The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report – Bridge B-9

(Hwy. 3 East Ramp Underpass at Hwy. 401 Sta. 9+549L to 9+676L, 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 (PIC). This team includes Dragados Canada, Inc., Acciona Infrastructure Canada Inc., and Fluor Canada Ltd. This combination brings a wide range of local and international experience to the project.

1.2 Report Introduction

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, and other structures.

This report presents the geotechnical design of Bridge B-9 located at Highway 401 Sta. 9+549L to 9+676L in the LaSalle sector of the proposed Windsor-Essex Parkway (WEP) project. Bridge B-9 will carry traffic of Highway 3 East Bound Exit Ramp (EBR8) over Highway 401 with one 4.75 m wide lane as shown on Drawing 285380-03-060-SEG1-0901. Several retaining walls (HRW-21L, MSHP-22L, MSHP-23R, MSEW-22L and MSEW-23R) are situated in the immediate vicinity of the Bridge B-9 abutments as components of the approachway embankments and slope stabilization around the abutments.

The report includes the results of the additional geotechnical investigation carried out to support the design available at the time of preparation of this report and other relevant background information. The report includes the results of the additional geotechnical investigation carried out to support the design (i.e., the layout and configuration) available at the time of preparation of this report and addresses review comments from peer reviews and MTO. This is the final report and is issued for construction (IFC).

The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines, and the Parkway Infrastructure Constructors (PIC). The WEMG proposal design for Bridge B-9 incorporated a single cell trapezoidal reinforced concrete box structure with true integral abutments and two piers founded on deep end bearing piles (ref. R-43). Although the same initial design concept has been maintained through final design, details of the retaining walls connecting to the bridge abutment wing walls have not been finalized yet.

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

The design of Bridge B-9 complies with the requirements of the execution version of the Project Agreement (PA), Schedule 15-2 Part 2, Article 5.

2 Background Information

2.1 Geological Setting

The WEP project site is located within the Essex Clay Plain, a part of the St. Clair Clay Plain physiographic region described in references R-17, R-19, R-20 and R-26. 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-26) 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) (ref. R-26). 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 all consisting of Limestone, Dolostone, and Shale.

2.2 Site Seismic Background

Windsor-Tecumseh area is described in the Canadian Highway Bridge Design Code (CHBDC, ref. R-9) by a seismic hazard associated to a Velocity Zone Zv = 0 and Acceleration seismic zone Za = 0. Zonal Velocity ratio V and Zonal Acceleration ratio A are both 0.

In accordance with the Canadian Highway Bridge Design Code (CHBDC, ref. R-9) the soil profile at the project site meets in general the description for Soil Profile Type III (soft clay and silts greater than 12 m in depth). A limited number of cross-hole tests was completed during the background investigation program (ref. R-23) at locations distributed strategically along the project alignment between Howard Road (east end) and Matchette Road (west end). The measured velocities of the shear waves were consistently over 200 m/s, with the bulk of results ranging between 200 and 300 m/s.

2.3 Existing Site Conditions and Proposed Bridge Layout

The existing ground topography immediately adjacent Bridge B-9 is generally flat with elevations ranging from approximately 181.9[1] in the area of west abutment to 183.5 at the east abutment. Adjacent land use is typically residential (see Appendix G for selected site photos).

Bridge B-9 site is situated in the western half of LaSalle segment of the Parkway. The bridge structure is a single cell trapezoidal reinforced concrete box structure and carries Highway 3 East Bound Exit Ramp (EBR8) over Highway 401 (Drawing 285380-03-060-SEG1-0901).

2.4 Frost Depth

In accordance with MTO–SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-38) and OPSD 3090.101 the frost depth below the ground surface in Windsor area is estimated at 1.0 m[2]. This estimate is considered applicable for natural soils and/or conventional pavement materials where the ground surface is usually cleaned from the snow cover.

The insulation effects of riprap and other coarse rockfill covers are considered to be one half of the insulation offered by soil deposits/cover, and the depth of frost penetration will have to be increased accordingly.

[1] Elevations are in metres and are referred to geodetic datum.
[2] Ontario Provisional Standard Drawings are included at the end of the report text.

3 Geotechnical Investigations

3.1 Scope and Procedures of Geotechnical Investigations

Geotechnical investigations involving a number of boreholes, cone penetration tests (CPT) and Nilcon vane tests had been carried out in 2007-09 by Golder Associates (ref. R-16 to R-23) as part of background information for development of the WEP proposal designs. Additional geotechnical investigation was undertaken in 2011 to supplement the available subsurface soil data, as required to support the detailed design development of the WEP embankment and structures. The additional investigation program at and around the proposed location of Bridge B-9 comprised a total of 3 boreholes, 1 Nilcon vane, 3 cone penetration tests and 2 Flat Blade Dilatometer probes (DMT). Table 3-1 lists the test holes put down at or in close proximity of the bridge site during both the previous and the current geotechnical investigations.

Table 3-1: Test Holes at and around Bridge B-9 Site

Reference	Boreholes	Nilcon Vane Tests	CPT's	DMT's
This Investigation (2011)	BH B9-1	NIL B9-1	CPT B9-1	DMT B9-1
	BH B9-2		CPT B9-2	DMT B9-2
	BH B9-3		CPT B9-3	
Previous Studies (2007-09)	BH-122	BH-122	CPT-10	
	BH-321		BH/CPT-123	
			BH/CPT-319	

Drawing 285380-04-090-WIP0-0901 shows the locations of the test holes and an interpreted soil stratigraphic profile along the WEP centreline for the general area at and around Bridge B-9. The test hole locations and stratigraphic sections at the bridge location are illustrated on Drawings 285380-04-090-WIP1-0902 and 285380-04-091-WIP1-0903.

3.2 Fieldwork

The boreholes were advanced using track-mounted CME 55 auger rigs, owned and operated by Marathon Drilling Co. Ltd., under contract to AMICO and under technical supervision by AMEC engineers and technicians. Boreholes were generally advanced using 215 mm OD hollow stem augers, followed by wash boring with NW casing. The depth at which the drilling methods transition occurred is noted on the borehole logs.

Soil sampling was generally carried out using a 50 mm diameter split spoon sampler and 70 mm diameter by 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 were transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing[3]. Rock coring of the bedrock was completed using a 1.5 m long NQ (OD=75.7 mm) or HQ (OD=96.0 mm) sized core barrels.

Standard Penetration Tests (SPT, ASTM D1586[4]) 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. Table 3-2 summarizes the depths of overburden penetration and rock coring as well as the list of instruments and the accompanying Nilcon vane tests. The Nilcon vane tests were carried out typically adjacent the boreholes.

Table 3-2: Overburden Thickness and Instrumentation in Boreholes

Borehole	Location[5]	Overburden Thickness, m	Test, Instrument & Elevation					
			Rock Coring	Nilcon Vane	S-Piez.	VWP	MSG	IN
BH B9-1 (AMEC)	N 4,679,235 E 332,594	33.7	148.2 to 146.8	159.3 to 154		172.8 & 165.4		X
BH B9-2 (AMEC)	N 4,679,219 E 332,622	33.5	148.9 to 146.2			171.4, 164.1 & 150.4	171.4 & 164.1	
BH B9-3 (AMEC)	N 4,679,135 E 332,674	33.2	149.4 to 147.3			168.3 & 158.4		X
BH-122 (Golder)	N 4,679,265 E 332,538	35.1	146.5 to 141.3	175.9 to 162.9	143.2 to 141.7			
BH-122A (Golder)	N 4,679,265 E 332,538	> 9.1				172.8		
BH-321 (Golder)	N 4,679,180 E 332,649	34.0	149.1 to 143.7			143.7		

Legend: S-Piez. Screen elevations for Standpipe Piezometer
VWP Sensor elevation for Vibrating Wire Piezometer
MSG Spider Magnet Heave/Settlement Gauge
IN Slope Inclinometer

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 recovery and RQD values are given on the borehole logs. The rock cores were photographed in the laboratory. Compression strength testes 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.

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[3] Advanced laboratory tests (one dimensional consolidation and drained direct shear tests) were carried out in AMEC's Scarborough Laboratory.

[4] American Society for Testing and Materials

[5] Location coordinates are in UTM-NAD 83 (Zone 17).

The Nilcon vane tests and CPT were carried out in cohesive soil strata after augering through the stiff/dense surficial materials. The Nilcon tests were carried out at 0.5 to 1.0 m depth intervals at an appropriate rate of rotational strain (ASTM D2573). The CPT cone was pushed at a constant rate into the ground using hydraulic ram system of the drill rig (ASTM D5778). All CPT were advanced to refusal. Pore pressure dissipation tests were carried out at selected depths thought the project. At Bridge B-9 site, dissipation test was carried out at BH B9-3 at 5.5 m depth.

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

The locations of test holes and the inferred soil profile along the WEP alignment between Sta. 10+400L and Sta. 11+000L (i.e., in the general area of the bridge) are shown on Drawing 285380-04-090-WIP1-0901. The test hole location in plan and soil stratigraphic section at the bridge location are shown on Drawings 285380-04-090-WIP1-0902 and 285380-04-091-WIP1-0903.

Borehole, DMT, Nilcon and CPT logs from the additional 2011 investigation are included in Appendix A. Relevant borehole logs from the previous investigation are included in Appendix B. Borehole logs illustrate the interpreted soil conditions, field test results and laboratory index test results.

3.3 Instrumentation

Geotechnical instruments were installed at selected locations on completion of boreholes to monitor pore water pressure and deformation behaviour of the soil strata during and after construction. A brief description follows:

Vibrating Wire Piezometers (VWP): The VWP transducers (RST Model VW2100, 0.35 MPa for shallow to mid-depth and 0.7 MPa for deep installations) were installed at the selected depths and electrical wires extended to the monitoring station located 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.

Spider Magnet Heave/Settlement Gauges (MSG): Magnetic targets were anchored to the ground around a PVC pipe, and are free to move with the soil. An estimate of ground heave/settlement can be made by measurement of MSG Ring elevations. Installation Ring/Gauge elevations are provided in Table 3-2and applicable borehole logs.

Slope Indicator (IN): Snap-seal 2.75 inch inclinometer casings with groves were installed in selected boreholes (Table 3-2) to measure the lateral movement of the soil. The boreholes were backfilled with bentonite-cement grout to ground surface.

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.

3.4 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. Following these soil classification tests, three representative soil samples were selected for two one-dimensional consolidation tests and one direct shear test. The index test results are shown on borehole logs included in Appendix A. The results of consolidation and shear tests are included in Appendix C.

Selected samples of the silty clay to clayey silt obtained from Boreholes B9-1, B9-2 and B9-3 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 are included in Appendix D.

3.5 Data Interpretation

Field Vane Test Data Correction: The chart in Figure 3-1[6] (developed initially by Bjerrum (1972) and updated subsequently by Ladd et al (1977) based on circular arc failure analyses of embankment failures) suggests correction by multiplying the field vane data by 1.05 to 1.10 for soils with plasticity index of about 15 (ref. R-33 and R-6). However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), the Canadian Foundations Manual suggests that the vane test data for clays with PI<20 should not be corrected (ref. R-1 and R-8, Figure 3-2). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI. Interestingly, the undrained shear strength (S_u) profiles inferred from the DMT and the Su values obtained from the conventional field vane tests in boreholes were consistently higher than the Nilcon vane test values.

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

$$S_{uCPT} = \frac{Q_t - \sigma_{vo}}{N_{kt}}$$

Where:

S_{uCPT} is the undrained shear strength estimated from the CPT test;

Q_t is the corrected total cone tip resistance;

σ_{vo} is the total vertical stress at the corresponding depth of measurement of the Qt value; and

N_{kt} is an empirical factor, depends on soil type & test arrangement, typically between 8 & 20.

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

The CPT based Su profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles. In this regard, the N_{kt} factor values used to calibrate the CPT strength profiles varied slight for different segments of the WEP and the soil strata. Thus, an N_{kt} factor of 14 was used to estimate the undrained shear strength of the clay crust and transition layers. The N_{kt} factors used for the underlying grey silty clay to clayey silt stratum and the lower clayey silt stratum were 16, and 12, respectively. In CPTs indicating pore pressures higher than cone tip resistance (e.g., soft clay stratum in CPT B9-1), the undrained shear strength was estimated from the excess pore pressures (using the N_u method).

Pre-Consolidation Pressures from Cone Penetration Tests: The approach used for estimating the pre-consolidation pressures from the estimated Su 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:

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

Where:

S_u is the undrained shear strength;

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

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

S is the normalized strength ratio (S_u/σ'_v) of normally consolidated soil;

OCR is the overconsolidation ratio; and

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

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

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

Flat Blade Dilatometer (DMT) Test Data: DMT tests were conducted following the ASTM D6635-01 (2007) method. The soil properties from the results of these tests were developed in general using the guidelines layout in ISSMGE, 2001 (ref. R-27), except that the undrained shear strength values for the clay deposits were estimated using the relationship $S_u = S\,\sigma'_{vo}\,(0.5\,K_d)^{1.25}$, where S = 0.18and K_d. is the horizontal stress index represented by:

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

Where:

- p_0 is the corrected instrument lateral pressure reading at zero membrane deformation (null method)
- u_0 is the pore water pressure in the soil prior to the blade insertion

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

The undrained shear strength (S_u), pre-consolidation pressure (σ_p'), natural water content (w_N) and compression index (C_c) profiles based on field and laboratory testing from boreholes, CPTs and DMT carried out in the vicinity of Bridge B-9 are presented in Figure 3-3. Also included on these figures are $0.18 \times \sigma_{vo}'$ curve (representing undrained strength for OCR=1 condition) and simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units.

4 Subsurface Conditions

The general soil stratigraphy at the borehole locations consists of the following successive strata: surficial layers of occasional fills, topsoil, and upper granular deposit; extensive clayey silt to silty clay deposit below about elevation 181, and a lower granular deposit below about elevation 151, overlying limestone and dolostone bedrock below about elevation 148. The thickness of the clayey silt to silty clay deposit at the test hole locations varied between about 20.3 m and 32.4 m. The lower granular deposit (sandy silt / silty sand / sand and gravel) varied in thickness between 0.5 to 5.4 m. The bedrock was encountered at depths ranging from about 33.7 m to 38.0 m below the ground surface.

4.1 Surficial Fills, Topsoil and Upper Granular Deposit

All boreholes, except for BH B9-2 encountered an up to 0.5 m thick layer of brown to black topsoil. The thickness of the topsoil is expected to vary in quality and thickness through the project area.

Borehole BH B9-2 was put down on existing pavement and encountered 100 mm thick asphalt layer overlaying sand fill which extended to 0.4 m below existing grade. A non-cohesive fine silty sand was encountered in all boreholes, except Boreholes BH B9-1, CPT B9-1, BH-122/122A below fill and/ topsoil. The thickness of this unit varies between 0.5 to 2.1 m.

4.2 Silty Clay to Clayey Silt Stratum

The cohesive silty clay stratum was encountered directly underlying the surficial layers of topsoil or fill/granular deposit. The encountered depth below existing ground surface was from 0.2 to 2.4 m corresponding to elevation 181.1 to 181.5. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into 4 successive layers as follows: brown desiccated stiff to very stiff clay crust, transition zone, upper grey silty clay to clayey silt deposit (referred to hereafter as upper silty clay), and then a generally coarser lower grey clayey silt deposit (referred to as lower clayey silt). The natural water content, Atterberg limits, bulk unit weights and undrained shear strengths (from Nilcon vane tests) properties of the clay sub-strata based on tests carried out during the additional investigation are summarized in Table 4-1. The plasticity charts (Appendix C) suggest the silty clay deposit to be a low to medium plasticity material.

Table 4-1: Summary of Index Properties of Clay Sub-Strata

Property	Clay Crust	Transition	Grey Silty Clay	Clayey Silt
Elevation Range, m	183(*) to 177	177 to 175	175 to 163	163 to 151
Natural Water Content, w_N, %	6 to 32	12 to 25	15 to 47	9 to 33
Liquid Limit, w_L, %	31 to 44	30 to 31	26 to 35	20 to 41
Plastic Limit, w_P, %	14 to 22	15 to 16	14 to 18	13 to 21
Plasticity Index, PI, %	17 to 22	14 to 15	11 to 19	12 to 20
Liquidity Index, LI	0.05 to 0.21	0.09 to 0.33	0.06 to 0.91	0.02 to 0.62
Unit Weight, γ, kN/m^3	20.7 to 21.6	20.8	19.5 to 21.9	20.5 to 20.6

(*) Elevation varies

As illustrated on Figure 3-3, the undrained shear strength of the silty clay stratum varied with depth generally as follows:

- Crust layer: > 80±20 kPa
- Transition layer: 80±20 kPa to 65±10 kPa
- Upper silty clay: 65±10 kPa to 55±10 kPa
- Lower clayey silt: 75±15 kPa

The stress-strain properties and the effective shear strength properties of the silty clay to clayey silt soils were based on published correlations (Kulhawy and Mayne, 1990, ref. R-29, Leroueil et al, 2001, ref. R-34 and Terzaghi et al. ref. R-42) and confirmed by tests reported in Golder's Subsurface Condition Interpretation Report (ref. R-19) and the tests performed during the additional geotechnical investigation carried out as part of the detailed design development for the entire WEP length.

The stress-strain relationships are correlated to natural water content (w_N, expressed as percent) as illustrated in Figure 4-1 and Figure 4-2, and are summarized as follows:

$$C_c = 0.0086w_N – 0.0086$$

$$C_r = 0.11C_c$$

$$C_s = 0.25C_c$$

$$C_\alpha = 0.028C_c$$

The interpreted average values used for the clay substrata for the Bridge B-9 site are summarized in Table 4-2.

Table 4-2: Clay Interpreted Compressibility Properties

Property	Clay Crust	Transition	Grey Silty Clay	Clayey Silt
Average Natural Water Content, w_N, %	19	19	23	19
Average Unit Weight, kN/m^3	21	21	20	21
Virgin Compression Index, C_c	0.15	0.15	0.19	0.15
Recompression Index, C_r	0.017	0.017	0.021	0.017
Swelling Index, C_s	0.039	0.039	0.047	0.039
Secondary Compression Index, C_α	0.004	0.004	0.005	0.004

Oedometer testing carried out on two samples in the upper Grey Silty Clay from Borehole BH B9-3 (TW 11 and TW 13) indicated the following compressibility indexes: C_c = 0.17 & 0.31, C_r = 0.028 & 0.045, C_s = 0.042 & 0.067.

The effective shear strength properties applicable to the silty clay to clayey silt stratum were determined from triaxial and direct shear tests performed during the pre-bid studies and additional geotechnical investigations on samples obtained from the entire length of the WEP and supported by published PI versus ǿ relationships (ref. R-29, R-35 andR-42), and are summarized as follows (Figures 4-3 and 4-4):

Apparent cohesion, ć	0 kPa
Angle of internal friction, ǿ	30°
Friction angle at critical state, Φ_c	25 to 26° (*)

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

The modulus of elasticity has been correlated with the undrained shear strength of the material, published information and local experience (ref. R-19 and R-42). For the unweathered portion of the silty clay stratum the empirical relationship were used based on average shear strength profiles for the material, as shown in Table 4-3.

$E_u = 300\ S_u$

$E' = 0.9E_u$

Table 4-3: Clay Interpreted Elastic Moduli Properties

Soils Stratigraphy	Undrained Elastic Modulus, MPa	Undrained Poisson's Ratio (*)	Drained Elastic Modulus (E), MPa	Drained Poisson's Ratio (*)
Clay Crust	30	0.49	27	0.35
Transition	21		19	
Grey Silty Clay	16		14	
Clayey Silt	18		16	

(*) Assumed values

The hydraulic conductivity of the silty clay to clayey silt stratum was interpreted from pore pressure dissipation tests carried out in the CPT probes as well as the laboratory oedometer tests. The hydraulic conductivity values obtained from previous (2007-09) and additional (2011) investigations are plotted on Figure 4-5.

4.3 Lower Granular Deposit

Underlying the silty clay to clayey silt stratum and overlying the bedrock was a discontinuous and heterogeneous non-cohesive material varying from silty sand, sand and gravel, and silts with sand. Based on SPT N-values ranging generally from 13 to greater than 100, this material is considered to be in a compact to dense state of compactness (with the exception of Borehole B9-1 where the sand and silt were in very loose state). This layer was approximately 0.5 to 5.4 m thick and varies significantly throughout the Bridge site.

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. Bedrock was encountered at elevations ranging from 146.8 to 151.9 m in the vicinity of Bridge B-9. The Rock Quality Designation (RQD) of the recovered rock cores varied generally between 60 to 95%, 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. Boreholes cores show that rock quality generally improves with depth. Photographs of rock cores recovered from the additional investigation are provided in Appendix H.

It was found during the preliminary investigations (ref. R-19) 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.

Based on the rock mass classification and the strength properties assuming an mi = 12 for a crystalline limestone, a disturbance factor of 0.7, and a factor of safety of 3.0, an allowable bearing capacity of the rock has been calculated to range from 5.3 MPa to 13.5 MPa. The mean allowable bearing capacity is determined to be 9.2 MPa using the Hoek and Brown strength criterion for determining the bearing capacity of a fractured rock mass (Wyllie, 1999, ref. R-44).

Table 4-4: Summary of Intact Properties of Rock Core Samples

Parameters	**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

4.5 Groundwater Conditions

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

The highest piezometric and vibrating wire water levels within the overburden and the bedrock were reported to be between elevations 181.9 and 178.5, respectively (Table 4-5). These observations suggest a potentially downward gradient between the overburden and the bedrock. Nevertheless, given the general prevalence in the Windsor area, occurrence of artesian condition in bedrock cannot be ruled out.

Table 4-5: Summary of Measured Water Levels

Borehole	Surface El, m	Piezo. Type	Screen/Sensor El., m	Stratum at Screen / Sensor Depth	Measured Water level	
					Date	El, m
BH B9-1	181.9	VWP	172.8	Silty Clay	Aug. 6, 2011	181.9
		VWP	165.4	Silty Clay	Aug. 6, 2011	181.8
BH B9-2	182.3	VWP	171.7	Silty Clay	Aug. 6, 2011	181.0
		VWP	164.1	Silty Clay	Aug. 6, 2011	179.8
		VWP	150.4	Silty Clay	Aug. 6, 2011	177.6
BH B9-3	183.6	VWP	168.3	Silty Clay	Aug. 6, 2011	180.3
		VWP	158.4	Silty Clay	Aug. 6, 2011	178.9
BH-122	181.7	S-Piez	143.2 – 141.7	Limestone	Jul. 23, 2011	178.3
BH-122A	181.7	VWP	172.6	Silty Clay	Jul. 23, 2011	180.3
BH-321	183.1	VWP	143.7	Limestone	Feb. 24, 2010	178.5

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

Perched groundwater was encountered in Boreholes CPT B9-2 and CPT B9-3 near elevation 182 and is known to accumulate seasonally within the upper deposits of fill, topsoil and granular layers, and within the fissures in the silty clay crust. In adverse conditions, the perched groundwater levels can rise to near the ground surface.

4.6 Subsurface Gases

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

The H_2S gas can frequently be detected by odour at concentrations in the order of 0.5 ppm and can be corrosive at concentrations of about 2 ppm to 3 ppm in the groundwater. The gas odour was not detected during the drilling at the Bridge B-9 site.

Although the presence of the H_2S and CH_4 gases was not observed during the geotechnical investigation at Bridge B-9 site, their presence cannot be entirely ruled out. Pumping tests were conducted at three locations across the proposed parkway to determine concentration levels of hydrogen H_2S gas in the groundwater of the area. A summary of the results of these tests is provided in Table 4-6.

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-24 and R-41). It is, therefore, recommended that the design and construction methodologies should be developed in consideration of the potential presence of these gases (ref. R-14).

Table 4-6: Pumping Tests Data

Test #	Approximate Location	H_2S Gas Concentration, mg/L
TOW-1	Bridge B-11	< 0.02
TOW-2	North of Tunnel T-7	20.0
TOW-3	South of Tunnel T-4	7.9

Air quality and subgrade pore pressure monitoring should be carried out during construction. In general, it is recommended that equipment operating in confined spaces be selected to safely operate in a potentially gaseous environment.

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 to total stress changes can be very low. This phenomena leads to reduction in effective stress and hence shear strength (ref. R-25 and R-41).

5 Development of Geotechnical Designs

5.1 Bridge Configuration

Bridge B-9 is a three-span underpass which carries the traffic of Highway 3 East Ramp over Highway 401 between Sta. 9+549 and Sta. 9+677 and near Huron Church Line (Drawing 285380-03-060-SEG1-0901).

The proposed Bridge B-9 is a one-lane concrete box structure located on a curved alignment over Highway 401. The bridge structure comprises true abutments and piers supported on deep end-bearing vertical and batter HP 310×110 steel piles (Drawing 285380-03-060-SEG1-0904 and 0905).

Table 5-1 provides a summary of control elevations at the bridge abutments and piers used for the geotechnical design development.

Table 5-1: Summary of Control Elevations at Abutments

Location	Existing Ground Surface	Top of Pavement	Bottom of Pile Cap	Highway 401 Subgrade
West Abutment	181.7	184.2	176.8	175.3
Pier 1	182.4	184.3	173.1	
Pier 2	183.1	184.5	172.1	
East Abutment	183.5	184.5	177.2	

As Bridge B-9 comprises of conventional abutments (i.e., reinforced concrete deck/wall/cap founded on steel piles), and the design for retaining walls HRW-21L and MSEW-23R are extended to the abutment wing walls. The high wall report and drawings should be consulted for the detailed design of the retaining walls[7].

Based on the available information, it is considered that Bridge B-9 construction will involve the following successive earthwork, structural components, and loading stages:

- Bulk excavations along the Highway 401 corridor to near the pavement sub-grade,
- Temporary excavations for the west and east abutments and piers 1 and 2 down to about elevations 176.8, 177.2, 173.1 and 172.1, respectively;
- Installation of piles (HP310×110) for all bridge supports;
- Construction of HRW-21L and MSEW-23R retaining walls;
- Backfilling excavated areas, where required;
- Construction of piers and integral abutments;

[7] The walls design was underway at the time of preparation of this report.

- Completion of drainage works and backfill behind the abutment; and
- Completion of the pavements over EBR8 and Highway 401.

5.2 Geotechnical Design Criteria and Considerations

The geotechnical design has been completed in compliance with the requirements of the execution version of the Project Agreement Schedule 15-2 Part 2, Article 5 (PA) for the Windsor-Essex Parkway Project. The foundations' designs were as per the principles of Limit States Design (LS Method) based on Load and Resistance Factors (CHBDC and CFEM, ref. R-9 and R-8).

Working Stress Design (WS Method) was employed for global stability of the earthworks and soil mass containing earth retaining structures. The stability of the soil mass containing the bridge end abutments was checked for all potential surfaces of sliding and has a minimum factor of safety of 1.3 according to the PA.

5.3 Design Soil Properties

As indicated in Section 3.5, 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. Selected typical design values obtained from the profiles are summarized in Table 5-2. Effective cohesion for the upper clay crust and transition zone layers has been neglected due to long term weathering, moisture ingress and fissuring effects.

Table 5-2: Interpreted Design Clay Strength

Clay Substratum	Elevation, m	Undrained Shear Strength, S_u, kPa	Effective Stress Parameters	Pre-consolidation Pressure σ'_p, kPa	OCR
Clay Crust	> 177	75 (*)	Peak friction angle, ϕ'_{max} = 30º Cohesion, ć = 0	550	7
Transition	177 to 175	75 to 65		500 to 350	4.5
Upper Silty Clay-1	175 to 166	65 to 44		350 to 230	2
Upper Silty Clay-2	166 to 163	44 to 50		230 to 260	1.1
Lower Clayey Silt-1	163 to 161	50 to 65		230 to 400	1.5
Lower Clayey Silt-2	161 to 151	65		400	1.3

(*) Applicable for global stability verifications
Note: The undrained shear strength and pre-consolidation pressure profiles applicable to Bridge B-9 site are shown on Figure 3-3, and the effective shear strength parameters are based on the relationship presented in Section 4.2.

The design values of the coefficient of horizontal permeability (k_h) and the hydraulic conductivity anisotropy ratio used for the analysis of stress and deformation response of the soils are provided in Table 5-3. 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.

For design purposes the initial groundwater level in the overburden was considered at elevation 181 and 183 for west and east abutments, respectively.

Table 5-3: Other Interpreted Design Parameters for Clay

Clay Substratum	Horizontal Permeability, k_h		Anisotropy ratio, k_h/k_v	Initial Void Ratio, e_0
	m/days	cm/sec		
Clay Crust	2.9×10^{-4}	3.3×10^{-7}	1	0.5
Transition	9×10^{-5}	1.0×10^{-7}	2	0.5
Upper Silty Clay				0.6
Lower Clayey Silt				0.5

5.4 Excavations and Temporary Cut Slopes

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

Excavations are expected to encounter surficial fills, topsoil and water bearing granular soils and will be extended 6.7 and 8.2 m below existing grade (elevation 182 and 183.5) to about elevation 175.3 into the native firm silty clay for the west and east abutments, respectively.

Basal hydrostatic uplift was calculated based on the highest measured water level (elevation 178.5) in the rock, anticipated deepest excavation depth (Pier#2 underside of pile cap at elevation 172.1, and a silty clay to clayey silt layer thickness of 20.9 m (Borehole BH-321) below the deepest excavation. The calculated factor of safety against hydrostatic uplift was 1.6.

As described in Section 4.6, the presence of gassy soils near the bedrock surface could potentially be encountered, and that could impact the pore pressure and undrained shear strength condition of the lower part of the silty clay deposit. While no indications of gassy soils were recorded at this site during the pre-bid and additional investigations, it is recommended that in the case of excavations deeper than 5 m careful monitoring of basal heave and pore water pressures below the bottom of the excavations shall be carried out during construction. Adequate number of heave gauges and low-displacement type piezometers shall be installed prior to initiation of the major excavations. If warranted by the monitoring of the excavation progress performance, the excavation rates will have to be adjusted to allow sufficient time to dissipate the pore pressures to safe levels. The excavation guidelines can be revised based on on-site experience.

5.5 Pile Foundations

5.5.1 ULS and SLS Resistance to Axial Loads

It is understood that HP310×110 steel H piles will be used at this project. The pile driving equipment and installation procedure should be established in the field. A number of static load tests should be carried out at key locations along the alignment of WEP in conjunction with PDA testing to facilitate proper calibration of the PDA, and determine the hammer performance and appropriate driving criteria (set).

The piles are expected to be driven to bedrock as per OPSS 903 and accordingly they will mobilize a Ultimate Limit States (ULS) axial geotechnical resistance in excess of 4000 kN. A factored ULS resistance of at least 2000 kN is anticipated.

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

Based on the available borehole data at this structure, the bedrock surface at Bridge B-9 location varies between elevations 146.8 and 151.9, where the tips of piles are anticipated to be set. In cases where some of the piles cannot be driven to bedrock due to presence of dense till lying immediately above the bedrock and/or a perceived risk of damaging the piles by overdriving is apparent, consideration should be given to supplementing the field testing to prove the actual mobilized resistance. If lower mobilized pile resistances are proven, options based on the most economical approaches may be considered (e.g., changes to the driving method and equipment, or addition of more piles).

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

The following general pile installation recommendations should be considered:

- The steel H piles should be installed and monitored in accordance with OPSS 903 requirements. The piles should be reinforced with Type I shoe flanges as shown in OPSD 3000.100, or approved alternatives.
- Survey of all the pile head elevations should be completed at the end of driving and just prior to forming the pile cap. Re-tapping of the piles will be necessary where uplift exceeding 5 mm is noted, or as directed by engineer.
- While unlikely to occur at this site, considering the general geologic conditions in the region, indications of natural gas venting, water, and fines washout should be monitored during driving. Provision to mitigate such occurrences (by heavy mud, grouting of the cavities, etc.) should be in place. It is recommended that the pile splicing be completed by butt-welding (OPSD 3000.150, Section A-A) to minimize the pathways for upward flow of artesian water along the piles to the surface.
- Consideration should be given to potential driving difficulties due to the presence of dense to very dense lower granular soils and potential presence of cobbles and boulders above the bedrock.
- Vibrations generated by piling should be monitored. It is not expected that the vibrations during piling will have a significant impact on the stability of temporary slopes. Nonetheless, if the vibration intensities at the toe and top of the slopes exceed 10 mm/s, appropriate mitigation

measures (slope flattening or vibration dampening by dumping sand around the piles) should be considered.

- Noise monitoring should be carried out during pile driving at the site.

5.5.2 ULS and SLS Resistance to Lateral Loads

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

The SLS geotechnical resistance to lateral loads is dependent on the acceptable levels of the lateral pile deflections under the design loads and should be obtained on the basis of field load tests. In the absence of field tests, the preliminary design can be based on a conventional SLS resistance of 65 kN along the strong axis and 45 kN along the weak axis of the HP310×110. This conventional SLS resistance represents the lateral shear force applied on a free-head pile at the level of the ground surface that causes a lateral deflection of 10 mm measured at the ground surface.

The ULS lateral resistance is defined as the lateral force applied to the pile shaft causing unstabilized pile displacements due to soil failure or pile structural failure. In the absence of field tests, the ULS lateral resistance based on pile structural failure can be assumed 185 kN and 85 kN along the strong axis and weak axis, respectively.

The above estimates were based on a pile model assumed to be embedded within stiff to firm silty clay below elevation 179.5. The above resistances were estimated using the "p-y" model (LPile 5.0 model Ensoft 2010, ref. R-15). The "p-y" curves were generated using the Reese method described in the Technical manual for LPILE, using the Reese "Stiff-Clay without free water" and Matlock "Soft Clay" models in conjunction with the soil parameters defined in Table 5-4.

Table 5-4: Soil Parameters for p-y Curve Calculation

Soils Around the Piles	Elevation Range	Design Bulk Unit Weight, kN/m^3	Undrained Shear Strength (S_u), kPa	ε_{50}
Silty Clay Crust	Above 177	21	75	0.005
Transition Clay	177 to 175	21	75 to 65	0.007
Upper Silty Clay - 1	175 to 166	20	65 to 44	0.010
Upper Silty Clay – 2	166 to 163	20	44 to 50	0.010
Lower Clayey Silt - 1	163 to 161	21	50 to 65	0.007
Lower Clayey Silt - 2	Below 161	21	65	0.005

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

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

The actual SLS and ULS lateral resistances will increase in the case of piles with structural restraints at the pile head due to embedment within the pile caps. Both the ULS and SLS to lateral loads resistances are also strongly dependent on the structural and load configuration and on the acceptable deformations.

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

Horizontal Subgrade Reaction Method:

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

$k_h = n_h (z/d)$ for cohesionless soils; and

$= 67 (S_u/d)$ for cohesive soils.

Where:

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

n_h (MPa/m) = Soil coefficient;

S_u (MPa) = Undrained shear strength;

z (m) = Depth below finished grade; and

d (m) = Pile diameter/width.

The recommended ranges of soil parameters are tabulated in Table 5-4.

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

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, as indicated in Table 5-5 for abutment pile groups. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed here.

The pile spacing space in the direction of loading under the abutments and piers is about 6 and 5 times pile diameter, which result in 0.7 and 0.55 reduction factors in the lateral resistance, respectively.

Table 5-5: Lateral Resistance Reduction Factor for Pile Groups For Subgrade Reaction Method

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

d = pile diameter
Source: NAVFAC DM-7.2 (ref. R-13)

Alternative Nonlinear 'p-y' Curve Method:

Alternative pile design methods can be considered using the nonlinear "p-y" interaction method and elastic continuum theory as discussed in the Canadian Foundation Engineering Manual (ref. R-8).

The p-y curves describe the lateral soil resistance along the pile depth. For each soil layer along the pile shaft, the p-y curves describe lateral soil pressure 'p' (kPa) per unit length mobilized by the pile lateral deflection 'y' (m). Where only pile head loads are applied and there are no lateral movements of the surrounding soil mass, 'y' is the absolute lateral deflection. Where lateral ground movements occur, 'y' is the relative movement between the pile and the soil. The p-y curves reflect the non-linear soil behaviour under moderate to high stress levels where the more traditional elastic modeling of the soil response is considered to be insufficient.

The general procedure for computing p-y curves is summarized in the Canadian Foundation Engineering Manual (ref. R-8). A detailed description for the generation of the p-y curves can be found in the Technical Manual for the commercial software LPile Plus by Ensoft Inc (ref. R-15). For a given foundation configuration, pile size, and soil stratification, the soil properties required for the generation of the p-y curves are provided in Table 5-9. "Stiff clay" p-y curves as given in the LPile manual should be developed appropriate for either static or cyclic loading conditions in absence of free water. For p-y curves below the water table, submerged unit weights in the soil mass shall be used.

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

In the case of batter of 1H:3V (abutments), the p-y curve modifier will be $\boldsymbol{B_m}$ = 0.5 and 1.5 for the batter in the direction of the lateral load, and opposite direction of the lateral load, respectively. For the batter of 1H:5V in piers, the p-y curve modifier are $\boldsymbol{B_m}$ = 0.75 and 1.25.

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

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

Where:

β_{ki} = the influence factor of pile 'k' in the group on pile 'i', with k ≠ i, and is calculated with one of the following expressions (depending on the relative position of pile 'k' in the group with respect to pile 'i' (Table 5-6).

Table 5-6: Lateral Load Capacity Reduction Factor for Pile Groups using p-y Method

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

The closest spacing between the piles is 1525 mm at the piers and 1500 mm at the abutments. Accordingly, some reduction factors would apply only for the tailing piles.

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

5.5.3 Soil Pile Interaction Assessment

The influence of ground loads and deformations developed due to construction of abutment on pile capacity is further discussed in the following paragraphs.

Downdrag Loads (Negative Skin Friction – NSF):

Potential for downdrag loads on piles was considered in response to ground movements (rebound and settlements) that are estimated to occur at Bridge B-9.

Soil stress-deformation analyses described in Section 5.6.2 were conducted using the SIGMA/W software. The net estimated ground vertical movement (settlement/heave) after excavation in the vicinity of the pile shaft were analyzed at the following representative stages: after completion of the backfill behind the bridge diaphragm (End of Construction - EC) and in long-term (LT), and associated vertical effective stresses were presented in Figures F-11 and F-12 (Appendix F). The analyses indicated the following:

- Ground settlements is expected to occur along the pile shaft during construction of the abutment and completion of the associated backfill; and
- Ground rebound is expected to occur after the substantial completion of the ground surface loading.

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

- Maximum transient downdrag of 230 kN plus structural dead load only (pile cap and bridge deck) occurring during completion of the backfilling against the bridge abutment; and
- Residual (long-term) downdrag of 165 kN plus total design dead loads (structural and pavement materials over bridge deck) after the completion of construction.

In accordance with the Canadian Foundation Engineering Manual (ref. R-8), the service loads should not be reduced by any portion of the drag loads unless required by insufficient structural strength of the pile. Downdrag load and live load do not combine and two separate loading cases should be considered:

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

Shaft Bending:

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

- The ground lateral movement along the pile shaft anticipated to occur after the installation of the piles was estimated using the stress-deformation analysis described in Section 5.6.2.
- The pile head was assumed to be a free head.
- The above soil deformation field was imposed as "loads" along the pile shaft. The calculation was conducted using the "p-y" model (LPile 5.0 model Ensoft 2010, ref. R-15). The "p-y" curves were generated using the Reese method (for Stiff Clay without free water) described in the Technical manual for LPile, using undrained shear strength of 50 kPa and effective unit weight of 10 kN/m^3.
- The earth pressures from backfill and surcharge loads against the pile cap were not considered in the analyses.
- The shear force, bending moment and displacement within the pile were calculated from LPile model.

Based on the above approach and anticipated lateral ground displacement, the estimated maximum unfactored bending moment in the shaft was 20 kN-m for the strong axis pile loadings.

The above bending moments, shear forces and deflections are in addition to those caused by bridge loads applied to the piles. The maximum computed moment in the pile under assumed pile head load equal to the conventional SLS resistance (65 kN) was 80 kN-m for the strong axis pile loadings. Accordingly, a potential combination of the maximum bending stresses from pile head shear force and ground displacement field would lead to a maximum bending moment of 100 kN-m, which is less than the yield moment of the pile.

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

5.6 Abutment Walls

5.6.1 Global Stability

Slope stability analyses (Limit Equilibrium) were carried out using SLOPE/W Version 2007 software and the Morgenstern-Price method of analysis.

The global stability analyses have been carried out for short-term (undrained soil properties), end of construction (EOC) undrained soil properties) and long-term (drained soil properties) with steady state loading conditions.

The short-term condition refers to a temporary stage during construction when the pavement box material over Hwy 401 was not yet placed while the approachway embankment is completed and in operation.

The EOC condition refers also to a transient – undrained situation at, and shortly after the works are entirely completed and fully operational. The drained analyses refers to the long-term condition after completion of the project.

The presence of the piles and strength gain with time were not considered in the stability models (somewhat conservative approach). Surcharge of 12 kPa for short-term and long-term model was applied at the top of ground surface, while tension crack was assumed for short-term only.

As earlier discussed in Section 5.4, the global stability of temporary slopes is part of the Contractor's responsibilities. The calculated factors of safety (FS) for circular slip surfaces are in excess of 1.3 against global instability of the abutments, as shown in Figures E-1 to E-6 and summarized in Table 5-7.

Table 5-7: Summary of Abutment Slope Stability Analyses

Abutment	Loading Conditions			Figure
	Short-term	End of Construction	Long-term	
East	1.42 (1.28)	1.61 (1.42)	1.35 (1.46)	E-1 to E-3
West	1.46 (1.36)	1.65 (1.46)	1.42 (1.53)	E-4 to E-6

(*) Values in brackets refer to non-circular failure surface

5.6.2 Stress Deformation Analysis Models

Stress-deformation analyses (SDA) were carried out using the SIGMA/W software Version 2007. The main focus of the SDA was to assess the ground deformations in the vicinity of the bridge structure and along the approachway embankments. Also, the SDA results were used to evaluate the effects of the ground movement on the pile shafts in terms of deformations and structural stressing.

SDA has been carried out for two representative sections cut at the west abutment where the soil stress increase is anticipated to be higher due to the vicinity of the embankment for the realigned Highway 3. The two sections referred to hereafter as ‘Wall’ and ‘Abutment’ sections, are aligned respectively perpendicular to and along the EBR8 centreline at the west abutment centreline. Model geometries and boundary condition are illustrated for these two sections in Figure F-1. The model is based on the following construction sequence simulations:

a) Generation of the initial (in-situ) stress condition for level ground assuming an average bulk soil unit weight of 21 kN/m^3, and an at-rest earth pressure coefficient K_0 of 0.75 for the soil deposit[8];

b) Excavation to the final grade at the Highway 401 subgrade level (i.e., to elevation 175.3), followed by H-pile installation at the bridge abutments and piers;

c) Construction of the reinforced concrete pile caps, abutment/retaining walls and piers, and completion of the fill behind abutment and retaining walls;

d) Completion of the pavement structure for Highway 401; and

e) Long-term dissipation of excess pore pressure.

The stratigraphy and selection of the soil properties was based on the design soil properties discussed in Section 5.3.

The SDA was carried out for drained (effective stress) soil behaviour using a fully coupled stress-pore pressure analysis (coupled stress-deformation and seepage dissipation equations).

Modified Cam-Clay constitutive models were considered for the unweathered firm to soft clayey silt below the transition zone, and the elastic-plastic Mohr-Coulomb model for the remaining soil layers (i.e., crust, transition, and backfill). The drained Modified Cam-Clay model required as input the critical state friction angle, pre-consolidation pressure, initial void ratio, primary compression and unloading compression indices. The latter was selected as the rebound compression index given in Table 4-2. The drained elastic-plastic Mohr-Coulomb model required as input the peak friction angle, the drained initial Young’s modulus, and a Poisson’s ratio.

5.6.3 Serviceability Limit State (SLS) Performance

Ground deformations (i.e., heave/settlement, horizontal displacement, etc.) and stress distributions were estimated for the following elapsed times (days):

- 0 In Situ condition
- 0-90 Initial excavation

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[8] Based on published information and experience in Windsor area with DMT.

- 90-92 End of Construction (i.e., construction of abutment walls, backfilling and pavement)
- 10950 Long-term condition (complete pore pressure dissipation)

The SLS performance was assessed on the basis of the SDA model described above in Section 5.6.2. The computed deformations[9] representing temporary excavation and end of construction (short-term) loading conditions are shown in Figures F-2 and F-3. The computed deformations in the long-term condition are also shown in Figure F-4.

Figures F-5 and F-6 illustrate the lateral soil displacement contours for end of construction and long-term loading conditions, respectively.

Figure F-7 illustrates the long-term stabilized pore water pressure contours at the end of pore pressure dissipation.

Charts of calculated heave at the ground surface progressing from the abutments wall to the centreline of Highway 401 are shown in Figure F-8 at various loading conditions.

Plots of heave versus elevation (along vertical line) at the abutment and pier pile locations for various construction stages are shown in Figure F-9. The plots indicate long term gradual heave of the ground surface caused by a slight net unloading of the subsoils following removal of the existing abutment fills. Following initial excavation of the existing abutment fills (when abutment piles would be driven) there could be some soil settlement around the piles caused by re-application of new fills to construct the abutments.

Figure F-10 shows lateral ground movements versus elevation at various times acting on the abutment and pier piles.

Figures F-11 illustrates the elevation profile of vertical effective stress along the abutment and pier piles at various construction stages. The plots indicate a slight reduction in vertical effective stress versus time at both locations, suggesting long-term stress relaxation and soil heave. Fill placement during abutment construction results in short-term increases in vertical effective stress at the abutment locations, leading to short term soil settlement. In the long-term, net unloading of the soil around the abutment piles is indicated. At the centre pier, slight unloading in the longer term is seen to occur.

Figure F-12 shows the cumulative ground settlement behind the bridge abutment. Table 5-8 summarizes representative deformation response obtained for bridge B-9.

The SIGMA/W model used for stress-deformation analysis is a plane strain model and does not capture the 3-D excavation effects involved around the bridge. Based on 3-D elastic stress distribution theory, it is considered that the actual deformations near the abutments would be somewhat lower than calculated baserd on the 2-D model.

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[9] Negative and positive values of vertical displacements indicated in Appendix F figures refer to settlement and heave, respectively.

The ground movements generated by the construction loads are anticipated to stabilize within approximately 10 to 15 years following completion of construction. Ground movement monitoring should be undertaken during construction.

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

Table 5-8: Summary of Calculated Deformations

Loading Stage	Vertical Ground Movement at Various Distances (2) from the Bridge Abutment, mm					
	0 m	5 m	10 m	20 m	50 m	75 m
End of Bridge Construction (EOC) (1)	< 10	25	20	15	5	< 5
Long-term (LT) (1)	< 10	50	40	30	30	< 30
Post construction (LT – EOC) (3)	< 5	25	20	15	25	< 25

Notes:
(1) Cumulative values
(2) Distances measured perpendicular to the bridge abutment.
(3) Incremental values excluding deformation at completion of construction that are assumed to be compensated during construction

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 is expected to be generally small. Stringent compaction control will be required to minimize these risks.

5.6.4 Earth Pressures on Abutment Walls

Behind the concrete abutment and wing walls, non-frost susceptible free draining granular fill should be placed in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC).

The granular backfill should be compacted in maximum 200 mm thick loose lifts in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the backfill. Other aspects of the abutment backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150.

