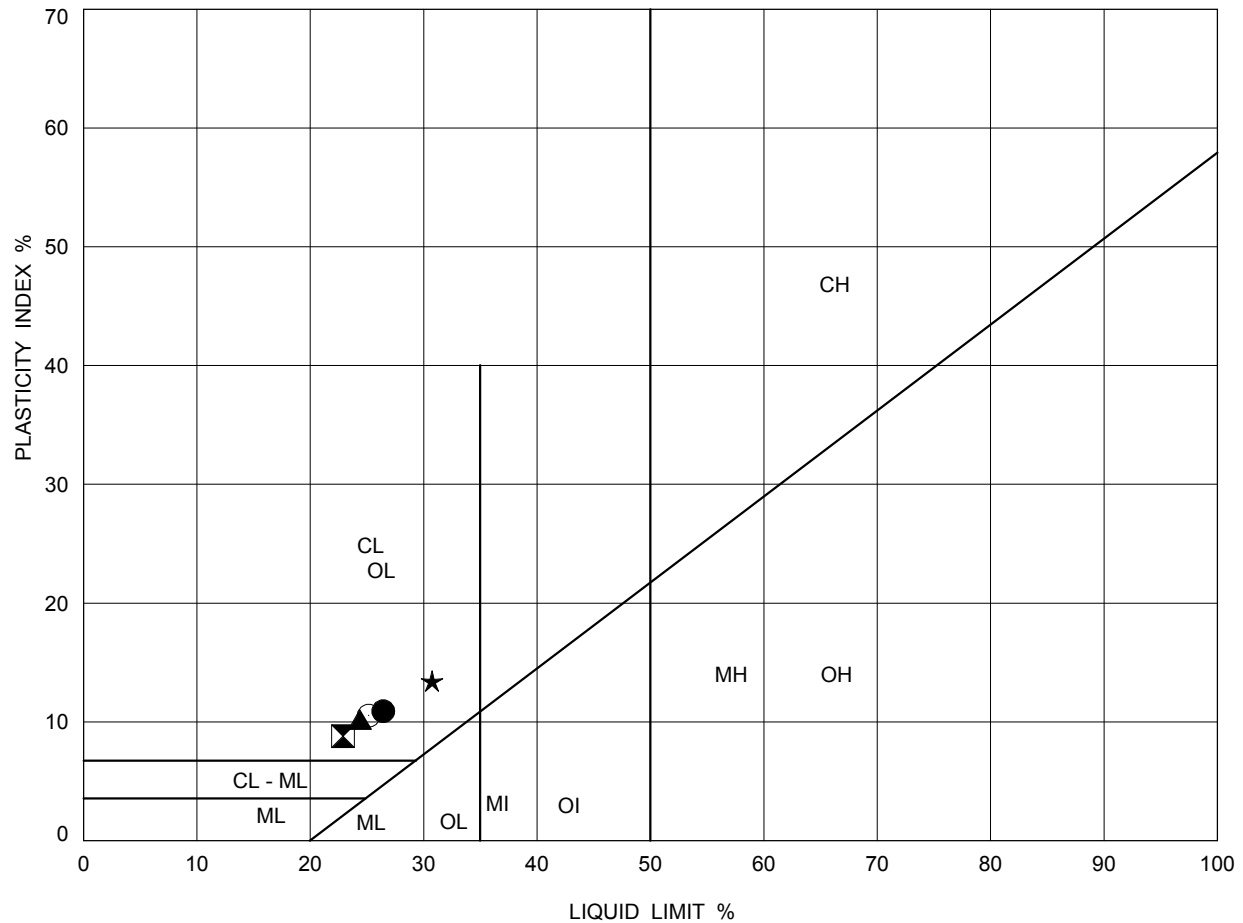
SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B10-7	13	13.7	25	15	10

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario			
TITLE		PLASTICITY CHART Upper Clay Layer			
  <small>Hatch Mott MacDonald</small>	PROJECT No. SW8801.1004.101		FILE No.		
			SCALE	REV.	
	DRAWN	SS	FIGURE 7		
	CHECK	GN			



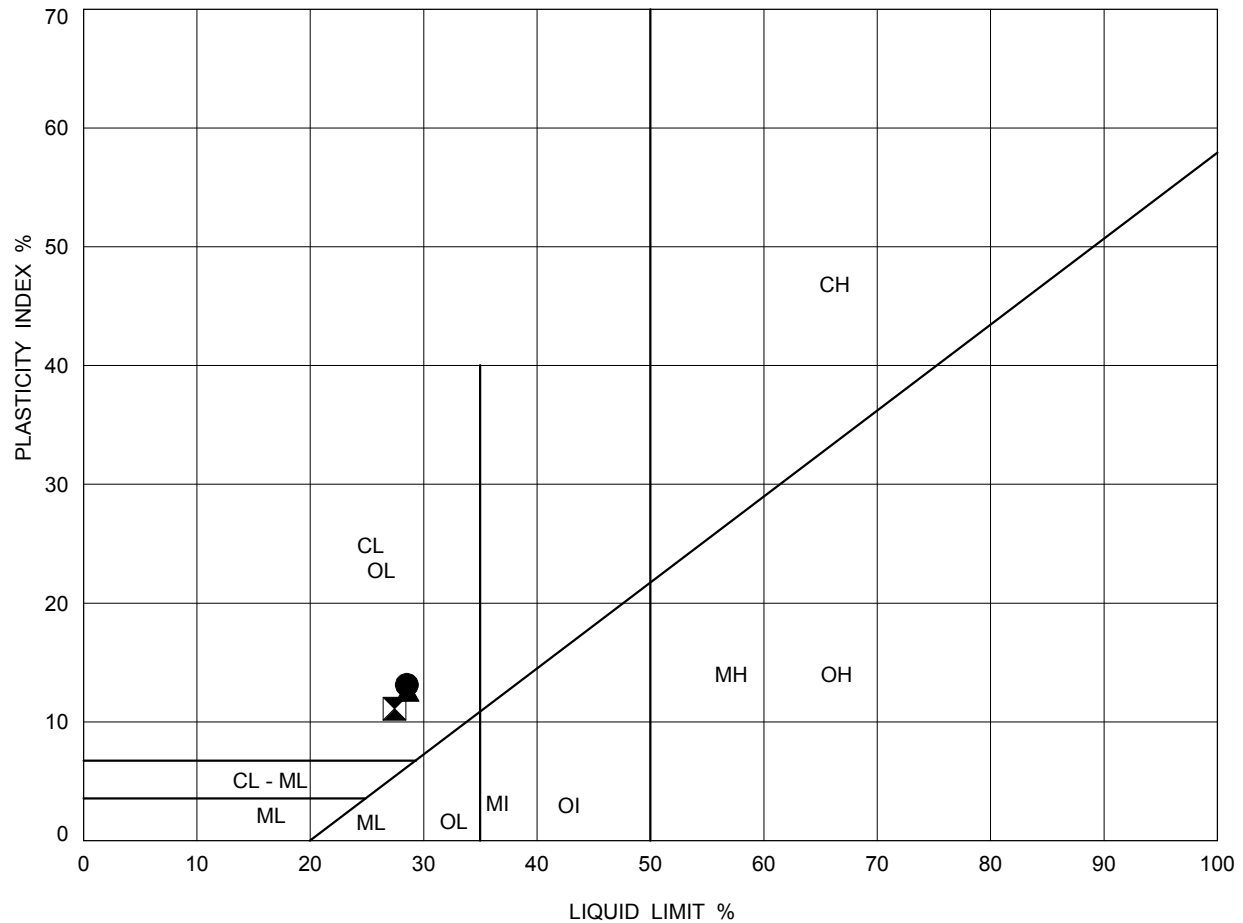
SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B10-2	19	22.9	26	16	10
⊠	B10-3	22	27.4	23	14	9
▲	B10-4	16	18.3	24	14	10
★	B10-4	22	27.4	31	17	14
○	B10-5	17	19.8	25	15	10