Heavy compaction equipment should not be used adjacent to the walls of the structure, where the backfill should be placed in maximum 100 mm thick loose lifts and compacted with small compactors. Effects of backfill compaction activities should be simulated as live load over and above the static lateral earth pressure for structural design in accordance with the CHBDC.

Earth pressures on abutment walls may be calculated on the basis of parameters given in Table 5-9.

Table 5-9: 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, Ø (degrees)	33 -35	29-32	22-30
Coefficients of Static Lateral Earth Pressure:			
'Active' or Unrestrained, $K_a^{(*)}$	0.27 to 0.30	0.310 to 0.35	0.33 to 0.45
'At Rest' or Restrained, $K_o^{(*)}$	0.43 to 0.46	0.47 to 0.52	0.50 to 0.62
'Passive', $K_p^{(*)}$	3.3 to 3.7	2.9 to 3.2	2.2 to 3.0

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

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

Group III soils can be used as general backfill in approved areas.

6 Construction Requirements

6.1 General Construction Requirements

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

As mentioned earlier, the Contractors are fully responsible for the design, construction methods, performance (stability, deformability and deterioration) and maintenance of the temporary slopes and temporary works. The following recommendations and comments are considered applicable:

- All excavation works should be carried out in accordance with the guidelines outlined in Occupational Health and Safety Act (OHSA) and OPSS 902. The native undisturbed soils may be classified as Type 3 soils. The excavations below the original ground levels may intersect water bearing backfill within trenches of active and/or abandoned utilities. In these cases, Type 4 soil conditions may occur and should be addressed accordingly.
- The silty clay soils at the project site are highly susceptible to rapid deterioration when exposed to elements, weathering and/ or subjected to direct construction traffic.
- Temporary slopes, permanent slopes, and subgrade areas must be appropriately protected at all times against surface erosion due to runoff, desiccation, freeze-thaw effects, etc.
- To protect the integrity of subgrade for foundations and pavements, the final excavation lift above the design elevation should not be less than 500 mm and should be carried out only when the Contractor is ready to prepare and cover the subgrade with the materials specified in the design same day the final excavation is exposed and approved. No construction traffic should be permitted over subgrade without approved protective covers.
- The final excavation lift above the design subgrade should be carried out using buckets equipped with smooth lips. Once exposed, the subgrade must be immediately inspected. Upon approval, the subgrade should be immediately protected; depending on the type of construction, geofabrics, granular mats, a skim coat (minimum 75 mm thick) of lean concrete protection (mud mat), etc. should be used.
- As indicated earlier, pore pressures, heave/settlement behaviour and presence of gassy soils below the excavation should be monitored diligently during excavation. 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.
- Regular monitoring and inspections of the condition of the temporary slopes for signs of instability, deterioration, sloughing, etc should be carried out by qualified personnel. Appropriate mitigation measures should be implemented.

- Excavations in this area should be limited in size in the area and appropriate monitoring of the residence should take place. Monitoring should consist of a precondition survey along with regular surveying conducted of the nearby utilities, residences, etc.

6.2 Construction Dewatering

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

Due to the relatively low permeability of the silty clay deposit, minor groundwater seepage is anticipated, which should be controllable by conventional temporary dewatering methods. However, runoff and seepage into the excavations from perched groundwater from the fill, old farm tiles and / or utility trenches, and upper granular layers should also be anticipated. In addition, random water bearing seams, pockets and lenses of fine sand may be intersected by the excavation slopes. In adverse conditions, the runoff and seepage from perched groundwater and sand/silt lenses can be significant and accompanied by piping and wash-outs of the fines causing sloughing of the slopes.

Accordingly, provision should be made to prevent runoff and piping erosion of the slope surfaces by blanketing of the slopes with filter fabric and free-draining granular material. The seepage flow should be directed to collection sumps by temporary drainage ditches properly sized, filtered and lined to accommodate the flow rates.

Effective drainage is an important aspect in the life expectancy and performance of any abutment wall, wing wall, or pavement structure associated with the bridge. Permanent sub-drainage should be installed behind abutment and wing walls. Free draining granular material (Granular B Type 1 or approved equivalent) should be installed immediately adjacent to walls to prevent water pressures acting on the walls and to permit downward flow of surface water down into the wall sub-drains. The subdrains should be surrounded by approved granular material and discharged via gravity flow to the storm drain or road ditch system along Highway 401.

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.

6.3 Instrumentation and Monitoring during Construction

As mentioned earlier in Section 3.3, a program of site instrumentation and monitoring of the temporary works during construction should be implemented by the Contractor in addition to the limited instrumentation already installed during the geotechnical investigation.

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

Instrumentation:

A limited number of geotechnical instruments have been installed during the recent geotechnical investigation at the locations of boreholes as indicated in Table 3-2.

Additional instruments should be installed at strategic locations to adequately cover the footprint of the construction area and the adjacent zone of influence. A suggested outline of the additional instruments is provided in Figure 6-1 and consists of:

- Heave/ settlement gauges placed typically at about 1.5 m below the subgrade level;
- Low displacement type of piezometers placed typically below the subgrade level at depths of 0.5 to 1.5 times the depth of excavation; and
- Shallow survey pins (stakes) typically driven > 600 mm into the ground.

The outline in Figure 6-1 is only for general information provided for Contractor's consideration. The type, number and locations of the instrumentations should be developed and revised in consideration of the observations during construction. Particular attention must be given to the nature and condition of the nearby facilities (residences, utilities, etc.) that may be affected by construction and may require additional and / or different type of instruments.

The instruments should be installed, and baseline monitoring (generally 3 readings) should be completed before significant excavation has been occurred.

The monitoring should be completed on a regular basis. As a general guideline, the following schedule should be considered after the completion of the baseline survey (Table 6-1).

Table 6-1: Monitoring Schedule of the Instruments

Instruments	Active Excavation	Active Construction inside Excavation	Backfilling	Post-Construction
Piezometer	EOD	D	W	M
Heave/Settlement Gauge	EOD	EOD	W	M
Inclinometer	TPW	EOD	BW	M
Survey Pins	TPW	EOD	BW	M

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

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

Monitoring Alert Levels and Contingencies:

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

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

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

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

6.4 Corrosion Potential

Analytical testing was carried out on samples of the silt and clay obtained in boreholes BH B9-1 (Sample 10), BH B9-2 (Sample 25) and BH B9-3 (Sample 12). Table 6-2 provides the results of various analyses carried out on the soil samples to assess the potential for corrosion on concrete.

Table 6-2: 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 BH B9-1 (Sample 10)	172.6	8.06	110	3070	<0.2	144
Borehole BH B9-2 (Sample 25)	151.0	7.77	134	1920	<0.2	429
Borehole BH B9-3 (Sample 12)	170.8	8.07	105	4170	<0.2	34

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 (ref. R-10).

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 (ref. R-2).

The above results and recommendations should be reviewed by a corrosion specialist.

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.

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

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

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

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

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

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

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

8 Closure

The design for Bridge B-9 was developed by Dr. Siavash Farhangi, P.Eng. and checked by Dr. Dan Dimitriu, P.Eng. (Lead Designer). The project was executed under the technical direction of Dr. Narendra S. Verma, P.Eng. (Technical Director) who also provided the senior review of the report. Mr. Matt Oldewening, P.Eng., managed the geotechnical investigation and Mr. Brian Lapos, P.Eng., is the project manager.

Mr. Zuhtu Ozden, P.Eng. and Dr. Andrew Smith of Coffey Geotechnics provided the peer review. The cooperation received from Ms. Biljana Rajlic, P.Eng. and Mr. Philip Murray, P.Eng. of Hatch Mott McDonald and Mr. Daniel Muñoz, P.Eng. of PIC during the design study is gratefully acknowledged.

Yours truly,
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Drawings

METRIC

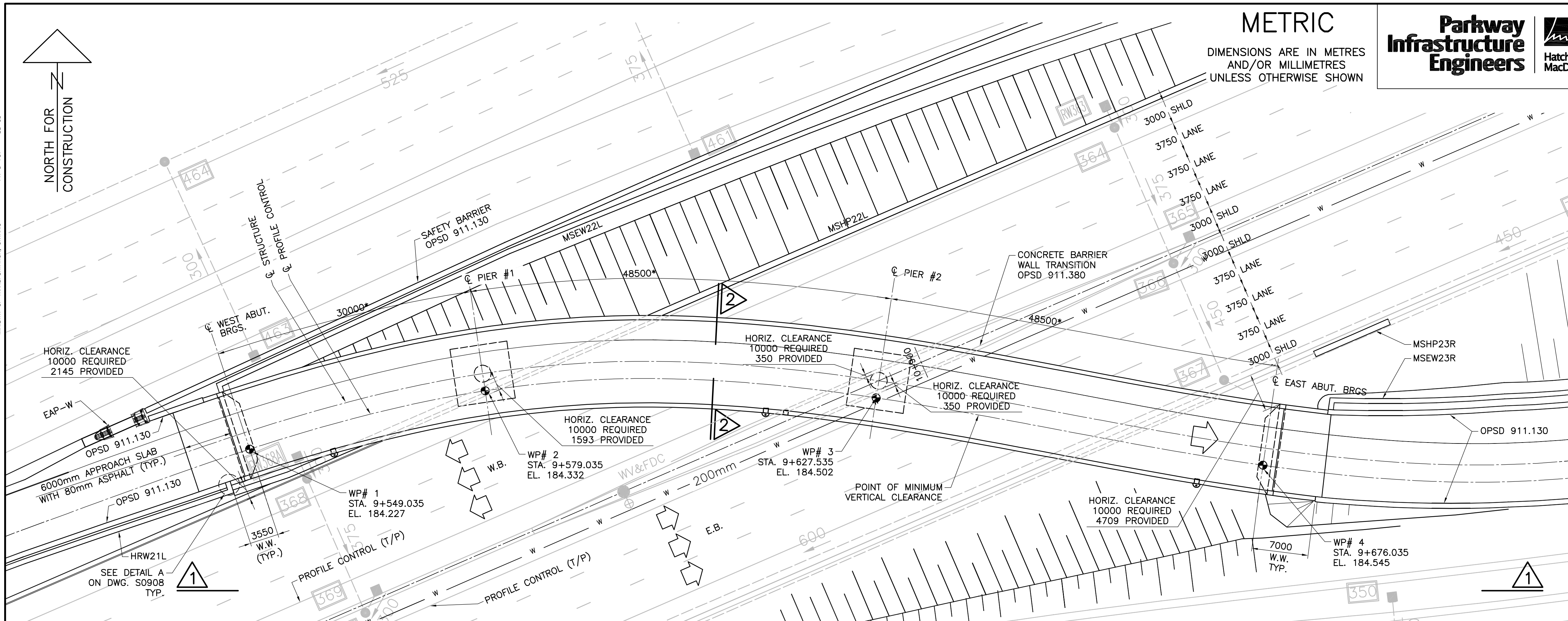
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | Hatch Mott MacDonald

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

NEW CONSTRUCTION
BRIDGE B–9
EASTBOUND RAMP UNDERPASS NEAR HURON CHURCH LINE
GENERAL ARRANGEMENT

SHEET S0901
Phase 1
IFC



PLAN
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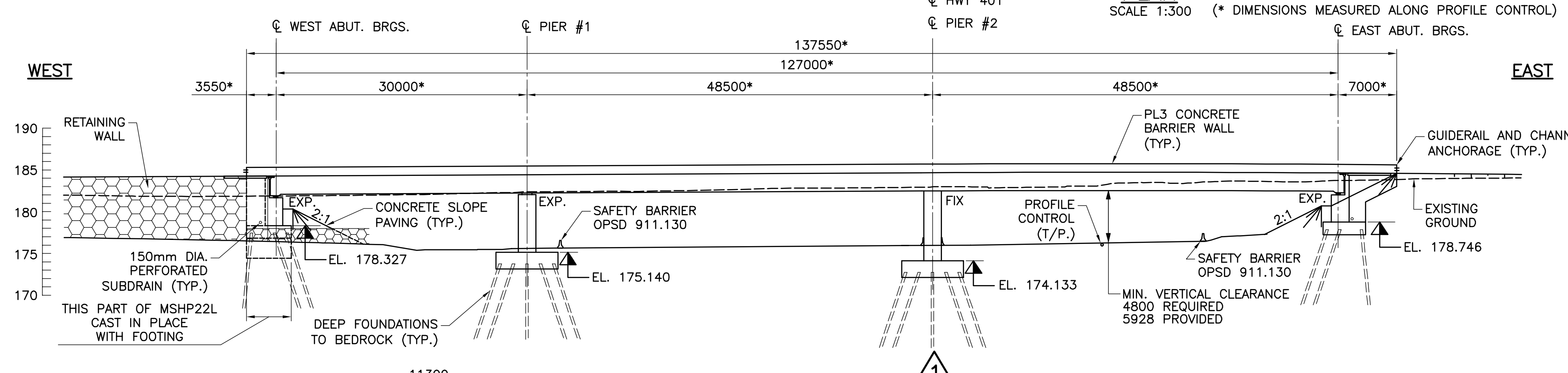

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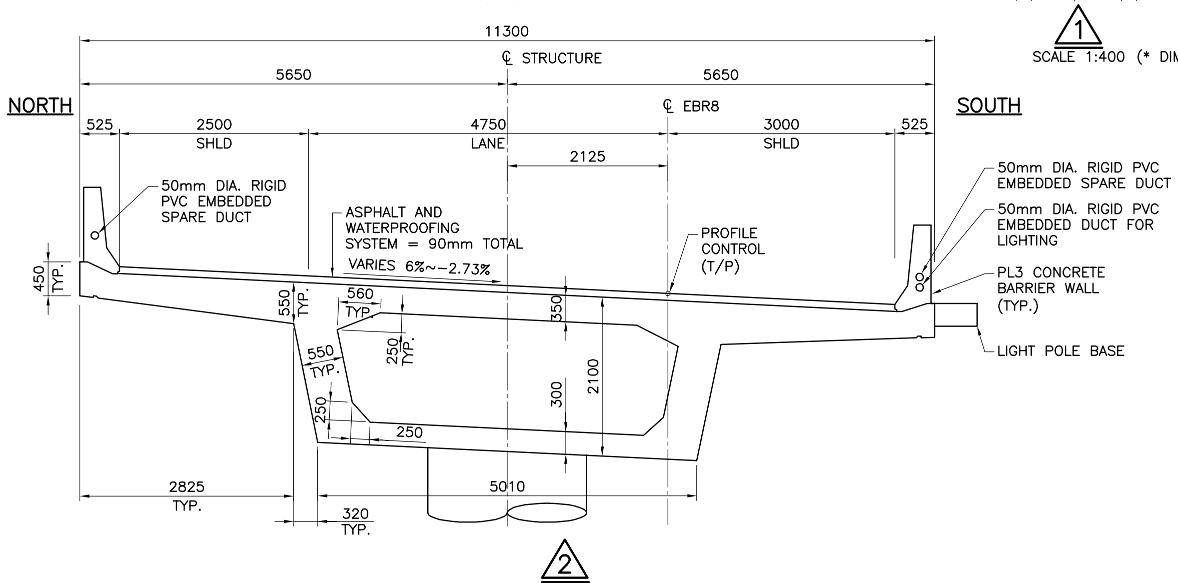

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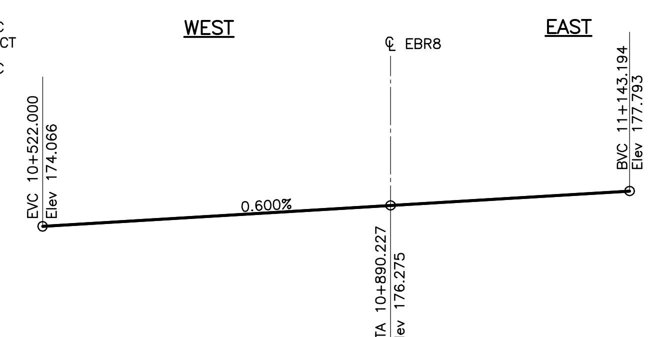
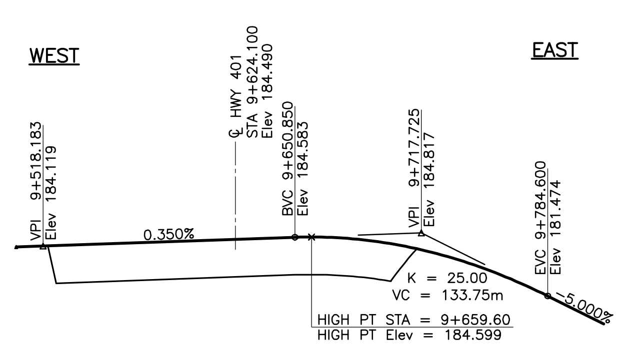


PROFILE OF HWY 401
N.T.S.



PROFILE OF EBR 8
N.T.S.

GENERAL NOTES:

1. CLASS OF CONCRETE:
   - DECK 50MPa
   - MASS CONCRETE 20MPa
   - REMAINDER 30MPa

2. CLEAR COVER TO REINFORCING STEEL:
   - FOOTING .... 100 ± 25
   - DECK : TOP SLAB TOP .... 70 ± 20
     - TOP SLAB BOT .... 50 ± 10
     - BOTTOM SLAB TOP .... 50 ± 10
     - BOTTOM SLAB BOT .... 60 ± 10
     - WEB .... 60 ± 10

   REMAINDER .... 70 ± 20
   UNLESS OTHERWISE NOTED.

3. REINFORCING STEEL:
   - REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
   - STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 OR TYPE XM–28 AND HAVE A MINIMUM YIELD STRENGTH OF 500MPa, UNLESS OTHERWISE SPECIFIED.
   - BAR MARKS WITH PREFIX 'C' DENOTE COATED BARS.
   - BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
   - UNLESS SHOWN OTHERWISE TENSION LAP SPLICES SHALL BE CLASS B
   - BARS HOOKS SHALL HAVE STANDARD HOOK DIMENSION USING MINIMUM BEND DIAMETER, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12–1 AND SS12–2, UNLESS INDICATED OTHERWISE.

4. FOR ALL HIGHWAY WORKS REFER TO HIGHWAY NEW CONSTRUCTION DRAWINGS.
5. FOR ALL ELECTRICAL AND ATMS WORKS REFER TO ELECTRICAL AND ATMS NEW CONSTRUCTION DRAWINGS.
6. FOR ALL UTILITY WORKS REFER TO UTILITY NEW CONSTRUCTION DRAWINGS.
7. APPROVED RSS WALL SUPPLIER TO REFER TO UTILITIES NEW CONSTRUCTION DRAWINGS AND CONFIRM LOCATION OF ALL UTILITIES. RSS WALL DESIGN SHALL ACCOUNT FOR ALL INTERFERENCE WITH UTILITIES.
8. RSS WALL SHALL BE DESIGNED AND CONSTRUCTED IN ACCORDANCE WITH THE 'MTO RSS DESIGN GUIDELINES' AND SPECIAL PROVISIONS SP599S22 AND SP599S23.
9. THE FACTOR–OF–SAFETY AGAINST EXTERNAL MODES OF FAILURE RSS WALLS SHALL BE AS CANADIAN FOUNDATION ENGINEERING MANUAL (CFEM).

APPLICABLE STANDARD DRAWINGS

- OPSD 3101.150 WALLS, ABUTMENT, BACKFILL, MINIMUM GRANULAR REQUIREMENT
- OPSD 3121.150 WALLS, RETAINING, BACKFILL, MINIMUM GRANULAR REQUIREMENT
- OPSD 3360.100 DECK LIGHT POLE BASES STRUCTURES WITH BARRIER WALLS
- OPSD 3370.100 DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
- OPSD 3370.101 DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS
- OPSD 3419.100 BARRIERS AND RAILINGS, STEEL GUIDERAIL AND CHANNEL ANCHORAGE
- OPSD 3941.200 FIGURES IN CONCRETE, SITE NUMBER AND DATE, LAYOUT
- OPSD 3950.100 JOINTS, CONCRETE EXPANSION AND CONSTRUCTION, ON STRUCTURE

CONSTRUCTION NOTES:

1. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
2. THE CONTRACTOR IS FULLY RESPONSIBLE FOR PROTECTION OF ALL EXISTING UTILITIES DURING CONSTRUCTION OPERATIONS UNLESS THE EXISTING UTILITIES ARE TO BE RELOCATED.
3. THE CONTRACTOR IS FULLY RESPONSIBLE FOR GROUNDWATER CONTROL ON TIMING OF CONSTRUCTION AND PREVAILING WEATHER CONDITIONS.
4. TEMPORARY EXCAVATIONS, SUBGRADE EXPOSURE AND PROTECTION, AND BACKFILLING SHALL CONFORM TO OPSS 902.
5. SETTLEMENTS AND GROUND DEFORMATIONS SHALL BE MONITORED DURING AND AFTER CONSTRUCTION.
6. VIBRATIONS SHALL BE MONITORED AT STRATEGIC LOCATIONS ON TEMPORARY SLOPES AND ADJACENT TO UTILITIES DURING PILING AND CONSTRUCTION.

LIST OF ABBREVIATIONS

ABUT.	ABUTMENT	SCL	SPEED CHANGE LANE
BRGS.	BEARINGS	STA.	STATION
DIA.	DIAMETER	SHLD	SHOULDER
E.B.	EASTBOUND	T/P	TOP OF PAVEMENT
EL.	ELEVATION	THK.	THICK
EXP.	EXPANSION	TYP.	TYPICAL
HORIZ.	HORIZONTAL	VERT.	VERTICAL
N.T.S.	NOT TO SCALE	W.B.	WESTBOUND
RW	RETAINING WALL	WP	WORKING POINT

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

REVISIONS

DATE	REV.	BY	DESCRIPTION
16–MAR–12	0	CW	ISSUED FOR CONSTRUCTION

DESIGN CW	CHK BR	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN YZ	CHK MAS	SITE 6–609	DATE 12–JUL–11

DOC: 285380–03–060–WIP1–0901

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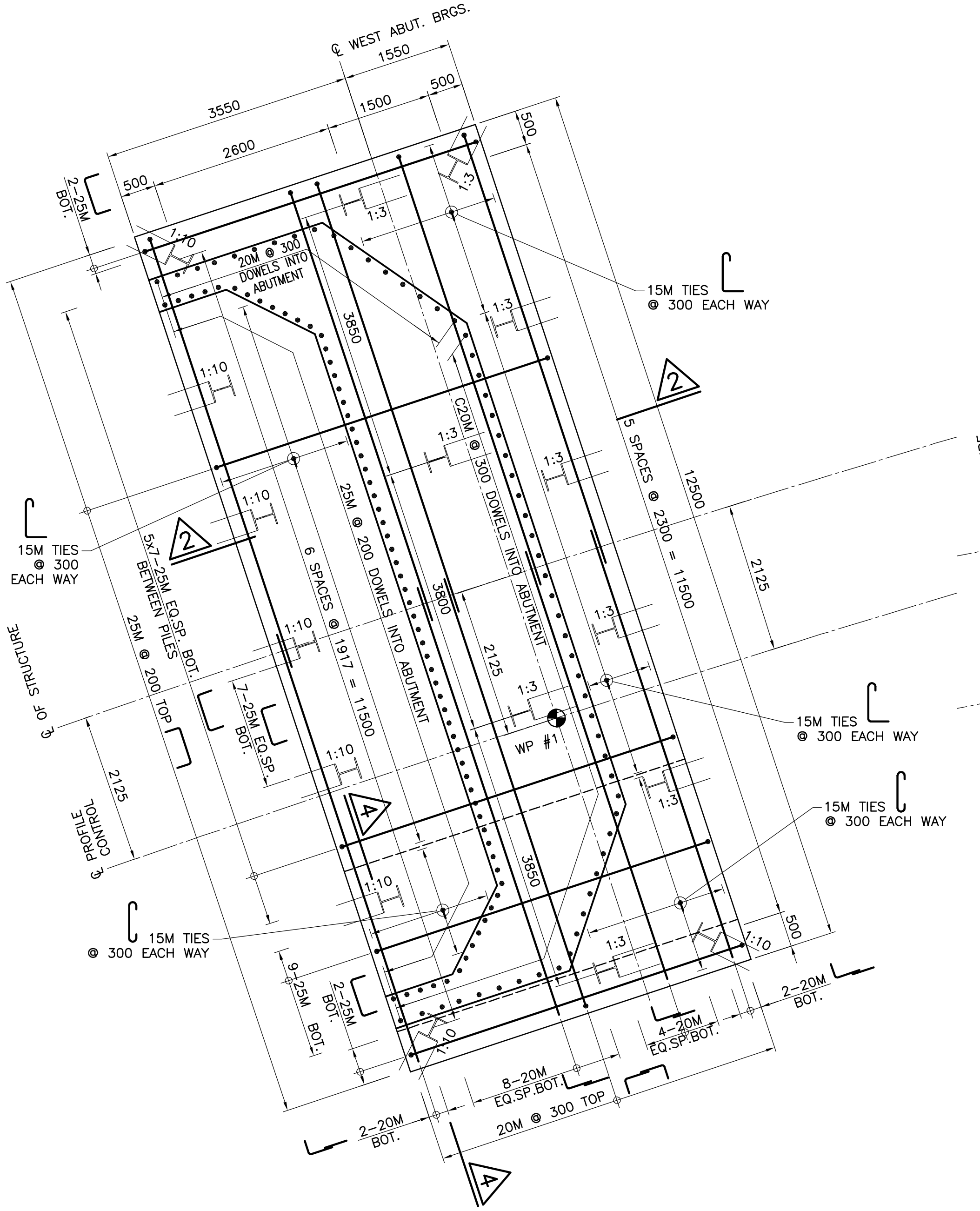
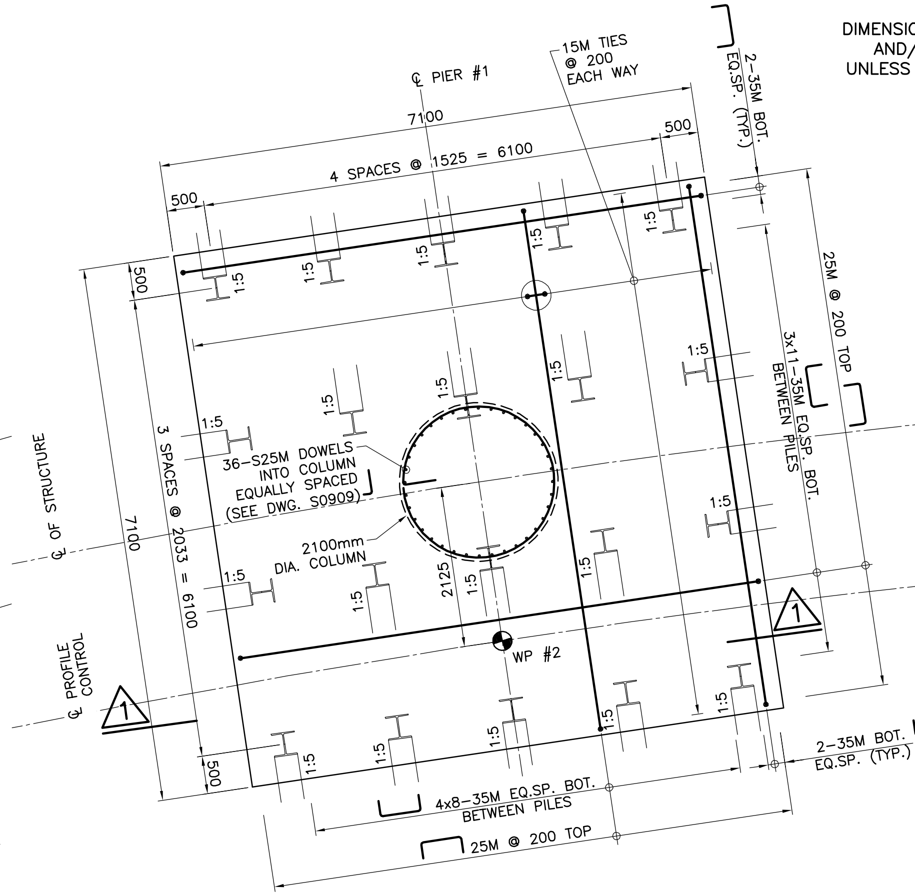
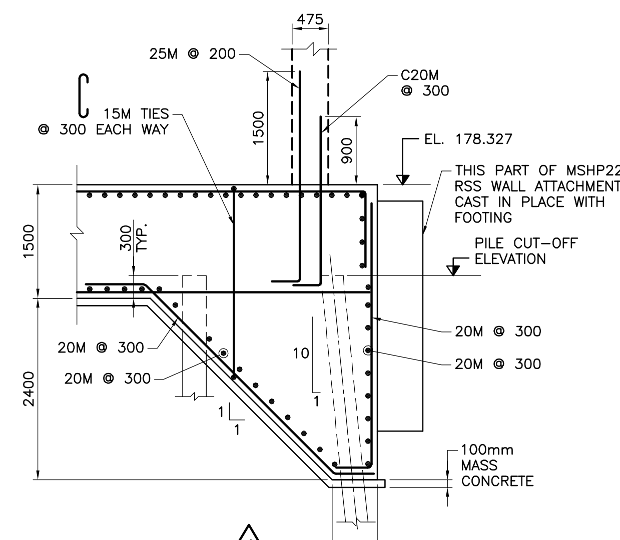
MINISTRY OF TRANSPORTATION, ONTARIO PH–D–707 88–05



WEST ABUTMENT PLAN
SCALE 1:50



PIER #1 PLAN
SCALE 1:50



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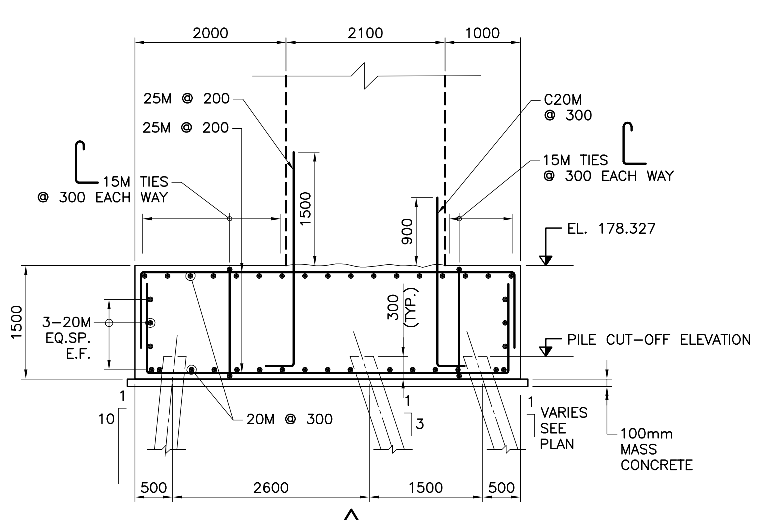

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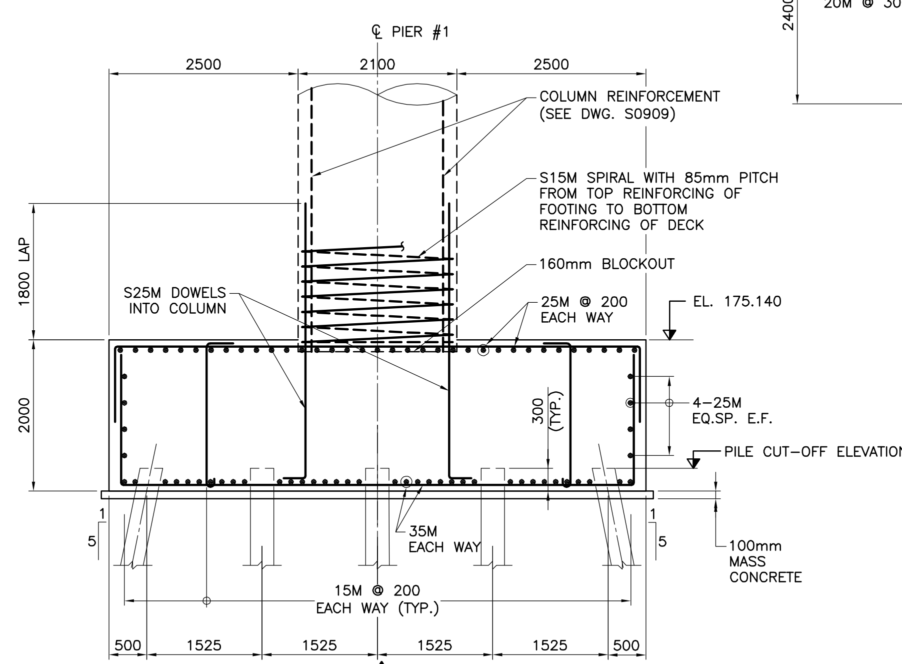

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DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers
Hatch Mott MacDonald

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

NEW CONSTRUCTION
BRIDGE B-9
EASTBOUND RAMP UNDERPASS NEAR HURON CHURCH LINE
FOUNDATION LAYOUT & REINFORCEMENT I

SHEET S0904

Phase 1

IFC

WORKING POINT DATA		
LOCATION	NORTHING	EASTING
WP #1	4 679 240.368	332 598.628
WP #2	4 679 221.802	332 622.151

PILE DATA			
LOCATION	No. REQUIRED	LENGTH (m)	BATTER
WEST ABUTMENT	17	32	SEE PLAN
PIER #1	20	26	SEE PLAN

NOTES: (STANDARD)

1. FOR GENERAL NOTES SEE GENERAL ARRANGEMENT DRAWING.
2. THIS DRAWING TO BE READ IN CONJUNCTION WITH THE RETAINED SOIL SYSTEM AND ABUTMENT LAYOUT DRAWINGS.

PILE NOTES:

1. PILE LENGTHS SHOWN ARE ESTIMATED LENGTHS FROM THE CUT-OFF TO THE ESTIMATED BEDROCK / REFUSAL SURFACE.
2. ALL PILES ARE HP 310X110 STEEL H PILES.
3. ALL PILES SHALL BE FITTED WITH TYPE I DRIVING SHOE PER OPSD 3000.100 OR APPROVED EQUIVALENT.
4. PILE SPLICES SHALL BE BUTT WELDED AS PER OPSD 3000.150 AND OPSS 903. SPLICE PLATES ARE NOT PERMITTED.
5. ALL PILES ARE TO BE DRIVEN TO BEDROCK OR TO REFUSAL IN THE VERY DENSE COHESIONLESS DEPOSIT OVERLYING BEDROCK IN ACCORDANCE WITH SS103-11 TO DEVELOP AN ULTIMATE GEOTECHNICAL RESISTANCE OF 4000 KN, GIVING A DESIGN FACTORED ULS RESISTANCE OF 2000 KN.
6. THE PILE ULTIMATE GEOTECHNICAL RESISTANCE AND REFUSAL CRITERIA SHALL BE CONFIRMED ON AT LEAST 3% OF THE PILES BY PDA METHOD SUPPLEMENTED WITH STATIC LOAD TESTS IN THE AREA OF THE STRUCTURE.
7. PILE DRIVING EQUIPMENT SHALL BE APPROPRIATE TO THE DRIVING CONDITIONS TO DEVELOP THE ULTIMATE GEOTECHNICAL RESISTANCE, AND PREVENT DAMAGES TO THE PILES DURING DRIVING. CONSIDERATION SHOULD BE GIVEN TO POTENTIAL DRIVING DIFFICULTIES DUE TO THE PRESENCE OF COBBLES OR BOULDERS.
8. HAMMER DETAILS (HAMMER TYPE AND MODEL, RATED ENERGY, HELMET AND CUSHION DETAILS) SHALL BE SUBMITTED 10 DAYS PRIOR TO THE EQUIPMENT MOBILIZATION TO THE SITE.
9. SURVEY ALL PILE HEAD ELEVATIONS AT END OF DRIVING AND JUST PRIOR TO FORMING OF PILE CAP. RE-TAP PILES WHERE UPLIFT >5 MM OR AS DIRECTED BY THE SITE ENGINEER.
10. THE CONTRACTOR SHALL MONITOR FOR POTENTIAL EMISSIONS OF NATURAL GASES AND GROUNDWATER SEEPAGE DURING PILE DRIVING AND IMPLEMENT MITIGATION MEASURES AS REQUIRED.
11. THE CONTRACTOR SHALL MONITOR VIBRATIONS AT STRATEGIC LOCATIONS (E.G. TEMPORARY SLOPES, UTILITIES AND STRUCTURES) AND ESTABLISH APPROPRIATE FREQUENCY BASED LIMITS ON PEAK PARTICLE VELOCITIES IN ORDER TO PREVENT DAMAGE CAUSED BY PILE DRIVING.

APPLICABLE STANDARD DRAWINGS:

OPSD-3000.100 FOUNDATION PILES – STEEL H-PILE DRIVING SHOE

OPSD-3000.150 FOUNDATION PILES – STEEL H-PILE SPLICE

LIST OF ABBREVIATIONS

BOT.	BOTTOM
O.F.	OUTSIDE FACE
I.F.	INSIDE FACE
E.F.	EACH FACE
EQ.SP.	EQUALLY SPACED
T/F	TOP OF FOOTING
TYP.	TYPICAL
EL.	ELEVATION
WP	WORKING POINT

NOTES:

THIS DRAWING TO BE READ IN CONJUNCTION WITH DWG. S0901, S0905 AND S0906

REVISIONS			
16-MAR-12	0	CW	ISSUED FOR CONSTRUCTION
DATE	REV.	BY	DESCRIPTION

DESIGN CW	CHK BR	CODE CAN/CSA S6-06	LOAD CL-625-ONT
DRAWN YZ	CHK MAS	SITE 6-609	DATE 12-JUL-11

DOC: 285380-03-061-WIP1-0904

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

METRIC

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Parkway Infrastructure Engineers | Hatch Mott MacDonald

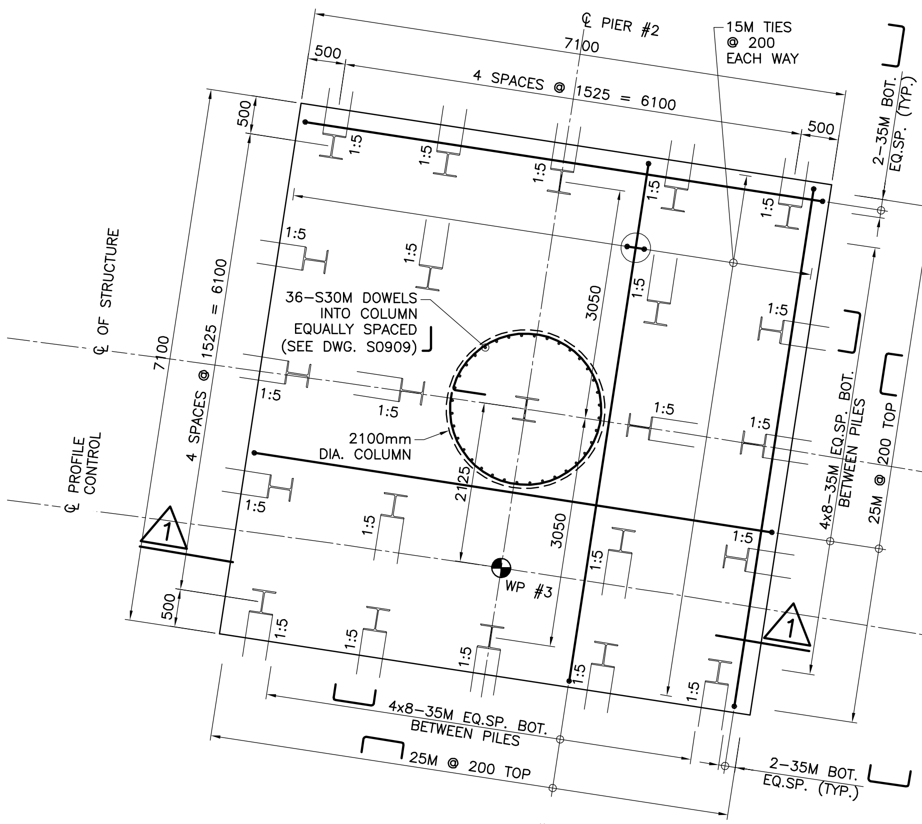
Windsor–Essex Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
BRIDGE B–9
EASTBOUND RAMP UNDERPASS NEAR HURON CHURCH LINE
FOUNDATION LAYOUT & REINFORCEMENT II

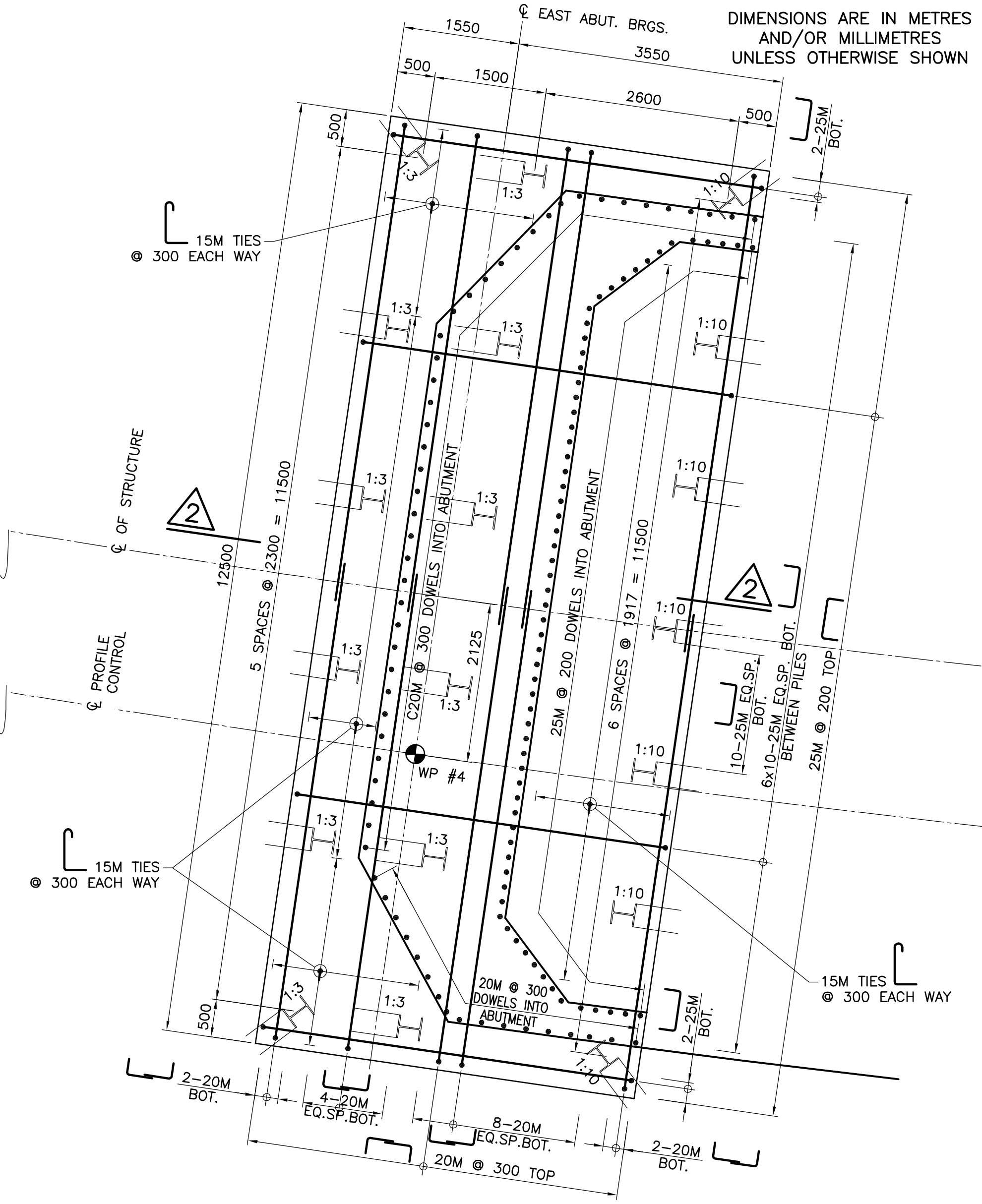
SHEET S0905

Phase 1

IFC



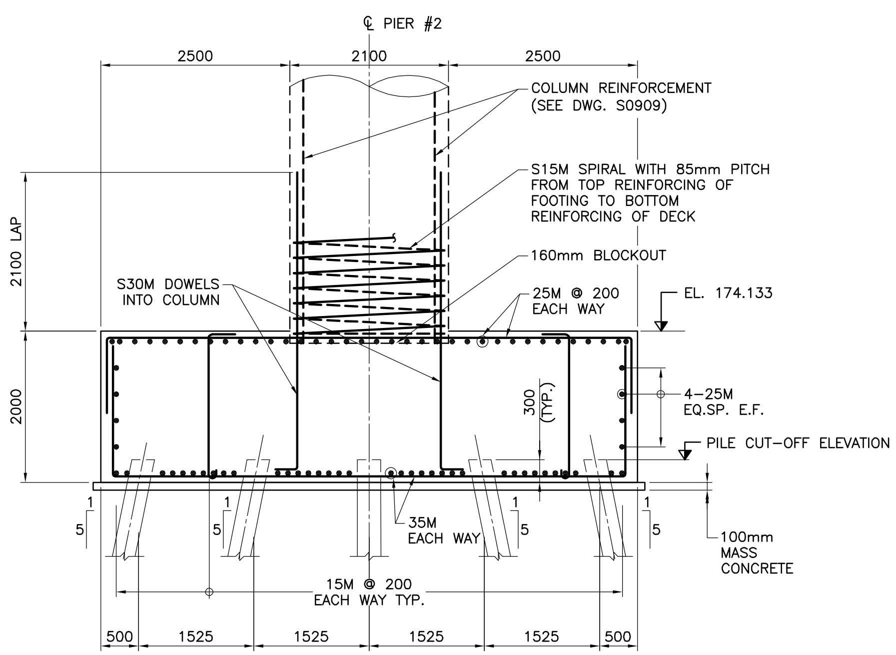
PIER #2 PLAN
SCALE 1:50



EAST ABUTMENT PLAN
SCALE 1:50

WORKING POINT DATA		
LOCATION	NORTHING	EASTING
WP #3	4 679 183.095	332 651.050
WP #4	4 679 140.246	332 673.762

PILE DATA			
LOCATION	No. REQUIRED	LENGTH (m)	BATTER
PIER #2	23	24	SEE PLAN
EAST ABUTMENT	19	30	SEE PLAN



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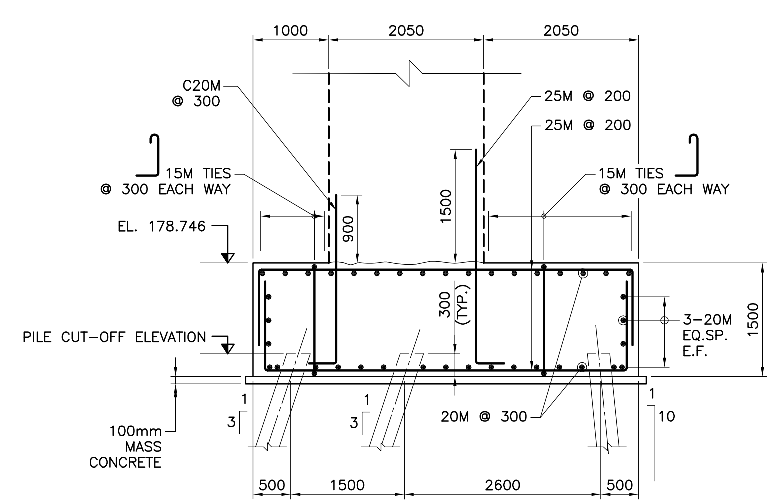

2
1:50

LIST OF ABBREVIATIONS

BOT.	BOTTOM
O.F.	OUTSIDE FACE
I.F.	INSIDE FACE
E.F.	EACH FACE
EQ.SP.	EQUALLY SPACED
T/F	TOP OF FOOTING
TYP.	TYPICAL
EL.	ELEVATION
WP	WORKING POINT

APPLICABLE STANDARD DRAWING:

OPSD 3000.150 FOUNDATION PILES STEEL H–PILES SPLICE

NOTES:

THIS DRAWING TO BE READ IN CONJUNCTION WITH DWG. S0901, S0904 AND S0907

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

REVISIONS			
16–MAR–12	0	CW	ISSUED FOR CONSTRUCTION
DATE	REV.	BY	DESCRIPTION

DESIGN CW	CHK BR	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN YZ	CHK MAS	SITE 6–609	DATE 12–JUL–11

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MINISTRY OF TRANSPORTATION, ONTARIO PH–D–707 88–05

METRIC

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

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

REVISIONS	DATE	REV.	BY	DESCRIPTION
	16–MAR–12	0	SF	ISSUED FOR CONSTRUCTION

DESIGN SF APR NSV DATE 15–JUL–11

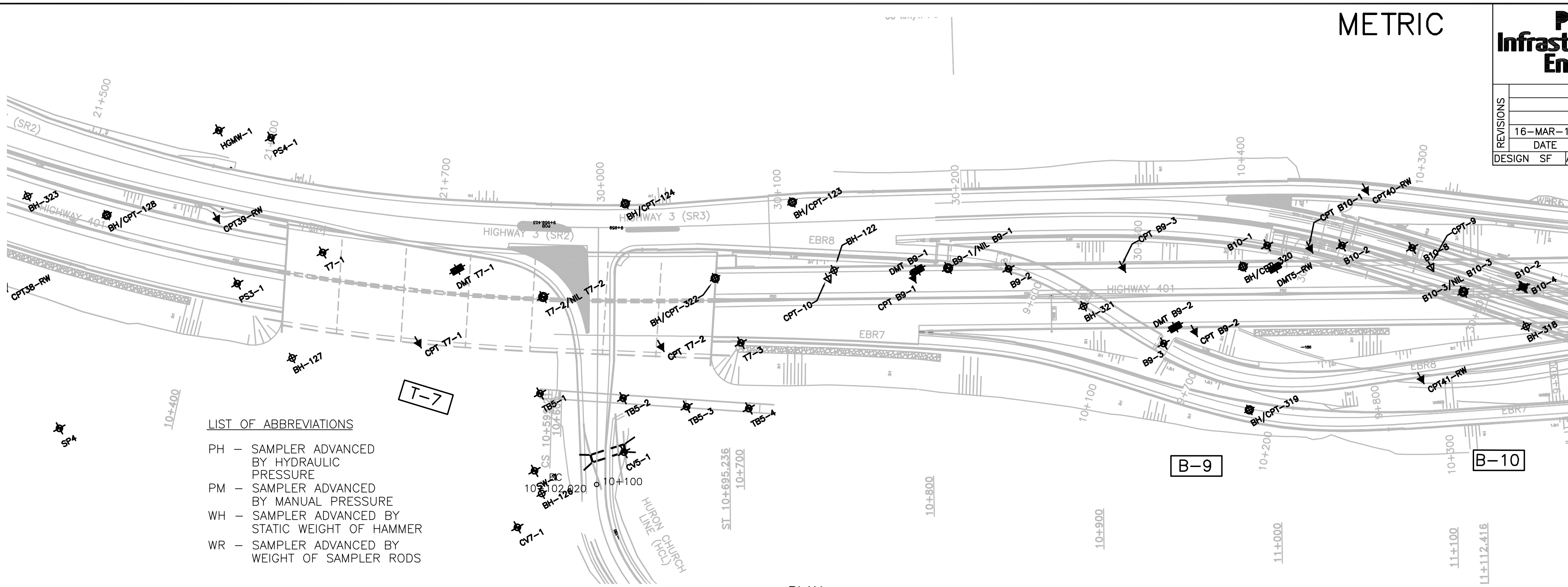
LOCATION PLAN & INTERPRETED STRATIGRAPHIC PROFILE

STA 10+400L TO STA 11+000L

SHEET G0901

Phase 1

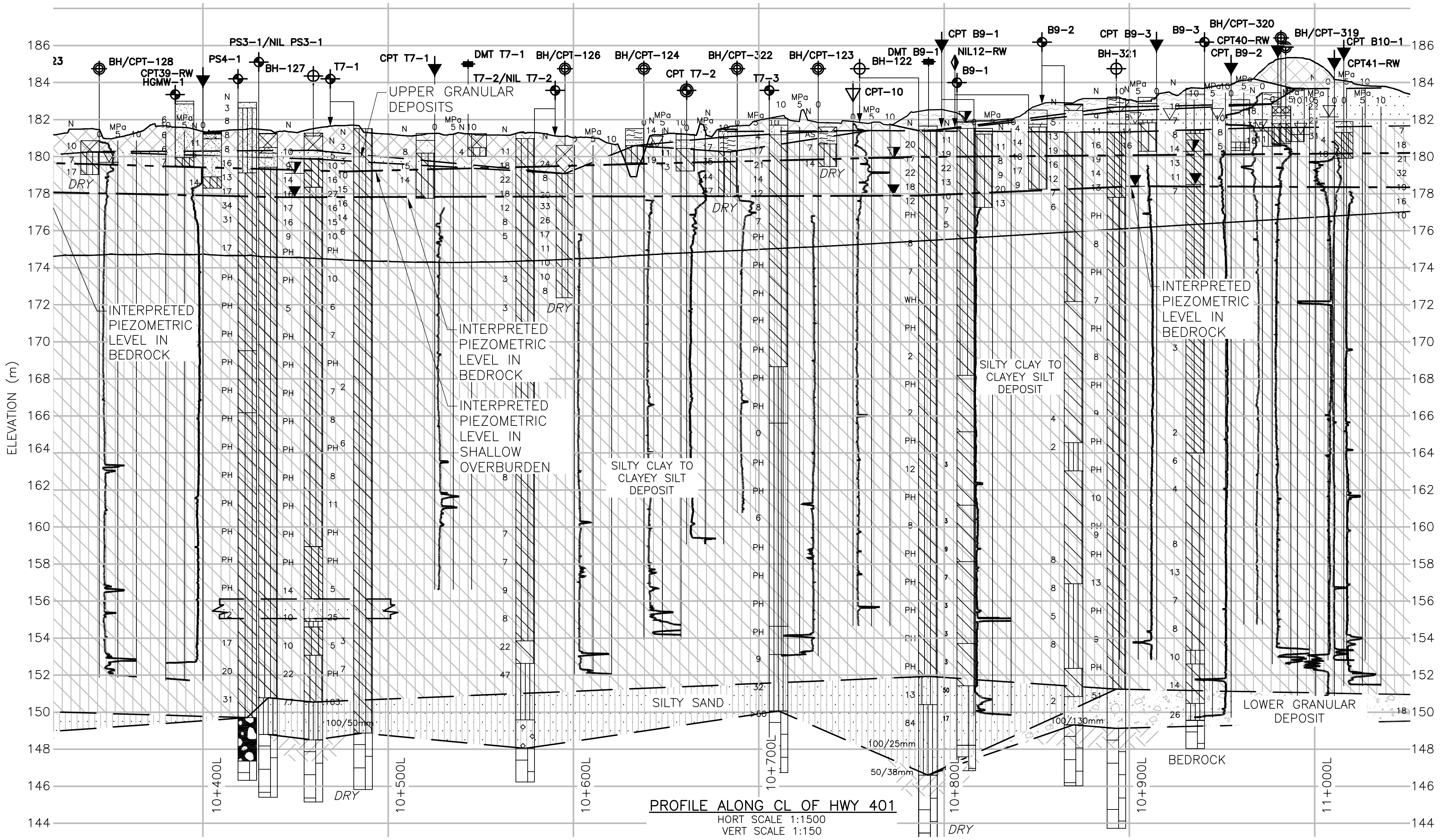
IFC



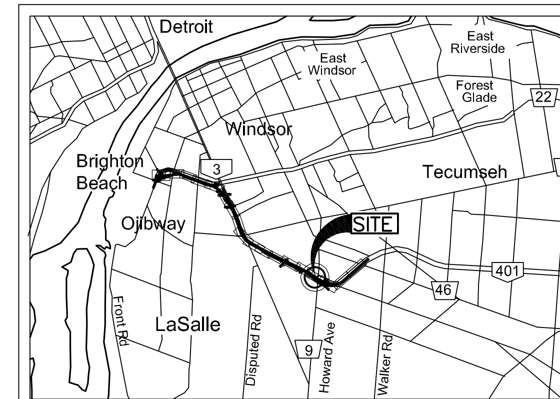
LIST OF ABBREVIATIONS

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

PLAN
SCALE 1:1500



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



KEY PLAN
SCALE
1 0 2 4Km

LEGEND

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

NOTES

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

SCALES
15 0 30m Horizontal
1.5 0 3 Vertical

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DOC: 285380-04-090-WIP1-0901

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

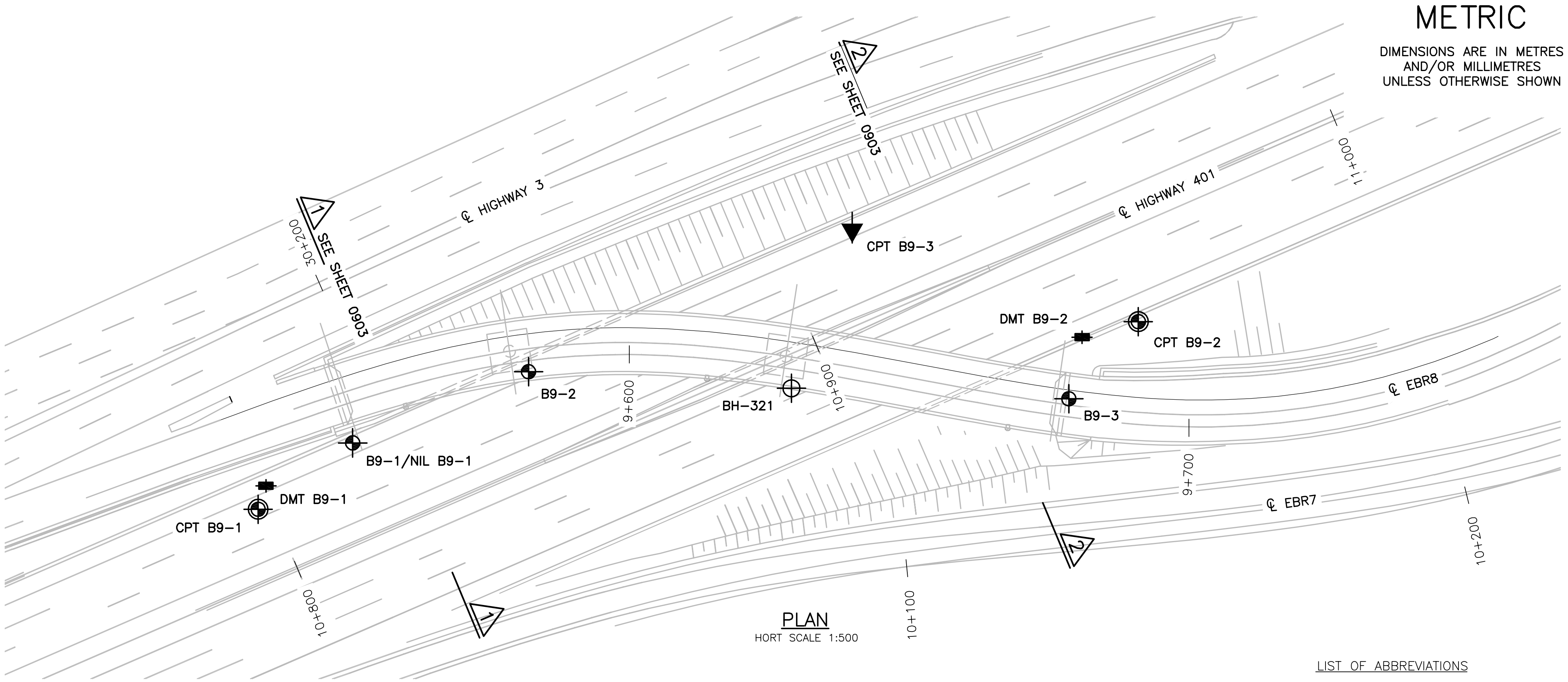
Windsor–Essex Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
BRIDGE B–9
EAST BOUND RAMP UNDERPASS NEAR HURON CHURCH LINE
BOREHOLE LOCATIONS & SOIL STRATA

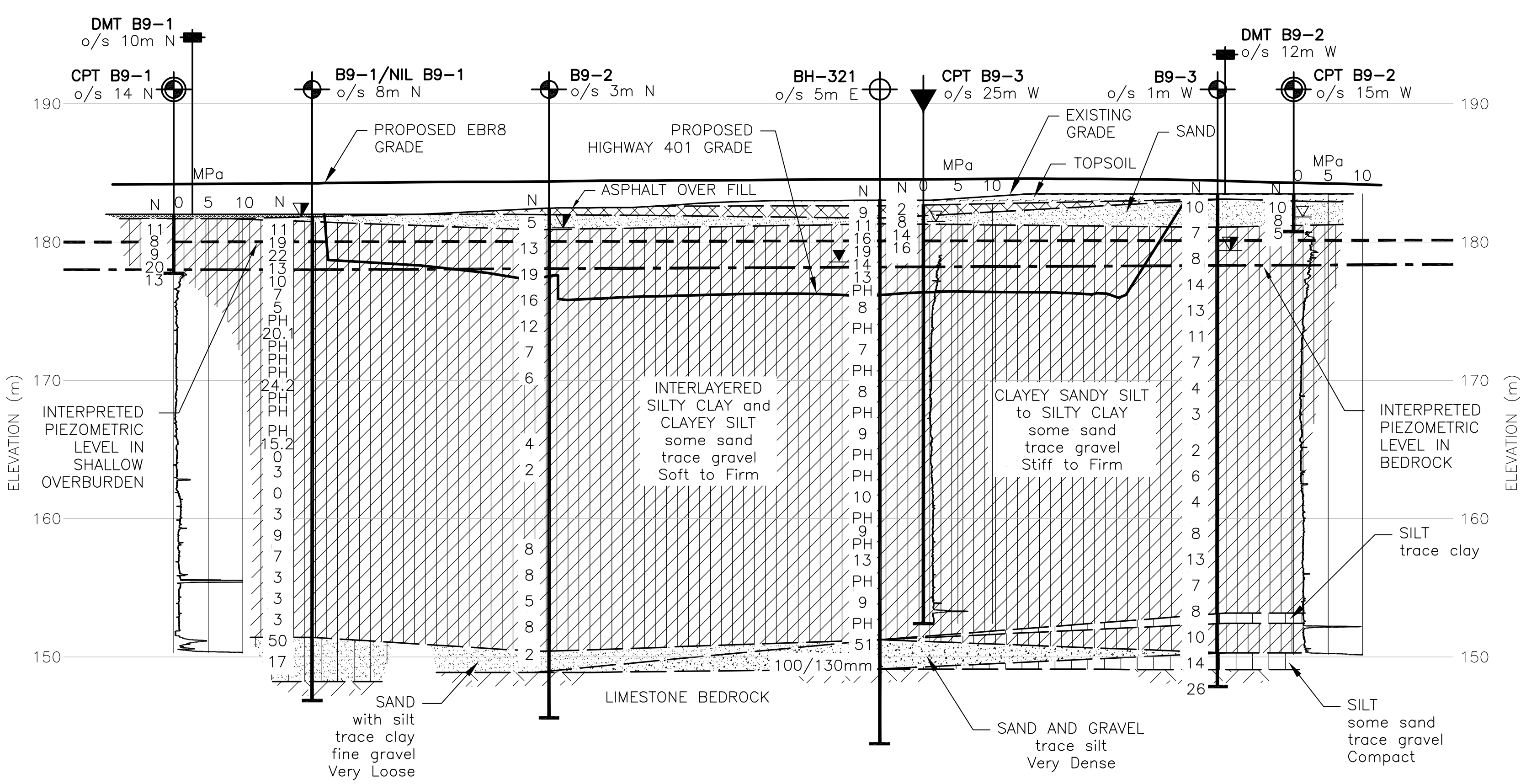
SHEET G0902

Phase 1

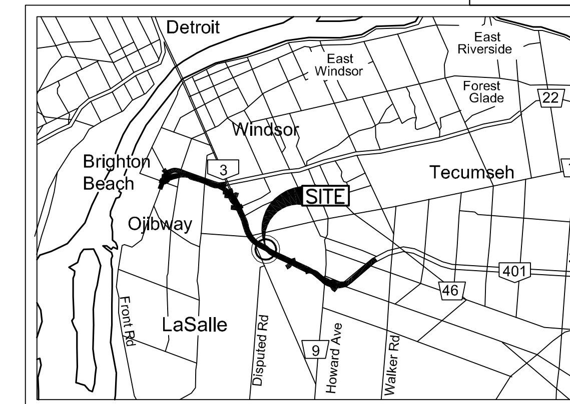
IFC



PLAN
HORT SCALE 1:500



PROFILE ALONG CL OF EBR8
HORT SCALE 1:500
VERT SCALE 1:250



KEY PLAN
SCALE
1 0 2 4Km

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

No.	ELEVATION	CO–ORDINATES (UTM, NAD 83 ZONE 17) NORTHING	EASTING
AMEC BOREHOLES			
B9–1/NIL B9–1	181.9	4679235.3	332593.8
B9–2	182.4	4679218.9	332622.2
B9–3	183.5	4679140.0	332677.6
CPT B9–1	182.4	4679241.3	332574.3
CPT B9–2	183.9	4679138.6	332696.0
CPT B9–3	182.7	4679189.2	332678.6
DMT B9–1	183.0	4679242.7	332578.4
DMT B9–2	183.5	4679144.8	332687.7
PREVIOUS BOREHOLES			
BH–321	183.1	4679179.9	332649.0

LEGEND

- BOREHOLE CURRENT INVESTIGATION
- BOREHOLE AND NILCON VANE CURRENT INVESTIGATION
- SW/SP HOLE (HYDROGEOLOGY) CURRENT INVESTIGATION
- NILCON VANE CURRENT INVESTIGATION
- CPT – CURRENT INVESTIGATION
- DMT – CURRENT INVESTIGATION
- BOREHOLE PREVIOUS INVESTIGATION
- BOREHOLE, CPT AND NILCON VANE PREVIOUS INVESTIGATIONS
- CPT –PREVIOUS INVESTIGATION
- N SPT N–VALUE
- 16 BLOWS/0.3m UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
- PH – SAMPLE OBTAINED UNDER HYDRAULIC PRESSURE
- MHSG – MAGNETIC HEAVE/SETTLEMENT GAUGE
- P – VIBRATING WIRE PIEZOMETER
- DRY BOREHOLE DRY DURING DRILLING
- WATER LEVEL DURING DRILLING
- WATER LEVEL (SHALLOW PIEZO)
- WATER LEVEL (DEEP PIEZO)
- MPa 0 5 10 CPT–qc

NOTES

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

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS

DATE	REV.	BY	DESCRIPTION
16–MAR–12	0	SF	ISSUED FOR CONSTRUCTION

DESIGN SF	CHK NSV	CODE CAN/CSA S6–06	LOAD CL–625–ON
DRAWN MM	CHK DD	SITE 6–609	DATE 14–JUL–11

DOC: 285380–04–090–WIP1–0902

DATE PLOTTED: 3/16/2012 4:44:37 PM
FILE LOCATION: C:\pwworking\hmmg_285380\stephen.labute@amec.com\dms09829\285380-04-090-WIP1-0902.dwg

MINISTRY OF TRANSPORTATION, ONTARIO PH–D–707 88–05

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

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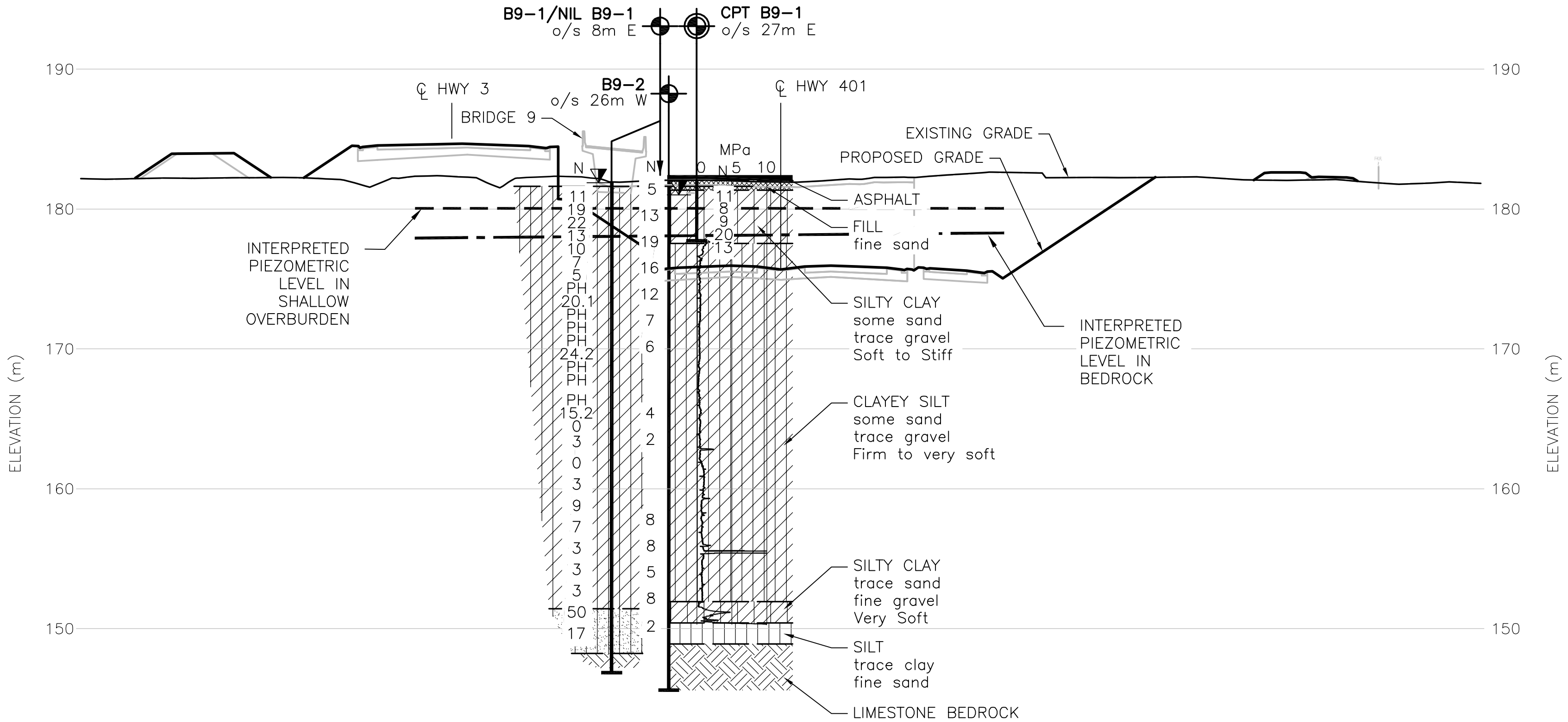
Windsor–Essex Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
BRIDGE B–9
EAST BOUND RAMP UNDERPASS NEAR HURON CHURCH LINE
SOIL STRATIGRAPHY

SHEET G0903

Phase 1

IFC



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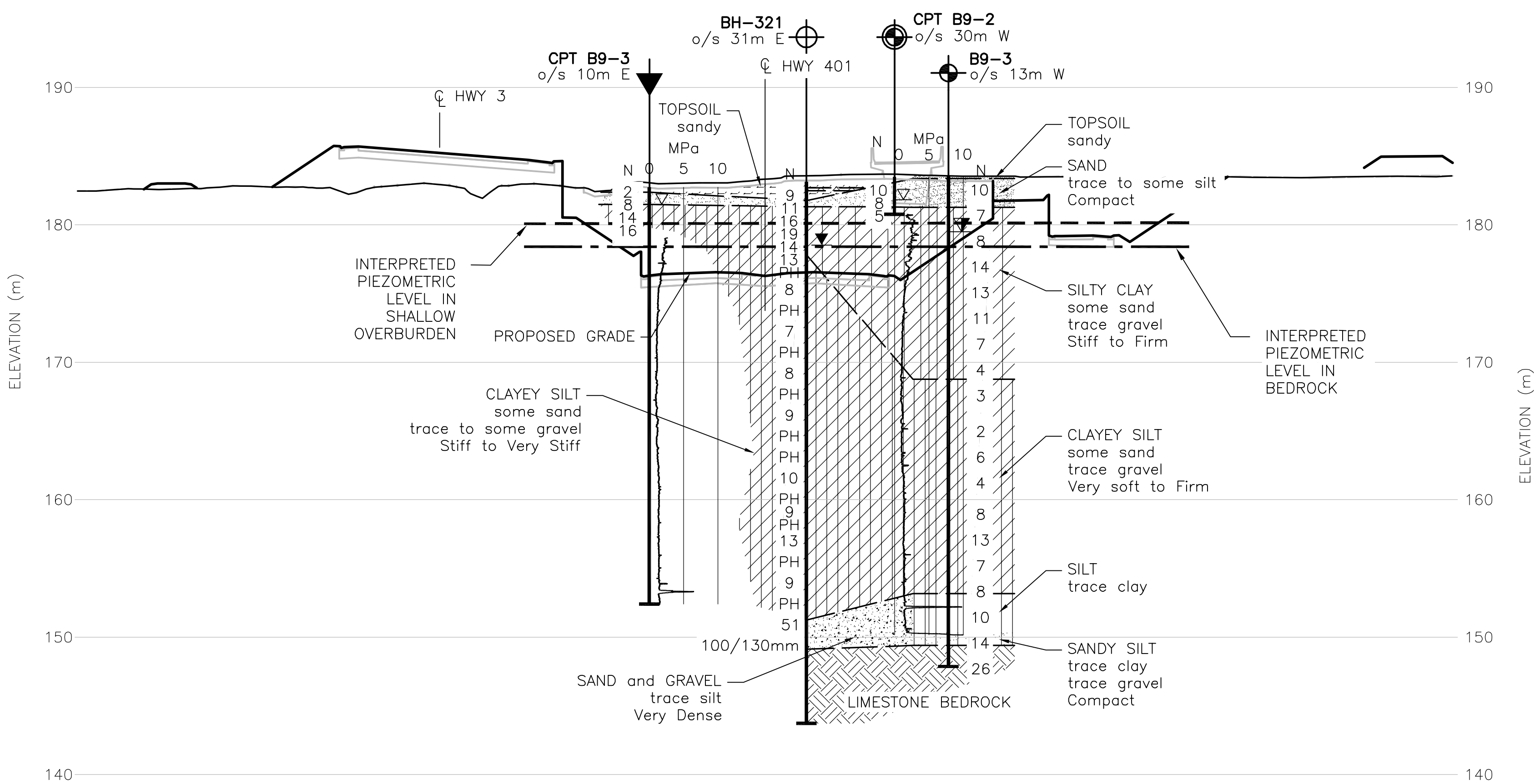
VERT SCALE 1:250



HORT SCALE 1:500
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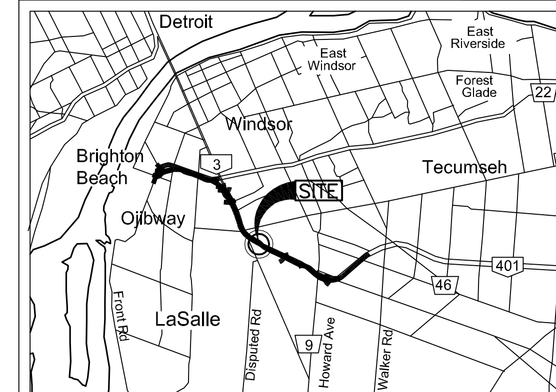

KEY PLAN
SCALE
1 0 2 4Km

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

No.	ELEVATION	CO–ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC BOREHOLES			
B9–1/NIL B9–1	181.9	4679235.3	332593.8
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DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

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
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- P – VIBRATING WIRE PIEZOMETER
- DRY BOREHOLE DRY DURING DRILLING
- WATER LEVEL DURING DRILLING
- WATER LEVEL (SHALLOW PIEZO)
- WATER LEVEL (DEEP PIEZO)
- MPa 0 5 10 CPT–qc

NOTES

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3. ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

REVISIONS DATE	REV.	BY	DESCRIPTION
16–MAR–12	0	SF	ISSUED FOR CONSTRUCTION

DESIGN SF	CHK NSV	CODE CAN/CSA–S6–06	LOAD CL–625–ON
DRAWN MM	CHK DD	SITE 6–609	DATE 14–JUL–11

DOC: 285380–04–091–WIP1–0903

DATE PLOTTED: 3/16/2012 4:45:11 PM
FILE LOCATION: C:\pwworking\hmmg_285380\stephen.labute@amec.com\dms09829\285380–04–091–WIP1–0903.dwg

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

Figures

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PARKWAY INFRASTRUCTURE CONSTRUCTORS

Figure 3-1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures

(Ladd & DeGroot, 2004)

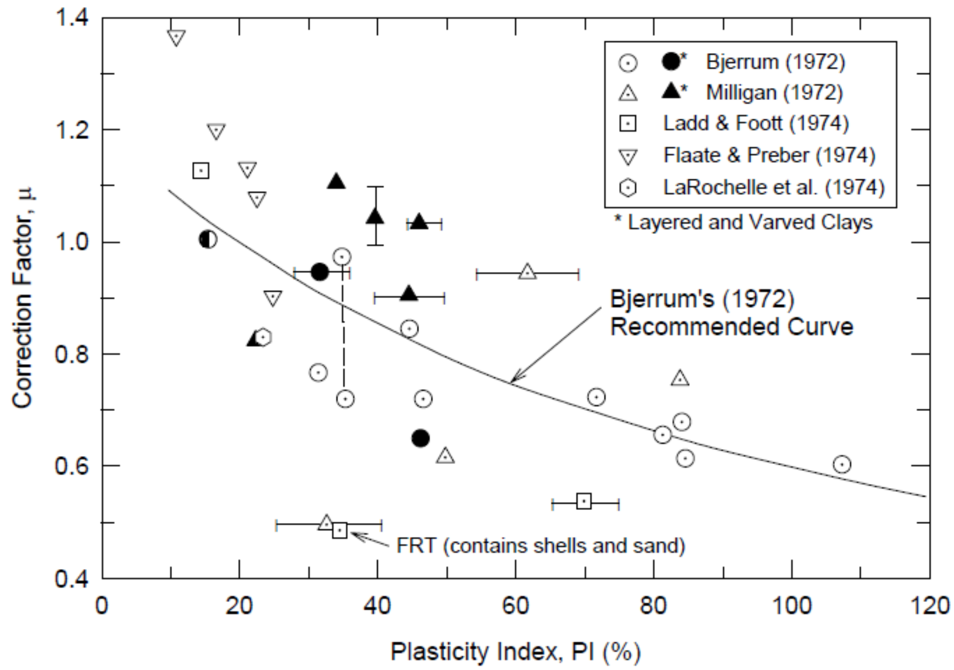
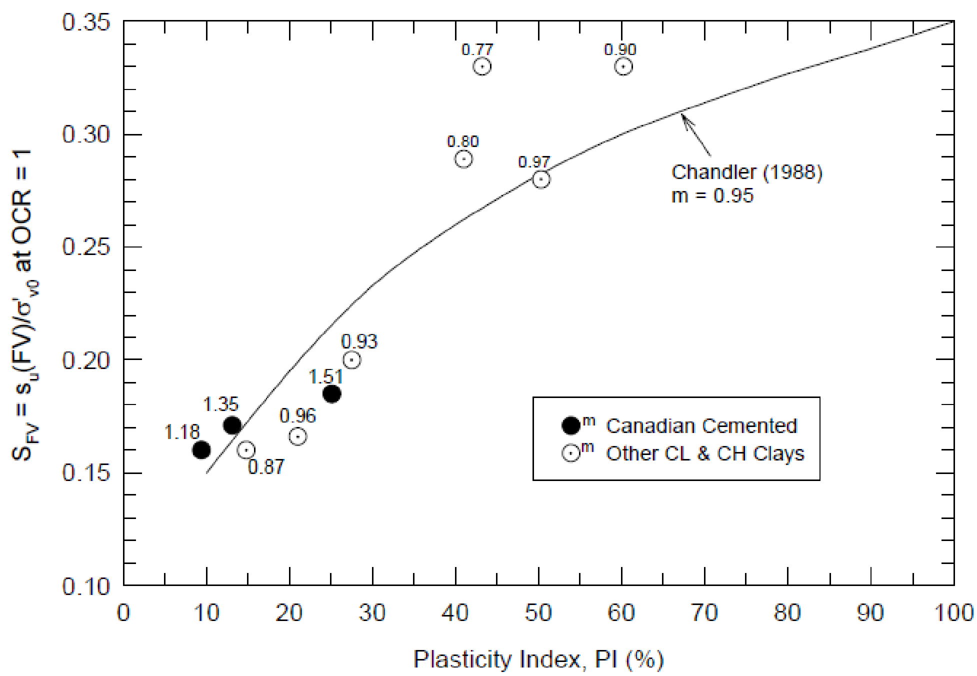
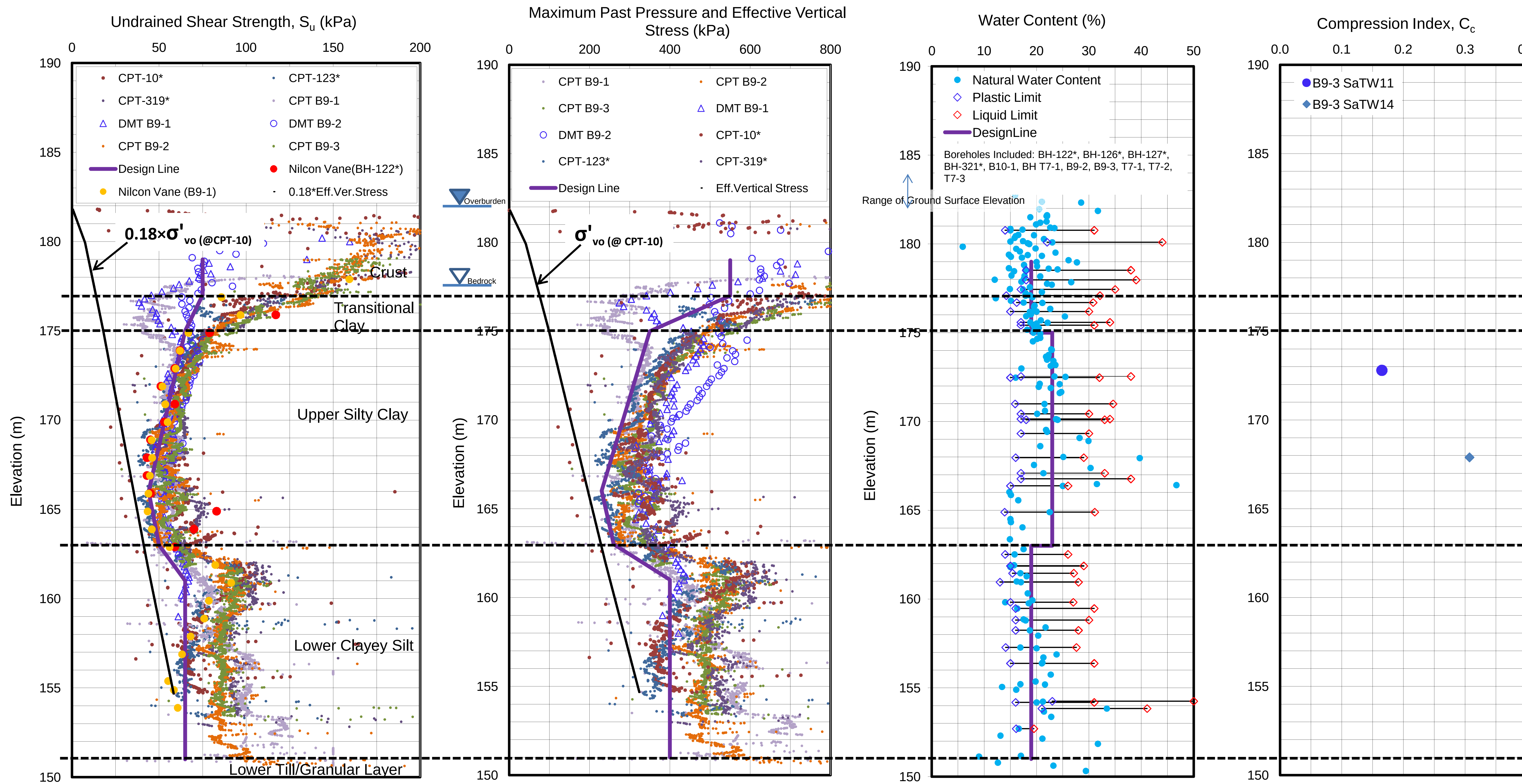


Figure 3-2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays

(Ladd & DeGroot, 2004)



Project:	Windsor-Essex Parkway	**Date:**	March / 2012
Document:	Geotechnical Investigation and Design Report – Bridge B-9 (Hwy. 3 East Ramp Underpass at Hwy. 401 Sta. 9+549L to 9+676L, LaSalle)	**Rev:**	0
Doc No.:	285380-04-119-0025 (Geocres No. 40J6-31)	**Page No.:**	Figures - Page 1 of 9



Notes:

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

* From previous investigations.

amec Earth & Environmental	PROJECT:	WINDSOR ESSEX PARKWAY
CLIENT :	TITLE:	SOIL PROPERTIES PROFILES BRIDGE B-9

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.:	REV.
Mar 2012	SW8801.1002		3-3	B

C:\Users\siavash.farhangi\Documents\AMEC Projects\WEP\Bridge B-9\90% Design - Rev B\Anlayses\Soil\Profile 10_Rev3_20120314

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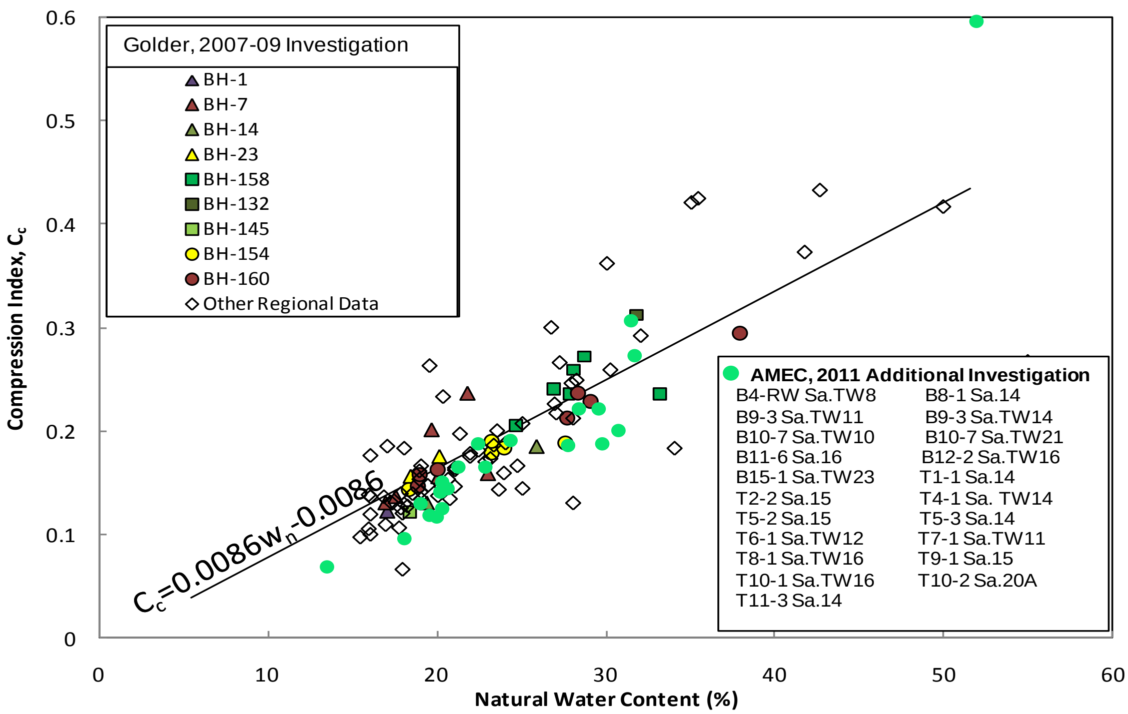
PARKWAY INFRASTRUCTURE CONSTRUCTORS

Figure 4-1: Compressibility Parameters at WEP



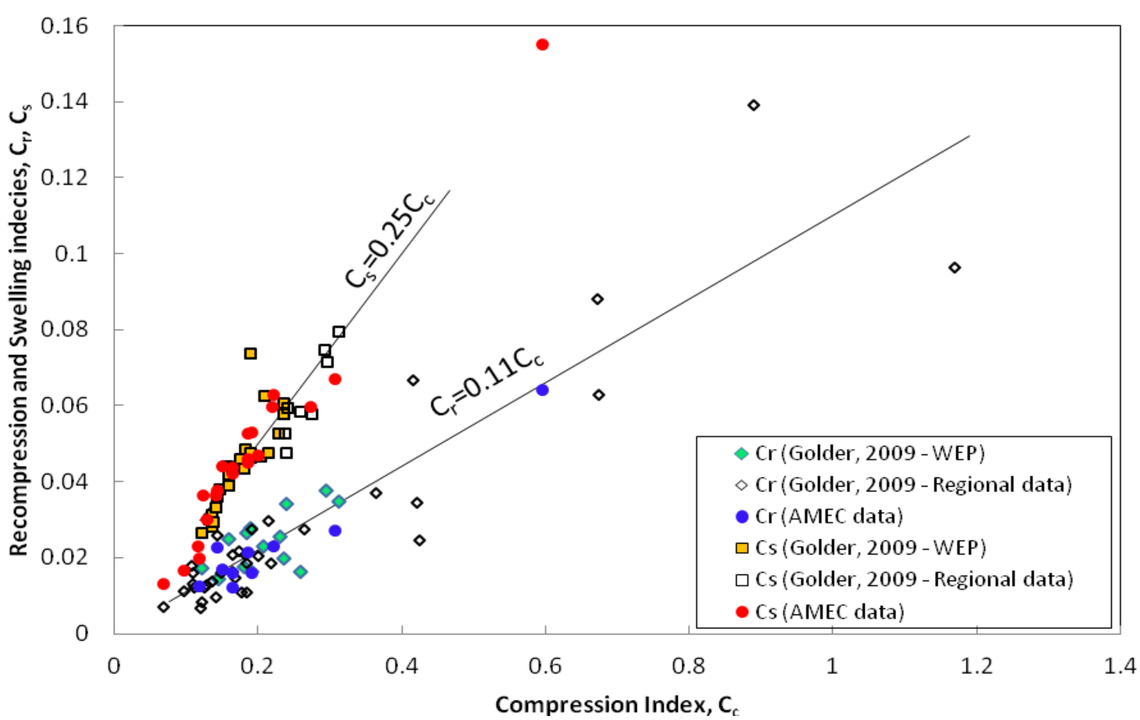


Figure 4-2: Cc versus Cα Relationship at WEP

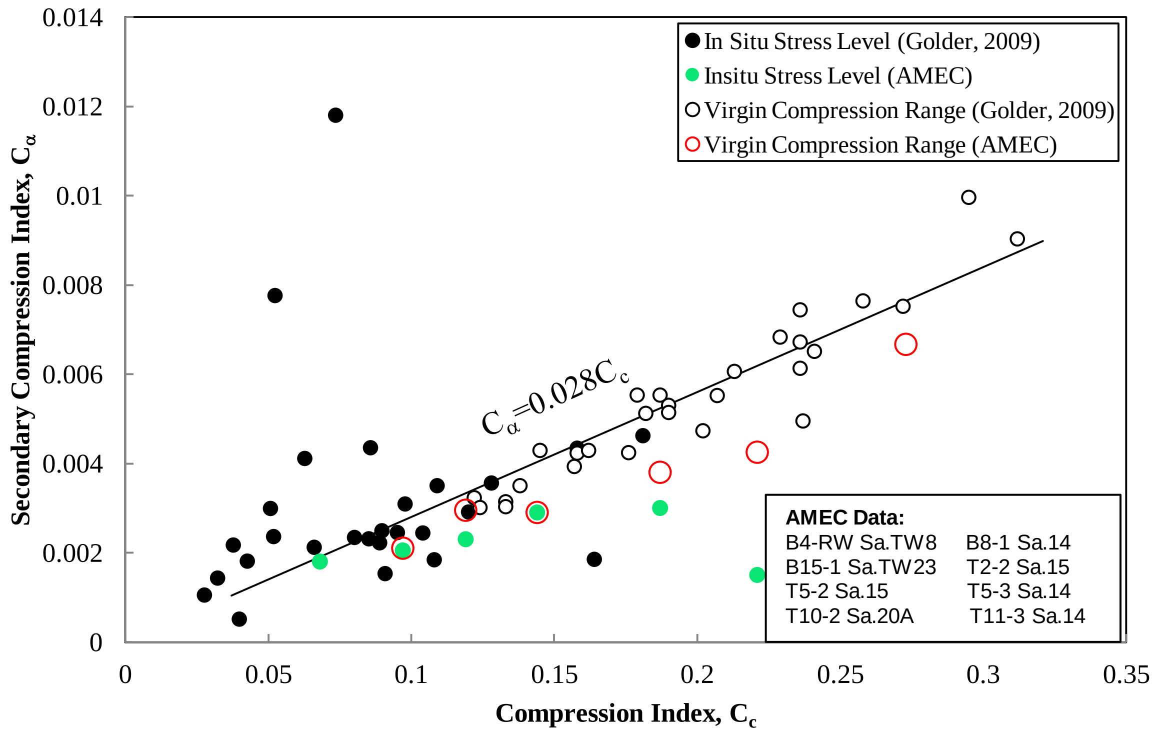
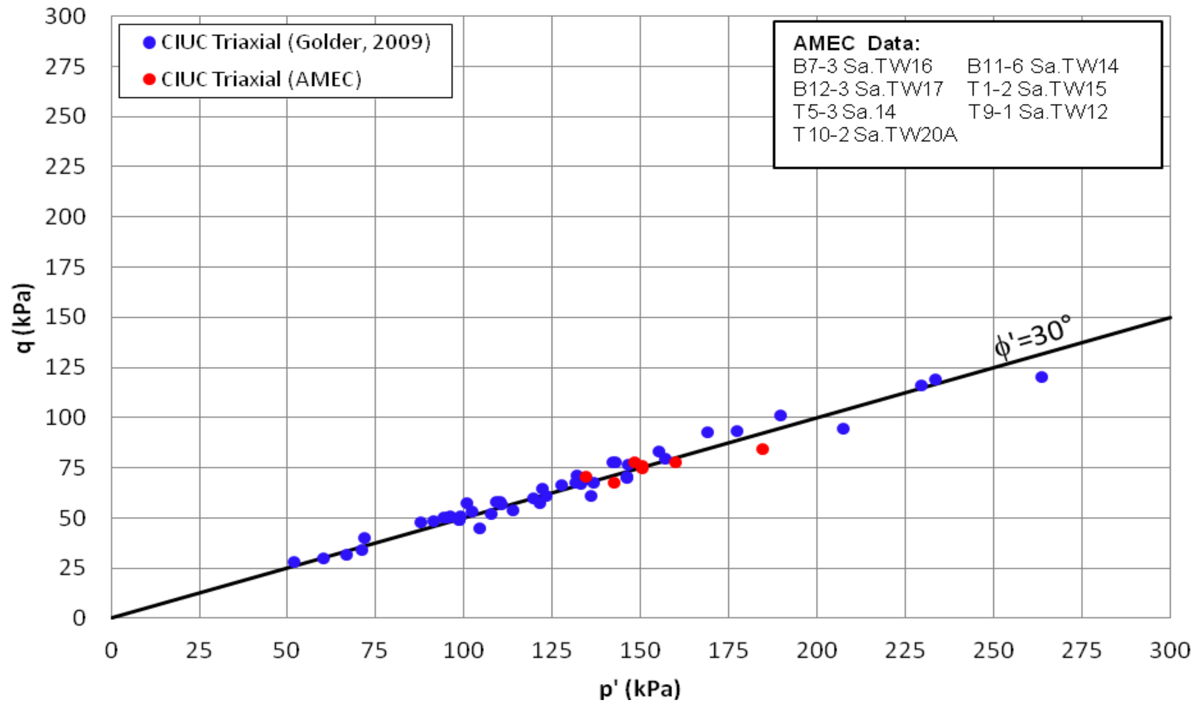