PROJECT				Windsor Essex Parkway (WEP) Windsor, Ontario			
TITLE				PLASTICITY CHART Lower Clay Layer			
		PROJECT No. SW8801.1004.101		FILE No.			
		DRAWN	SS	SCALE		REV.	
		CHECK	GN	FIGURE 8			


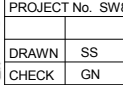


SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND:

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

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART Lower Clay Layer	
  	PROJECT No. SW8801.1004.101		FILE No.
	DRAWN	SS	SCALE
CHECK	GN		REV.
FIGURE 9			

Appendix D Analytical Laboratory Test Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
Doc No.: 285380-04-119-0009 (Geocres No. 40J3-18)

Date: August/2012
Rev: 0
Page No.: Appendix D



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

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

Client Phone: 519-735-2499

Certificate of Analysis

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

Gayle Braun
Senior Account Manager

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ADDRESS: 309 Exeter Road Unit #29, London, ON N6L 1C1 Canada | Phone: +1 519 652 6044 | Fax: +1 519 652 0671
ALS CANADA LTD Part of the ALS Group A Campbell Brothers Limited Company

ALS ENVIRONMENTAL ANALYTICAL REPORT

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

Reference Information

Test Method References:

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

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

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

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

Chain of Custody Numbers:

092978

GLOSSARY OF REPORT TERMS

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

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

mg/kg 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: L1012450

Report Date: 09-JUN-11

Page 1 of 3

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

Contact: Brian Lapos

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

Quality Control Report

Workorder: L1012450

Report Date: 09-JUN-11

Page 2 of 3

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Workorder: L1012450

Report Date: 09-JUN-11

Page 3 of 3

Hold Time Exceedances:

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

Legend & Qualifier Definitions:

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

Notes*:

Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1012450 were received on 03-JUN-11 10:45.

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

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

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



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

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

Client Phone: 519-735-2499

Certificate of Analysis

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

Gayle Braun
Senior Account Manager

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ADDRESS: 309 Exeter Road Unit #29, London, ON N6L 1C1 Canada | Phone: +1 519 652 6044 | Fax: +1 519 652 0671
ALS CANADA LTD Part of the ALS Group A Campbell Brothers Limited Company

ALS ENVIRONMENTAL ANALYTICAL REPORT

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

Reference Information

Test Method References:

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

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

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

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

Chain of Custody Numbers:

092732-6

GLOSSARY OF REPORT TERMS

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

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

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

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

mg/L - milligrams per litre.

< - Less than.

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

N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.

UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.

Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Quality Control Report

Workorder: L1025377

Report Date: 08-JUL-11

Page 1 of 3

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

Contact: Brian Lapos

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

Quality Control Report

Workorder: L1025377

Report Date: 08-JUL-11

Page 2 of 3

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Workorder: L1025377

Report Date: 08-JUL-11

Page 3 of 3

Hold Time Exceedances:

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

Legend & Qualifier Definitions:

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

Notes*:

Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1025377 were received on 30-JUN-11 11:00.

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

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

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



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

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

Client Phone: 519-735-2499

Certificate of Analysis

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

Gayle Braun
Senior Account Manager

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

ALS ENVIRONMENTAL ANALYTICAL REPORT

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

Reference Information

Test Method References:

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

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

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

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

Chain of Custody Numbers:

092982

GLOSSARY OF REPORT TERMS

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

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

mg/kg 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: L1018322

Report Date: 22-JUN-11

Page 1 of 3

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

Contact: Brian Lapos

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

Quality Control Report

Workorder: L1018322

Report Date: 22-JUN-11

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Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Workorder: L1018322

Report Date: 22-JUN-11

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Hold Time Exceedances:

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

Legend & Qualifier Definitions:

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

Notes*:

Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1018322 were received on 16-JUN-11 10:15.

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

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

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

Appendix E Stress-Deformation Analyses

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
Doc No.: 285380-04-119-0009 (Geocres No. 40J3-18)

Date: August/2012
Rev: 0
Page No.: Appendix E

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

Submerged Culvert S-2
LongTerm
Last Solved Date: 8/15/2012

Name: ClayCrust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Crust Vol. WC. Function: Volumetric WC_Crust Load Response Ratio: 1
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'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Transition Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.65 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.008 Initial Void Ratio: 0.55 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(LC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: ClayBackfill (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 ° Vol. WC. Function: Volumetric WC_Sand
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: 1.2 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1

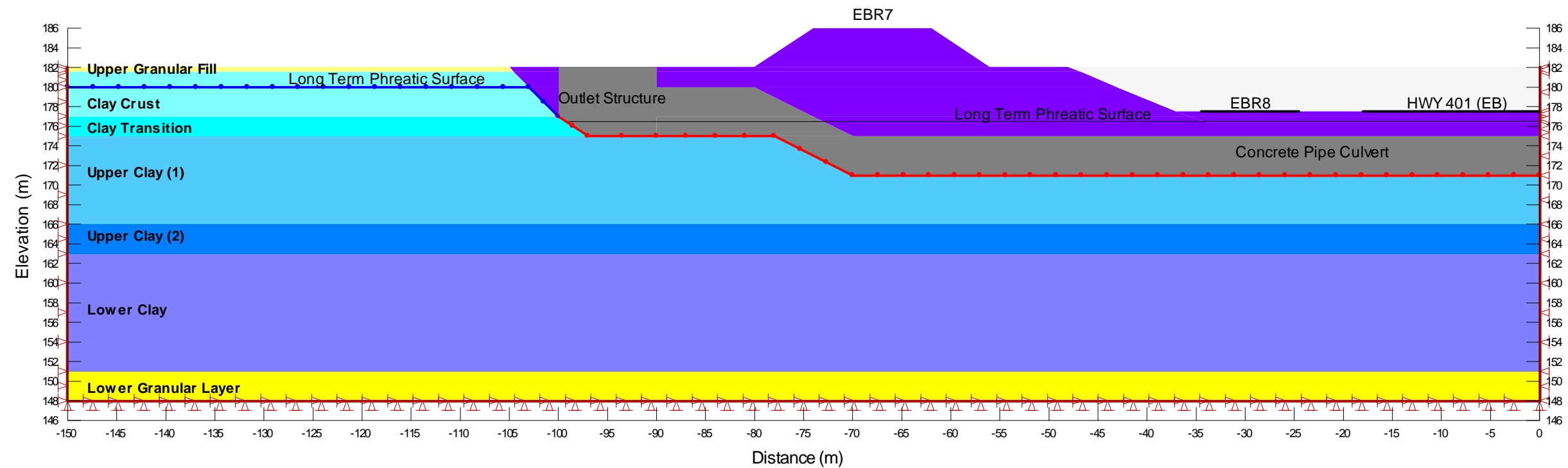


Figure E-2: Sigma/W Model (Along Culvert) – End of Highway 401 Excavation

Submerged Culvert S-2
Excav.401
Last Solved Date: 8/15/2012

Name: ClayCrust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Crust Vol. WC. Function: Volumetric WC_Crust Load Response Ratio: 1
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: ClayTransition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Transition Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.65 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.008 Initial Void Ratio: 0.55 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(LC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
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 ° Vol. WC. Function: Volumetric WC_Sand
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1

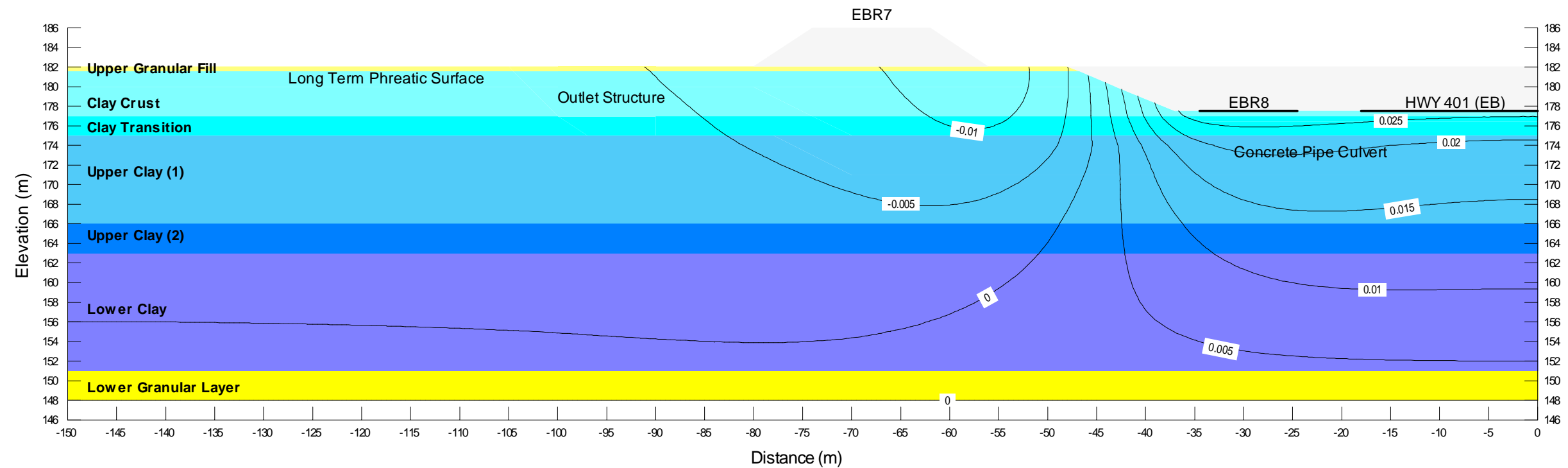


Figure E-3: Cumulative Heave/Settlement – End of Construction

Submerged Culvert S-2
EBR7
Last Solved Date: 8/15/2012

Name: ClayCrust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Crust Vol. WC. Function: Volumetric WC_Crust Load Response Ratio: 1
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'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Transition Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.65 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.008 Initial Void Ratio: 0.55 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(LC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: ClayBackfill (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 ° Vol. WC. Function: Volumetric WC_Sand
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: 1.2 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1

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

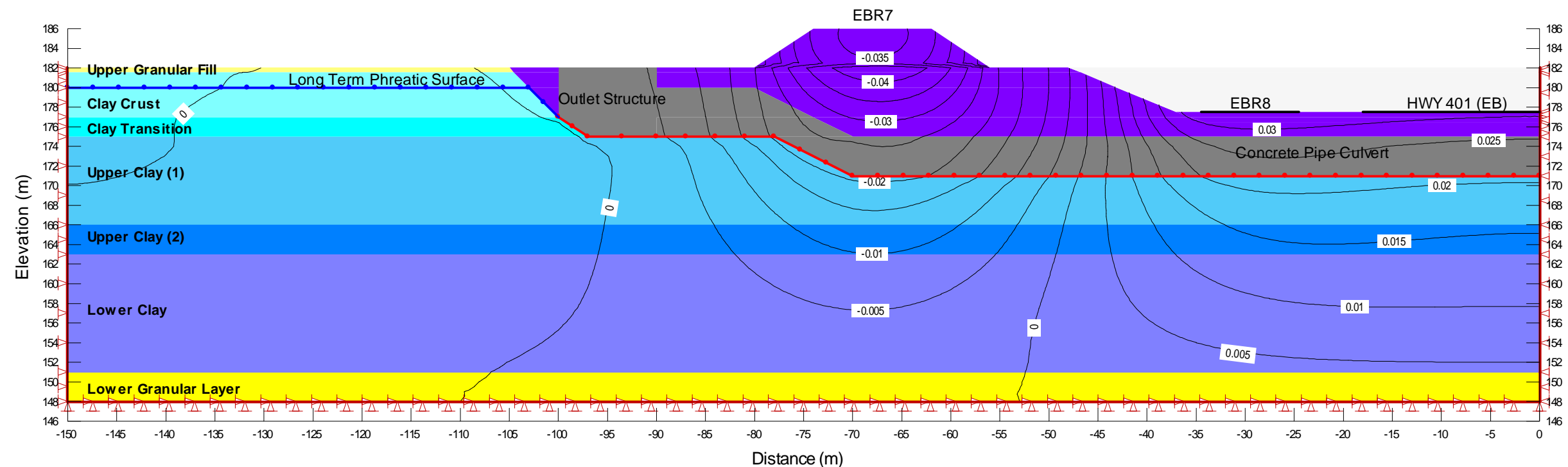


Figure E-4: Cumulative Heave/Settlement - Long-term (Drained)

Submerged Culvert S-2
LongTerm
Last Solved Date: 8/15/2012

Name: ClayCrust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Crust Vol. WC. Function: Volumetric WC_Crust Load Response Ratio: 1
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'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Transition Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.65 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.008 Initial Void Ratio: 0.55 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(LC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: ClayBackfill (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 ° Vol. WC. Function: Volumetric WC_Sand
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: 1.2 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1