Figure 4-3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP



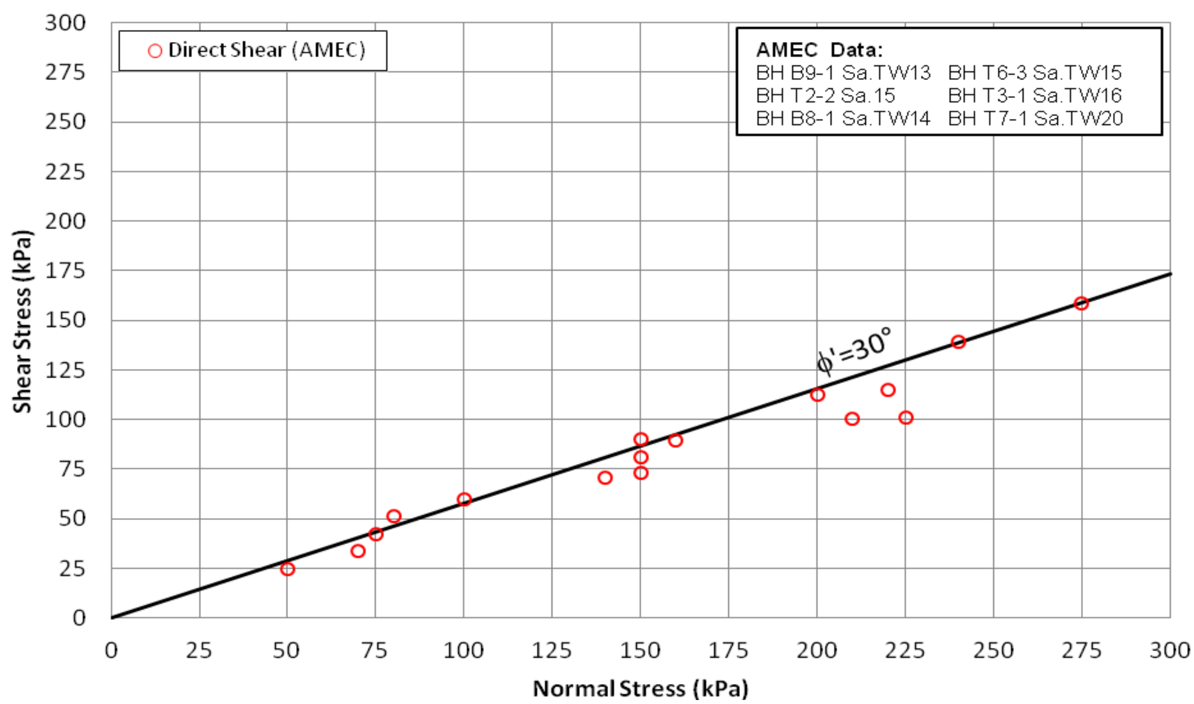


Figure 4-4: Relationship between sin ϕ′ and Plasticity Index for Normally Consolidated Soils

(Kenney, 1959)

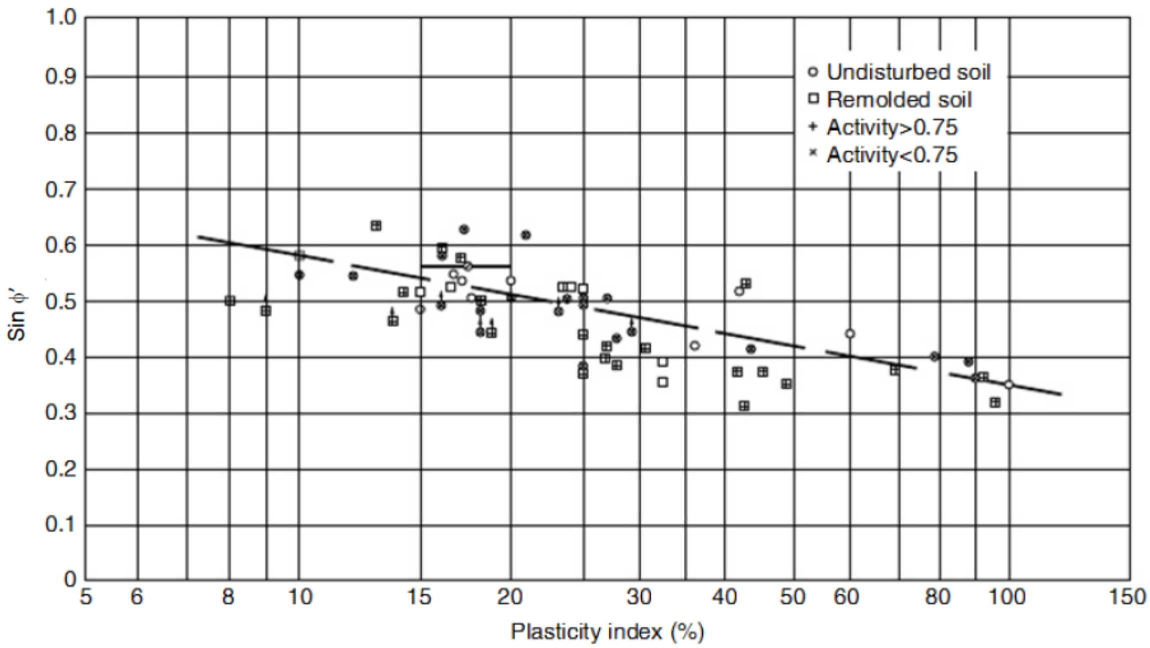
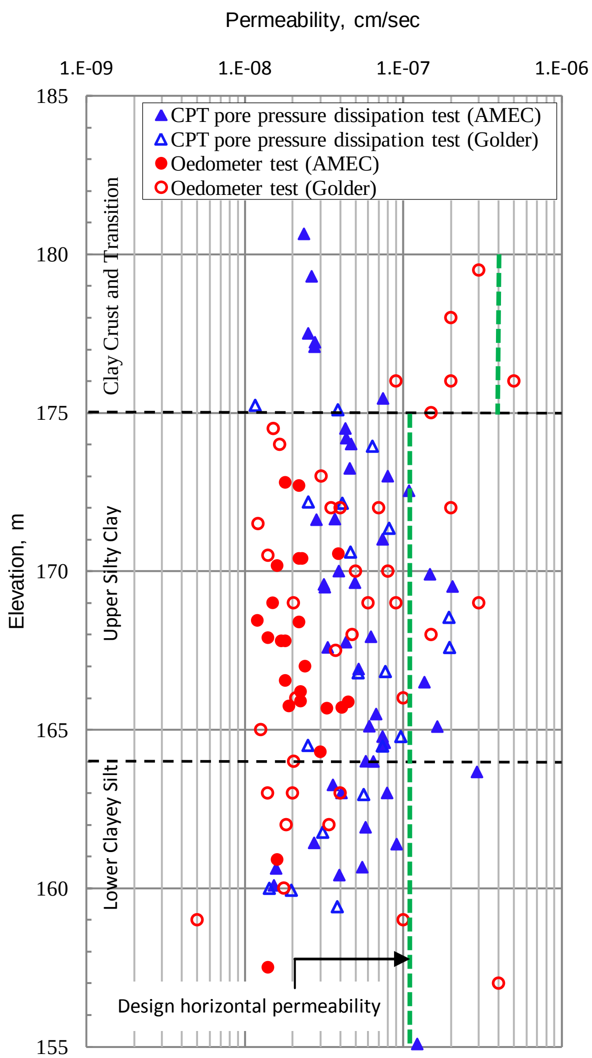
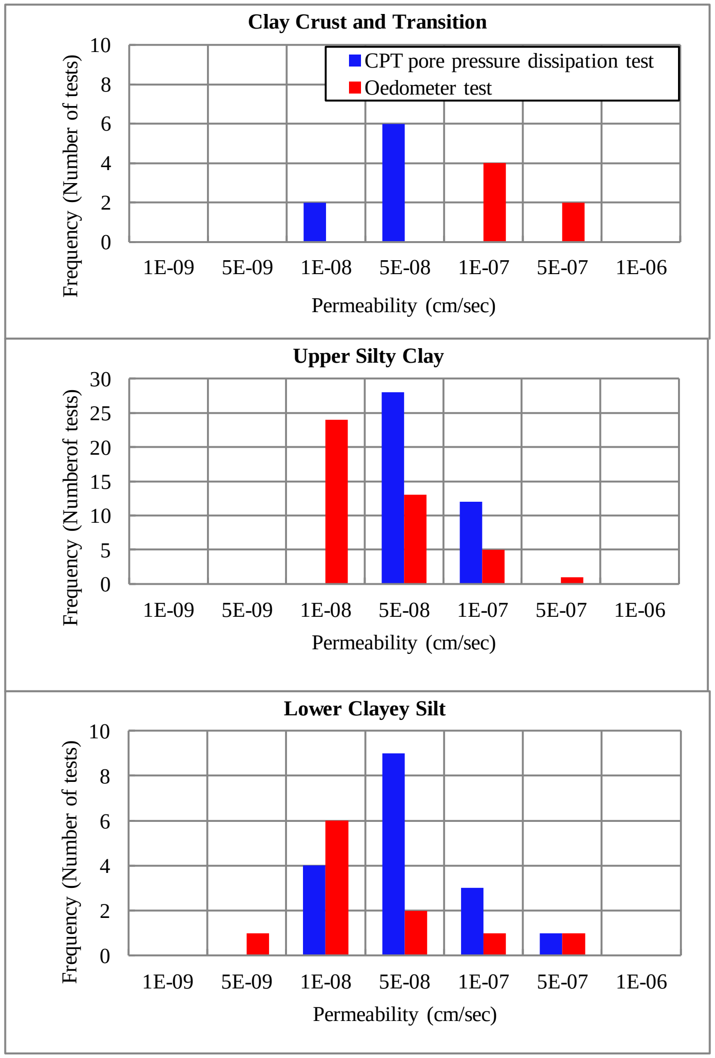
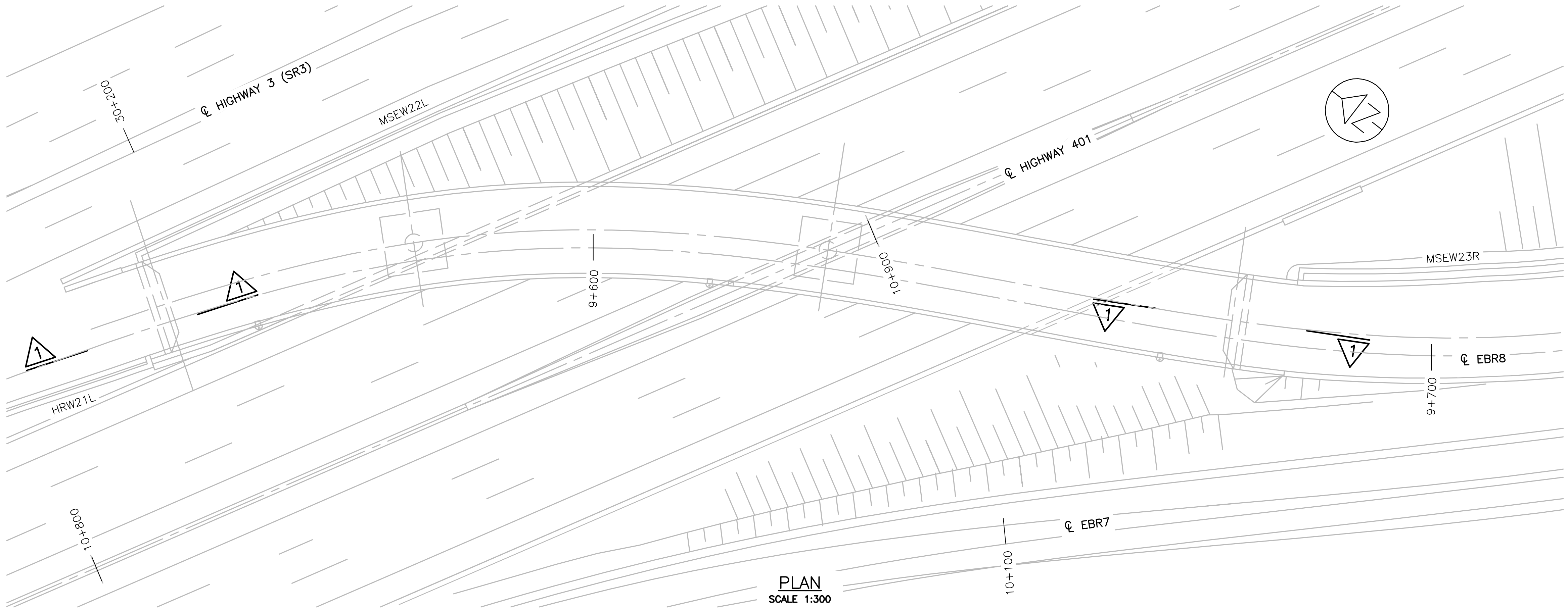


Figure 4-5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests





Project:	Windsor-Essex Parkway	**Date:**	March / 2012
Document:	Geotechnical Investigation and Design Report – Bridge B-9 (Hwy. 3 East Ramp Underpass at Hwy. 401 Sta. 9+549L to 9+676L, LaSalle)	**Rev:**	0
Doc No.:	285380-04-119-0025 (Geocres No. 40J6-31)	**Page No.:**	Figures - Page 7 of 9



PLAN
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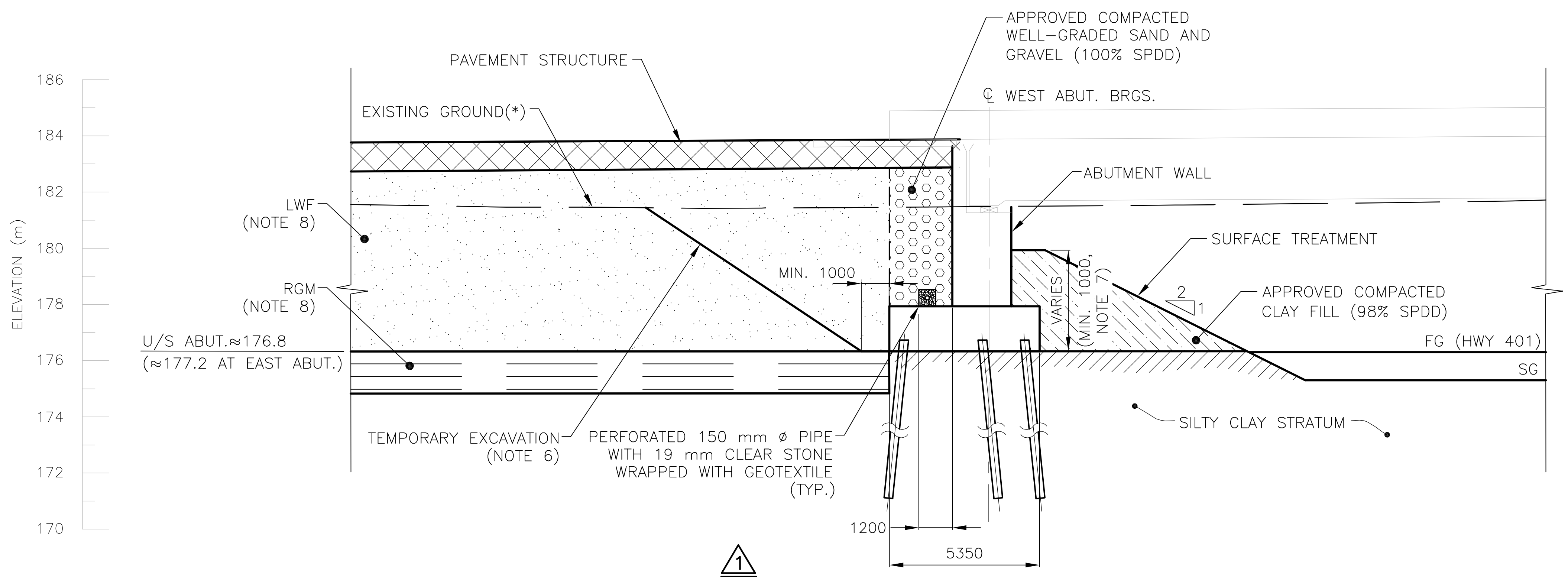

1

HORT SCALE 1:100
VERT SCALE 1:100

NOTES:

1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
2. THIS DRAWING ILLUSTRATES THE GENERAL BACKFILL ARRANGEMENT AT SELECTED REPRESENTATIVE LOCATION (WEST ABUTMENT) OF BRIDGE B–9 BASED ON GEOTECHNICAL DESIGN ANALYSES.
3. THE EAST ABUTMENT BACKFILL ARRANGEMENT IS SIMILAR TO THE WEST ABUTMENT.
4. ABUTMENT ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGN WERE OBTAINED FROM STRUCTURAL DRAWINGS AVAILABLE IN AUGUST 2011. ABUTMENT ELEVATIONS VARY ALONG THE BRIDGE.
5. 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 IMPLEMENTED.
6. CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS. EXCAVATED SLOPES ARE SUSCEPTIBLE TO DETERIORATION AND MAY EXPERIENCE DEFORMATIONS AND INSTABILITY. THE TEMPORARY SLOPES MUST BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED, MONITORED AND TREATED AS REQUIRED.
7. BACKFILL IN FRONT OF ABUTMENT WALL SHOULD BE SUBSTANTIALLY COMPLETED ABOVE THE PILE CAP BEFORE PLACING BACKFILL BEHIND BEHIND THE ABUTMENT WALL ABOVE THE SEAT LEVEL.
8. DESIGN OF HRW21L INCORPORATES RGM AND LWF. FOR FURTHER DETAILS REFER TO THE RETAINING WALL DESIGN.
9. SEE RELEVANT DRAWINGS FOR APPLICABLE CONSTRUCTION NOTES.

LEGEND:

RGM – REINFORCED GRANULAR MAT

LWF – LIGHT WEIGHT FILL

(*) – VARIES

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

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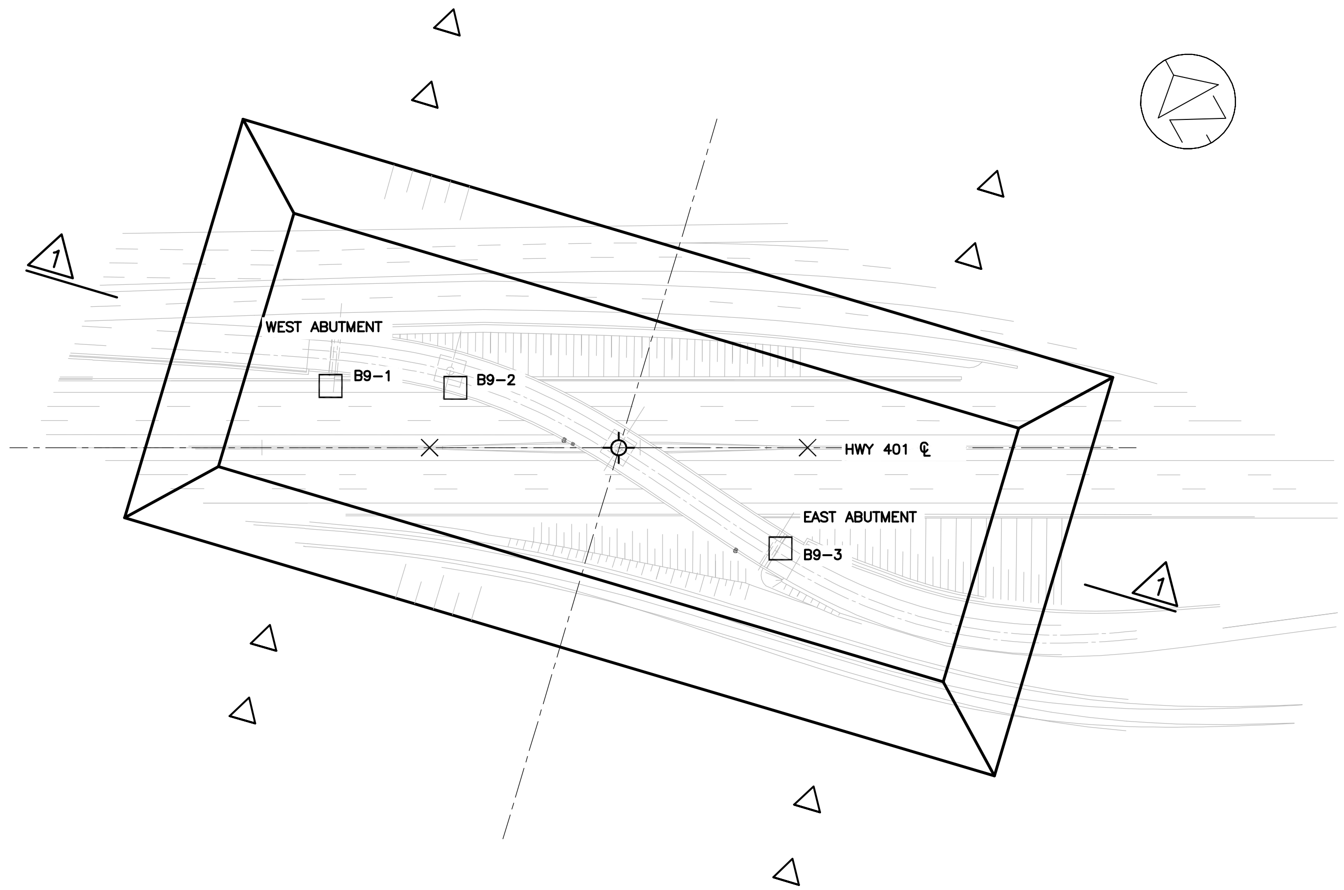
BRIDGE B–9
EAST BOUND RAMP UNDERPASS NEAR HURON CHURCH LINE
ABUTMENT EXCAVATION AND BACKFILL DETAILS

DWG. BY: SJL	CHK. BY: JF	FIGURE NO.: 5–1
DATE: Mar–12	SHEET: 1 OF 1	

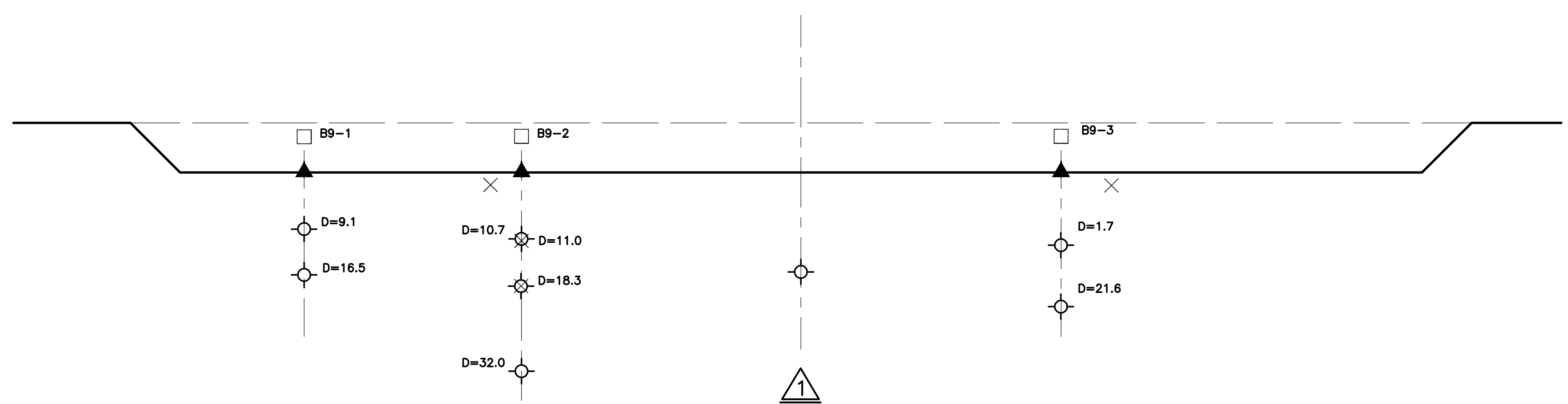
DOC: 285380-04-094-WIP1-0931_FIG 5-1



PLAN

LEGEND:

- EXISTING INSTRUMENTED BOREHOLES
- HEAVE GAUGE
- PIEZOMETER (VWP)
- SURVEY PINS
- INCLINOMETER



SECTION

NOT TO SCALE

NOT FOR CONSTRUCTION

Parkway Infrastructure Engineers

amec

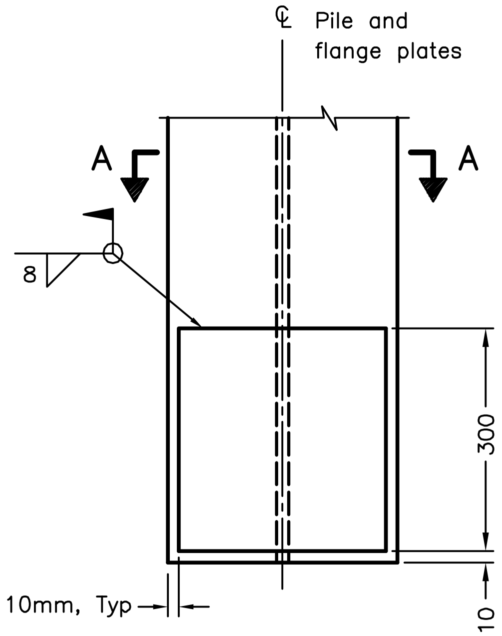
Hatch Mott MacDonald

BRIDGE B-9
INSTRUMENTATION FOR EXCAVATION MONITORING DURING CONSTRUCTION

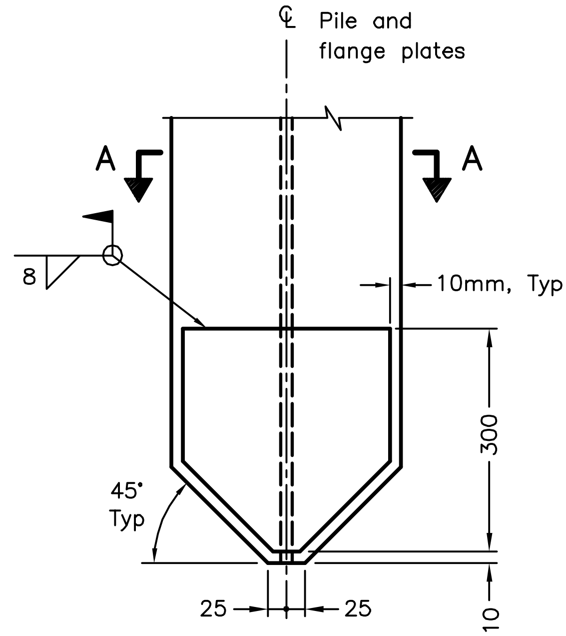
DWG. BY: MM	CHK. BY: SF	FIGURE NO.: 6-1
DATE: Feb-12	SHEET: 1 OF 1	

DOC: 285380-04-096-WIP1-0980_FIG 6-1

Applicable OPSDs

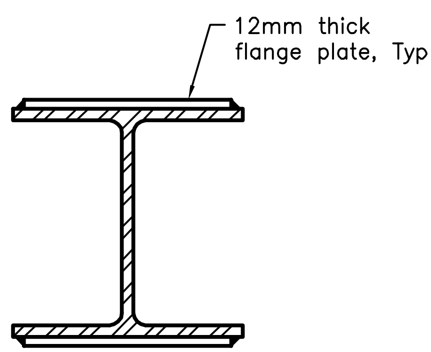


TYPE I



TYPE II

ELEVATION



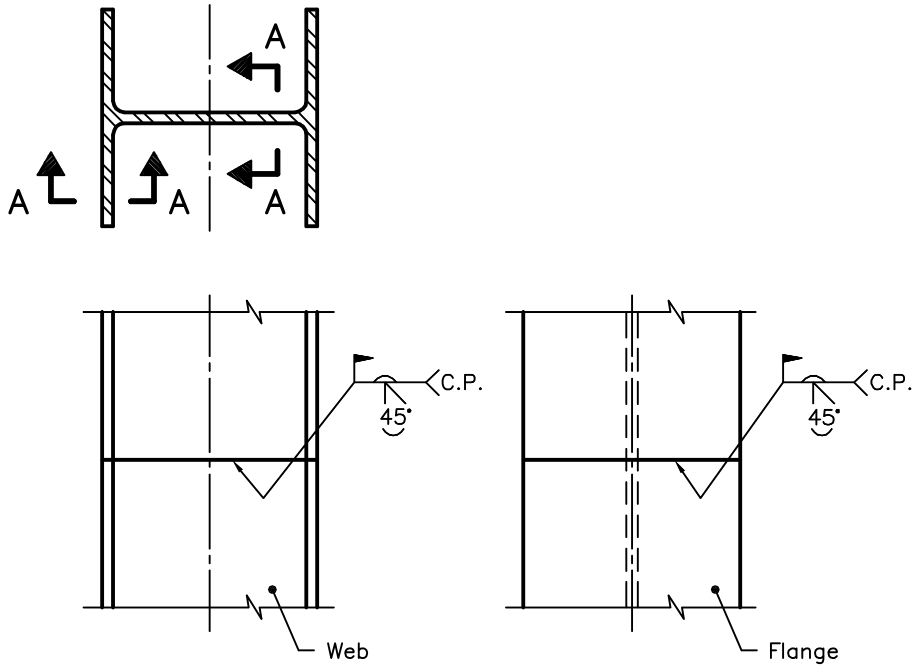
PILE DRIVING SHOE

SECTION A–A

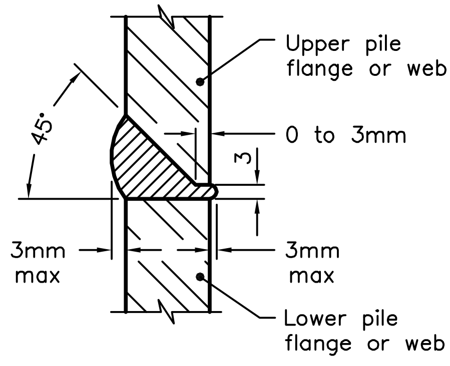
NOTES:

A Flange plates shall be according to CSA G40.20/G40.21, Grade 300W.
B Welding shall be according to CSA W59.
C Driving shoe Type I shall be used unless Type II is specified.
D All dimensions are in millimetres unless otherwise shown.

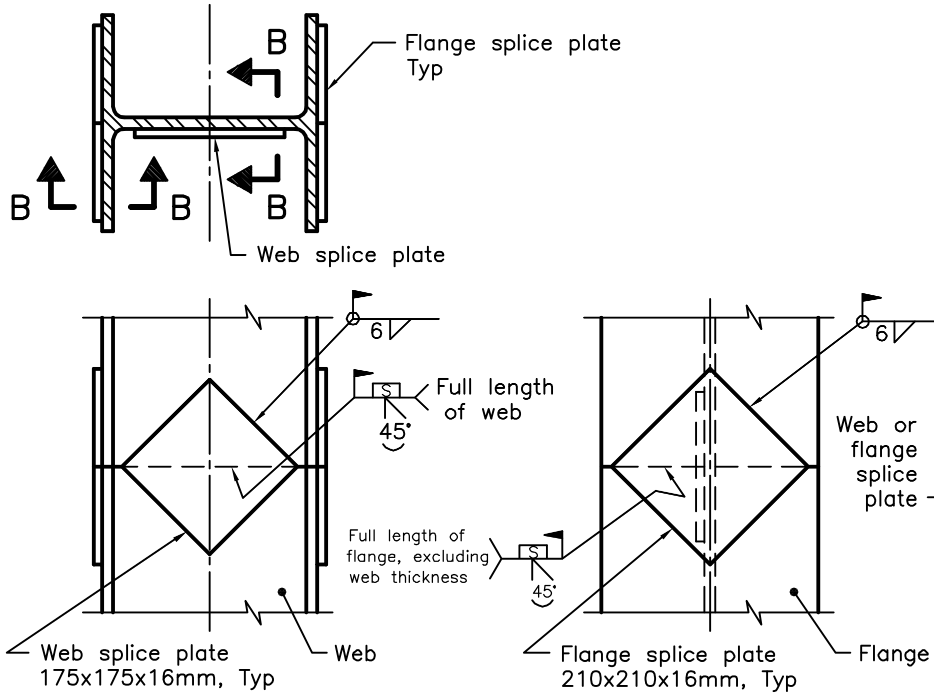
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 2
FOUNDATION PILES STEEL H-PILE DRIVING SHOE	OPSD 3000.100	



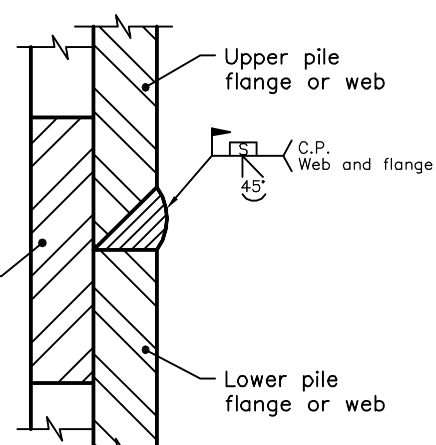
BUTT WELD



SECTION A–A



BUTT WELD WITH SPLICE PLATES

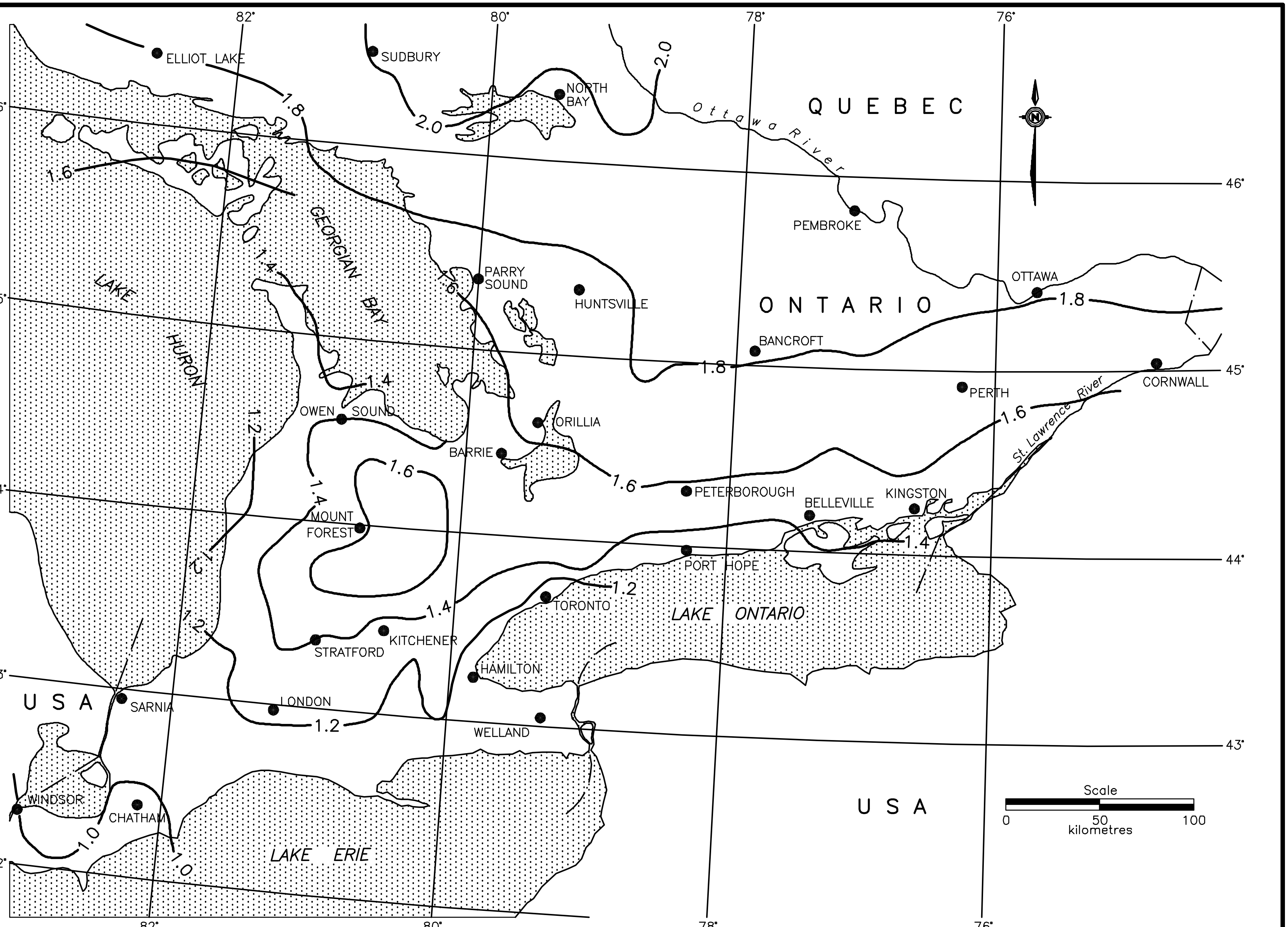


SECTION B–B

NOTES:

A The pile splice shall be perpendicular to the centreline of pile.
B Splice plates shall be according to CSA G40.20/G40.21, Grade 300W.
C Welding shall be according to CSA W59.
D Splice plate alternative is only applicable to H–pile sizes HP310x79, HP310x110, and HP310x132.
E All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
FOUNDATION PILES STEEL H–PILE SPLICE		
	OPSD 3000.150	



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 RR225 "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 penetration depths are in metres.

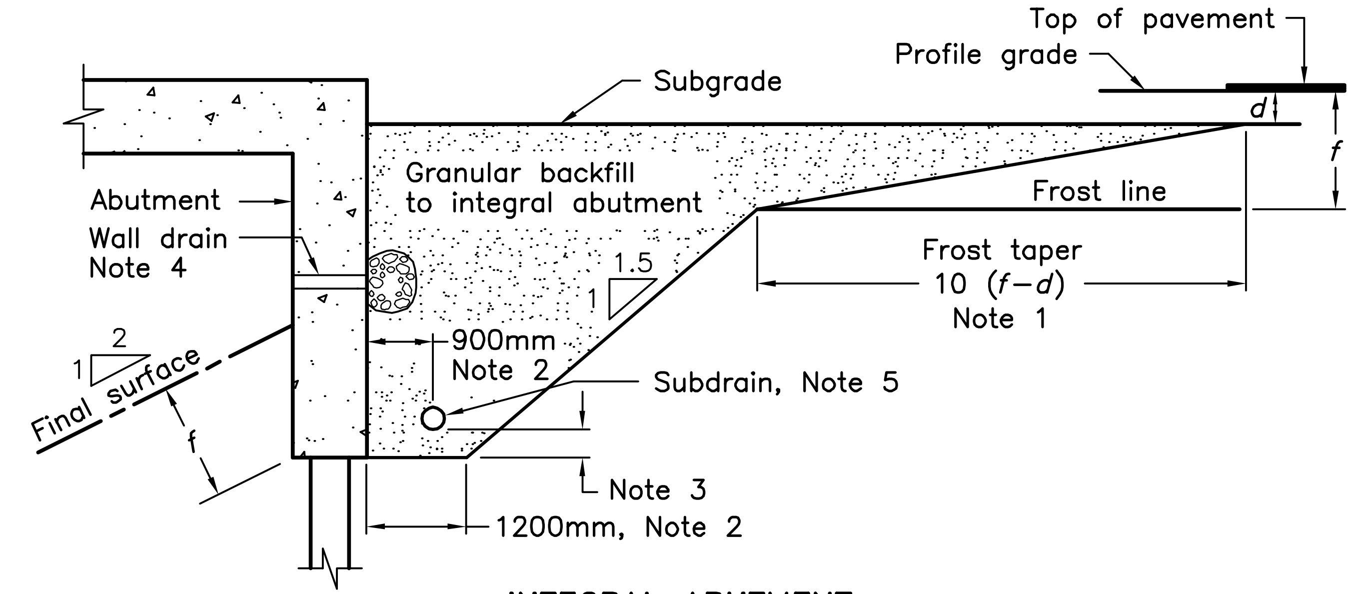
ONTARIO PROVINCIAL STANDARD DRAWING

FOUNDATION FROST PENETRATION DEPTHS FOR SOUTHERN ONTARIO

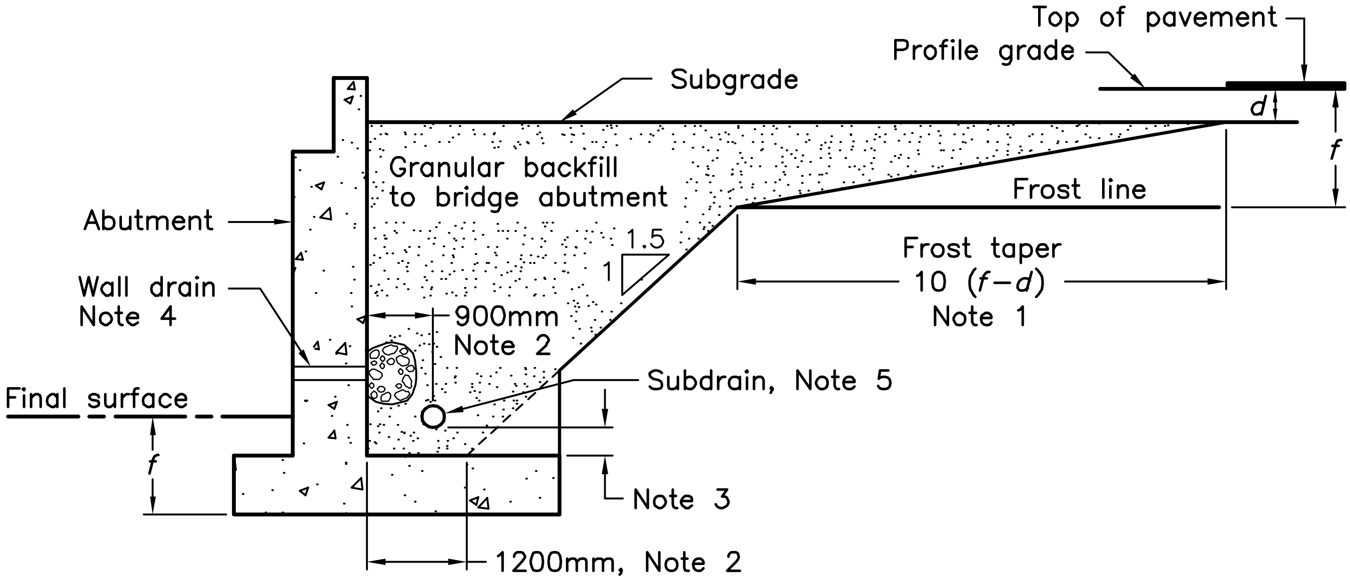
Nov 2010 | Rev 1

ONTARIO STANDARDS · MUNICIPAL – PROVINCIAL

OPSD 3090.101



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

1 d = depth of combined base and subbase courses
f = frost penetration depth as specified

2 Dimensions perpendicular to back face of abutment.

3 Height to be consistent with positive drainage of subdrain as specified.

4 Where specified, wall drains shall be installed according to OPSD 3190.100.

5 150mm dia perforated pipe subdrain wrapped with geotextile.

A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the backfill unless interrupted by the retaining wall or wingwall.

B Sections shown are parallel to centreline of roadway.

C Subdrain shall be installed with a 2% gradient behind wall.

D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
WALLS **ABUTMENT, BACKFILL** **MINIMUM GRANULAR REQUIREMENT**		ONTARIO STANDARDS · MUNICIPAL – PROVINCIAL · OPS
	OPSD 3101.150	

Appendix A: Borehole and CPT Logs from Additional Geotechnical Investigation

EXPLANATION OF BOREHOLE LOG

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

GENERAL INFORMATION

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

SOIL LITHOLOGY

Elevation and Depth

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

Lithology Plot

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

Description

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

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

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

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

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

Soil Sampling

Sample types are abbreviated as follows:

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

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

Field and Laboratory Testing

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

Instrumentation Installation

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

Comments

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

BEDROCK DESCRIPTION

STRENGTH CLASSIFICATION

Term (Grade)	Field Identification	Approximate Range of Uniaxial Compressive Strength (MPa)
Extremely Weak (R0)	Indented by thumbnail.	0.25 – 1.0
Very Weak (R1)	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife.	1.0 – 5.0
Weak (R2)	Can be peeled with a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.	5.0 – 25
Medium Strong (R3)	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of geological hammer.	25 – 50
Strong (R4)	Specimen requires more than one blow of geological hammer to fracture it.	50 – 100
Very Strong (R5)	Specimen requires many blows of geological hammer to fracture it.	100 – 250
Extremely Strong (R6)	Specimen can only be chipped with geological hammer.	>250

JOINT SPACING CLASSIFICATION

Term	Average Joint Spacing (m)
Extremely close	< 0.02
Very close	0.02 – 0.06
Close	0.06 – 0.20
Moderately close	0.20 – 0.6
Wide	0.6 – 2.0
Very wide	2.0 – 6.0
Extremely wide	> 6.0

ROCK QUALITY CLASSIFICATION

Rock Quality Designation, RQD (%)	Description of Rock Quality
0 – 25	Very Poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

Reference: Deere et al, 1967

WEATHERING CLASSIFICATION

Term (Grade)	Description
Fresh (W1)	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.
Slightly Weathered (W2)	Discoloration indicates weathering of rock material on discontinuity surfaces. Less than 5 % of rock mass altered.
Moderately Weathered (W3)	Less than half of the rock material is decomposed and/or disintegrated into a soil. Fresh or discoloured rock is present either as a continuous framework or as core stones.
Highly Weathered (W4)	More than half of the rock material is decomposed and/or disintegrated into a soil. Fresh or discoloured rock is present either as a discontinuous framework or as core stones.
Completely Weathered (W5)	All rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil (W6)	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume but the soil has not been significantly transported.

Reference: Brown, 1981, "Suggested Methods for Rock Characterization Testing and Monitoring". International Society for Rock Mechanics.

TERMINOLOGY

Rock Quality Designation (RQD) is defined as the percentage of intact core pieces longer than 100 mm (4 inches) to the total length of core. The core should be at least NW size (54.7 mm or 2.15 inches in diameter) and typically 5 ft (nominally 1.5 m) in length.