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

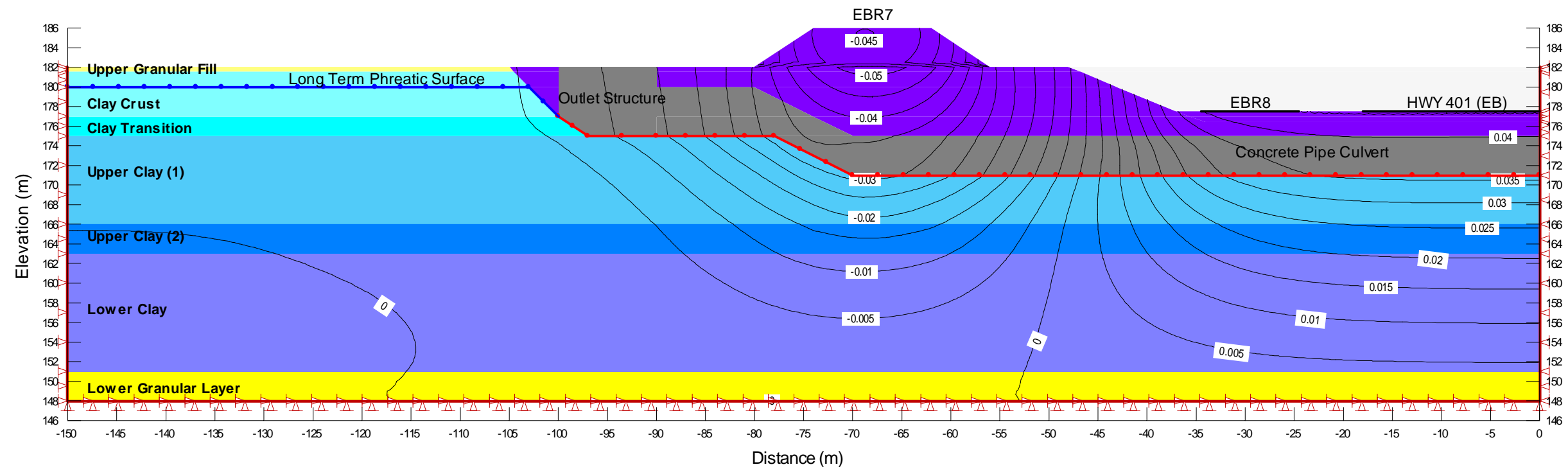


Figure E-5: Stabilized Pore Water Pressure Contours – Long-term (Drained)

Submerged Culvert S-2
LongTerm
Last Solved Date: 8/15/2012

Name: ClayCrust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Crust Vol. WC. Function: Volumetric WC_Crust Load Response Ratio: 1
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: ClayTransition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Transition Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.65 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.008 Initial Void Ratio: 0.55 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(LC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: ClayBackfill (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 ° Vol. WC. Function: Volumetric WC_Sand
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: 1.2 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1

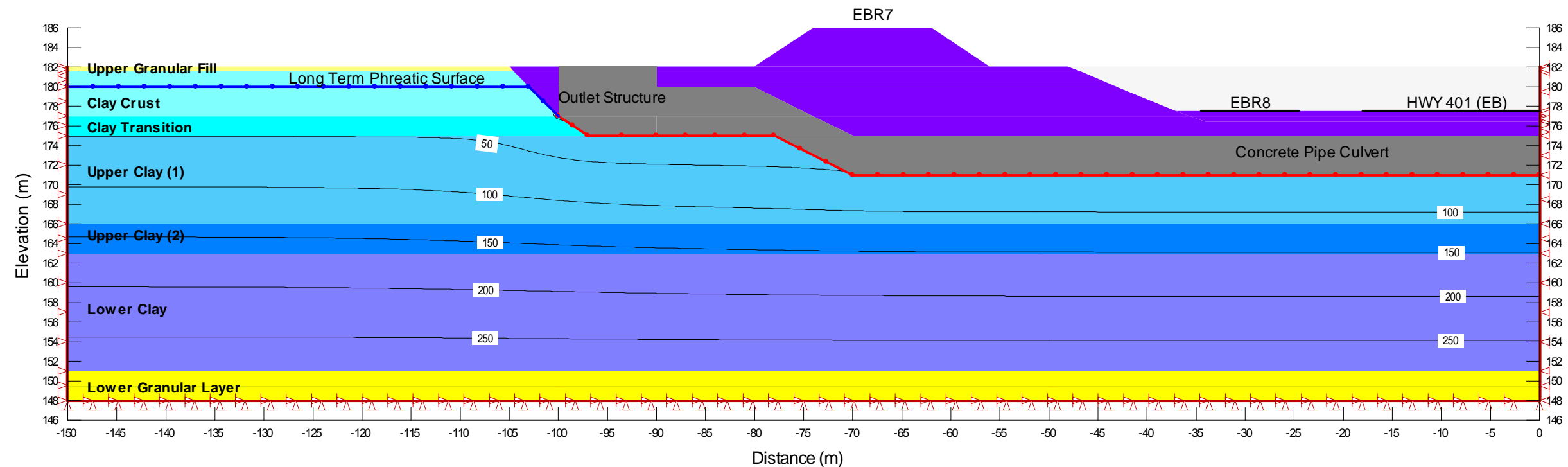


Figure E-6: Cumulative Heave/Settlement at Pipe Invert

Positive (+) Y-Displacement indicates heave movement and negative (-) Y-Displacement indicates settlement.

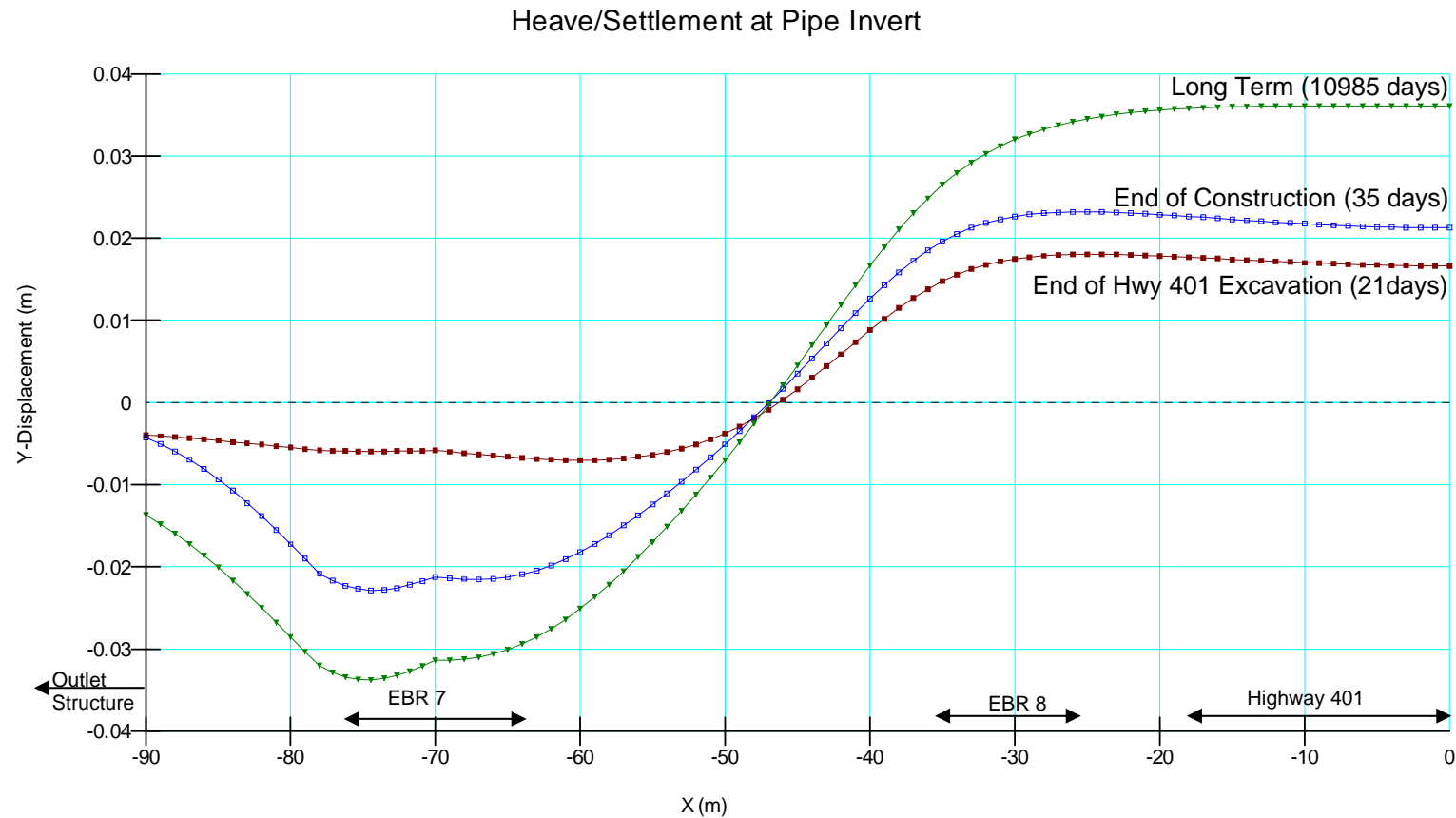


Figure E-7: Estimated Timeline for Cumulative Heave/Settlement at Pipe Invert

28 days = End of Construction

14600 days = Long-term Condition

The positive (+) Y-Displacement indicates heave movement and negative (-) Y-Displacement indicates settlement.

Heave/Settlement along Pipe base profile

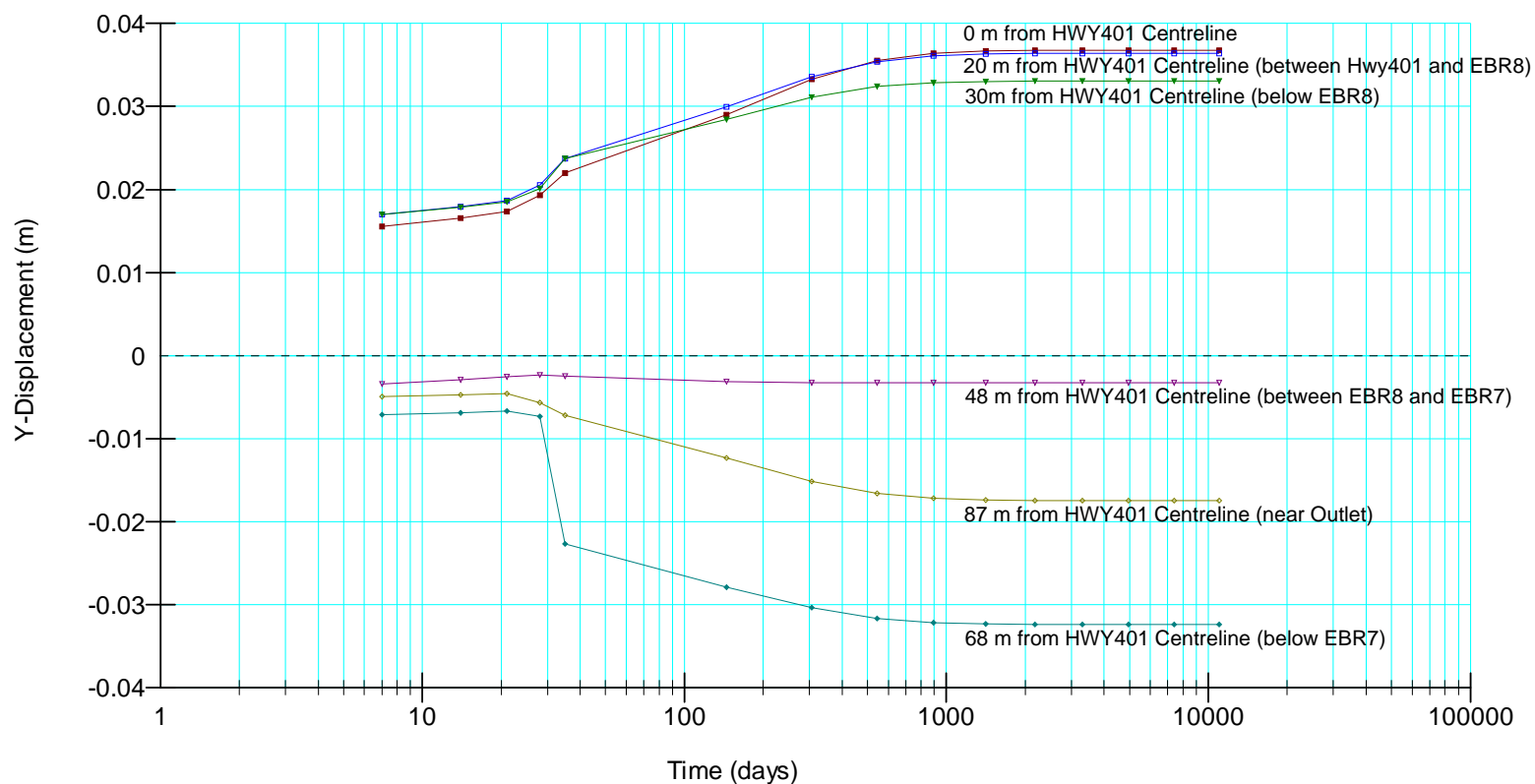


Figure E-8: Sigma/W Model for Culvert Construction (short-term conditions) at EBR7 Centreline (before completion of EBR7 Embankment)

Submerged Culvert S-2
BackFill
Last Solved Date: 8/14/2012

Name: ClayBackfill 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: ClayCrust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30° Unit Weight: 21 kN/m³ Dilation Angle: 0° K-Function: Conductivity_Crust Vol. WC. Function: Volumetric WC_Crust Load Response Ratio: 1
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'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30° Unit Weight: 21 kN/m³ Dilation Angle: 0° K-Function: Conductivity_Transition Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.65 Unit Weight: 20 kN/m³ Phi': 26° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.008 Initial Void Ratio: 0.55 Unit Weight: 20.5 kN/m³ Phi': 26° K-Function: Conductivity_Unweathered(LC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
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° Vol. WC. Function: Volumetric WC_Sand
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20 kN/m³ Phi': 26° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Pipe/Inlet (equivalent Sand) Model: Linear Elastic Effective Young's Modulus (E'): 100000 kPa Poisson's Ratio: 0.15 Unit Weight: 15 kN/m³ K-Function: Conductivity_Sand Vol. WC. Function: Volumetric WC_Sand Load Response Ratio: 1
Name: GranularBackfill (Eff.Stress) Model: Elastic-Plastic Effective Young's Modulus (E'): 30000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 33° Unit Weight: 22 kN/m³ Dilation Angle: 0° K-Function: Conductivity_Sand Vol. WC. Function: Volumetric WC_Sand Load Response Ratio: 1