Solid Core Recovery (SCR) is defined as the percentage of intact cylindrical core pieces to the total length of core.

Total Core Recovery (TCR) is defined as the percentage of intact core pieces to the total length of core.

MTC SOIL CLASSIFICATION
Based on MTC Soil Classification Manual

Ontario

MAJOR DIVISION				GROUP SYMBOL	TYPICAL DESCRIPTION	INFORMATION REQUIRED FOR DESCRIBING SOILS	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (LITTLE OR NO FINES)	WIDE RANGE IN GRAIN SIZE & SUBSTANTIAL AMOUNTS OF ALL INTERMEDIATE PARTICAL SIZE	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GIVE TYPE, NAME, IF NECESSARY, INDICATE APPROX % OF SAND & GRAVEL ; MAX SIZE; ANGULARITY, SURFACE CONDITION, & HARDNESSOF THE COARSE GRAINS, LOCAL OR GEOLOGICAL NAME & OTHER PERTINENT DESCRICTIVE INFORMATION, & SYMBOL IN PARENTHESIS.	$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 4; $C_C = \frac{(D_{30})^2}{D_{10} X D_{60}}$ BETWEEN 1 AND 3
			PREDOMINANTLY ONE SIZE OF A RANGE OF SIZES WITH STONE INTERMEDIATE SIZES MISSING	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR GW
		GRAVEL WITH FINES (APPLICABLE AMOUNT OF FINES)	NON PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND- SILT MIXTURES		ATTERBERG LIMITS BELOW A-LINE OR Ip LESS THAN 4 / ABOVE A-LINE WITH Ip BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE A- LINE WITH Ip GREATER THAN 7
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (LITTLE OR NO FINES)	WIDE RANGE IN GRAIN SIZE & SUBSTANTIAL AMOUNT OF ALL INTERMEDIATE PARTICLE SIZES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	FOR UNDISTURBED SOILS ADD INFORMATION ON STRATIFICATION, DEGREE OF COMPACTNESS, CEMENTATION, MOISTURE CONDITION & DRAINAGE CHARACTERISTICS	$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 6; $C_C = \frac{(D_{30})^2}{D_{10} X D_{60}}$ BETWEEN 1 AND 3
			PREDOMINANTLY ONE SIZE OR A RANGE OF SIZES WITH SOME INTERMEDIATE SIZE MISSING	SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		NOT MEETING ALL GRADATION FOR SW
		SANDS WITH FINES (APPLICABLE AMOUNT OF FINES)	NON PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES		ATTERBERG LIMITS BELOW A- LINE OR Ip LESS THAN 4 / ABOVE A-LINE WITH Ip BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE A- LINE WITH Ip GREATER THAN 7

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

DETERMINE PERCENTAGE OF GRAVEL & SAND FROM GRAIN SIZE CURVE. DEPENDING ON PERCENTAGE OF FINES (FRACTION SMALLER THAN 75 µm) COARSE GRAINED SOILS ARE CLASSIFIED AS FOLLOWS:

LESS THAN 5%	GW, GP, SW, SP
MORE THAN 12%	GM, GC, SM, SC
5% TO 12%	BORDER LINE CASES REQUIRE USE OF DUAL SYMBOL.

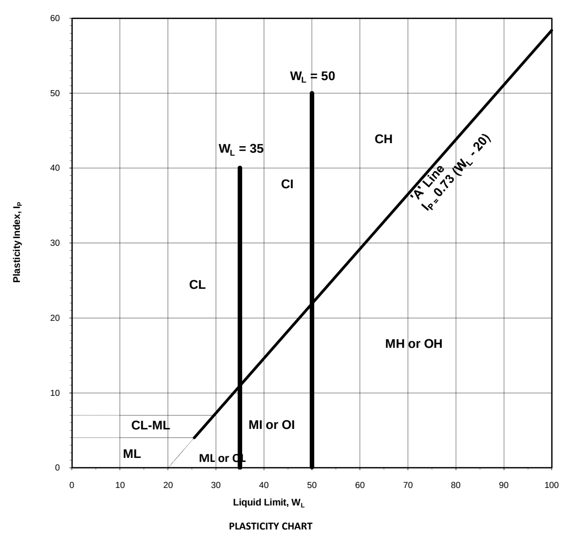
IDENTIFICATION PROCEDURE ON FRACTION SMALLER THAN 425µm

MAJOR DIVISION			DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	GROUP SYMBOL	TYPICAL DESCRIPTION	INFORMATION REQUIRED FOR DESCRIBING SOILS
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	SILT AND CLAYS	LIQUID LIMIT LESS THAN 35	NONE	QUICK	NONE	ML	INORGANIC SILTS & SANDY SILTS OR SLIGHTLY PLASTICITY, ROCK FLOUR	GIVE TYPE, NAME, IF NECESSARY, INDICATE DEGREE AND CHARACTER OF PLASTICITY, AMOUNT AND MAXIMUM SIZE OF COURSE GRAINS, COLOUR IN WET CONDITION, ODOUR, IF ANY, LOCAL OR GEOLOGIC NAME & OTHER PERTINENT DESCRICTIVE INFORMATION & SYMBOL IN PARENTHESIS.
			MEDIUM TO HIGH	NONE TO VERY SLOW	MEDIUM	CL	SILTY CLAYS (INORGANIC), GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS	
			SLIGHT TO MEDIUM	SLOW	SLIGHT	OL	ORGANIC SILT OF LOW PLASTICITY, ORGANIC SANDY SILTS	
		LIQUID LIMIT BETWEEN 35 AND 50	NONE TO SLIGHT	SLOW TO QUICK	SLIGHT	MI	INORGANIC COMPRESSIBLE FINE SANDY SILT WITH CLAY OF MEDIUM PLASTICIT, CLAYEY SILTS	
			HIGH	NONE	MEDIUM TO HIGH	CI	SILTY CLAYS (INORGANIC) OF MEDIUM PLASTICITY	FOR UNDISTURBED SOILS AND INFORMATION ON STRUCTURE, STRATIFICATION, CONSISTANCY IN UNDISTURBED AND REMOLDED STATES, MOISTURE & DRAINAGE CONDITION.
			SLIGHT TO MEDIUM	VERY SLOW	SLIGHT	OI	ORGANIC SILTY CLAYS OF MEDIUM PLASTICITY	
		LIQUID LIMIT GREATER THAN 50	SLIGHT TO MEDIUM	SLOW TO NONE	MEDIUM	MH	INORGANIC SILTS, HIGHLY COMPRESSIBLE MICACEOUS OR DIATOMECACOUS FINE SANDY SILTS, ELASTIC SILTS	
			HIGH TO VERY HIGH	NONE	HIGH	CH	CLAYS (INORGANIC) OF HIGH PLASTICITY, FAT CLAYS	
			MEDIUM TO HIGH	NONE TO VERY SLOW	SLIGHT TO MEDIUM	OH	ORGANIC CLAYS OF HIGH PLASTICITY	
HIGH ORGANIC SOILS			READILY IDENTIFIED BY COLOUR, ODOUR, SPONGY FEEL & FREQUENTLY BY FIBROUS TEXTURE			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	

FRACTION	U.S STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		PASSING	RETAINED	PERCENT	DESCRIPTOR
GRAVEL	COARSE	75 mm	26.5 mm	40-50	AND
	FINE	26.5 mm	4.75 mm	30-40	Y/EY
SAND	COARSE	4.75 mm	2.00 mm	20-30	WITH
	MEDIUM	2.00 mm	425 µm	10-20	SOME
	FINE	425 µm	75 µm	1-10	TRACE
FINES (SILT OR CLAY BASED ON PLASTICITY)		75 µm			

OVERSIZED MATERIAL

ROUNDED OR SUBROUNDED: COBBLES 75 mm TO 200 mm BOULDERS > 200 mm	NOT ROUNDED: ROCK FRAGMENTS > 75 mm ROCKS > 0.76 CUBIC METRE IN VOLUME



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

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

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ENGINEERING PROPERTIES OF SOIL

TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	PERMEABILITY WHEN COMPACTED	STRENGTH WHEN COMPACTED	COMPRESSIBILITY WHEN COMPACTED	WORKABILITY AS A CONSTRUCTION MATERIAL	SCOUR RESISTANCE	SUSCEPTIBI LIIY TO SURFICIAL EROSION	SUSCEPTIBILI TY TO FROST ACTION	DRAINAGE CHARACTERISTICS
WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	MEDIUM	NEGLIGIBLE	NEGLIGIBLE	EXCELLENT
POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GP	VERY PERVI0US	GOOD	NEGLIGIBLE	GOOD	MEDIUM	NEGLIGIBLE	NEGLIGIBLE	EXCELLENT
SILTY GRAVELS, POORLY GRADED GRAVEL- SAND-SILT MIXTURES	GM	SEMI-PERVIOUS TO IMPERVIOUS	GOOD	NEGLIGIBLE	GOOD	LOW TO MEDIUM	SLIGHT	SLIGHT	FAIR TO SEMI IMPERVIOUS
CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	GC	IMPERVIOUS	GOOD TO FAIR	VERY LOW	GOOD	MEDIUM	SLIGHT	NEGLIGIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	LOW TO MEDIUM	SLIGHT	NEGLIGIBLE	EXCELLENT
POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SP	PERVIOUS	GOOD	VERY LOW	FAIR TO GOOD	LOW TO MEDIUM	MODERATE	NEGLIGIBLE TO SLIGHT	EXCELLENT
SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES	SM	SEMI-PERVIOUS TO IMPERVIOUS	GOOD	LOW	FAIR	LOW	MODERATE	SLIGHT TO MODERATE	FAIR TO SEMI IMPERVIOUS IMPERVIOUS
CLAYEY SANDS, POORLY GRADED SAND WITH SOME CLAY MIXTURES	SC	IMPERVIOUS	GOOD TO FAIR	LOW	GOOD	VERY LOW TO LOW	MODERATE TO SLIGHT	NEGLIGIBLE	PRACTICALLY IMPERVIOUS
INORGANIC SILTS AND SANDY SILTS OF SLIGHT PLASTICITY, ROCK FLOUR	ML	SEMI-PERVIOUS TO IMPERVIOUS	FAIR	MEDIUM	FAIR	VERY LOW	SEVERE	SEVERE	FAIR TO POOR
INORGANIC CLAYEY SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS	CL	IMPERVIOUS	FAIR	MEDIUM	GOOD TO FAIR	LOW TO MEDIUM	SLIGHT TO MODERATE	MODERATE TO SEVERE	PRACTICALLY IMPERVIOUS
ORGANIC SILTS OF LOW PLASTICITY	OL	SEMI-PERVIOUS TO IMPERVIOUS	POOR	MEDIUM	FAIR TO POOR	VERY LOW TO LOW	SEVERE	SEVERE	POOR
INORGANIC COMPRESSIBLE SILTS OF MEDIUM PLASTICITY	MI	SEMI-PERVIOUS TO IMPERVIOUS	FAIR	MEDIUM TO HIGH	FAIR TO POOR	LOW	MODERATE	MODERATE TO SEVERE	FAIR TO POOR
INORGANIC SILTY CLAYS OF MEDIUM PLASTICITY	CI	IMPERVIOUS	FAIR TO POOR	HIGH	FAIR	LOW TO MEDIUM	SLIGHT	MODERATE TO SEVERE	SEMI IMPERVIOUS TO PRACTICALL Y
ORGANIC SILTY CLAY OF MEDIUM PLASTICITY	OI	SEMI-PERVI0US TO IMPERVIOUS	POOR	HIGH	POOR	VERY LOW TO LOW	SEVERE	MODERATE TO SEVERE	POOR TO PRACTICALLY IMPERVIOUS
INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	MH	SEMI-PERVIOUS TO IMPERVIOUS	FAIR TO POOR	HIGH	POOR	VERY LOW	MEDIUM	SEVERE	POOR
INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	CH	IMPERVIOUS	POOR	HIGH	FAIR TO POOR	LOW TO MEDIUM	SLIGHT TO NEGLIGIBLE	NEGLIGIBLE	PRACT I.CALL Y IMPERVIOUS
ORGANIC CLAYS OF HIGH PLASTICITY	OH	IMPERVIOUS	POOR	HIGH	POOR	LOW	MODERATE	NEGLIGIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
PEAT AND OTHER HIGHLY ORGANIC SOILS	Pt	-	-	-	-	LOW	SEVERE	-	FAIR TO GOOD

RECORD OF BOREHOLE No B9-1

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4679235.3, E332593.8		ORIGINATED BY DG
DIST	HWY WEP	BOREHOLE TYPE CME 55 Track Mounted Drill - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 13 Jul 11 - 14 Jul 11		CHECKED BY MSO

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.9	Ground Surface							
0.0	**TOPSOIL**							-Slope inclinometer casing installed in sampled borehole; Vibrating Wire Piezometers (VWP installed in adjacent boring at N4679234.3, E332595.5 -Nilcon vane advanced in adjacent hole from 4m to 28m (EL. 177.9m to EL. 153.9m)
181.5 / 0.4	**CLAYEY SILT** Trace fine-coarse gravel Stiff Mottled brown-grey	1	SS	11	181			
	Brown -Trace fine sand	2	SS	19	180			
	Trace sand, fine-medium gravel Trace fissures	3	SS	22	179			
		4	SS	13				
	Grey	5	SS	10	178			
	Trace fine-coarse gravel, trace sand	6	SS	7	177			
	-Trace pink clay nodules	7	SS	5	176			-began wash boring
	Trace fine-medium gravel	8	TW	PH			20.8	1 21 41 37
			VT		175	1.2		
		9	TW	PH	174			
		10	TW	PH	173			-corrosivity sample -VWP B9-1-P10 installed 9.1m below ground surface (EL. 172.8m)
			VT		172	1.4		
		11	TW	PH	171		20.0	1 18 42 39
	-Trace pink clay nodules	12	TW	PH	170 / 169			
			VT			2.2		
168.2 / 13.7	**CLAYEY SILT** Trace fine-medium gravel Grey Moist to Wet	13	TW	PH	168 / 167	52.0		1 11 38 50

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

Continued Next Page

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

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RECORD OF BOREHOLE No B9-1

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4679235.3, E332593.8		ORIGINATED BY DG
DIST	HWY WEP	BOREHOLE TYPE CME 55 Track Mounted Drill - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 13 Jul 11 - 14 Jul 11		CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH (kPa)	WATER CONTENT	UNIT WEIGHT γ (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT Trace fine-medium gravel, Grey, Moist to Wet *(continued)*	14	TW	PH			19.4	1 16 47 35
			VT		1.9			
165.1 / 16.8	**CLAYEY SILT** Trace fine-coarse gravel, Some sand, Grey, Firm to Stiff, Moist to wet	15	SS	0				-no blow counts, weight of rods pushed spoon 0.45m -VWP B9-1-P17 installed at 16.5m below ground surface (EL. 165.4m)
		16	SS	3				
			VT		1.5			
	-Trace fine-medium gravel -Some sand, wet	17	SS	0				-no blow counts, weight of hammer pushed spoon 0.45m
161.2 / 20.7	**SILTY SAND** Some clay, trace gravel, saturated, Loose to very loose	18	SS	3				1 50 29 20
			VT		>> 1.5			
		19	SS	9				
158.1 / 23.8	**CLAYEY SILT** Trace sand, trace fine-coarse gravel, Firm, Grey, Moist-wet	20	SS	7				
		21	SS	3				
		22	SS	3				
153.7 / 28.2	**CLAYEY SILT** Soft, Grey	23	SS	3		48.0		

Continued Next Page

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

RECORD OF BOREHOLE No B9-1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679235.3, E332593.8 ORIGINATED BY DG

DIST HWY WEP BOREHOLE TYPE CME 55 Track Mounted Drill - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 13 Jul 11 - 14 Jul 11 CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
151.4 / 30.5	**SANDY SILT** Trace fine-medium gravel Compact to Very dense Grey	24	SS	50	0 26 64 10
	Trace clay, trace fine-medium gravel	25	SS	17	
					-SPT refusal at 33.6m
148.2 / 33.7	-Rock fragments	26	SS		-end of drilling July 13
	LIMESTONE Fine Grained Non-calcareous inclusions, porous Grey				
147.7 / 34.2	**LIMESTONE** Medium to coarse grained, porous Laminated, stylolites present Fracture between 34.8m and 35.1m, running parallel to the core length	1	RC		RQD = 86% TCR = 100% SCR = 98%
146.8 / 35.1	**END OF BOREHOLE**				

Piezometric Levels in VWP B9-1-P10 (Shallow)
July 23, 2011: EL. 181.6m
August 6, 2011: EL. 181.9m

Piezometric Levels in VWP B9-2-P17 (Mid-depth)
July 23, 2011: EL. 182.2m
August 6, 2011: EL. 181.8m

No groundwater observed during drilling July 13 to July 14, 2011 due to wash boring

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (151 to 137); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No B9-2

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4679218.9, E332622.2		ORIGINATED BY DG
DIST	HWY WEP	BOREHOLE TYPE Track Mounted Drill - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 10 Jul 11 - 12 Jul 11		CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR SA SI CL	REMARKS
182.4	Pavement Surface						
0.0	50mm **ASPHALT**						-Shallow Vibrating Wire Piezometer (VWP) installed in adjacent boring at N4679218.5, E3326228. Mid-Depth and bedrock VWP installed in sampled borehole. Spider Magnets (MG) installed in adjacent boring at N4679218.0, E332623.4
182.0 / 0.4	Over 350mm Crushed Limestone Sand and gravel **FILL**						
181.3 / 1.1	Brown Fine Sand **FILL**	1	SS	5			
	CLAYEY SILT Some sand, trace gravel Soft to very stiff Brown Mottled	2	SS	13			
	Brown	3	SS	19			
	Grey	4	SS	16			
	-Some pink clay nodules	5	SS	12			
	-Trace pink clay nodules	6	SS	7		1 18 42 39	
		7	SS	6			
		8	SS	PH			-no recovery with shelby tube; sample retrieved by pushing split spoon
			VT				
		9	SS	PH			-no recovery with shelby tube; sample retrieved by pushing split spoon
		10	TW	PH			
			VT				
		11	SS	PH			-VWP B9-2-P11 installed at 10.67m below ground surface (EL. 172.0m) -MG installed at 10.97m below ground surface (EL. 171.7m)
	-Increasing silt content	12	TW	PH	20.3	2 18 41 39	
			VT				
	Laminated (thickness approx. 10mm) Some pink clay nodules	13	TW	PH	20.3		

Plot columns: GROUND WATER CONDITIONS; ELEVATION SCALE (182 to 168); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● POCKET PEN., × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30).

Field vane labels: >> 1.4 (EL. approx. 175.2); 1.8 (EL. approx. 172.1); 1.6 (EL. approx. 169.1)

Continued Next Page

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

RECORD OF BOREHOLE No B9-2

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679218.9, E332622.2 ORIGINATED BY DG

DIST HWY WEP BOREHOLE TYPE Track Mounted Drill - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 10 Jul 11 - 12 Jul 11 CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
	CLAYEY SILT Some sand, trace gravel Soft to very stiff *(continued)*	14	TW	PH	167	19.5	1	17	44	38	
			VT		166						
		15	SS	4	165						
	-Some sand, trace gravel	16	SS	2	164						-installed MG and VWP B9-2-P18 at 18.29m below ground surface (EL. 164.1m)
			VT		163						
	-Trace fine-medium gravel, trace sand	17	TW	PH	162	21.6	4	26	46	24	
		18	TW	PH	161						
			VT		160						
	-Trace sand, fine-medium gravel	19	TW	PH	159	21.3	4	22	45	29	
	Fine-coarse gravel	20	SS	8	158						-end of drilling July 11; continue July 12
		21	SS	8	157, 156						
		22	SS	5	155, 154						
		23	SS	8	153						

Field vane values plotted: 1.7 (near 166), 1.4 (near 163), >>1.5 (near 160).

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; GROUND WATER CONDITIONS; STRAT PLOT; REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No B9-2

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4679218.9, E332622.2	ORIGINATED BY	DG
DIST	HWY WEP	BOREHOLE TYPE	Track Mounted Drill - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	10 Jul 11 - 12 Jul 11	CHECKED BY	MSO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
151.9 / 30.5	**SILTY CLAY** Trace sand, fine gravel. Very soft. Grey	24	SS	0	152, 151	○ (~31)	0 5 50 45; -sampler advanced under weight of hammer and rods
150.4 / 32.0	**SILT** Trace clay and fine sand. Very loose. Grey. Moist-wet	25	SS	2	150	○ (~30)	0 0 93 7; -installed VWP B9-2-P32 at 32.00m below ground surface (EL. 150.4m)
148.9 / 33.5	**LIMESTONE COBBLES and BOULDERS** (Inferred from sample fragments). Trace clay, trace fine gravel	26	SS	50/150mm	149		
148.4 / 34.0	**LIMESTONE** Medium to coarse grained, laminated. Fractured at locations between 34.35m and 36.39m. Fracture is running parallel to the core length. Brown	1	RC		148		RQD = 31.8% TCR = 100% SCR = 33%
		2	RC		147		RQD = 17.4% TCR = 100% SCR = 33%
		3	RC		146		RQD = 88.7% TCR = 100% SCR = 88%
146.0 / 36.4	**LIMESTONE** Laminated, porous. Grey						
145.6 / 36.8	**END OF BOREHOLE**				145		

Piezometric Levels in VWP B9-2-P11 (Shallow)
July 23, 2011: EL. 181.1m
August 6, 2011: EL. 181.0m

Piezometric Levels in VWP B9-2-P19 (Mid-depth)
July 23, 2011: EL. 180.9m
August 6, 2011: EL. 179.8m

Piezometric Levels in VWP B9-2-P32 (Deep)
July 23, 2011: EL. 177.4m
August 6, 2011: EL. 177.6m

No groundwater observed during drilling July 10 to July 12, 2011 due to wash boring

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

Elevation scale continues: 144, 143, 142, 141, 140, 139, 138

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No B9-3

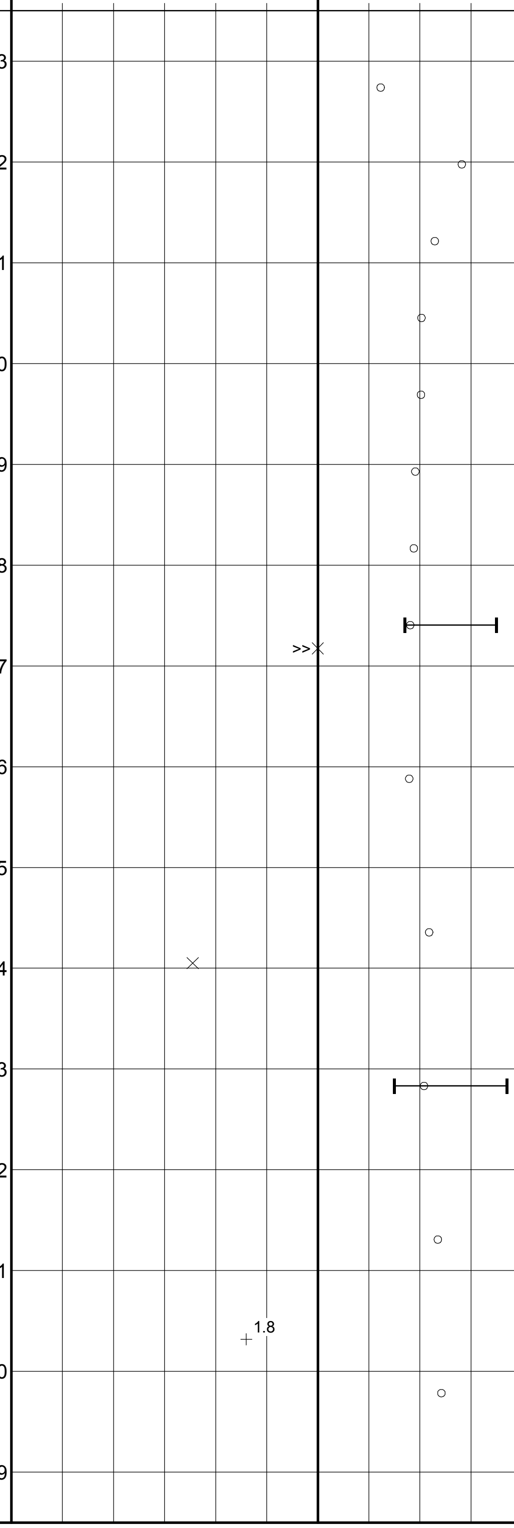
METRIC

W.P. RFP No. 09-54-1007	LOCATION N4679140.0, E332677.6		ORIGINATED BY LC
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 11 Jul 11 - 12 Jul 11		CHECKED BY MSO

SOIL PROFILE / SAMPLES

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
183.5	Ground Surface										
0.0 / 183.2 / 0.3	Sandy **TOPSOIL**				183						- bulk sample taken
	Fine Brown **SAND** Trace to some silt Compact	1	SS	10							-Slope inclinometer casing installed in sampled borehole; Vibrating Wire Piezometers (VWP) installed in adjacent boring at N4679140.8, E332679.2
	Loose -Black laminations Saturated	2	SS	7	182						
181.1 / 2.4	**SILTY CLAY** Some sand, trace gravel Very soft to stiff Grey	3A, B	SS	8	181						
		4	SS	14	180						
		5	SS	13							
		6	SS	11	179						
		7	SS	7	178						
		8	TW	PH	177	20.7	3	20	37	40	
	Moist to wet	9	TW	PH	176 / 175						
	Moist to wet	10	TW	PH	174	20.8	1	17	38	44	
	Wet	11	TW	PH	173		2	20	35	43	
		12	TW	4	172 / 171						
			VT								
	Wet	13	SS	PH	170 / 169						-no recovery with shelby tube; sample retrieved by pushing split spoon

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30



Continued Next Page

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

RECORD OF BOREHOLE No B9-3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679140.0, E332677.6 ORIGINATED BY LC

DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 11 Jul 11 - 12 Jul 11 CHECKED BY MSO

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

Soil profile

- **SILTY CLAY** — Some sand, trace gravel; Very soft to stiff; Grey *(continued)*
  - Pink-Grey
- 164.0 / 19.5 — **CLAYEY SILT** — Some sand, trace gravel; Firm; Grey; Moist to wet
  - Trace shale fragments below approx. 25m

Elevation scale: 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154

Number	Type	"N" Values	Unit Weight (kN/m³)	Grain size GR SA SI CL	Remarks
14	TW	PH		3 7 43 47	-end of HS augers, continue with NW Casing wash boring -VWP B9-3-P15 at 15.24m below ground surface (El. 168.3m)
	VT				
15	TW	PH	20.0	2 15 37 46	
16	SS	PH			-shelby tube damaged at tip
	VT				
17	SS	PH			-no recovery with shelby tube; sample retrieved by driving Split Spoon
18	SS	PH			-no recovery with shelby tube; sample retrieved by driving Split Spoon
	VT				
19	TW	PH			
20	SS	8			
					-VWP B9-3-P25 at 25.15m below ground surface(EL. 158.3m)
21	SS	13			-no recovery, inferred cobble recovery second try (N-values likely inflated)
22	SS	7		2 22 43 33	
23	SS	8			

Plot annotations: 48.0; 2.4; 1.8; 1.8

Continued Next Page

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

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RECORD OF BOREHOLE No B9-3

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4679140.0, E332677.6		ORIGINATED BY LC
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic		DATE 11 Jul 11 - 12 Jul 11	CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
153.1 / 30.4	**SILT** Trace clay, moist to wet Grey	24	SS	10	153	w plotted at approx. 18	
		25	SS	14	151		-no recovery
150.3 / 33.2	**SANDY SILT** Trace clay, trace gravel -Slightly cemented Compact Grey Moist to wet	26	RC	26	150		
149.4 / 34.1							
149.1 / 34.4	**LIMESTONE** Fine grained Partially crystallized, calcite crystallization is visible, vuggy at approx. 34.44m Grey	27	RC		149		RQD = 72% TCR = 100% SCR = 73% Rock Core UC = 77.8 MPa
147.8 / 35.7	Medium to coarse grained **LIMESTONE** Porous, laminated, fractured at approx. 35.63m				148		
	END OF BOREHOLE						

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

Elevation scale: 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143, 142, 141, 140, 139

Piezometric Levels in VWP B9-3-P15 (Shallow)
July 23, 2011: EL. 182.9m
August 29, 2011: EL. 180.3m

Piezometric Levels in VWP B9-3-P25 (Mid-depth)
July 23, 2011: EL. 181.1m
August 29, 2011: EL. 178.9m

No groundwater observed during drilling July 11, 2011 to July 12, 2011 due to wash boring

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

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RECORD OF BOREHOLE No CPT B9-1

METRIC

W.P. RFP No. 09-54-1007 | LOCATION N4679241.3, E332574.3 | ORIGINATED BY TA

DIST ___ HWY WEP | BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY SS

DATUM Geodetic | DATE 25 Jul 11 - 25 Jul 11 | CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
182.4	Ground Surface			
0.0	**TOPSOIL**			
182.1 / 0.3	**CLAYEY SILT** Some sand, trace gravel, trace organics in upper 1.0 m mottled brown-grey	1	SS	11
		2	SS	8
		3	SS	9
	-Trace fissures Brown	4	SS	20
	Grey	5	SS	13
178.1 / 4.3	**END OF SAMPLED BOREHOLE** Continue with CPT from 4.3 m to refusal at 31.7 m (El. 178.1 m to El. 150.7 m) No groundwater observed on July 25, 2011			

Chart columns: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (182 to 168); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● POCKET PEN., × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10, 20, 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 14/03/12

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No CPT B9-2

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4679138.6, E332696.0	ORIGINATED BY	TA
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	25 Jul 11 - 25 Jul 11	CHECKED BY	MSO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)
183.9	Ground Surface					
0.0	**TOPSOIL**					
183.3 / 0.6	**FINE SAND** Poorly Graded, trace silt Brown	1	SS	10		
		2	SS	8	∇	-Groundwater on July 25, 2011
181.6 / 2.3	**SANDY SILT**, brown	3	SS	5		
181.3 / 2.6	**CLAYEY SILT** Some sand Brown					
181.2 / 2.7	**END OF SAMPLED BOREHOLE** Continue with CPT from 2.7 m to refusal at 33.5 m (El. 181.2 m to El. 150.4 m) Groundwater observed at 1.7 m (El. 182.2 m) on July 25, 2011					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (183 to 169); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● POCKET PEN., × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; GR SA SI CL

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

RECORD OF BOREHOLE No CPT B9-3

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4679189.2, E332678.6		ORIGINATED BY TA
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 26 Jul 11 - 26 Jul 11		CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.7	Ground Surface						
0.0	**TOPSOIL**	1	SS	2			-sample from auger cuttings
182.4 / 0.3	**FINE SAND** Poorly Graded Trace silt, brown					182	
		2A, B	SS	8	▽		
181.5 / 1.2	**CLAYEY SILT** Trace to some sand Grey with trace oxidation in fissures						-Groundwater on July 26, 2011
		3	SS	14		181	
		4	SS	16		180	
179.8 / 2.9	**END OF SAMPLED BOREHOLE** Continue with CPT from 3.7 m to refusal at 30.4 m (El. 179.0 m to El. 152.3 m) Groundwater observed at 1.2 m (El. 181.5 m) on July 26, 2011					179	

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

Elevation scale: 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168

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

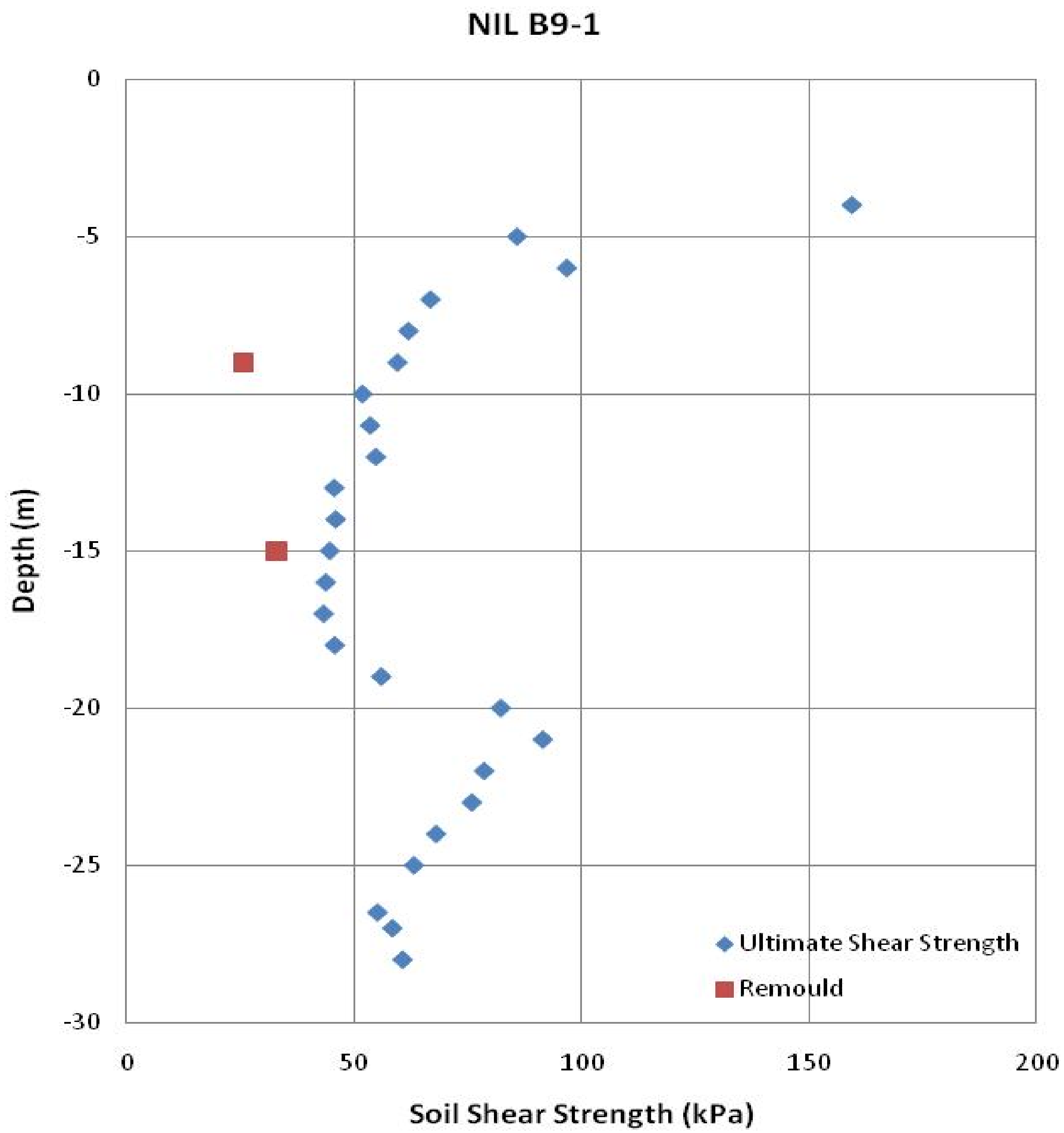
Foundation Design

RECORD OF NILCON VANE TEST NIL B9-1

Project : Windsor-Essex Parkway
Location: N 4679235.3; E 332593.8

Test Date: 7/18/2011
Predrill Depth : 4 m

Sheet 1 of 1
Datum Geodetic
El : 181.9



Operator: SO

Checked: DD

RECORD OF CONE PENETRATION TEST CPT B9-1

METRIC

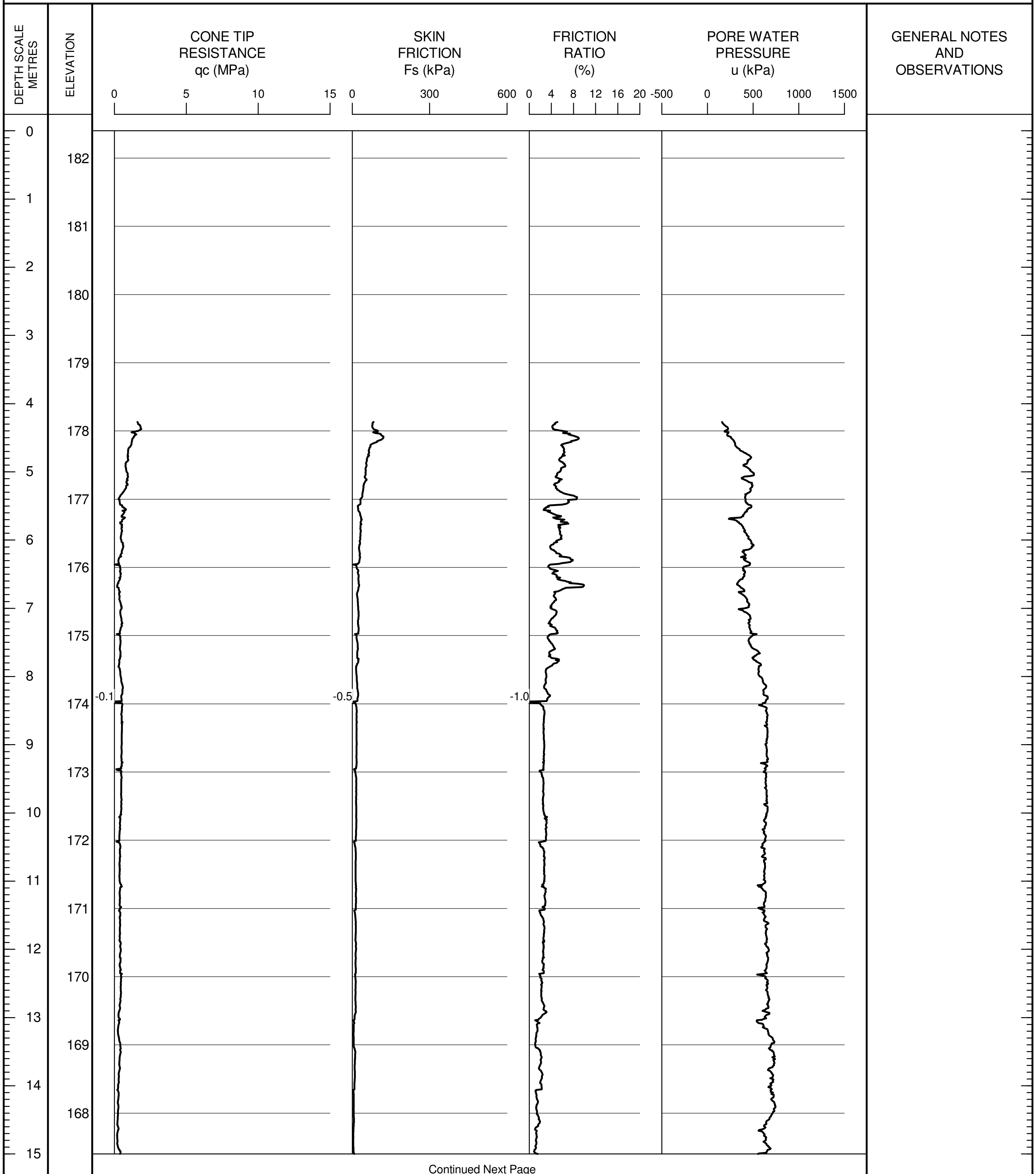
PROJECT Windsor-Essex Parkway

TEST DATE 7/25/2011 - 7/25/2011

LOCATION N 4679241.3; E 332574.3

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.4 PREDRILL DEPTH: 4.3 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



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OPERATOR: TA

CHECKED: NR

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Foundation Design

RECORD OF CONE PENETRATION TEST CPT B9-1

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 7/25/2011 - 7/25/2011

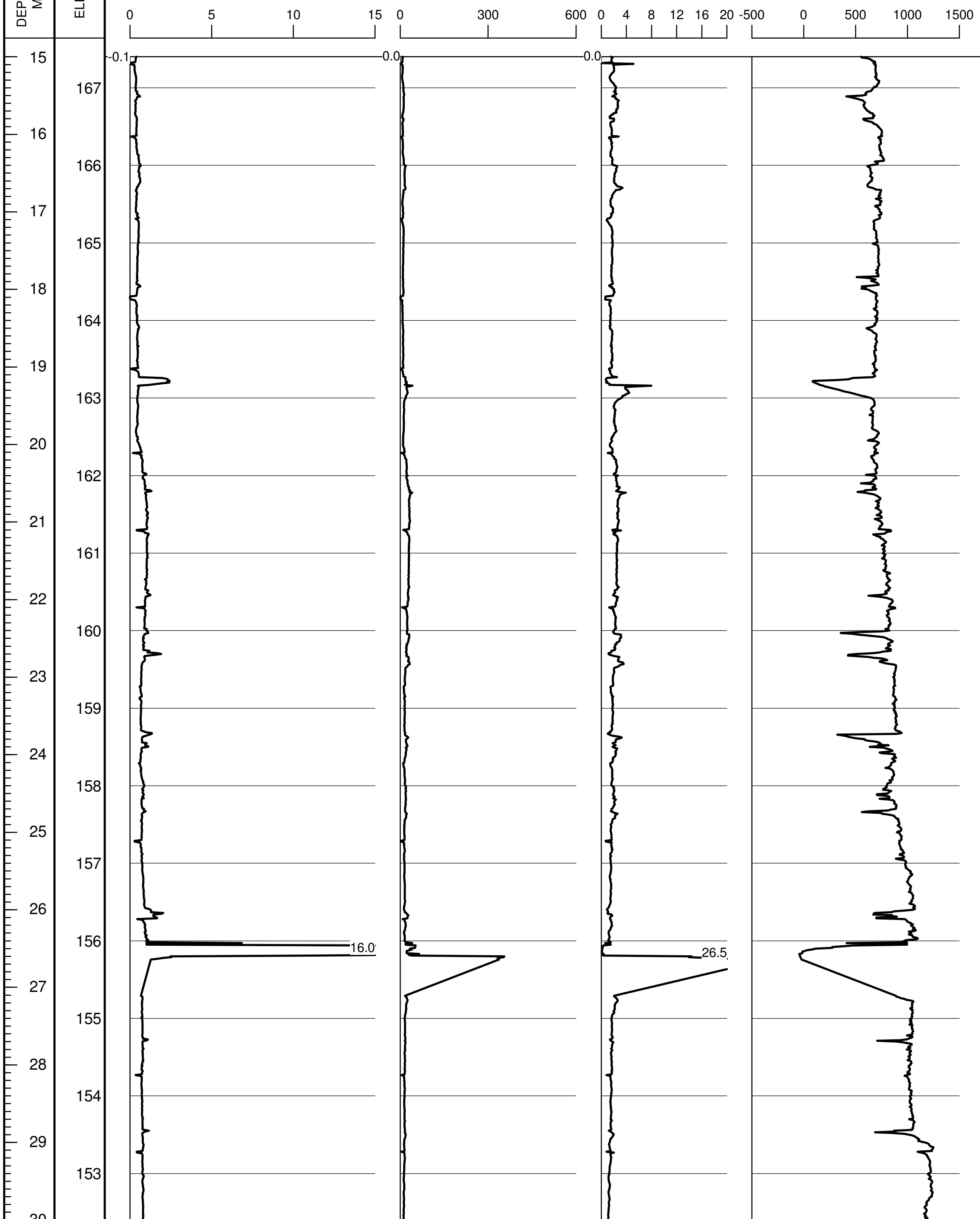
SHEET 2 OF 3

LOCATION N 4679241.3; E 332574.3

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.4 PREDRILL DEPTH: 4.3 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0

DEPTH SCALE METRES	ELEVATION	CONE TIP RESISTANCE qc (MPa)	SKIN FRICTION Fs (kPa)	FRICTION RATIO (%)	PORE WATER PRESSURE u (kPa)	GENERAL NOTES AND OBSERVATIONS



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OPERATOR: TA

CHECKED: NR

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RECORD OF CONE PENETRATION TEST CPT B9-1

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 7/25/2011 - 7/25/2011

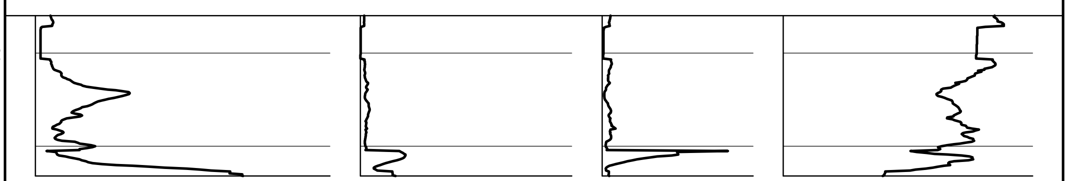
SHEET 3 OF 3

LOCATION N 4679241.3; E 332574.3

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.4 PREDRILL DEPTH: 4.3 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0

DEPTH SCALE METRES	ELEVATION	CONE TIP RESISTANCE qc (MPa) 0 5 10 15	SKIN FRICTION Fs (kPa) 0 300 600	FRICTION RATIO (%) 0 4 8 12 16 20	PORE WATER PRESSURE u (kPa) -500 0 500 1000 1500	GENERAL NOTES AND OBSERVATIONS
30	152					
31	151					



OPERATOR: TA

CHECKED: NR

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RECORD OF CONE PENETRATION TEST CPT B9-2

METRIC

PROJECT Windsor-Essex Parkway

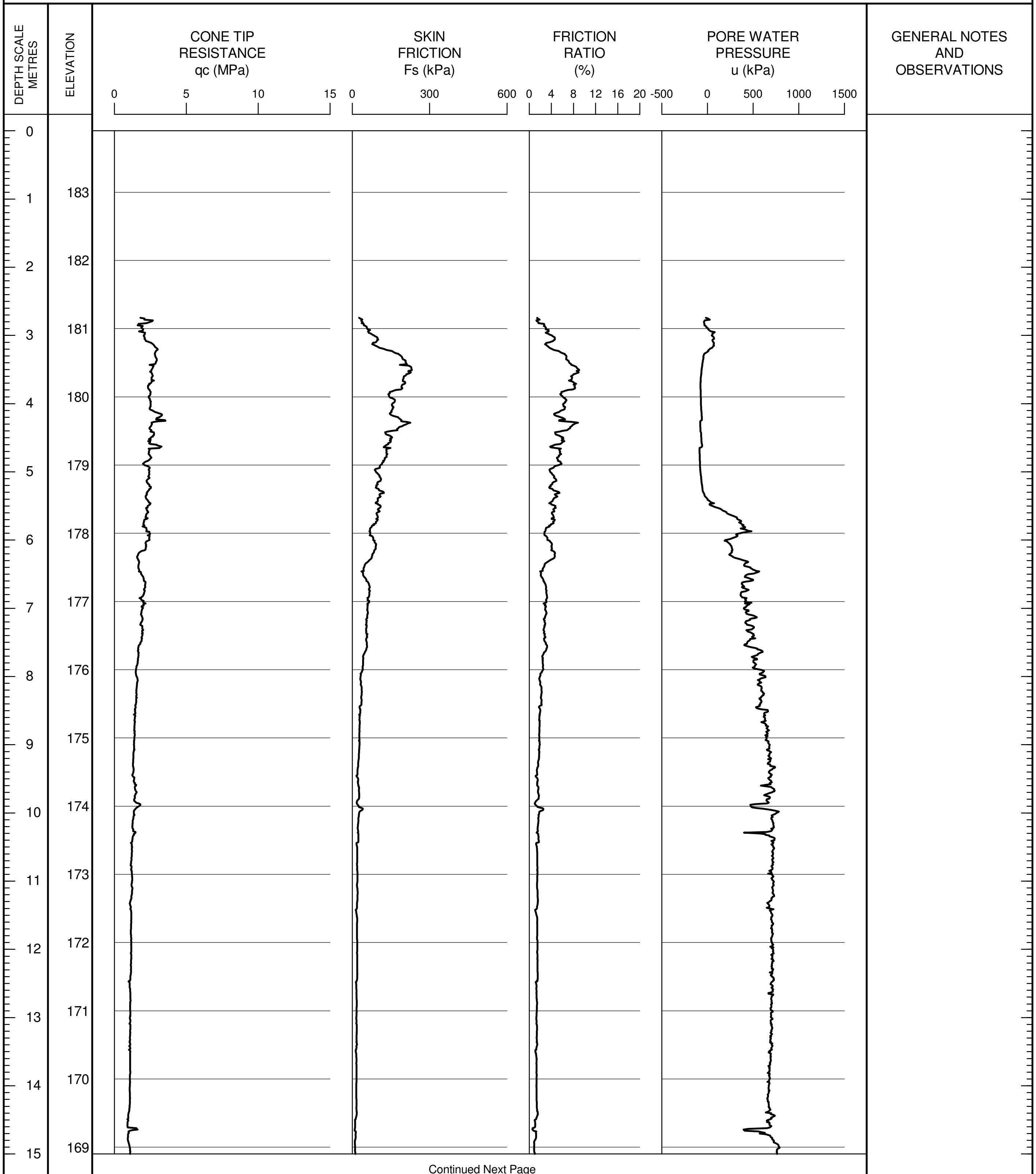
TEST DATE 7/25/2011 - 7/25/2011

LOCATION N 4679138.6; E 332696

DATUM Geodetic

GROUND SURFACE ELEVATION: 183.9 PREDRILL DEPTH: 2.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0