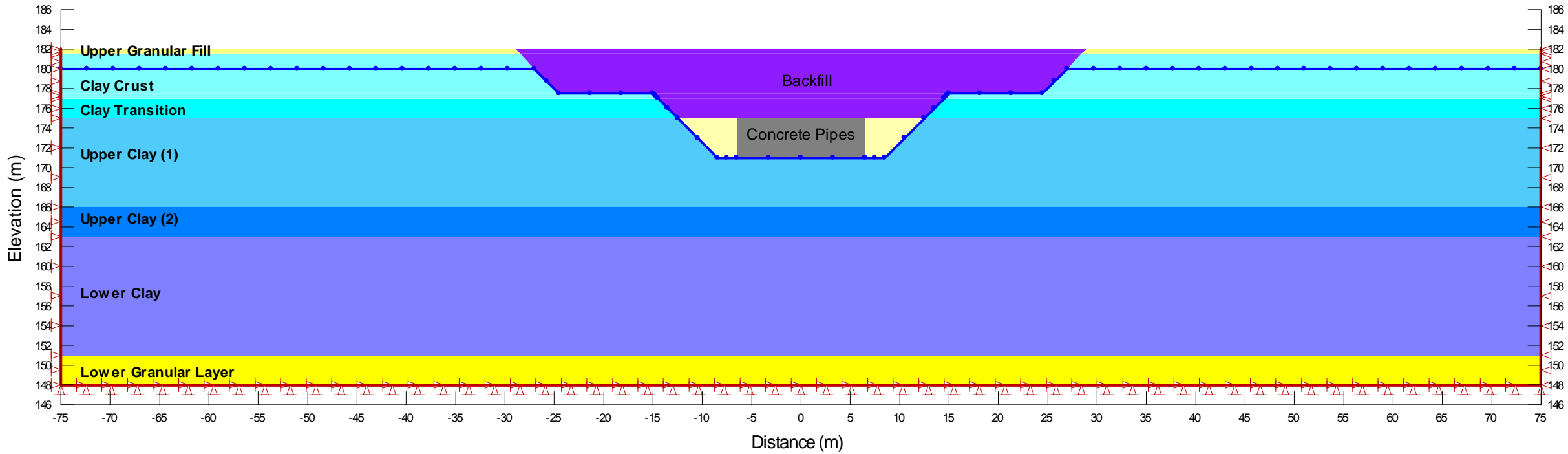


Figure E-9: Incremental short-term Heave/Settlement at End of Temporary Excavation for Culvert Construction at EBR7 Centreline (before completion of EBR7 Embankment)

Submerged Culvert S-2
Temp Excav
Last Solved Date: 8/14/2012

Name: ClayCrust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Crust Vol. WC. Function: Volumetric WC_Crust Load Response Ratio: 1
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'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Transition Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.65 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.008 Initial Void Ratio: 0.55 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(LC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
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 ° Vol. WC. Function: Volumetric WC_Sand
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1

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

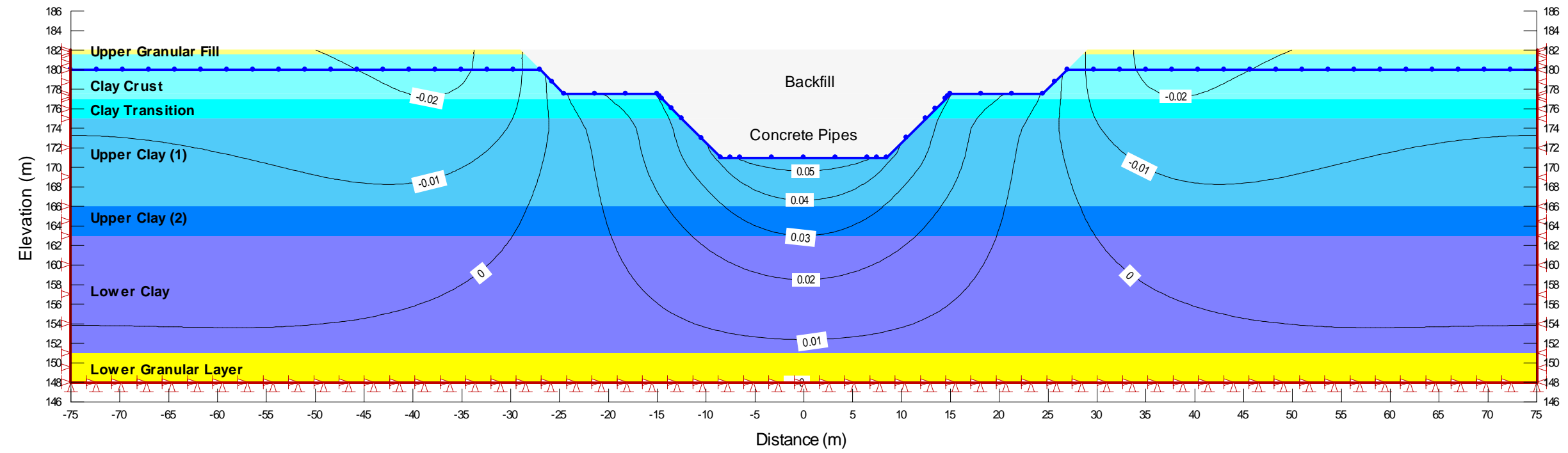


Figure E-10: Incremental short-term Heave/Settlement at End of Culvert Construction at EBR7 Centreline (before completion of EBR7 Embankment)

Submerged Culvert S-2
Backfill
Last Solved Date: 8/14/2012

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: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Crust Vol. WC. Function: Volumetric WC_Crust Load Response Ratio: 1
Name: Lower Granular Layer 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'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Transition Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.65 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.008 Initial Void Ratio: 0.55 Unit Weight: 20.5 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(LC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Granular Fill 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 ° Vol. WC. Function: Volumetric WC_Sand
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20 kN/m³ Phi': 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Pipe/Inlet (equivalent Sand) Model: Linear Elastic Effective Young's Modulus (E'): 100000 kPa Poisson's Ratio: 0.15 Unit Weight: 15 kN/m³ K-Function: Conductivity_Sand Vol. WC. Function: Volumetric WC_Sand Load Response Ratio: 1
Name: Granular Backfill (Eff. Stress) Model: Elastic-Plastic Effective Young's Modulus (E'): 30000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 33 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Sand Vol. WC. Function: Volumetric WC_Sand Load Response Ratio: 1