GENERAL NOTES AND OBSERVATIONS



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OPERATOR: TA

CHECKED: NR

WEP CPT LOG CPT 33-RW.GPJ ONTARIO MOT.GDT 10/08/11

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RECORD OF CONE PENETRATION TEST CPT B9-2

METRIC

PROJECT Windsor-Essex Parkway

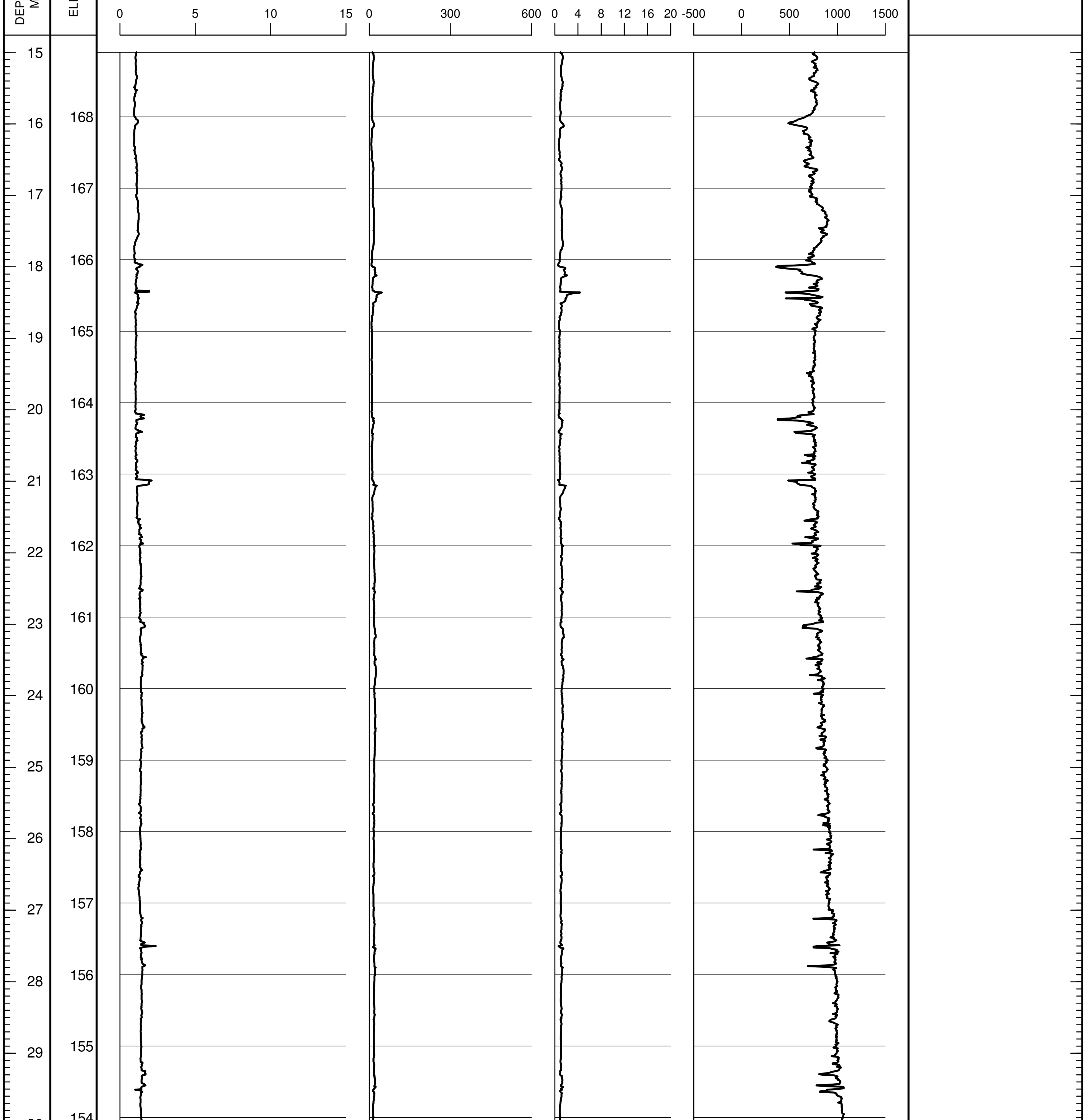
TEST DATE 7/25/2011 - 7/25/2011

LOCATION N 4679138.6; E 332696

DATUM Geodetic

GROUND SURFACE ELEVATION: 183.9 PREDRILL DEPTH: 2.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0

DEPTH SCALE METRES	ELEVATION	CONE TIP RESISTANCE qc (MPa)	SKIN FRICTION Fs (kPa)	FRICTION RATIO (%)	PORE WATER PRESSURE u (kPa)	GENERAL NOTES AND OBSERVATIONS



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OPERATOR: TA

CHECKED: NR

WEP CPT LOG CPT 33-RW.GPJ ONTARIO MOT.GDT 10/08/11

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Foundation Design

RECORD OF CONE PENETRATION TEST CPT B9-2

METRIC

PROJECT Windsor-Essex Parkway

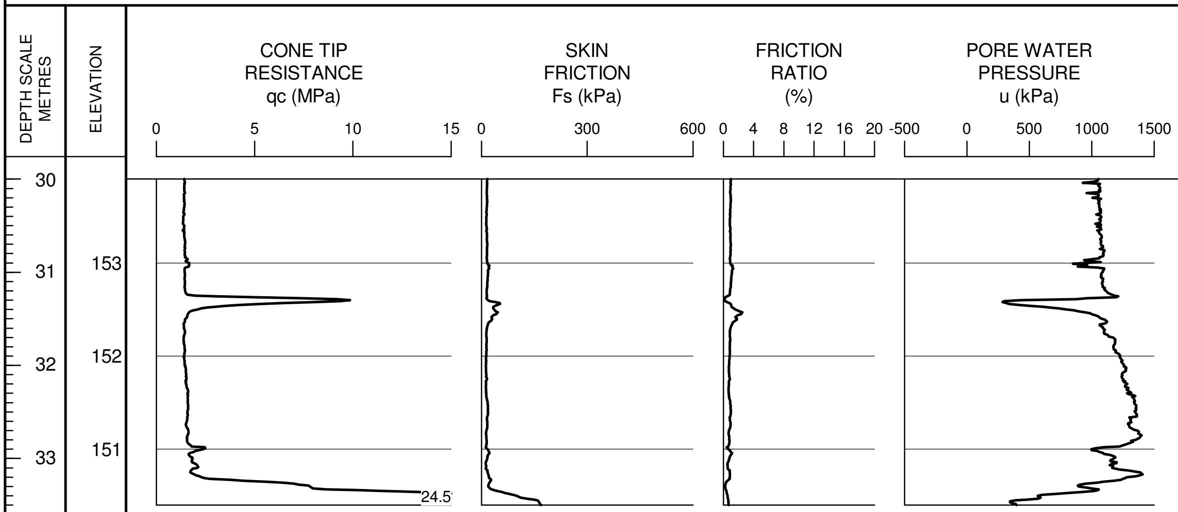
TEST DATE 7/25/2011 - 7/25/2011

SHEET 3 OF 3

LOCATION N 4679138.6; E 332696

DATUM Geodetic

GROUND SURFACE ELEVATION: 183.9 PREDRILL DEPTH: 2.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

OPERATOR: TA

CHECKED: NR

WEP CPT LOG CPT 33-RW.GPJ ONTARIO MOT.GDT 10/08/11

RECORD OF CONE PENETRATION TEST CPT B9-3

METRIC

PROJECT Windsor-Essex Parkway

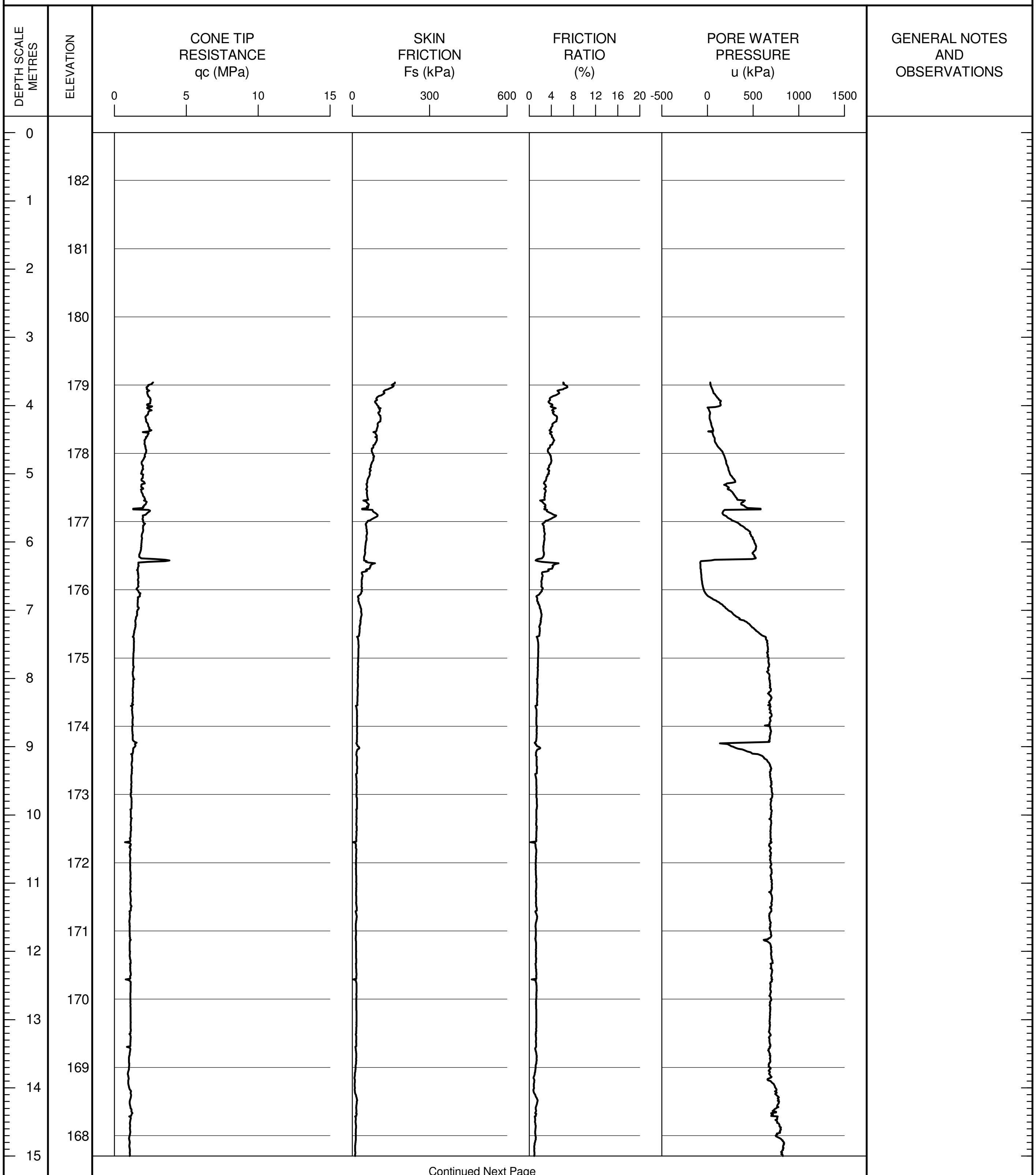
TEST DATE 7/26/2011 - 7/26/2011

SHEET 1 OF 3

LOCATION N 4679189.2; E 332678.6

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.7 PREDRILL DEPTH: 3.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



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OPERATOR: TA

CHECKED: NR

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

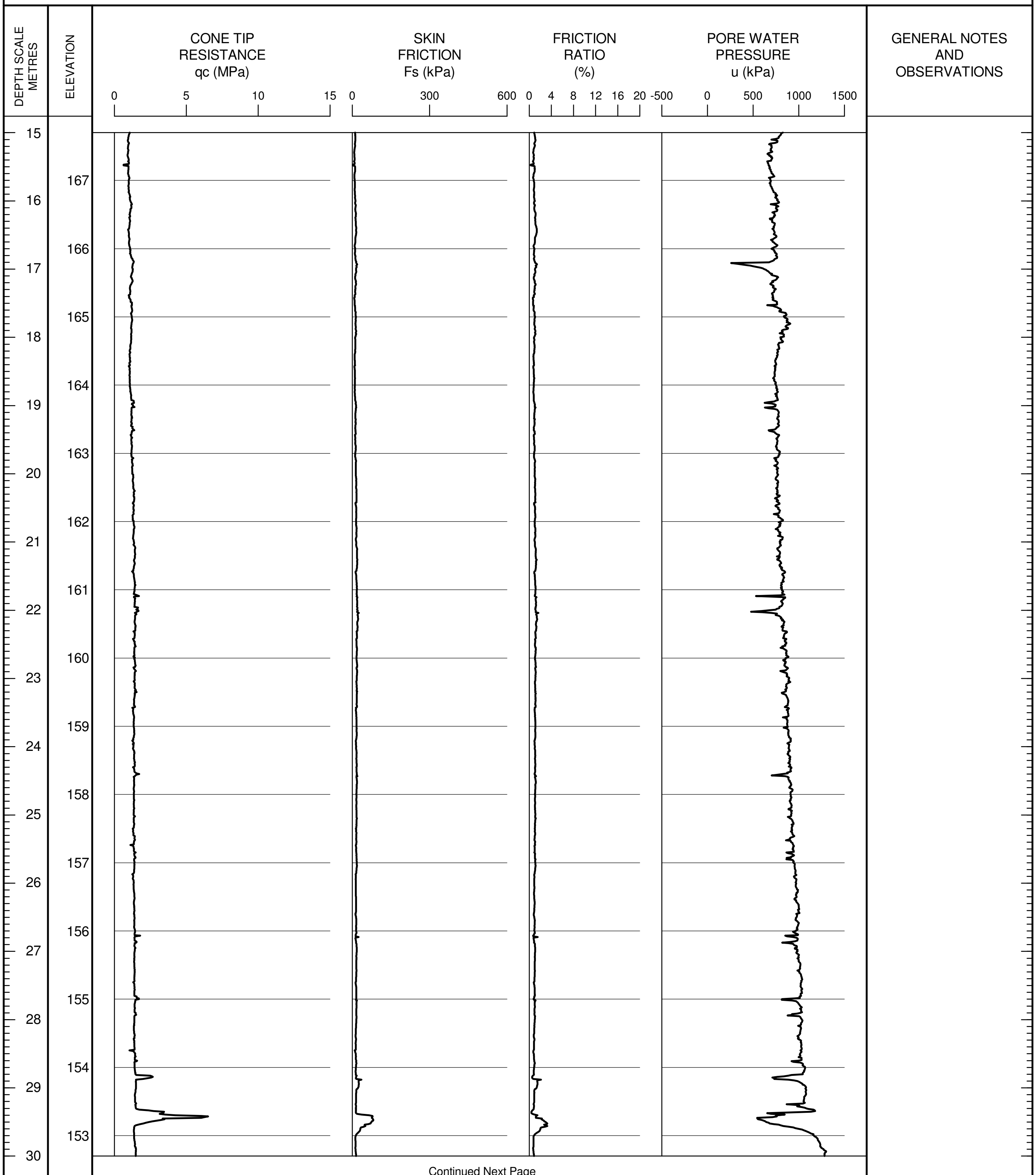
RECORD OF CONE PENETRATION TEST CPT B9-3

METRIC

PROJECT Windsor-Essex Parkway TEST DATE 7/26/2011 - 7/26/2011 SHEET 2 OF 3

LOCATION N 4679189.2; E 332678.6 DATUM Geodetic

GROUND SURFACE ELEVATION: 182.7 PREDRILL DEPTH: 3.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

CHECKED: NR

WEP CPT LOG CPT 33-RW.GPJ ONTARIO MOT.GDT 10/08/11

Foundation Design

RECORD OF CONE PENETRATION TEST CPT B9-3

METRIC

PROJECT Windsor-Essex Parkway

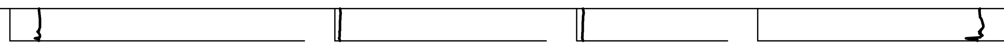
TEST DATE 7/26/2011 - 7/26/2011

LOCATION N 4679189.2; E 332678.6

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.7 PREDRILL DEPTH: 3.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0

DEPTH SCALE METRES	ELEVATION	CONE TIP RESISTANCE qc (MPa) 0 5 10 15	SKIN FRICTION Fs (kPa) 0 300 600	FRICTION RATIO (%) 0 4 8 12 16 20	PORE WATER PRESSURE u (kPa) -500 0 500 1000 1500	GENERAL NOTES AND OBSERVATIONS
30						
	152					



OPERATOR: TA

CHECKED: NR

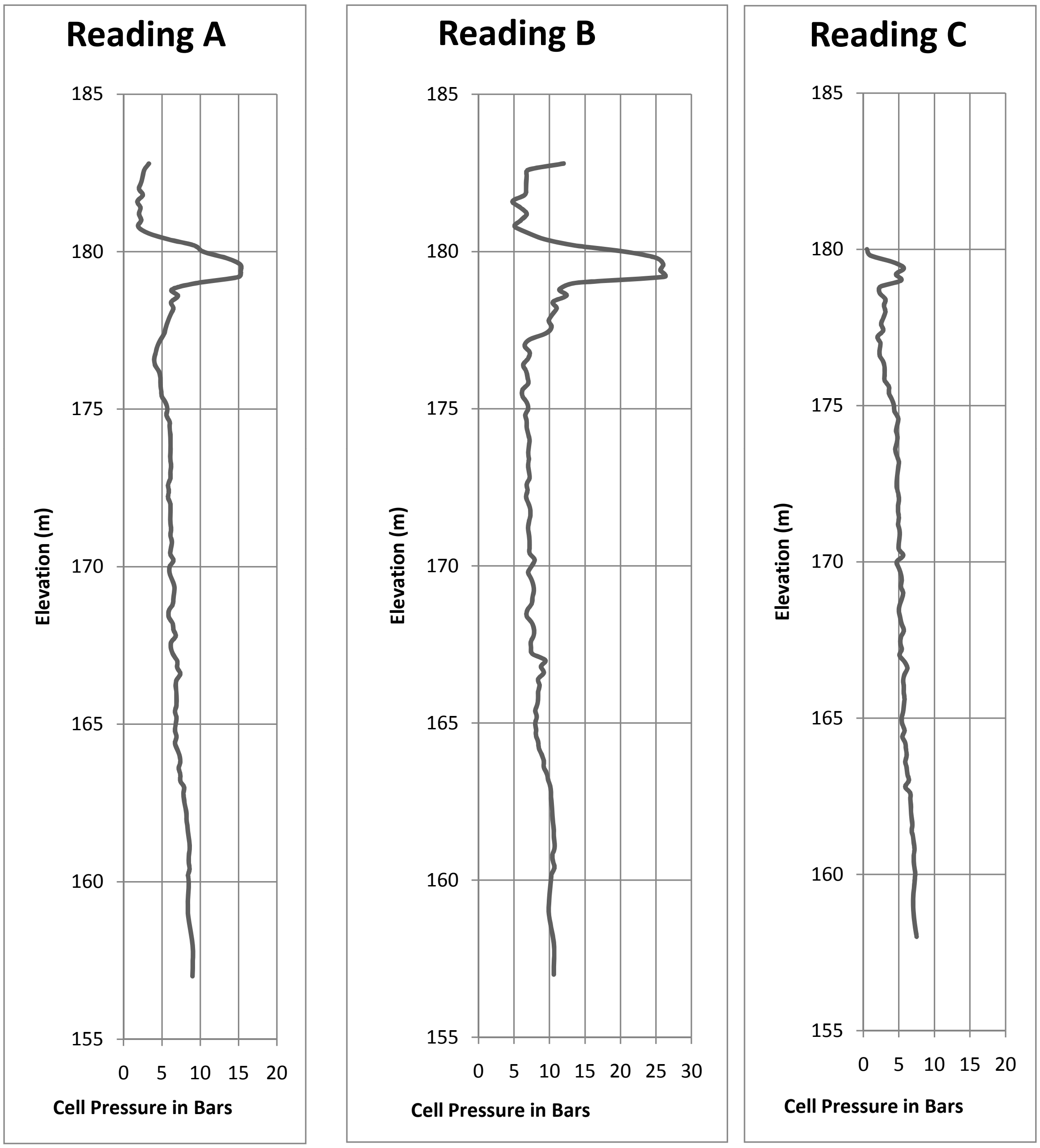
Foundation Design

RECORD OF DILATOMETER TEST DMT B9-1

Project : Windsor-Essex Parkway
Location: N 4679242.7; E 332578.4
Ground Surface Elevation : 183.0

Test Date: 7/16/2011
Predrill Depth : 0 m
Delta A: 0.13 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.23 Bar



Operator: LC

Checked: DD

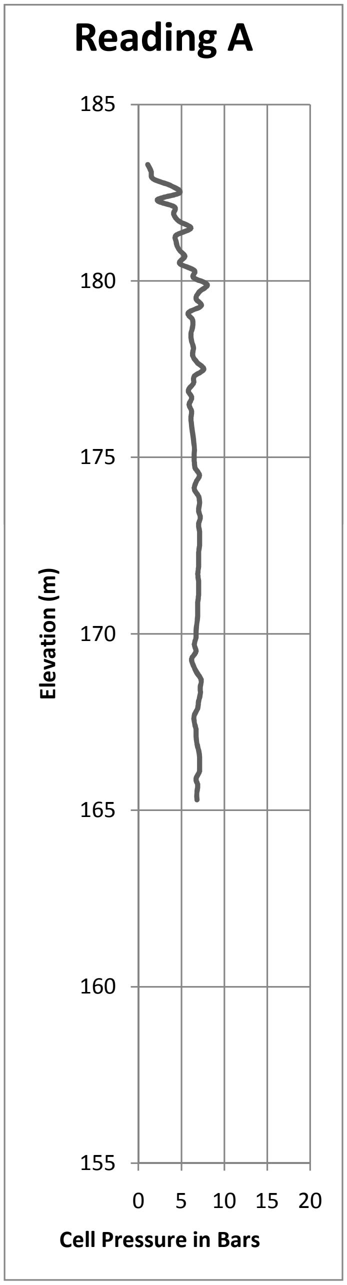
Foundation Design

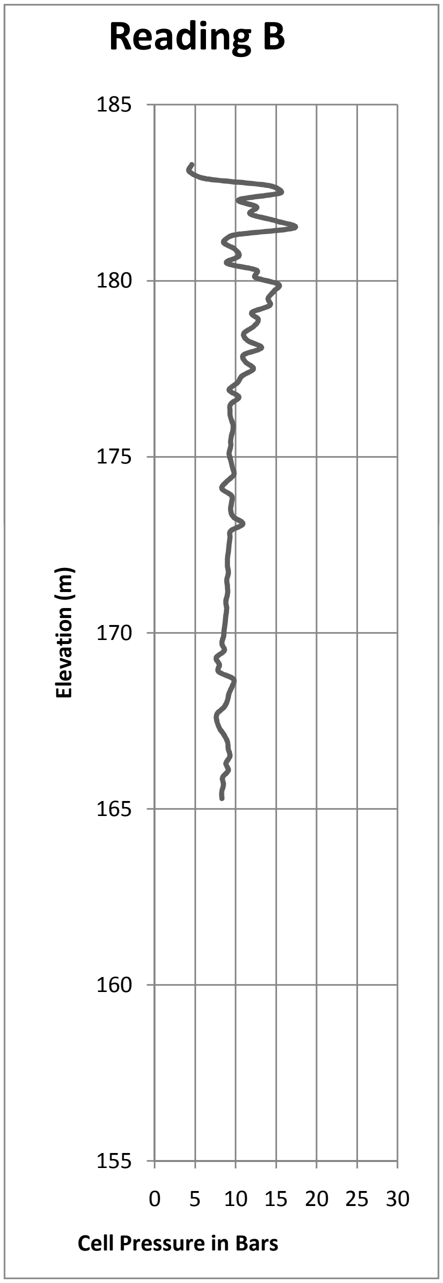
RECORD OF DILATOMETER TEST DMT B9-2

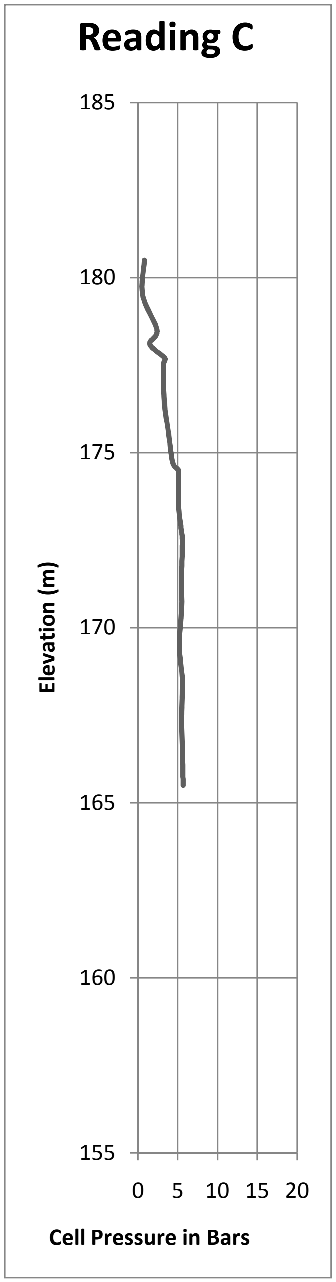
Project : Windsor-Essex Parkway
Location: N 4679144.8; E 332687.7
Ground Surface Elevation : 183.5

Test Date: 7/18/2011
Predrill Depth : 0 m
Delta A: 0.14 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.19 Bar







Operator: LC

Checked: DD

Appendix B: Borehole Logs from Previous Investigations

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 122

METRIC

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

LOCATION N 4679265.4 ;E 332537.9
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE January 24, 2008 - January 29, 2008

ORIGINATED BY SM
COMPILED BY BRS
CHECKED BY SBB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (sensitivity)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.66	GROUND SURFACE							
0.00	TOPSOIL, clayey Black				Bentonite			
0.20	CLAYEY SILT, some sand, trace gravel Very stiff Mottled brown and grey				181			
		1	SS	20				
		2	SS	17	180			
		3	SS	22	179			
		4	SS	18				
178.00 3.66	CLAYEY SILT, some sand, trace gravel Firm to stiff Grey				178			
		5	SS	12				
		6	TO	PH	177			1 21 41 37 CIUC
					176	122.1 (2.0); 137.9 (1.6)		
		7	SS	8	175	+3.9; +2.3		
		8	SS	7	174	+3.9; +2.8		
		9	SS	WH	173 / 172	+1.9; +1.5		
		10	TO	PH	171			1 19 45 35 CIUE
					170	+2.2; +1.9		
		11	SS	2	169	+; +3.4		
		12	TO	PH	168			
					167	+2.7		

Continued Next Page

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

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 122

METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4679265.4 ;E 332537.9		ORIGINATED BY SM
DIST WEST	HWY 401/3	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC	COMPILED BY BRS
DATUM GEODETIC	DATE January 24, 2008 - January 29, 2008		CHECKED BY SJB

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

Description: CLAYEY SILT, some sand, trace gravel
Firm to stiff
Grey

Sample Number	Type	"N" Values	Remarks / Grain Size (GR SA SI CL)
13	SS	2	
14	TO	PH	4 24 39 33 CIUC
15	SS	12	
16	TO	PH	
17	SS	8	
18	TO	PH	
19	TO	PH	5 26 44 25 CIUC
20	TO	PH	
21	TO	PH	
22	TO	PH	0 22 69 9 CIUC

Elevation scale: 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153, 152

Field vane sensitivity values plotted: 1.3, 1.6, 1.4, (>143.6), 122.1, 2.0, 1.6, 1.4, 1.1, 1.7, 1.4, 1.3, 1.4, 1.7, 1.3, 1.4, 1.6, 2.0

Water content annotation: 47

151.94
29.72

Continued Next Page

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

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 122

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION N 4679265.4 ;E 332537.9

ORIGINATED BY SM

DIST WEST HWY 401/3

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

COMPILED BY BRS

DATUM GEODETIC

DATE January 24, 2008 - January 29, 2008

CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND, fine to medium, trace clay Compact Grey	23	SS	13	Grout	151	(>143.6)	w plotted		(29)
150.42 31.24	SANDY SILT, trace clay, with clayey silt intrusions Very dense Grey	24	SS	84		150		w plotted		
		25	SS	100/ 12.5mm	Bentonite	149, 148, 147				
146.61 35.05	LIMESTONE, fresh, medium strong, thinly laminated to laminated, very fine to fine grained, faintly to strongly porous Brown to grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	26	SS	50/ 3.8mm						
		27	NQ RC			146	T.C.R. (%) 100; S.C.R. (%) 100; R.Q.D. (%) 73			
		28	NQ RC		Sand	145	T.C.R. (%) 90; S.C.R. (%) 90; R.Q.D. (%) 60			
		29	NQ RC			144, 143	T.C.R. (%) 100; S.C.R. (%) 100; R.Q.D. (%) 100			
		30	NQ RC		Screen	142	T.C.R. (%) 93; S.C.R. (%) 93; R.Q.D. (%) 93			UC
141.33 40.33	END OF BOREHOLE									

Borehole dry during drilling between January 24 and 29, 2008.

Water level measured in deep piezometer at elev. 178.01m on July 22, 2008.

Water level measured in deep piezometer at elev. 178.26m on August 11, 2008.

Water level measured in deep piezometer at elev. 178.26m on September 19, 2008.

Water level measured in deep piezometer at elev. 177.54m on November 11, 2008.

Water level measured in deep piezometer at elev. 177.21m on January 28, 2009.

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

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

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 122

LOCATION: N 4679265.4 ;E 332537.9

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: January 24, 2008 - January 29, 2008

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DATUM: GEODETIC

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
		ROCK SURFACE	146.61	
	MUD ROTARY / NO ROCK CORE	LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous, brown	35.05	JN,PL,Ro Si
36		LIMESTONE, fresh, medium strong, thinly laminated, very fine to fine grained, faintly porous, white to light grey	145.79 / 35.87	JN,UN,Ro Si
37		LIMESTONE, fresh, medium strong, laminated, fine grained, moderately porous, brown to grey	144.84 / 36.82	BD,PL,SM Cl
		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous, light grey to brown	144.54 / 37.12	
38		LIMESTONE, fresh, medium strong, laminated, very fine grained, faintly porous, grey	143.74 / 37.92	
		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, moderately porous, light brown to grey	143.41 / 38.25	JN,PL,SM Cl
39		LIMESTONE, fresh, medium strong, medium grained, strongly porous, light grey to brown	142.68 / 38.98	
40		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, moderately porous, light grey to brown	142.04 / 39.62	
		END OF DRILLHOLE	141.33 / 40.33	

Column headings: SYMBOLIC LOG; RUN No.; PENETRATION RATE (m/min); FLUSH COLOUR % RETURN; ELEVATION; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3; DIP w.r.t. CORE AXIS; HYDRAULIC CONDUCTIVITY k, cm/sec; DIAMETRAL POINT LOAD INDEX (MPa); NOTES WATER LEVELS INSTRUMENTATION

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: SG

CHECKED: SJB

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 122A

METRIC

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

LOCATION N 4679265.4 ;E 332537.9
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
DATE January 24, 2008

ORIGINATED BY SM
COMPILED BY BRS
CHECKED BY SSB

ELEV DEPTH	SOIL PROFILE: DESCRIPTION (SOIL CONDITIONS INFERRED FROM BOREHOLE No. 122)	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.66	GROUND SURFACE										
0.00	TOPSOIL, clayey Black					Bentonite					
0.20	CLAYEY SILT, some sand, trace gravel Very stiff Mottled brown and grey						181 180 179				
178.00 3.66	CLAYEY SILT, some sand, trace gravel Firm to stiff Grey					Grout Bentonite Sand Piezometer	178 177 176 175 174 173				
172.52 9.14	END OF BOREHOLE Water level measured in shallow piezometer at elev. 179.81m on August 11, 2008 Water level measured in shallow piezometer at elev. 179.53m on September 19, 2008 Water level measured in shallow piezometer at elev. 180.22m on January 28, 2009										

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 321

METRIC

PROJECT 09-1132-0080
W.P.
LOCATION N 4679179.9 ;E 332649.0
ORIGINATED BY MR
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY LMK/DMB
DATUM GEODETIC
DATE December 9, 2010 - December 14, 2010
CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.14	GROUND SURFACE				
0.00	TOPSOIL, sandy, Dark grey				
182.68 / 0.46	FILL, sand, fine, Yellowish brown				
0.67	TOPSOIL, some sand, trace rootlets, Stiff, Dark brown	1	SS	9	
181.77 / 1.37	SAND, fine to medium, some silt, Compact, Brown	2	SS	11	
181.31 / 1.83	SILTY CLAY, some sand, trace gravel, Very stiff, Grey	3	SS	16	
		4	SS	19	1 19 39 41
		5	SS	14	
		6	SS	13	
177.81 / 5.33	CLAYEY SILT, some sand, trace to some gravel, Stiff to very stiff, Grey	7	TO	PH	
		8	SS	8	
		9	TO	PH	
		10	SS	7	3 22 39 36
		11	TO	PH	
		12	SS	8	

Field vane annotations: (>143.6), (>143.6), 1.7, 1.8, 1.7, 1.6, 1.9
Liquid limit annotation: 44
Grout

Continued Next Page

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

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 321

METRIC

PROJECT 09-1132-0080
W.P.
LOCATION N 4679179.9 ;E 332649.0
ORIGINATED BY MR
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY LMK/DMB
DATUM GEODETIC
DATE December 9, 2010 - December 14, 2010
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, some sand, trace to some gravel Stiff to very stiff Grey	13	TO	PH	168	
					167	
		14	SS	9		1 21 49 29
					166	
		15	TO	PH	165	
					164	
		16	TO	PH		
					163	
		17	SS	10	162	
					161	
		18	TO	PH		
		19	SS	9	160	11 24 40 25
					159	
		20	TO	PH		
					158	
		21	SS	13	157	
					156	
		22	TO	PH		
					155	
		23	SS	9	154	

GROUND WATER CONDITIONS: Grout

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100

Field vane values (sensitivity): 2.2, 1.8, 1.6, 1.7, (>95.8), 2.1, (>143.6), 2.1, 1.9

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

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London, Ontario

PROJECT 09-1132-0080

RECORD OF BOREHOLE No 321

METRIC

W.P.
LOCATION N 4679179.9 ;E 332649.0
ORIGINATED BY MR

DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY LMK/DMB

DATUM GEODETIC
DATE December 9, 2010 - December 14, 2010
CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
	CLAYEY SILT, some sand, trace to some gravel Stiff to very stiff Grey	24	TO	PH	Grout	153										
						152				+ 1.5						
151.24 31.90	SAND AND GRAVEL, trace silt Very dense Grey	25	SS	51		151					o		21	69	7	3
					Bentonite											
		26	SS	100/130mm		150					o					
149.12 34.02	LIMESTONE, fresh, medium strong, weakly laminated, very fine grained, faintly porous Light grey to brown (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	27	NQ RC	-		149 / 148	97	92	87							
		28	NQ RC	-	Sand	147	98	87	77							
		29	NQ RC	-		146 / 145	100	100	98							
		30	NQ RC	-	Piezometer	144	100	95	95							
143.73 39.41	END OF BOREHOLE Groundwater encountered at about elev. 181.7m and at about elev. 151.7m during drilling between December 9 and 14, 2009. Water level measured at elev. 178.52 on February 24, 2010. Water level measured at elev. 178.26 on January 6, 2010.															

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

PROJECT: 09-1132-0080

RECORD OF DRILLHOLE: 321

SHEET 4 OF 4

LOCATION: N 4679179.9 ;E 332649.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 9, 2010 - December 14, 2010

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: LANTECH

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	ELEVATION	RECOVERY TOTAL CORE % / SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE		149.12												
	MUD ROTARY / NQ ROCK CORE	LIMESTONE, fresh, medium strong, weakly laminated, very fine grained, faintly porous, light grey		34.02				149					JN, CU, SM			
35		LIMESTONE, fresh, medium strong, thinly laminated to bedded, very fine to fine grained, faintly porous, brown to grey		148.73 / 34.41	1			148								
36					2			147								
37		LIMESTONE, fresh, medium strong, weakly laminated, very fine grained, faintly porous, light grey, occasional stylolites		146.81 / 36.33	3			146								
38								145								
39					4			144								
		END OF DRILLHOLE		143.73 / 39.41												
40																
41																
42																
43																
44																
45																
46																
47																
48																
49																

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: SG

CHECKED:

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-123

METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4679309.7 ;E 332536.3		ORIGINATED BY CC
DIST WEST	HWY 401/3	BOREHOLE TYPE POWER AUGER, SOLID STEM	COMPILED BY SJL
DATUM GEODETIC	DATE September 10, 2008		CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.60	GROUND SURFACE									
0.00	FILL, crushed gravel and recycled aggregate									
0.28	Grey and black TOPSOIL, clayey Black		1	AS			181	o		
180.69 0.91	CLAYEY SILT, trace sand, trace gravel Firm to stiff Mottled brown and grey		2	SS	7			o		
			3	SS	14		180	o		
179.47 2.13	END OF BOREHOLE Borehole dry during drilling on September 10, 2008.									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-319

METRIC

PROJECT 09-1132-0080

W.P.

LOCATION N 4679084.5 ;E 332701.0

ORIGINATED BY TA

DIST WEST HWY 401 / 3

BOREHOLE TYPE POWER AUGER, SOLID STEM

COMPILED BY DMB

DATUM GEODETIC

DATE December 21, 2009

CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.71	ROAD SURFACE										
0.05	ASPHALT PAVEMENT										
0.20	FILL, limestone gravel, crushed Grey										
	SAND, fine, some silt Compact Brown						183				
			1	SS	22				o		
						∇					
			2	SS	21		182				
181.58 / 2.13	CLAYEY SILT, some sand, trace gravel Hard Grey										
			3	SS	31				o		
180.81							181				
2.90	END OF BOREHOLE Groundwater encountered at about elev. 182.2m during drilling on December 21, 2009.										

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

O 3% STRAIN AT FAILURE

Depth (m)	Elevation (m)	Undrained Shear Strength (kPa) Natural	Post-Peak	Remoulded	Sensitivity
14.8	169.8	47	38	17	2.8
15.8	168.8	59	38	43	1.3
16.8	167.8	43	15	21	2.1
17.8	166.8	45	21	21	2.2
18.8	165.8	53	13	15	3.5
19.9	164.7	60	40	32	1.9
21.0	163.6	43	25		
Field Vane Location 119 (Borehole BH-119)					
5.6	176.9	119	83	59	2.0
6.6	175.9	115	91	40	2.9
9.6	172.9	64	53	32	2.0
10.6	171.9	60	47	30	2.0
11.6	170.9	43	23	31	1.4
12.6	169.9	45	32	30	1.5
13.6	168.9	51	38	25	2.1
14.6	167.9	43		16	2.8
15.6	166.9	51	28	21	2.5
16.6	165.9	51	30	15	3.4
17.7	164.9	45		26	1.7
18.7	163.9	42	26	23	1.8
19.7	162.9	45	42	26	1.7
20.7	161.9	76		49	1.5
Field Vane Location 122 (Borehole BH-122)					
5.8	175.9	117	74	53	2.2
6.8	174.9	79	51	40	2.0
7.8	173.9	62	42	26	2.4
8.8	172.9	59	32	26	2.2
9.8	171.9	51	34	21	2.5
10.8	170.9	59	26	23	2.6
11.8	169.9	53	32	26	2.0
12.8	168.9	45	19	9	4.8
13.8	167.9	43	23	13	3.3
14.8	166.9	43	28	19	2.3
15.8	165.9	45	36	26	1.7
16.8	164.9	83	68	47	1.8
17.8	163.9	70	62	59	1.2
18.8	162.9	59	38	47	1.2
Field Vane Location 129 (Borehole BH-129)					
4.7	176.1	117	55		
5.7	175.1	93	62	40	2.3
6.7	174.1	95	32	42	2.3
7.7	173.1	70	43	8	9.3
8.7	172.1	93	26	38	2.5
9.7	171.1	57	23	25	2.3
10.7	170.1	55	23	25	2.2
11.7	169.1	51	34	21	2.5

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-10

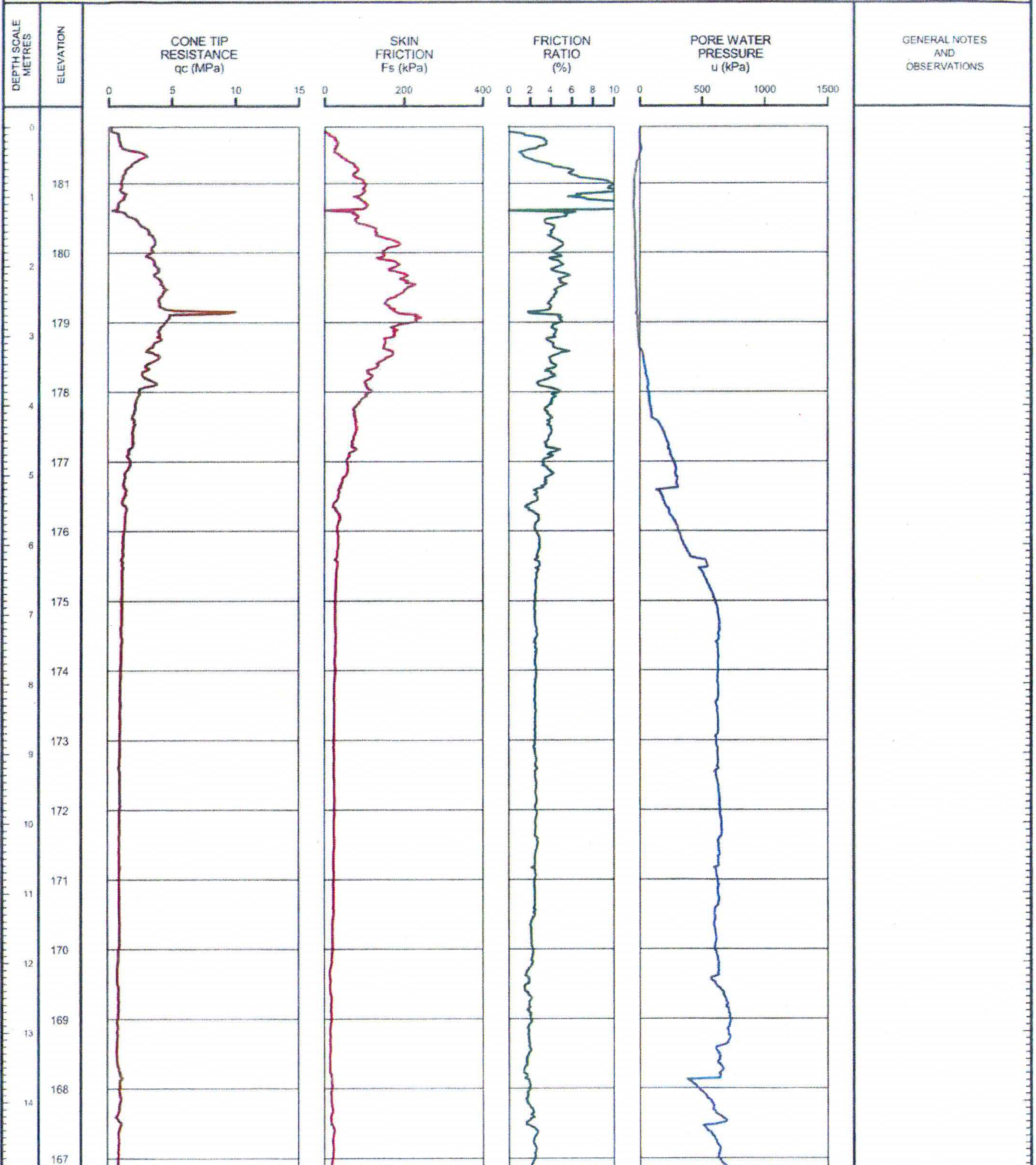
SHEET 1 OF 2

LOCATION: N 4679264.0 ;E 332533.0

TEST DATE: November 10, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



— CONTINUED NEXT PAGE —

DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

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

PROJECT: 07-1130-207-0

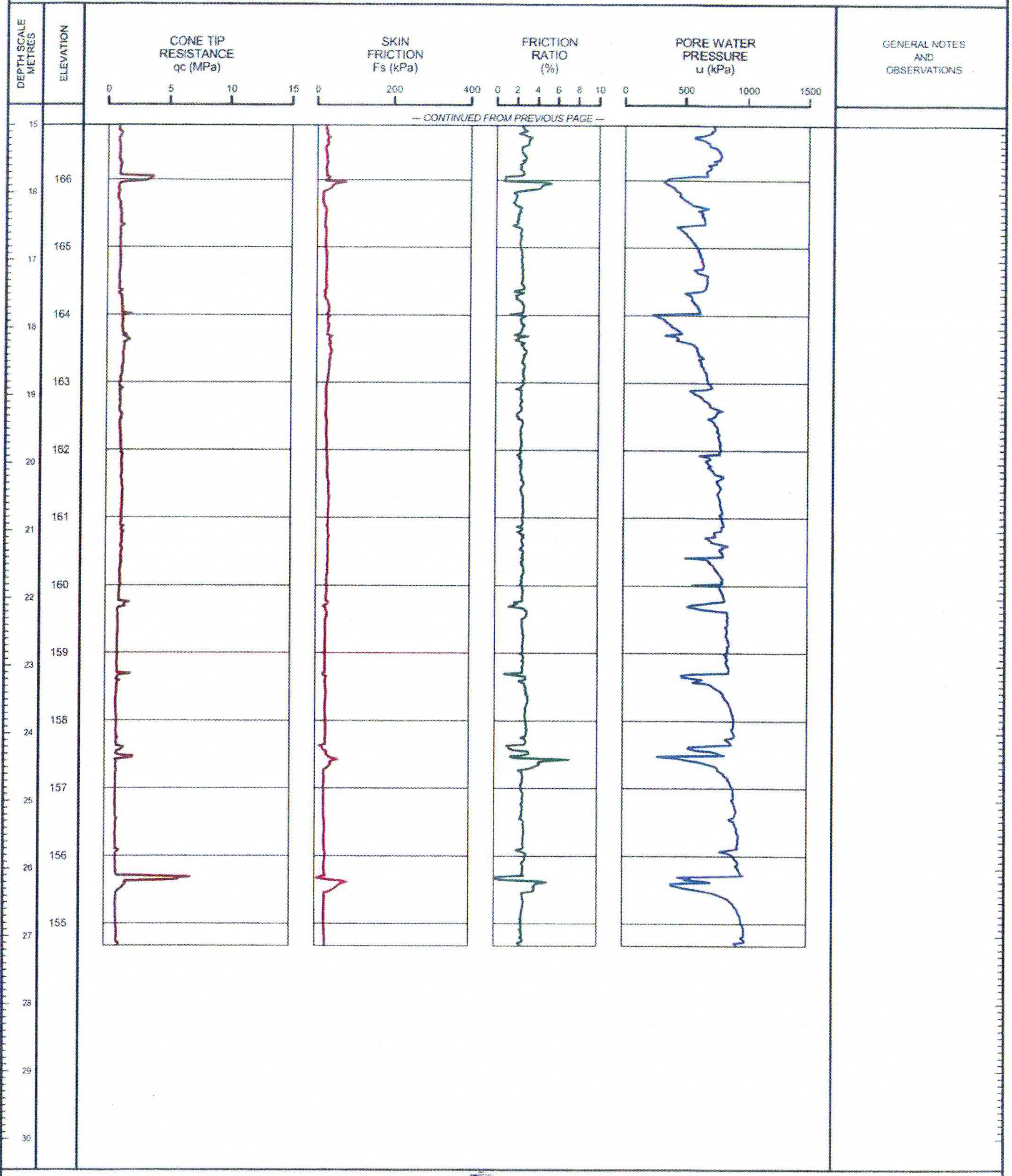
RECORD OF CONE PENETRATION TEST CPT-10

LOCATION: N 4679264.0 ;E 332533.0

TEST DATE: November 10, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-123

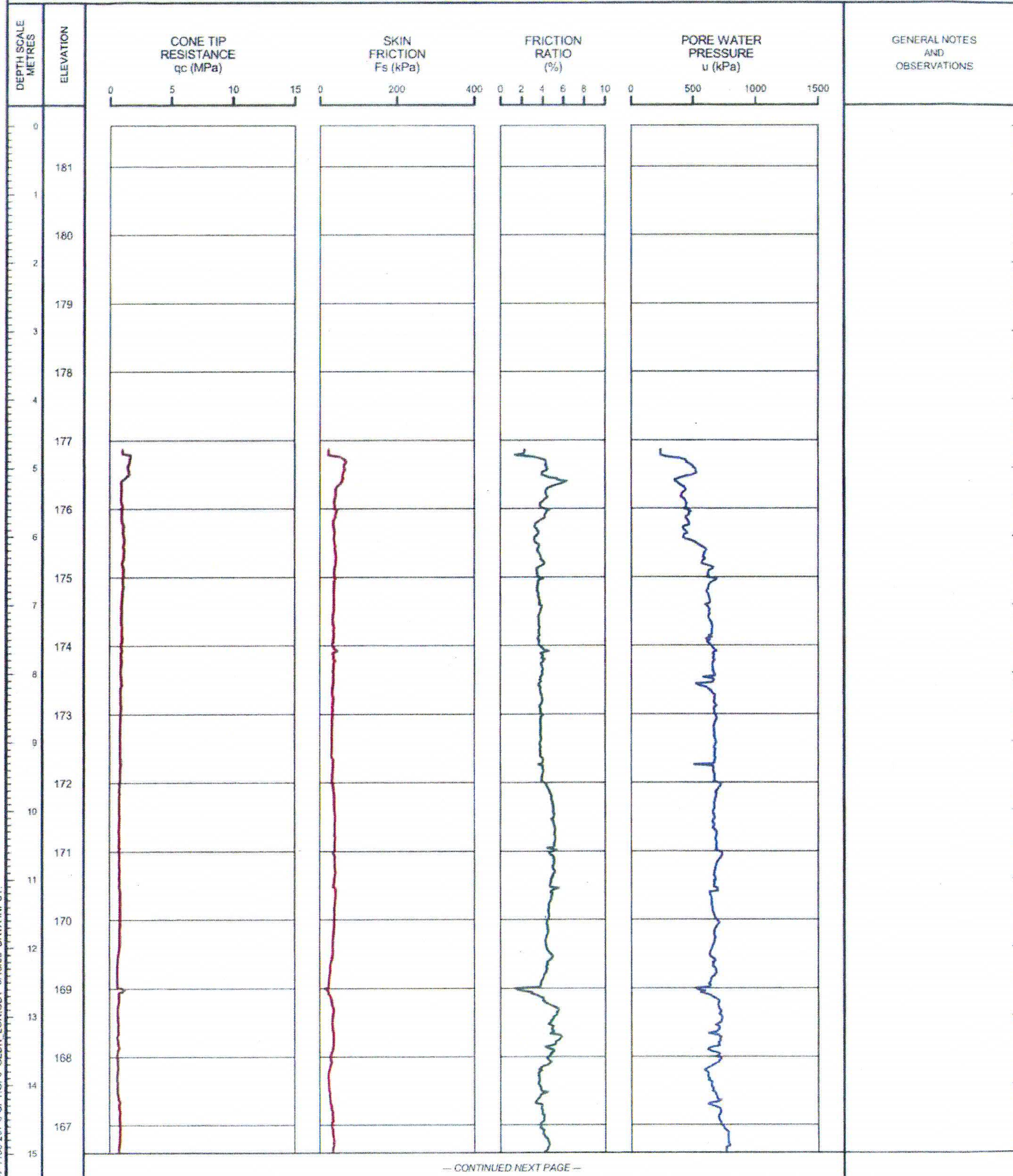
SHEET 1 OF 2

LOCATION: N 4679309.7 ;E 332536.3

TEST DATE: September 29, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION PREDRILL DEPTH: 4.75m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



— CONTINUED NEXT PAGE —

DEPTH SCALE
1 : 75

Golder Associates

OPERATOR: CC
CHECKED: SJB

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

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-123

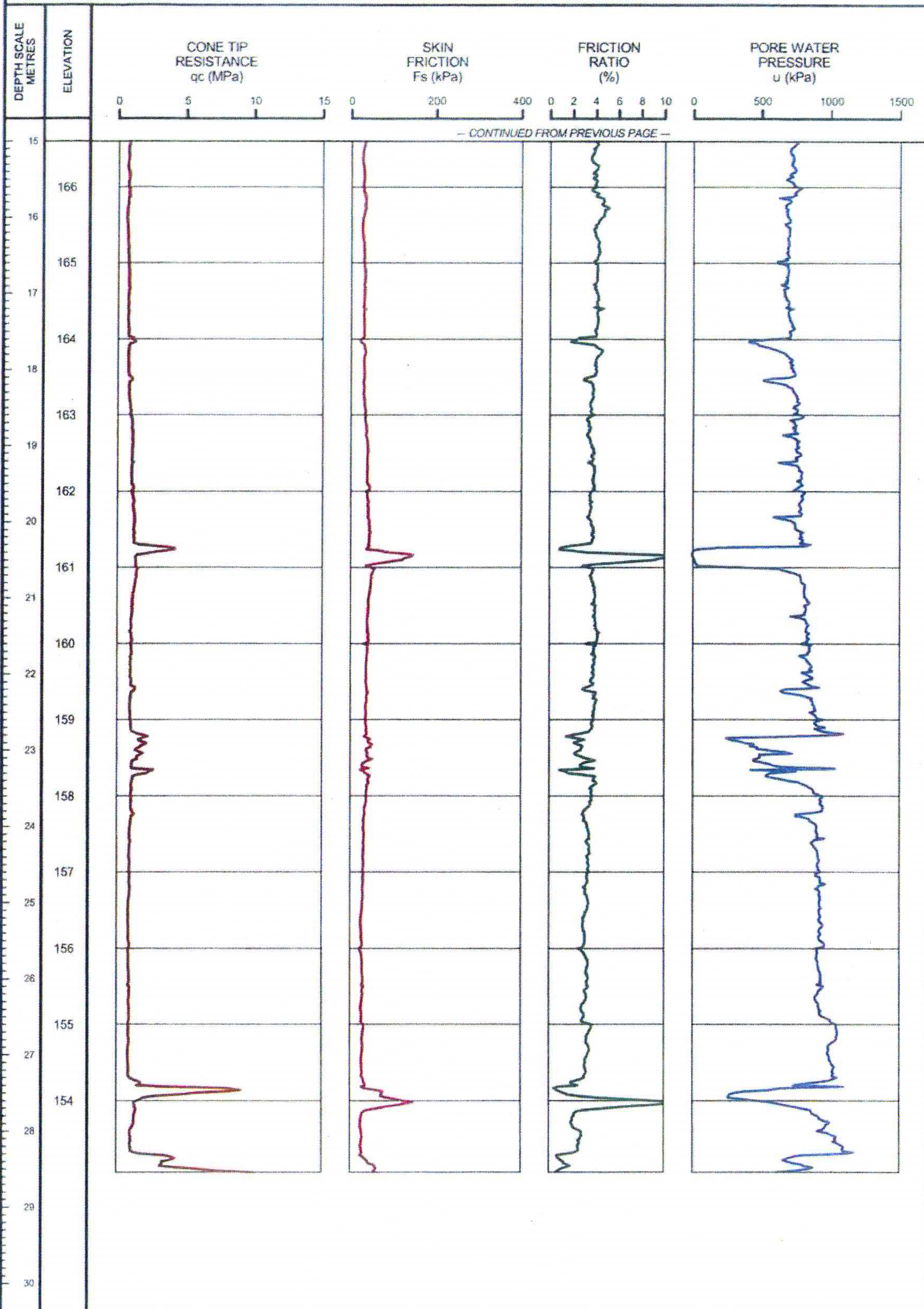
SHEET 2 OF 2

LOCATION: N 4679309.7 ;E 332536.3

TEST DATE: September 29, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 4.75m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



GENERAL NOTES AND OBSERVATIONS

DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

PROJECT: 09-1132-0080

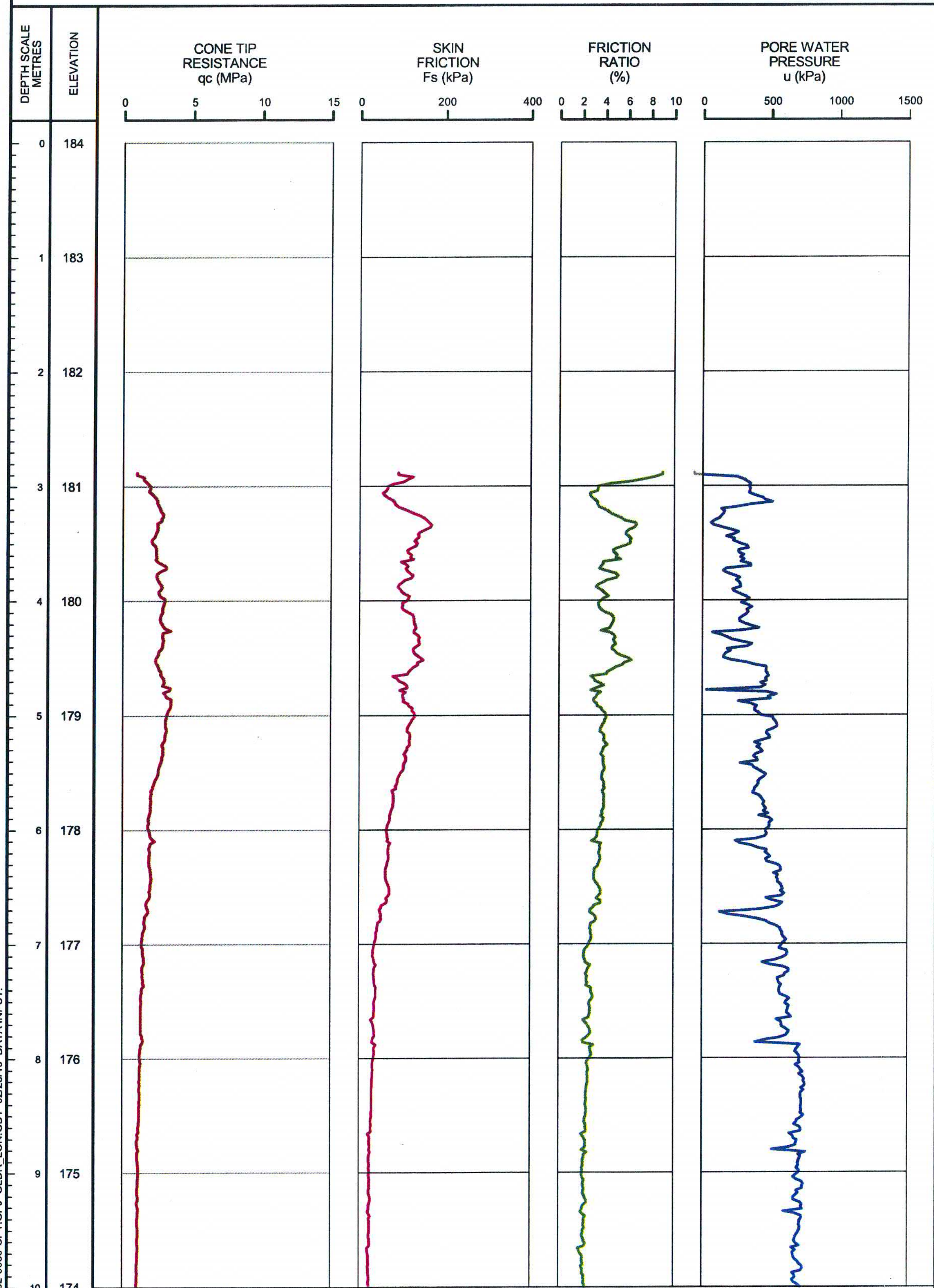
RECORD OF CONE PENETRATION TEST CPT-319

LOCATION: N 4679084.5 ;E 332701.0

TEST DATE: December 22, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 183.71m PREDRILL DEPTH: m CORRECTION FACTOR A: CORRECTION FACTOR B:



GENERAL NOTES AND OBSERVATIONS

CPT PUSHED AFTER PREDRILLING SAMPLED HOLE.
FOR SOIL PROFILE, SEE RECORD OF BOREHOLE No. CPT-319.

— CONTINUED NEXT PAGE —

DEPTH SCALE
1 : 50

Golder Associates

OPERATOR: TA
CHECKED:

PROJECT: 09-1132-0080

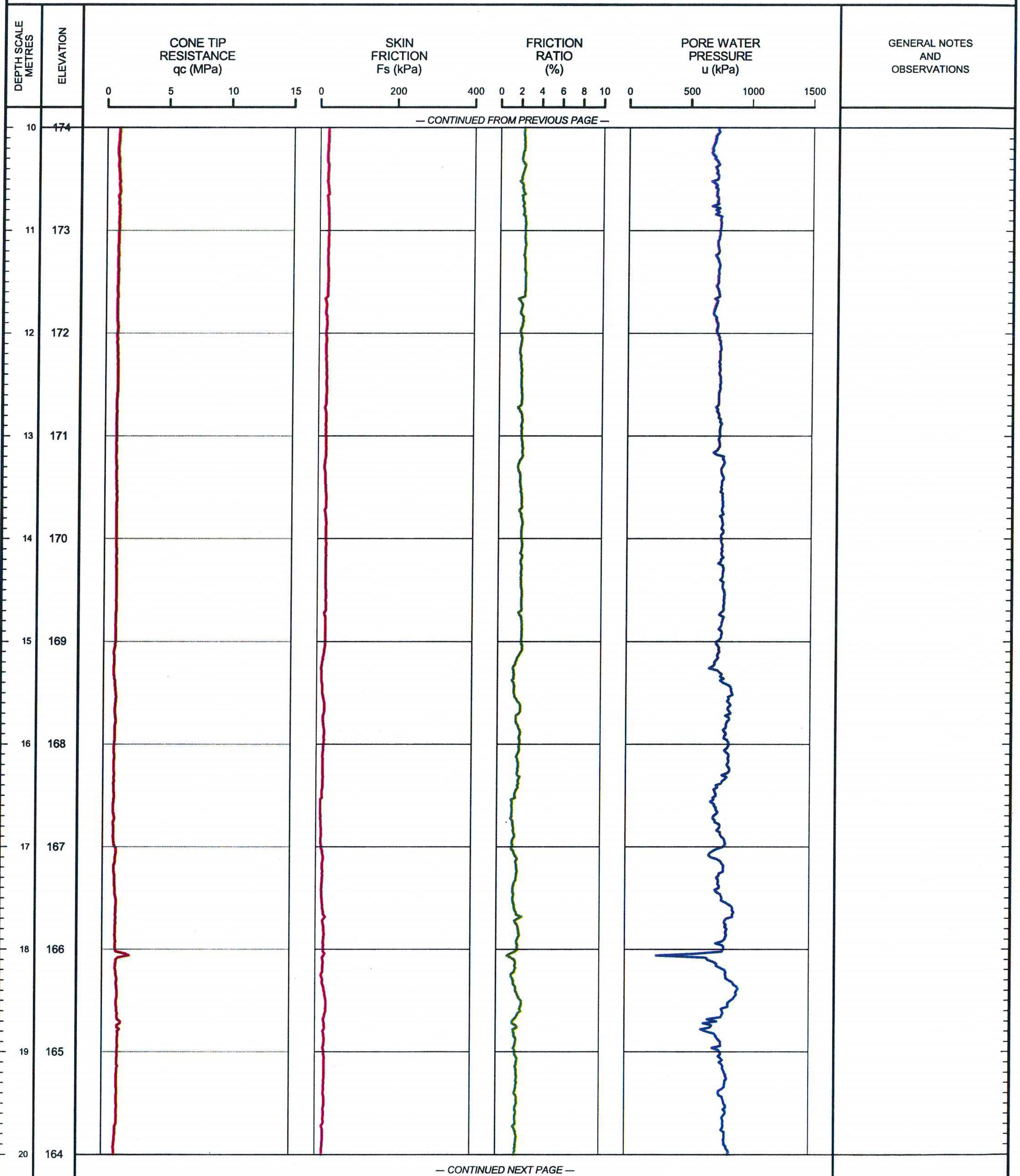
RECORD OF CONE PENETRATION TEST CPT-319

LOCATION: N 4679084.5 ;E 332701.0

TEST DATE: December 22, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 183.71m PREDRILL DEPTH: m CORRECTION FACTOR A: CORRECTION FACTOR B:



DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

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

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-319

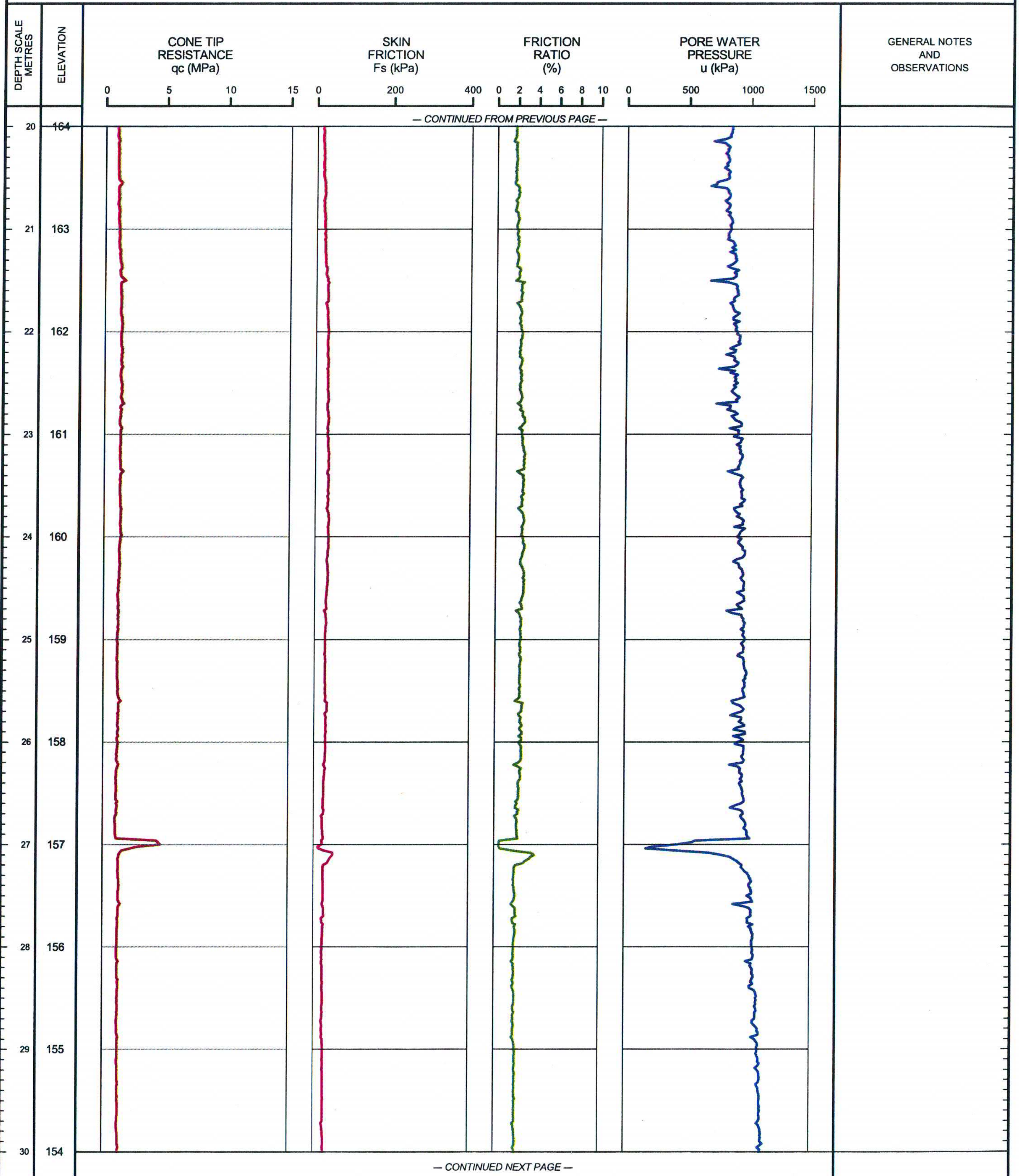
LOCATION: N 4679084.5 ;E 332701.0

TEST DATE: December 22, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 183.71m PREDRILL DEPTH: m CORRECTION FACTOR A: CORRECTION FACTOR B:

— CONTINUED FROM PREVIOUS PAGE —



— CONTINUED NEXT PAGE —

DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-319

LOCATION: N 4679084.5 ;E 332701.0

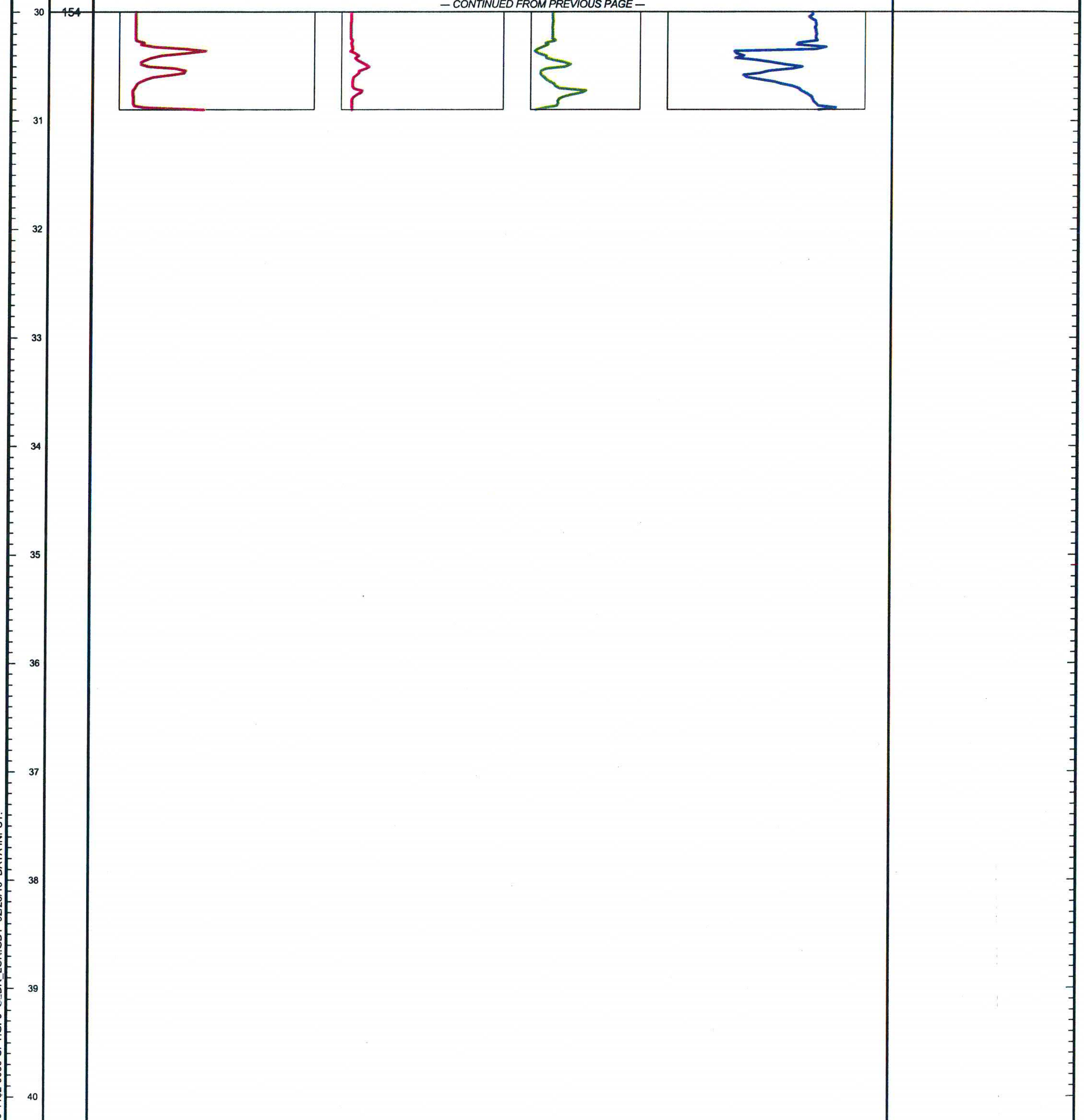
TEST DATE: December 22, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 183.71m PREDRILL DEPTH: m CORRECTION FACTOR A: CORRECTION FACTOR B:

DEPTH SCALE METRES	ELEVATION	CONE TIP RESISTANCE qc (MPa) 0 5 10 15	SKIN FRICTION Fs (kPa) 0 200 400	FRICTION RATIO (%) 0 2 4 6 8 10	PORE WATER PRESSURE u (kPa) 0 500 1000 1500	GENERAL NOTES AND OBSERVATIONS

— CONTINUED FROM PREVIOUS PAGE —



DEPTH SCALE

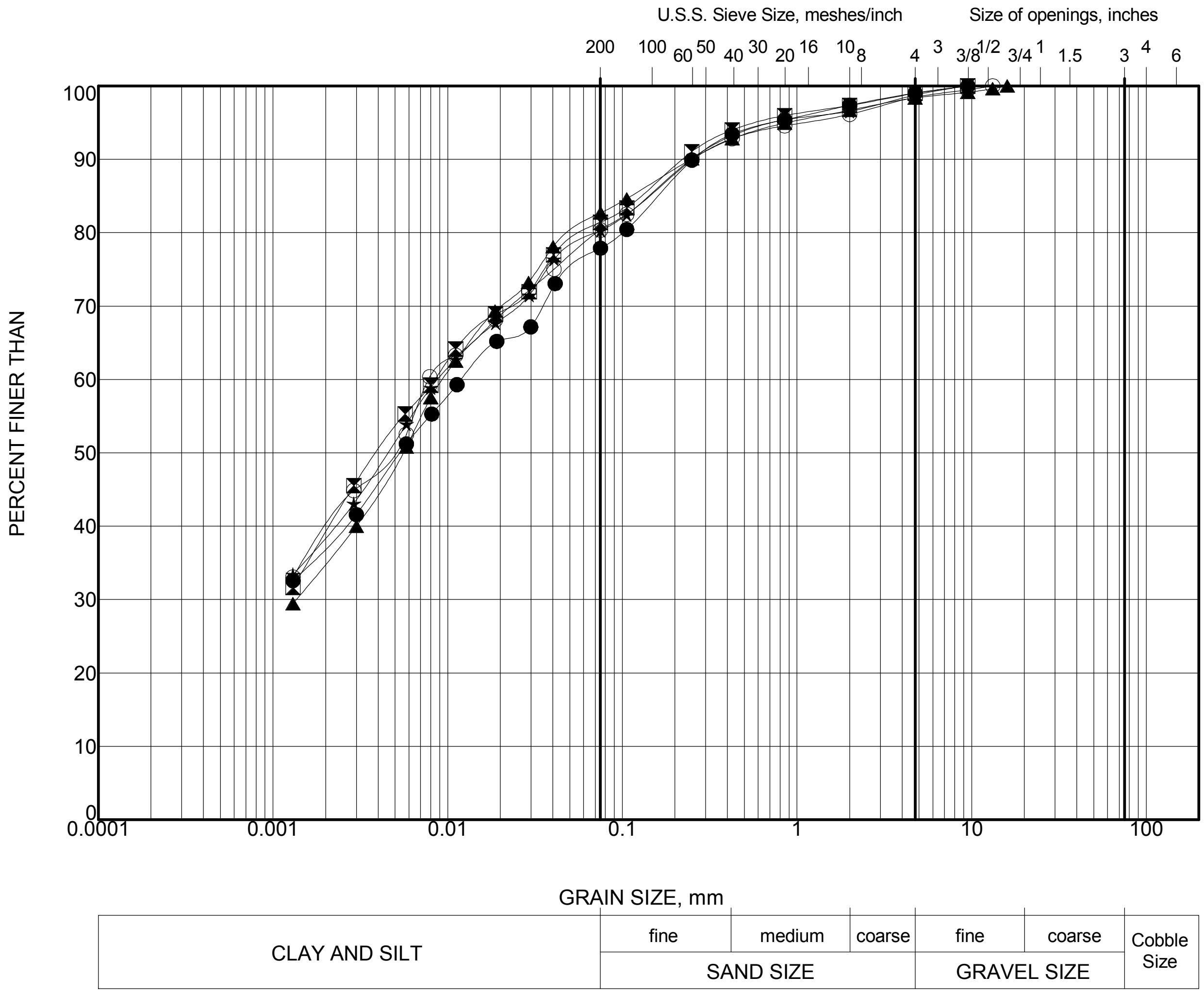
1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

Appendix C: Geotechnical Laboratory Test Results



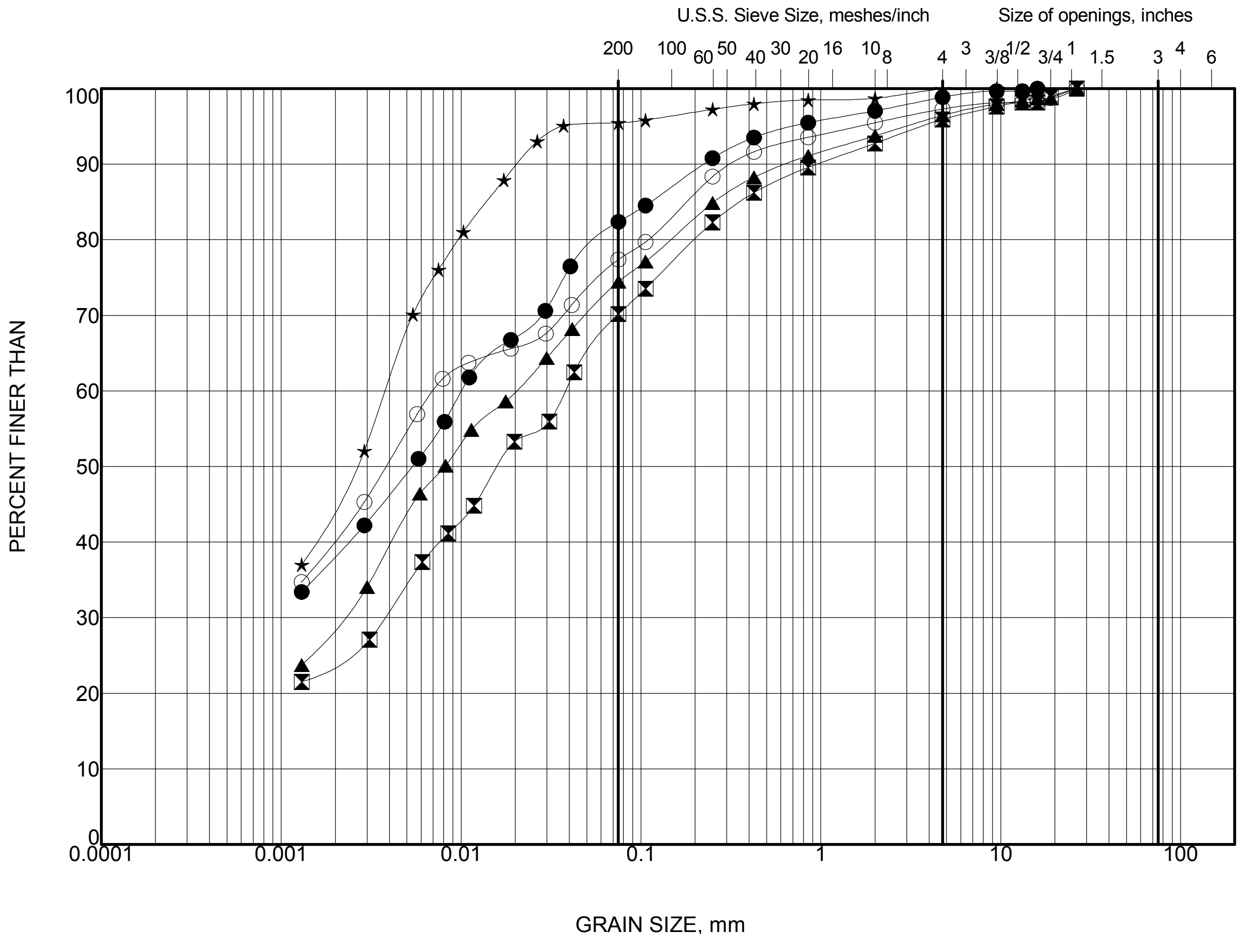
LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B9-1	8	6.1
⧗	B9-1	11	10.7
▲	B9-1	14	15.2
★	B9-2	6	4.6
○	B9-2	12	12.2

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario
TITLE	**GRAIN SIZE DISTRIBUTION** **Clayey Silt to Silty Clay**

Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PROJECT No. SW8801.1004.101	FILE No.
		SCALE / REV.
	DRAWN	
	CHECK	FIGURE **C-1**

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 14/03/12



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

LEGEND:

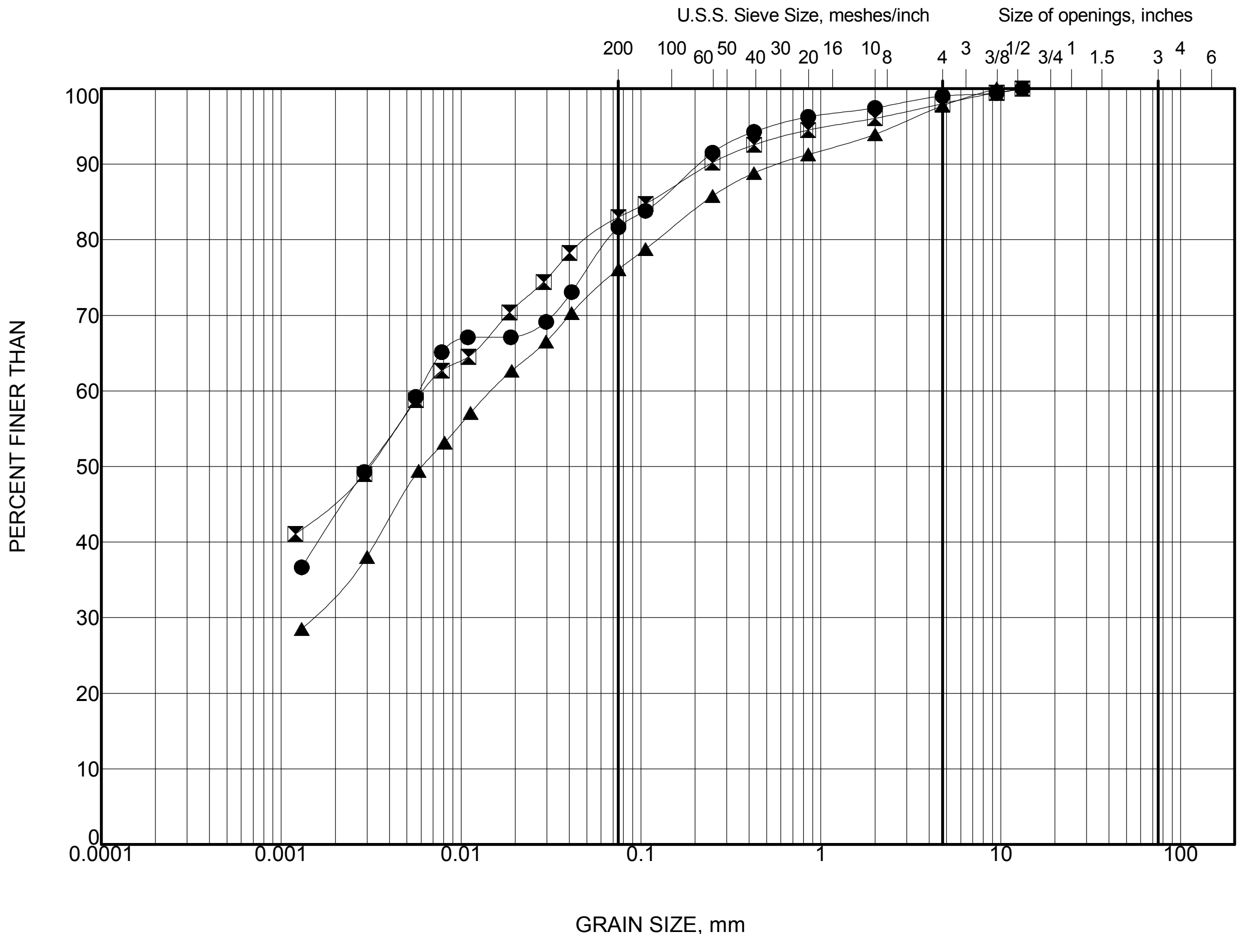
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B9-2	14	15.2
⧗	B9-2	17	19.8
▲	B9-2	19	22.9
★	B9-2	24	30.5
○	B9-3	8	6.1

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario
TITLE	**GRAIN SIZE DISTRIBUTION** **Clayey Silt to Silty Clay**

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101	FILE No.
	SCALE / REV.
DRAWN	
CHECK	FIGURE **C-2**

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 14/03/12



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

LEGEND:

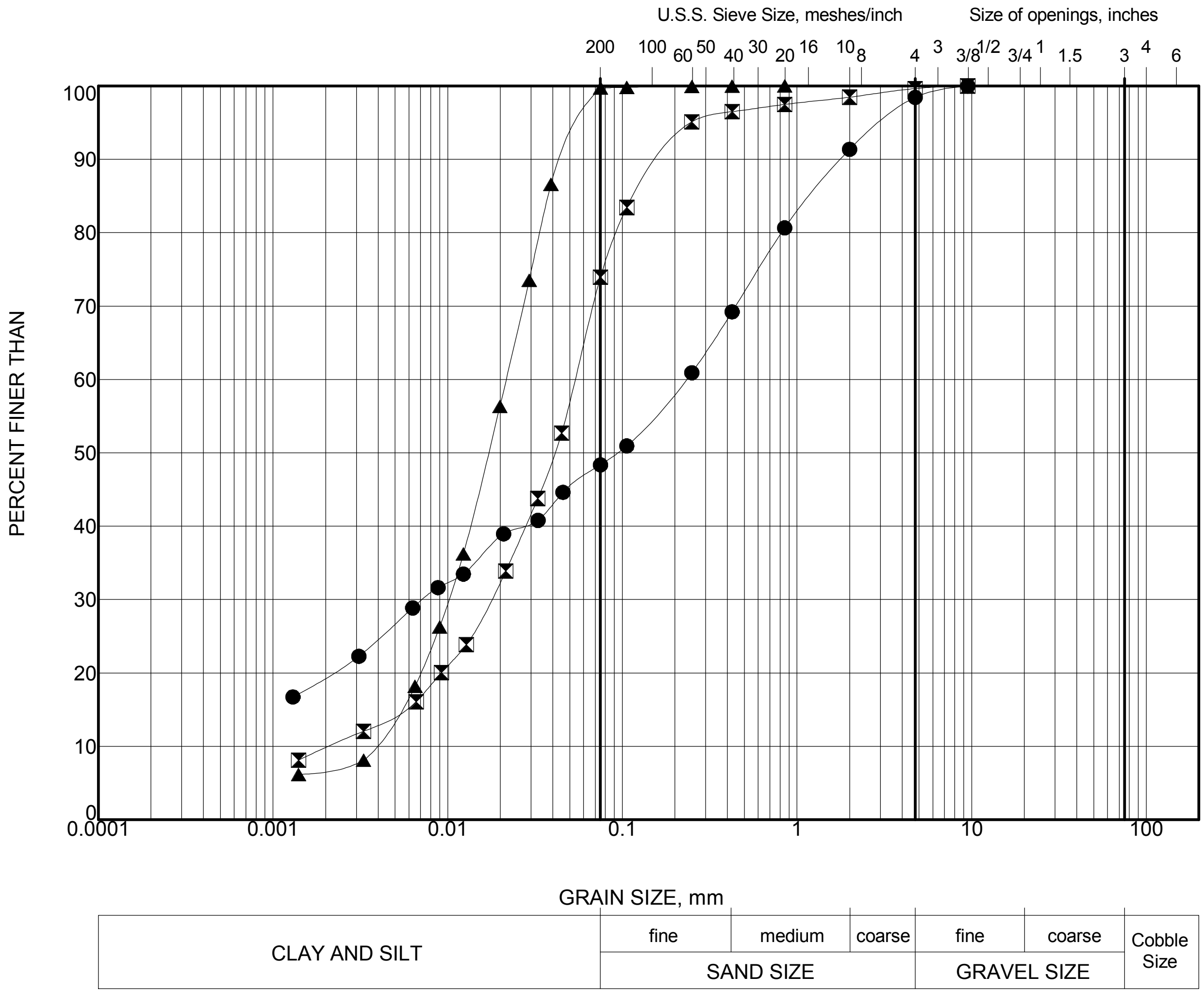
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B9-3	10	9.1
⧗	B9-3	15	16.8
▲	B9-3	22	27.4

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario
TITLE	**GRAIN SIZE DISTRIBUTION** **Clayey Silt to Silty Clay**

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101	FILE No.
	SCALE / REV.
DRAWN	
CHECK	FIGURE **C-3**

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 14/03/12



LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B9-1	18	21.3
⧗	B9-1	24	30.5
▲	B9-2	25	32

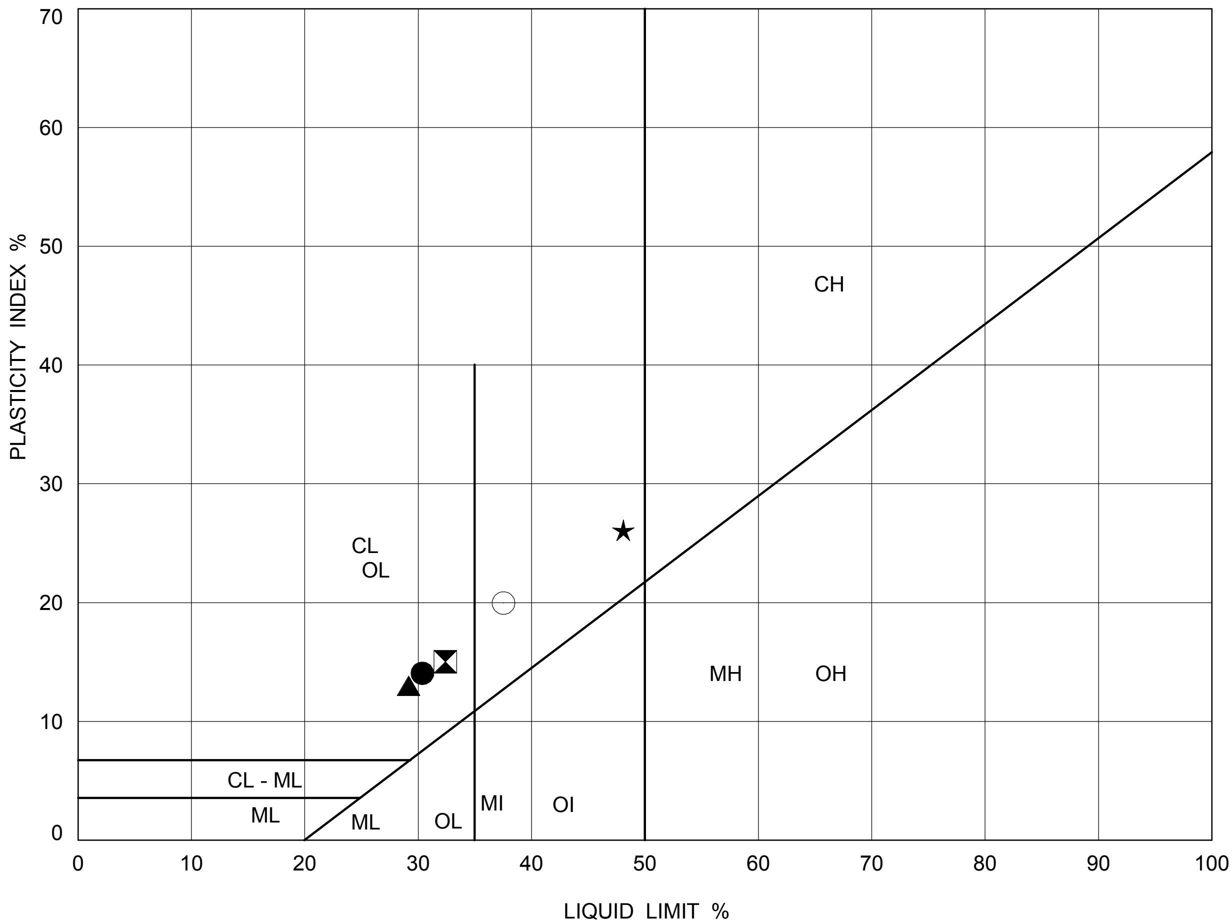
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION
Silty Sand to Silt

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101	FILE No.
	SCALE / REV.
DRAWN	
CHECK	FIGURE **C-4**

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 14/03/12



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B9-1	8	6.1	**30**	**16**	**14**
⧗	B9-1	11	10.7	**32**	**17**	**15**
▲	B9-1	14	15.2	**29**	**16**	**13**
★	B9-1	23	29	**48**	**22**	**26**
○	B9-2	5	3.8	**38**	**18**	**20**