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

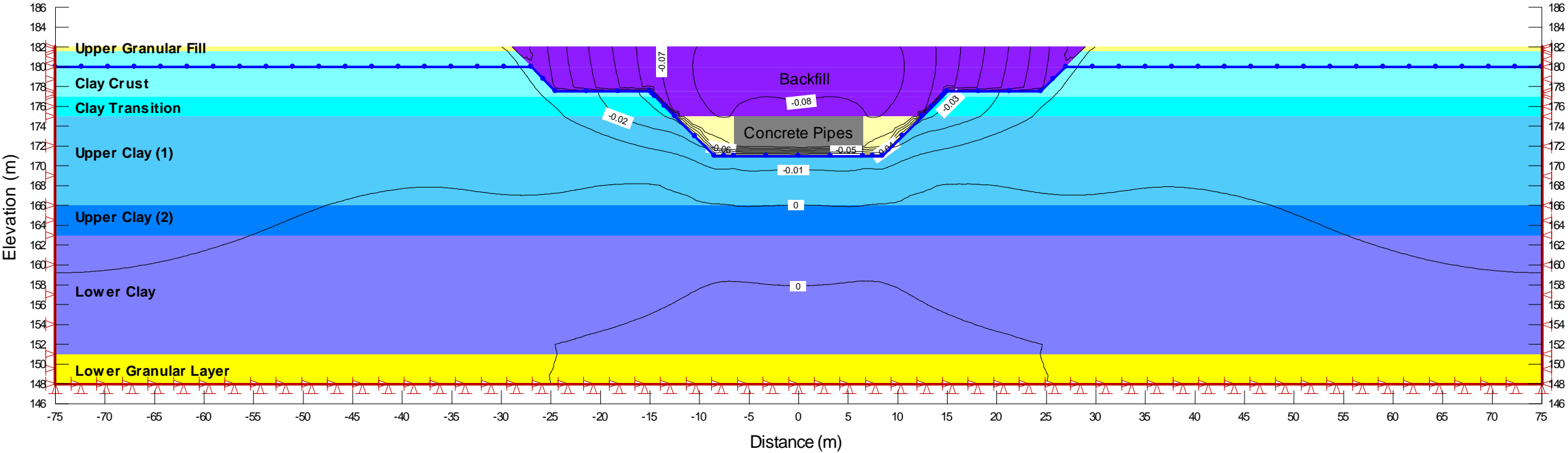


Figure E-13: Incremental short-term Heave/Settlement at End of Culvert Construction at Highway 401 Centreline (after Excavation for Highway 401)

Submerged Culvert S-2
Backfill
Last Solved Date: 8/14/2012

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: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Crust Vol. WC. Function: Volumetric WC_Crust Load Response Ratio: 1
Name: Lower Granular Layer 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'): 18000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Transition Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (1) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.65 Unit Weight: 20 kN/m³ Phi: 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Lower Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.008 Initial Void Ratio: 0.55 Unit Weight: 20.5 kN/m³ Phi: 26 ° K-Function: Conductivity_Unweathered(LC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Upper Clay (2) (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 20 kN/m³ Phi: 26 ° K-Function: Conductivity_Unweathered(UC) Vol. WC. Function: Volumetric WC_Unweathered Load Response Ratio: 1
Name: Pipe/Inlet (equivalent Sand) Model: Linear Elastic Effective Young's Modulus (E'): 100000 kPa Poisson's Ratio: 0.15 Unit Weight: 15 kN/m³ K-Function: Conductivity_Sand Vol. WC. Function: Volumetric WC_Sand Load Response Ratio: 1
Name: Granular Backfill (Eff.Stress) Model: Elastic-Plastic Effective Young's Modulus (E'): 30000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 33 ° Unit Weight: 22 kN/m³ Dilation Angle: 0 ° K-Function: Conductivity_Sand Vol. WC. Function: Volumetric WC_Sand Load Response Ratio: 1

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

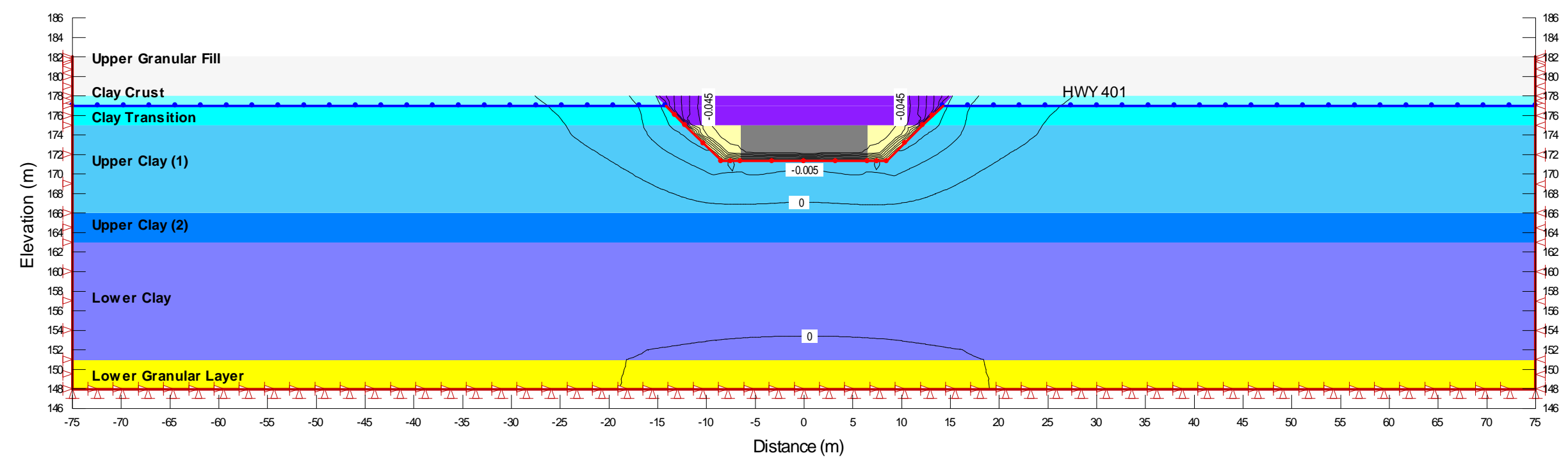
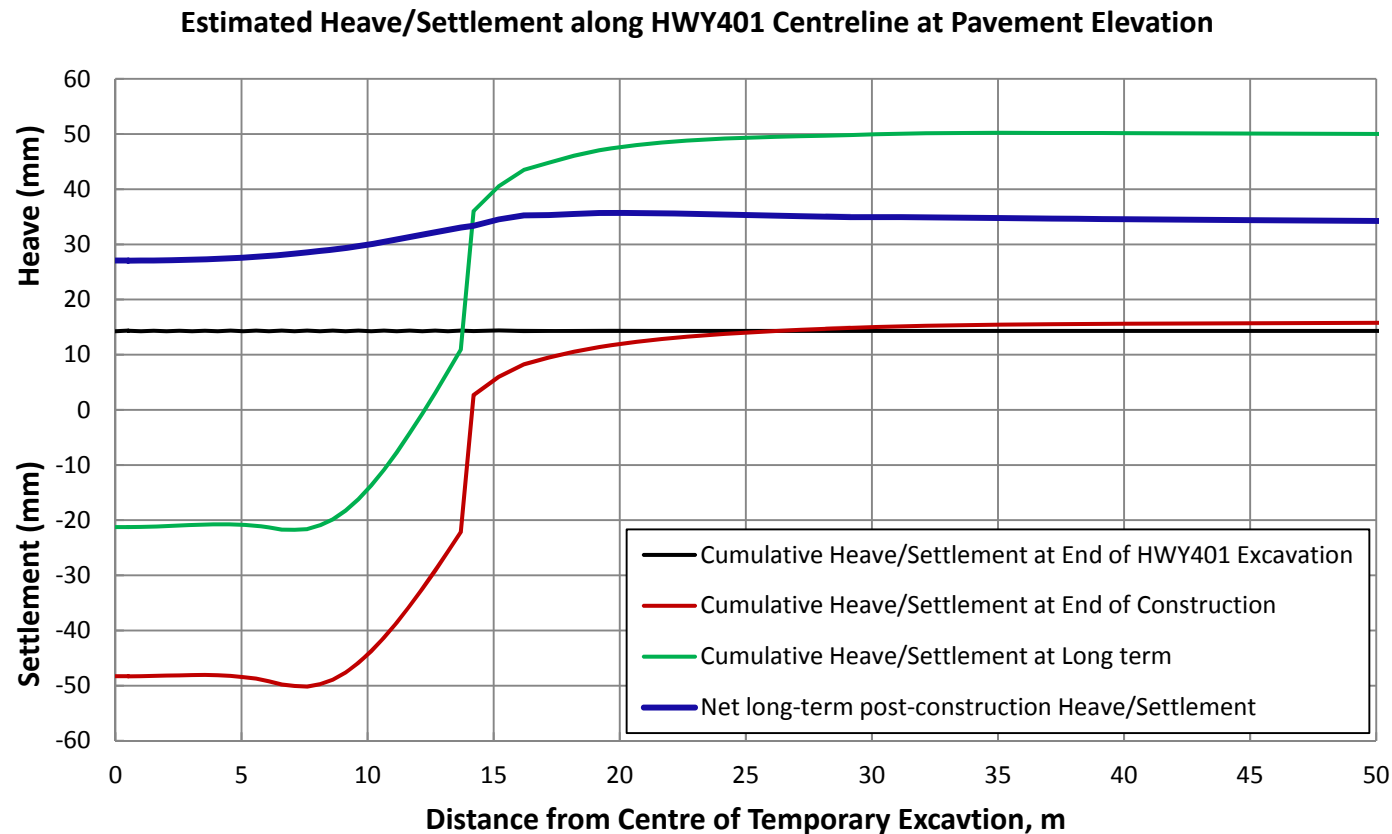


Figure E-14: Estimated Heave/Settlement along Highway 401 Centreline at Pavement Elevation



Appendix F Rock Core Photographs

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Submerged Culvert S-2 (Cahill Drain, Sta. 11+175 LaSalle)
Doc No.: 285380-04-119-0009 (Geocres No. 40J3-18)

Date: August/2012
Rev: 0
Page No.: Appendix F

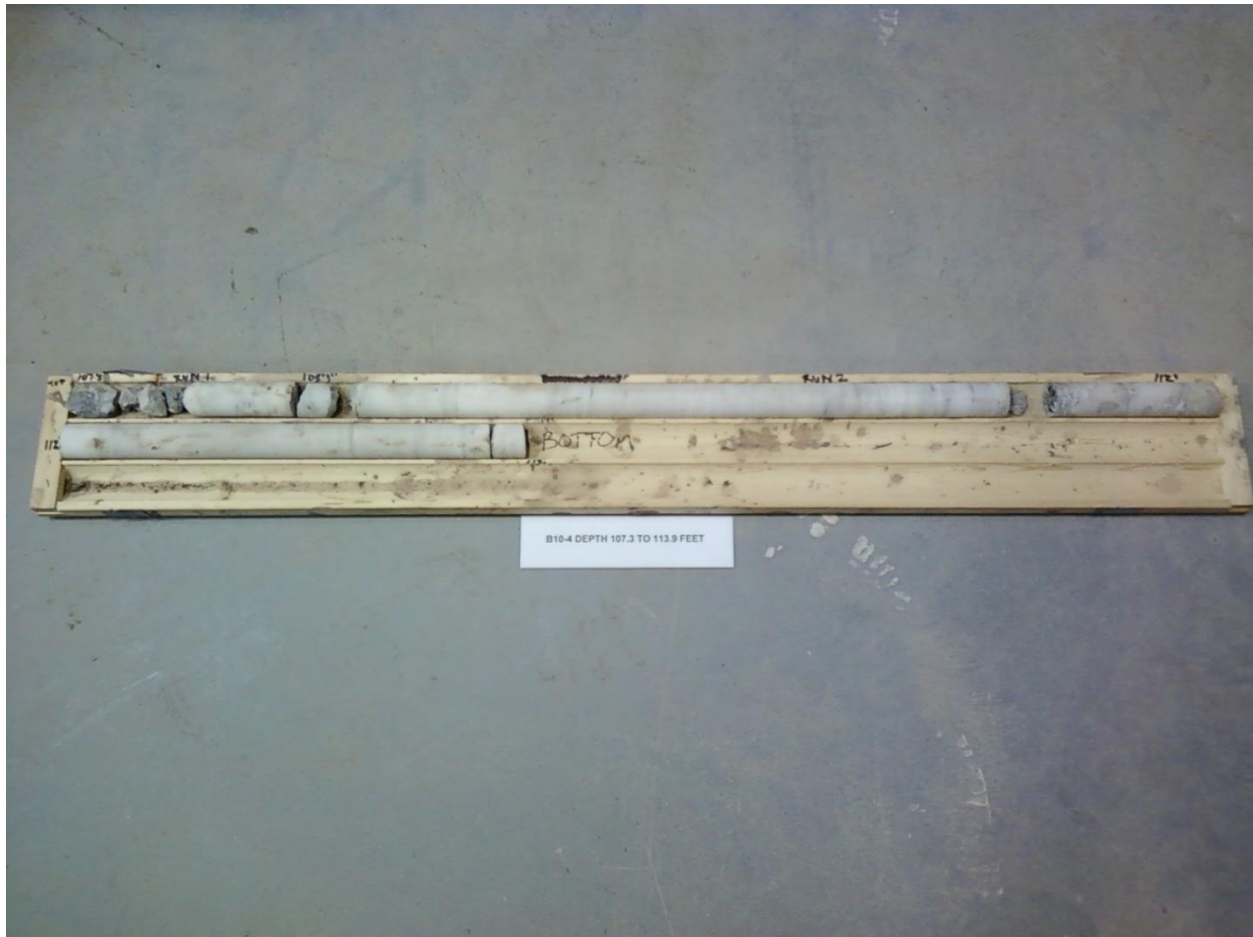
Photograph F-1: Borehole B10-2 – Rock Core Elevation 148.8 to 147.2 m



Photograph F-2: Borehole B10-3 – Rock Core Elevation 149.8 to 147.8 m



Photograph F-3: Borehole B10-4 – Rock Core Elevation 149.7 to 147.7 m



Photograph F-4: Borehole B10-5 – Rock Core Elevation 149.6 to 146.1 m



Photograph F-5: Borehole B10-5 – Rock Core Elevation 146.1 to 142.6 m



Photograph F-6: Borehole B10-5 – Rock Core Elevation 142.6 to 139.5 m



Photograph F-7: Borehole B10-6 – Rock Core Elevation 149.7 to 147.2 m



Photograph F-8: Borehole B10-7 – Rock Core Elevation 149.6 to 146.5 m