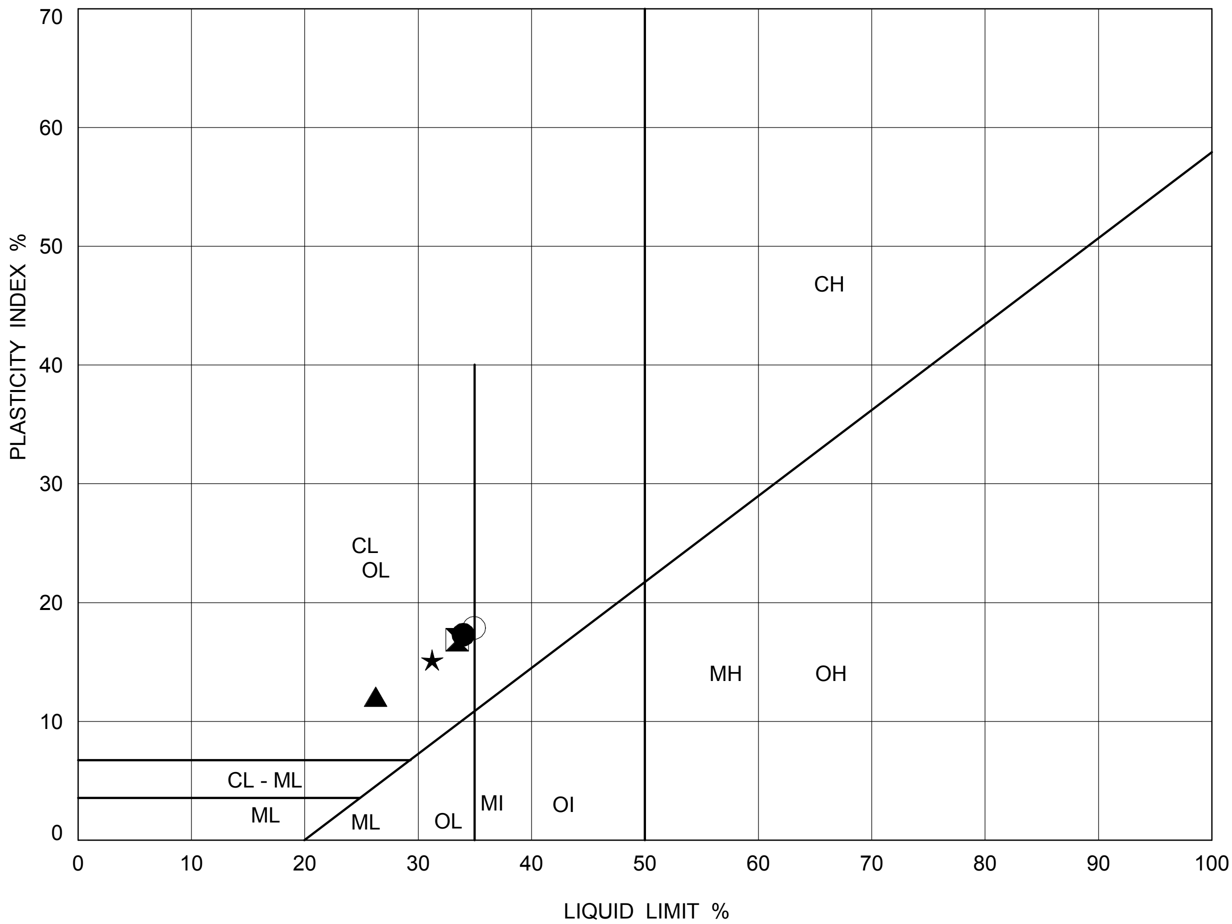
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
PLASTICITY CHART
Clayey Silt to Silty Clay

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101			FILE No.	
			SCALE	REV.
DRAWN			FIGURE **C-5**	
CHECK				

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 14/03/12



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B9-2	12	12.2	**34**	**17**	**17**
⧗	B9-2	14	15.2	**33**	**17**	**16**
▲	B9-2	17	19.8	**26**	**14**	**12**
★	B9-2	19	22.9	**31**	**16**	**15**
○	B9-3	8	6.1	**35**	**17**	**18**

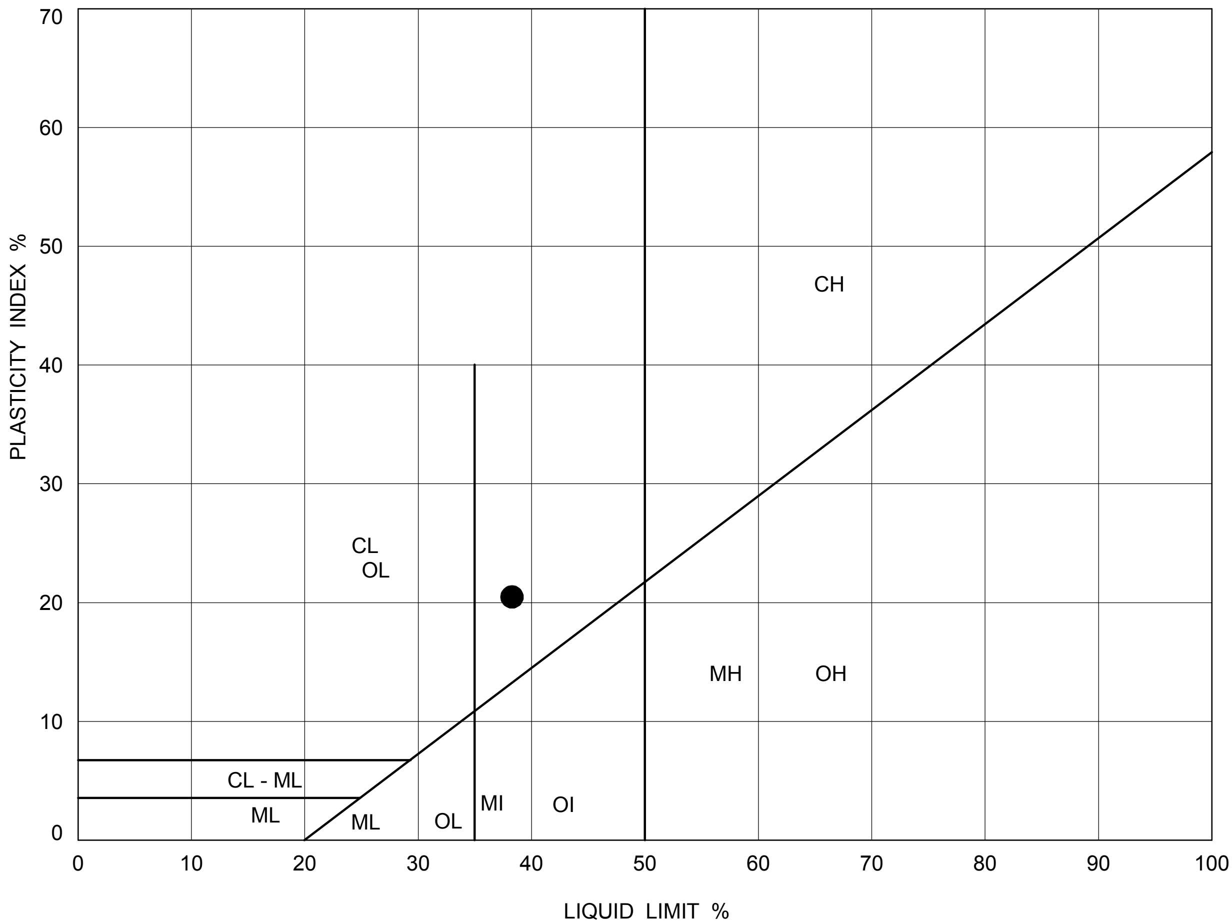
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
PLASTICITY CHART
Clayey Silt to Silty Clay

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101	FILE No.
	SCALE / REV.
DRAWN	
CHECK	FIGURE **C-6**

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 14/03/12



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B9-3	15	16.8	**38**	**18**	**20**

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 14/03/12

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario
TITLE	PLASTICITY CHART **Clayey Silt to Silty Clay**

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.
		SCALE / REV.
DRAWN		
CHECK		FIGURE **C-7**

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 21-Oct-11 **Sample ID:** B9-3_TW11 **Depth(m):** 10.7

Test Data

Ring # : B	Ring Height (in) = 0.756	Wt of dry filter paper (g)	0.69
Wet soil + Ring Wt (g)	202.20	Wt of ring (g)	76.54
Wet soil + Wet Paper + Ring (g)	201.33	Wet Paper (g)	2.13
Dry Soil + Dry Paper + Ring (g)	179.48	Ring Dia (in)	2.498
Initial moisture Content (%)	22.89	Final moisture Content (%)	19.96
Area of Ring (in^2)	4.90	Initial Volume (in^3)	3.7051
Initial Bulk Density (kg/m^3)	2070	Initial Dry Density (kg/m^3)	1684
Specific Gravity of Soil	2.74	Eqiv. Thick. of solids (mm)	11.811
Final Bulk Density (kg/m^3)	2126	Final Dry Density (kg/m^3)	1730
Initiall gauge reading for Load 1	0.2570	Gauge reading for last Loading	0.2195
Initial Voids Ratio	0.626	Final Void Ratio	0.545
Initial Degree of Saturation (%)	100	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	4.0	6.0	10.0	15.0	22.5	35.0	50.0
Load (tsf)	0.0416	0.0624	0.104	0.156	0.234	0.364	0.520
Gauge Reading (in)	0.2568	0.2567	0.2559	0.2542	0.2520	0.24862	0.2457
(H-Hs) mm	7.387	7.384	7.363	7.320	7.264	7.179	7.105
Voids ratio	0.625	0.625	0.623	0.620	0.615	0.608	0.602
t90 (min)			4.41	8.41	12.60	13.69	16.00
Cv (m^2/day)			0.025	0.013	0.009	0.008	0.007
k' (MPa)			3.642	2.260	2.568	2.777	3.867
Mv (mm^2 / N)			0.2746	0.4425	0.3894	0.3601	0.2586

Trial #	8	9	10	11	12	13	14
Load (kPa)	75	110.0	75.0	50.0	75.0	110.0	165.0
Load (tsf)	0.78	1.144	0.780	0.520	0.780	1.144	1.716
Gauge Reading (in)	0.2416	0.2368	0.2375	0.2382	0.2376	0.2365	0.2306
(H-Hs) mm	7.000	6.878	6.896	6.913	6.898	6.869	6.721
Voids ratio	0.593	0.582	0.584	0.585	0.584	0.582	0.569
t90 (min)	8.12	6.25					10.24
Cv (m^2/day)	0.013	0.017					0.010
k' (MPa)	4.519	5.400					6.938
Mv (mm^2 / N)	0.2213	0.1852					0.1441

Trial #	15	16	17	18	19	20	21
Load (kPa)	250.0	370.0	560.0	835.0	1250.0	625.0	315.0
Load (tsf)	2.6	3.848	5.824	8.684	13.000	6.500	3.276
Gauge Reading (in)	0.2214	0.2129	0.2009	0.1883	0.1746	0.1772	0.1814
(H-Hs) mm	6.487	6.271	5.966	5.646	5.298	5.364	5.471
Voids ratio	0.549	0.531	0.505	0.478	0.449	0.454	0.463
t90 (min)	6.25	7.84	6.25	8.41	7.56		
Cv (m^2/day)	0.011	0.013	0.016	0.011	0.012		
k' (MPa)	0.295	10.146	11.290	15.276	20.820		
Mv (mm^2 / N)	3.3871	0.0986	0.0886	0.0655	0.0480		

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

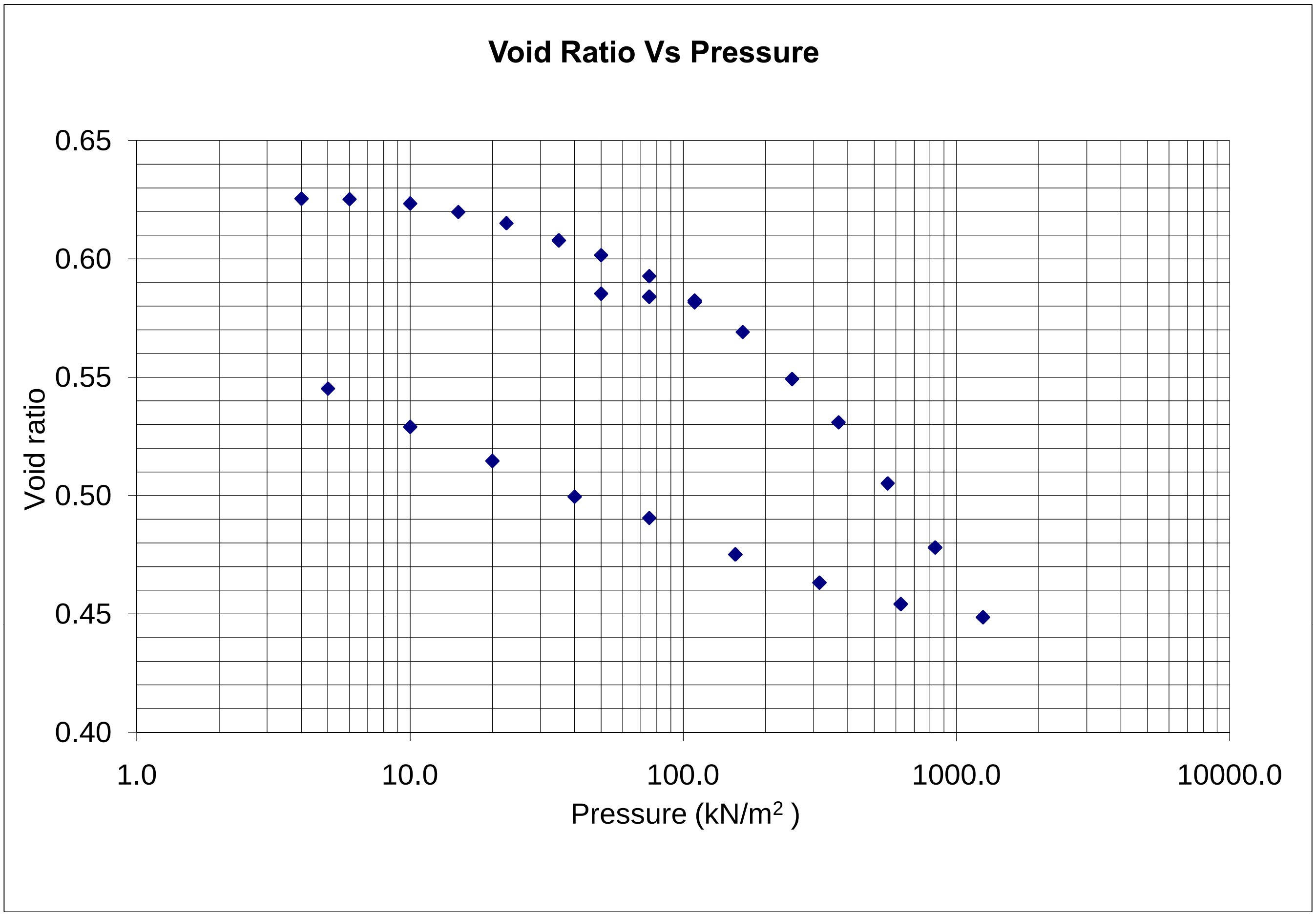
Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 21-Oct-11 **Sample ID:** B9-3_TW11 **Depth(m):** 10.7

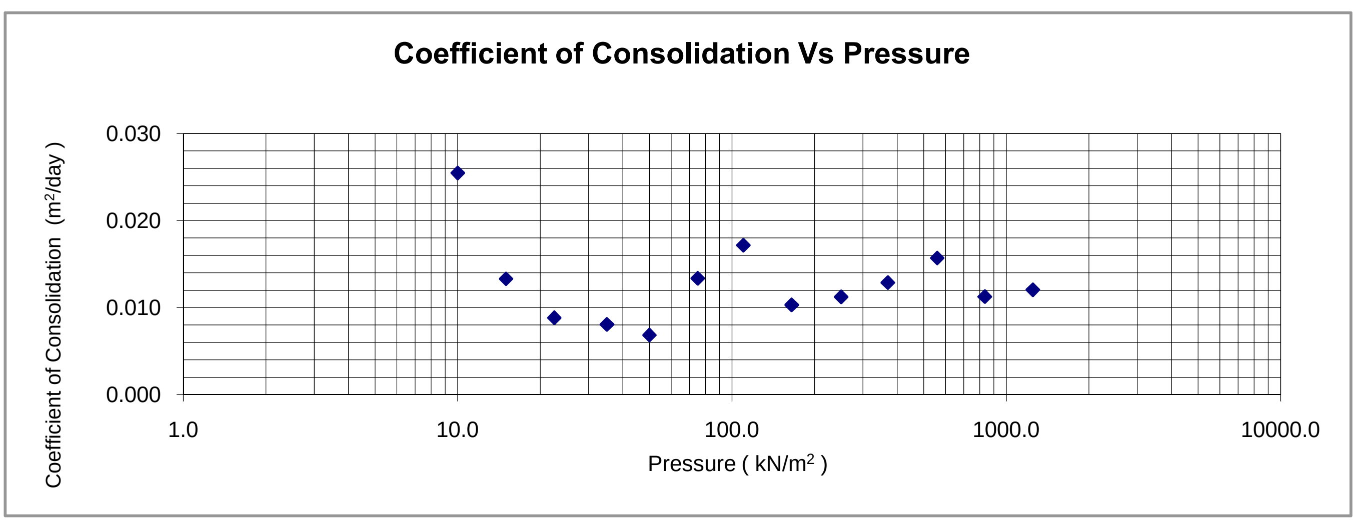
Trial #	22	23	24	25	26	27	
Load (kPa)	155	75.0	40.0	20.0	10.0	5.0	
Load (tsf)	1.612	0.780	0.416	0.208	0.104	0.052	
Gauge Reading (in)	0.18694	0.1941	0.1983	0.2053	0.2120	0.2195	
(H-Hs) mm	5.612	5.794	5.900	6.078	6.248	6.439	
Voids ratio	0.475	0.491	0.500	0.515	0.529	0.545	
t90 (min)							
Cv (m^2/day)							
k' (MPa)							
Mv (mm^2 / N)							

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 21-Oct-11 **Sample ID:** B9-3_TW11 **Depth(m):** 10.7

σ'_v versus e and c_v





ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 21-Oct-11 **Sample ID:** B9-3_TW11 **Depth(m):** 10.7

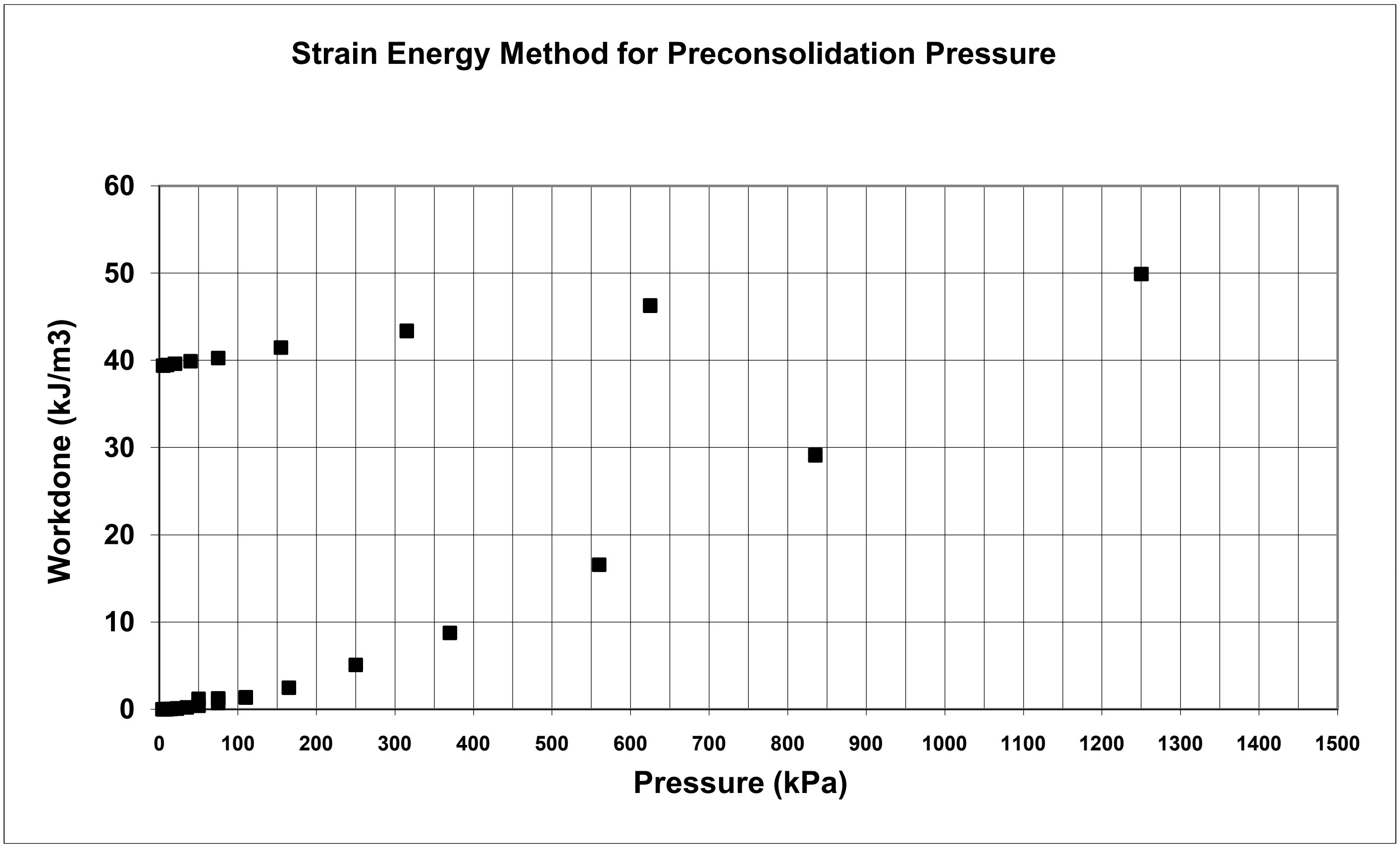
Strain Energy Data

Presssure (kN/m^2)	c_v (m^2/day)	Void ratio
4.0		0.625
6.0		0.625
10.0	0.025	0.623
15.0	0.013	0.620
22.5	0.009	0.615
35.0	0.008	0.608
50.0	0.007	0.602
75.0	0.013	0.593
110.0	0.017	0.582
75.0		0.584
50.0		0.585
75.0		0.584
110.0		0.582
165.0	0.010	0.569
250.0	0.011	0.549
370.0	0.013	0.531
560.0	0.016	0.505
835.0	0.011	0.478
1250.0	0.012	0.449
625.0		0.454
315.0		0.463
155.0		0.475
75.0		0.491
40.0		0.500
20.0		0.515
10.0		0.529
5.0		0.545

Presssure (KN/m^2)	Height mm	Total Work (KJ/m^3)
4.0	19.202	0.000
6.0	19.199	0.001
10.0	19.178	0.010
15.0	19.136	0.037
22.5	19.080	0.092
35.0	18.994	0.221
50.0	18.920	0.386
75.0	18.816	0.732
110.0	18.694	1.331
75.0	18.712	1.242
50.0	18.728	1.187
75.0	18.714	1.235
110.0	18.685	1.378
165.0	18.537	2.468
250.0	18.303	5.089
370.0	18.086	8.755
560.0	17.782	16.578
835.0	17.462	29.132
1250.0	17.114	49.907
625.0	17.180	46.289
315.0	17.286	43.377
155.0	17.427	41.461
75.0	17.609	40.261
40.0	17.715	39.914
20.0	17.894	39.612
10.0	18.064	39.470
5.0	18.255	39.390

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project:	WEP			**Job No.:**	SW8801.1004.101
Client:	Hatch Mott MacDonald Limited				
Date:	21-Oct-11	**Sample ID:**	B9-3_TW11	**Depth(m):**	10.7



ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 28-Oct-11 **Sample ID:** B9-3_TW14 **Depth(m):** 15.3 to 15.9

Test Data

Ring # :	B	Ring Height (in) =	0.755	Wt of dry filter paper (g)	0.64
Wet soil + Ring Wt (g)			195.14	Wt of ring (g)	76.53
Wet soil + Wet Paper + Ring (g)			193.02	Wet Paper (g)	1.93
Dry Soil + Dry Paper + Ring (g)			167.40	Ring Dia (in)	2.498
Initial moisture Content (%)			31.45	Final moisture Content (%)	26.96
Area of Ring (in^2)			4.90	Initial Volume (in^3)	3.7002
Initial Bulk Density (kg/m^3)			1956	Initial Dry Density (kg/m^3)	1488
Specific Gravity of Soil			2.79	Eqiv. Thick. of solids (mm)	10.228
Final Bulk Density (kg/m^3)			2025	Final Dry Density (kg/m^3)	1595
Initiall gauge reading for Load 1			0.2568	Gauge reading for last Loading	0.2063
Initial Voids Ratio			0.875	Final Void Ratio	0.750
Initial Degree of Saturation (%)			100	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	6.0	9.0	13.0	20.0	30.0	45.0	65.0
Load (tsf)	0.0624	0.0936	0.135	0.208	0.312	0.468	0.676
Gauge Reading (in)	0.2565	0.2561	0.2558	0.2542	0.2526	0.24931	0.2460
(H-Hs) mm	8.941	8.931	8.923	8.881	8.842	8.758	8.675
Voids ratio	0.874	0.873	0.872	0.868	0.864	0.856	0.848
t90 (min)			4.41	8.41	12.60	13.69	8.70
Cv (m^2/day)			0.025	0.013	0.009	0.008	0.013
k' (MPa)			10.057	3.199	4.823	3.433	4.530
Mv (mm^2 / N)			0.0994	0.3126	0.2074	0.2912	0.2207

Trial #	8	9	10	11	12	13	14
Load (kPa)	100	150.0	100.0	65.0	45.0	65.0	100.0
Load (tsf)	1.04	1.560	1.040	0.676	0.468	0.676	1.040
Gauge Reading (in)	0.24161	0.2363	0.2376	0.2395	0.2417	0.2409	0.2387
(H-Hs) mm	8.563	8.429	8.462	8.510	8.565	8.545	8.489
Voids ratio	0.837	0.824	0.827	0.832	0.837	0.835	0.830
t90 (min)	8.12	12.25					
Cv (m^2/day)	0.013	0.009					
k' (MPa)	5.920	7.006					
Mv (mm^2 / N)	0.1689	0.1427					

Trial #	15	16	17	18	19	20	21
Load (kPa)	150.0	225.0	335.0	505.0	760.0	1140.0	1710.0
Load (tsf)	1.56	2.340	3.484	5.252	7.904	11.856	17.784
Gauge Reading (in)	0.23571	0.2295	0.2195	0.2021	0.1793	0.1566	0.1350
(H-Hs) mm	8.413	8.256	8.002	7.559	6.981	6.404	5.854
Voids ratio	0.823	0.807	0.782	0.739	0.682	0.626	0.572
t90 (min)		12.25	9.00	10.24	12.25	9.92	8.70
Cv (m^2/day)		0.009	0.011	0.010	0.008	0.009	0.009
k' (MPa)		8.878	8.021	6.988	7.849	11.332	17.240
Mv (mm^2 / N)		0.1126	0.1247	0.1431	0.1274	0.0882	0.0580

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 28-Oct-11 **Sample ID:** B9-3_TW14 **Depth(m):** 15.3 to 15.9

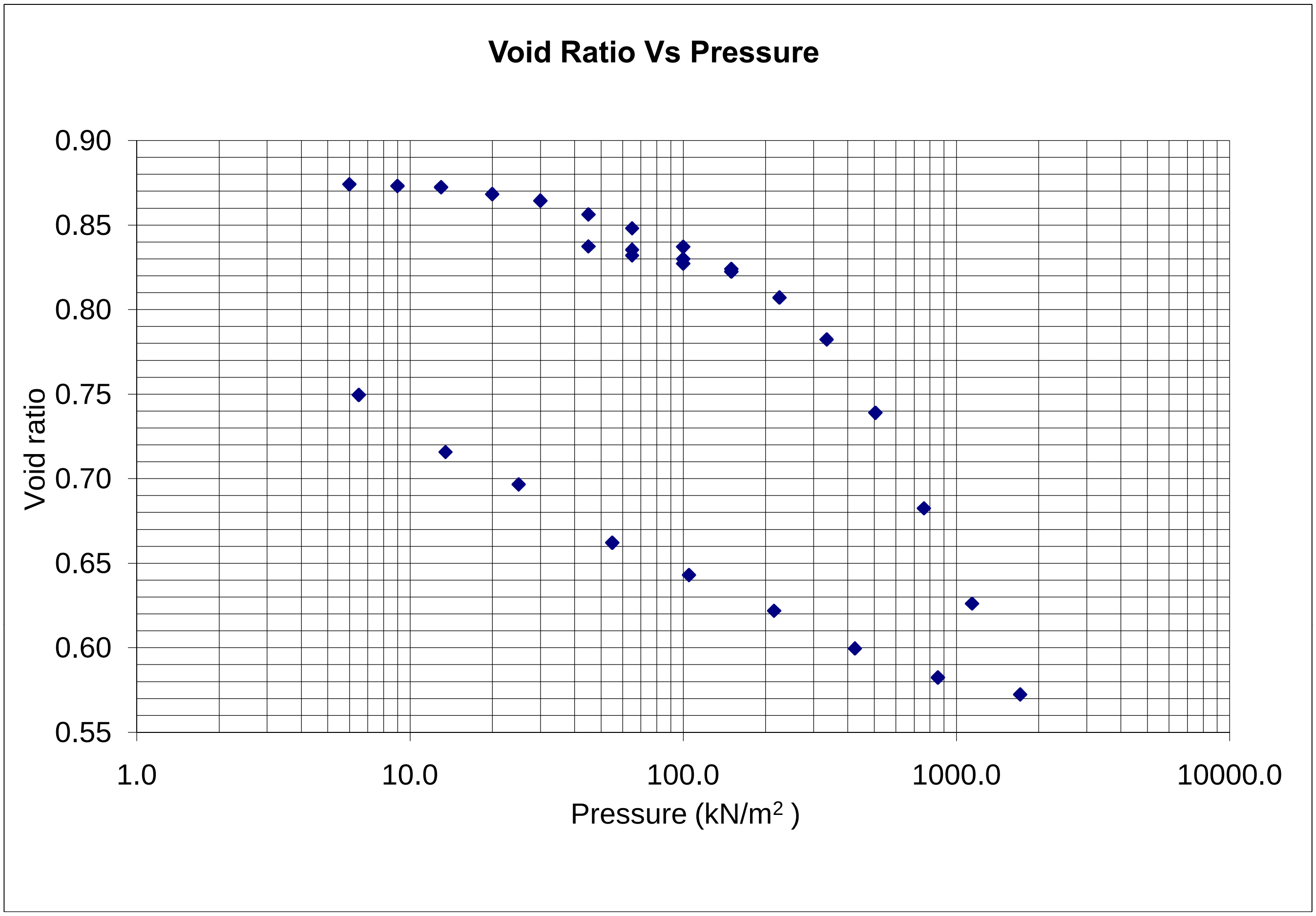
Trial #	22	23	24	25	26	27	28
Load (kPa)	855.0	425.0	215	105.0	55.0	25.0	13.5
Load (tsf)	8.892	4.420	2.236	1.092	0.572	0.260	0.140
Gauge Reading (in)	0.139	0.1459	0.1549	0.1634	0.1711	0.1850	0.1927
(H-Hs) mm	5.957	6.132	6.360	6.577	6.772	7.125	7.321
Voids ratio	0.582	0.599	0.622	0.643	0.662	0.697	0.716
t90 (min)							
Cv (m^2/day)							
k' (MPa)							
Mv (mm^2 / N)							

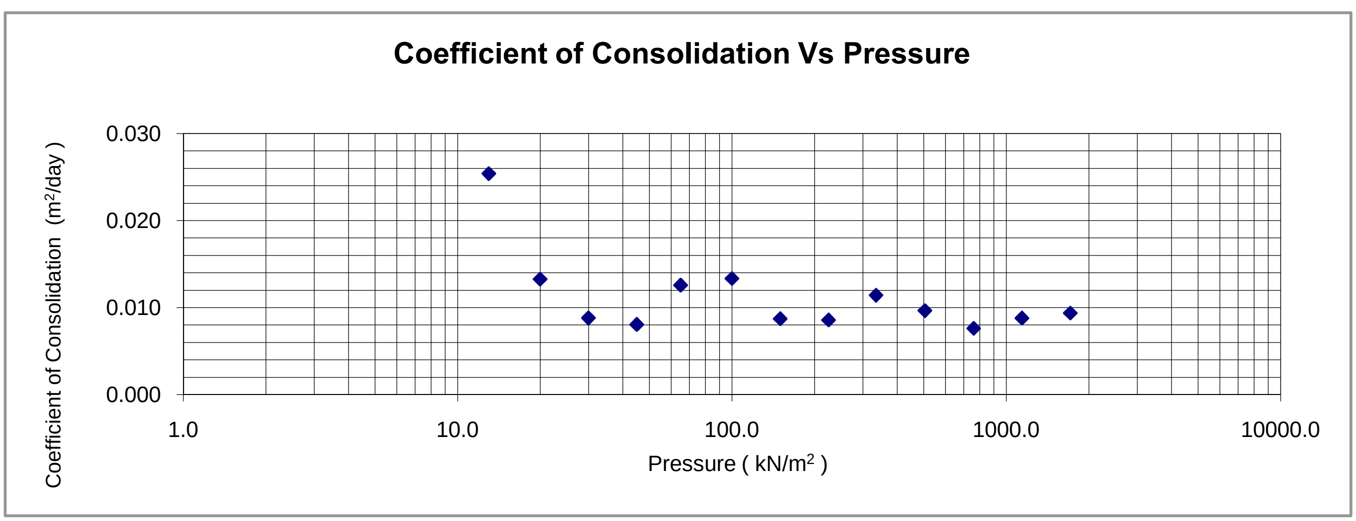
Trial #	29
Load (kPa)	6.5
Load (tsf)	0.068
Gauge Reading (in)	0.2063
(H-Hs) mm	7.666
Voids ratio	0.750
t90 (min)	
Cv (m^2/day)	
k' (MPa)	
Mv (mm^2 / N)	

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 28-Oct-11 **Sample ID:** B9-3_TW14 **Depth(m):** 15.3 to 15.9

σ'_v versus e and c_v





ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 28-Oct-11 **Sample ID:** B9-3_TW14 **Depth(m):** 15.3 to 15.9

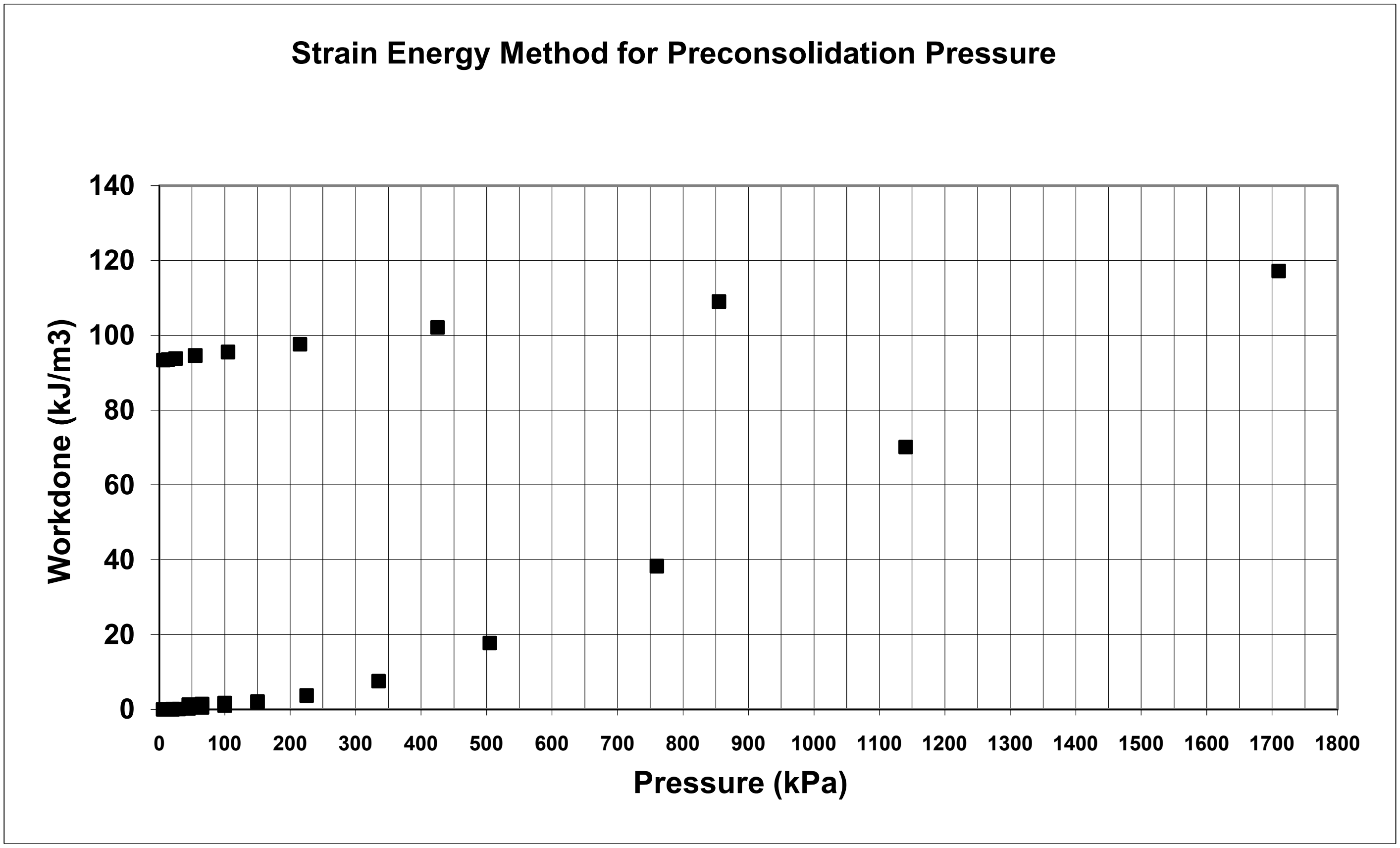
Strain Energy Data

Presssure (kN/m^2)	c_v (m^2/day)	Void ratio
6.0		0.874
9.0		0.873
13.0	0.025	0.872
20.0	0.013	0.868
30.0	0.009	0.864
45.0	0.008	0.856
65.0	0.013	0.848
100.0	0.013	0.837
150.0	0.009	0.824
100.0		0.827
65.0		0.832
45.0		0.837
65.0		0.835
100.0		0.830
150.0		0.823
225.0	0.009	0.807
335.0	0.011	0.782
505.0	0.010	0.739
760.0	0.008	0.682
1140.0	0.009	0.626
1710.0	0.009	0.572
855.0		0.582
425.0		0.599
215.0		0.622
105.0		0.643
55.0		0.662
25.0		0.697
13.5		0.716
6.5		0.750

Presssure (KN/m^2)	Height mm	Total Work (KJ/m^3)
6.0	19.177	0.000
9.0	19.167	0.004
13.0	19.159	0.008
20.0	19.117	0.044
30.0	19.078	0.096
45.0	18.994	0.260
65.0	18.911	0.503
100.0	18.799	0.990
150.0	18.665	1.882
100.0	18.697	1.663
65.0	18.746	1.449
45.0	18.801	1.286
65.0	18.781	1.346
100.0	18.725	1.590
150.0	18.649	2.100
225.0	18.491	3.683
335.0	18.238	7.522
505.0	17.794	17.735
760.0	17.217	38.274
1140.0	16.640	70.117
1710.0	16.090	117.211
855.0	16.193	109.012
425.0	16.368	102.085
215.0	16.596	97.615
105.0	16.813	95.531
55.0	17.008	94.602
25.0	17.361	93.772
13.5	17.556	93.555
6.5	17.902	93.358

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 28-Oct-11 **Sample ID:** B9-3_TW14 **Depth(m):** 15.3 to 15.9



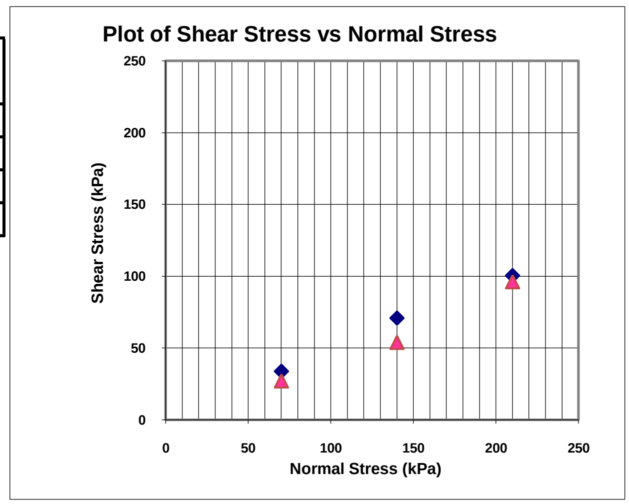
DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**19 October 2011**
Sample ID.:	**B9-1_TW13**	**Tested By:**	**CZ/SB**
Lab No.:	**AdS077_2011**	**Checked By:**	**SB**

Specimen ID	**1**	**2**	**3**
Date of Test	**19-Oct-11**	**20-Oct-11**	**21-Oct-11**
Normal Stress (kPa)	**70**	**140**	**210**
Rate of displacement (mm/min)	0.02	0.02	0.03
Initial thickness of specimen (mm)	24.10	24.10	24.10
Initial diameter of specimen (mm)	63.30	63.30	63.30
Initial moisture content (%)	42.1	33.2	36.7
Density (kN/m^3)	18.4	19.3	18.8
Final moisture (%)	38.3	27.2	29.2

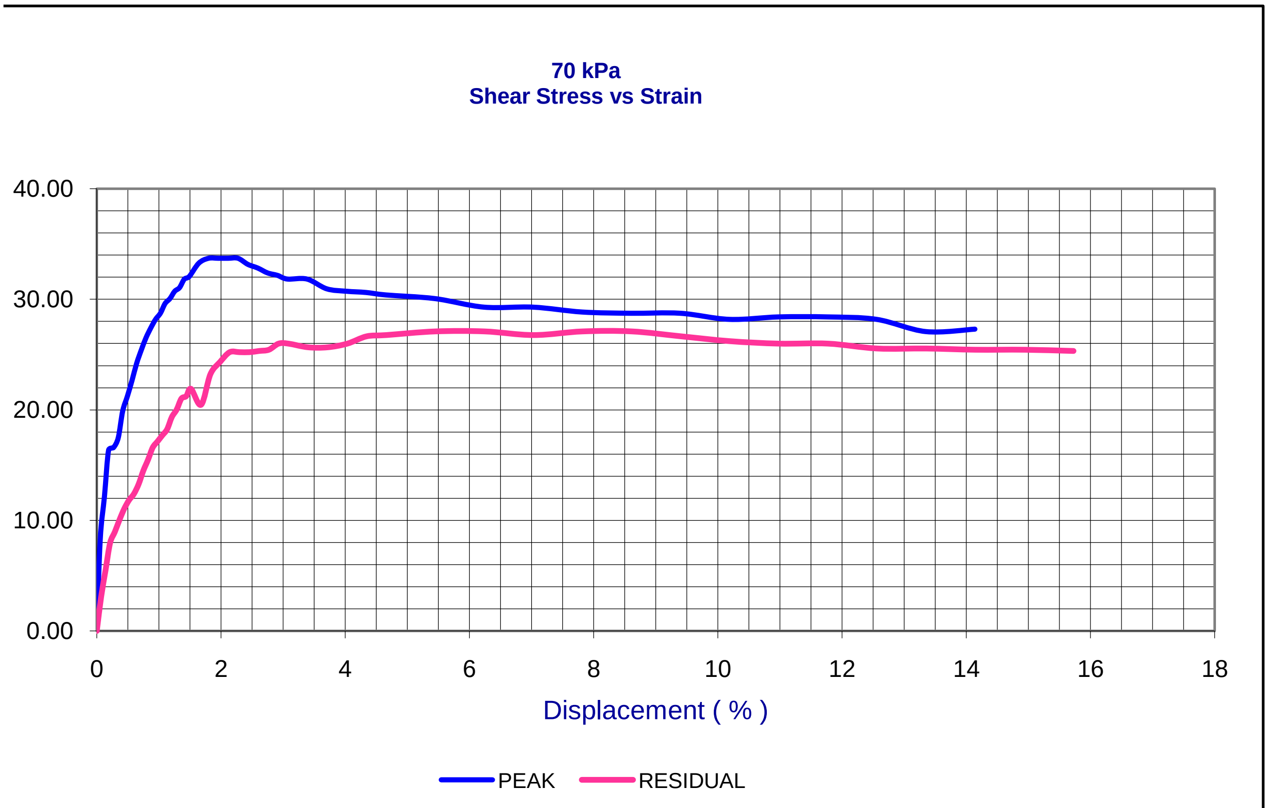
Specimen ID	**Normal Stress**	**Peak Shear Stress**	**Residual Shear Stress**
	kPa	**kPa**	**kPa**
1	70.0	33.7	27.1
2	140.0	70.9	53.9
3	210.0	100.5	96.3

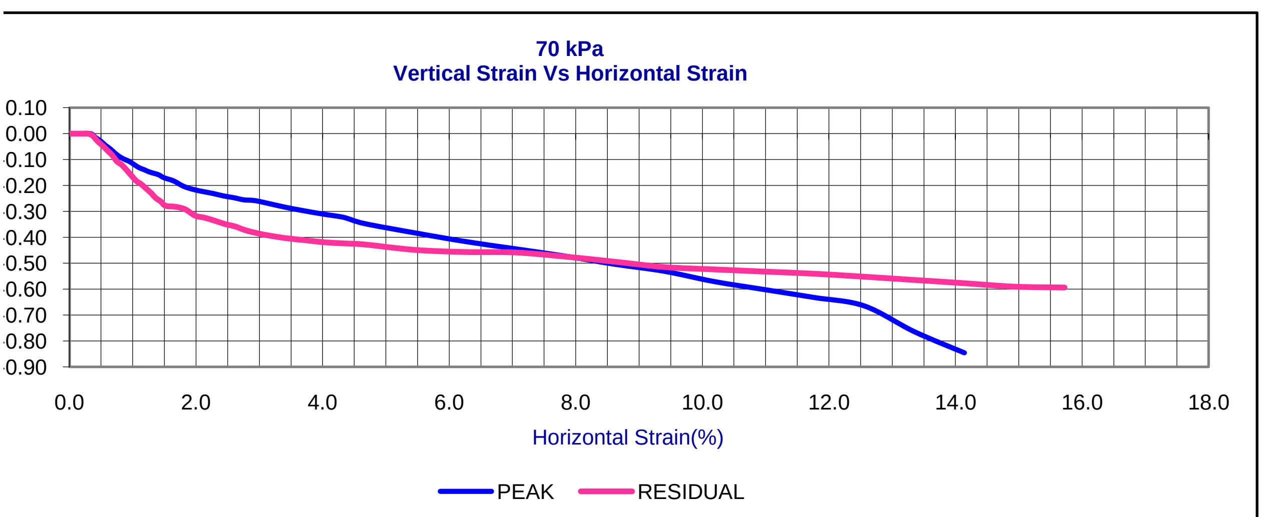
Note: Test specimens were inundated with water.



DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

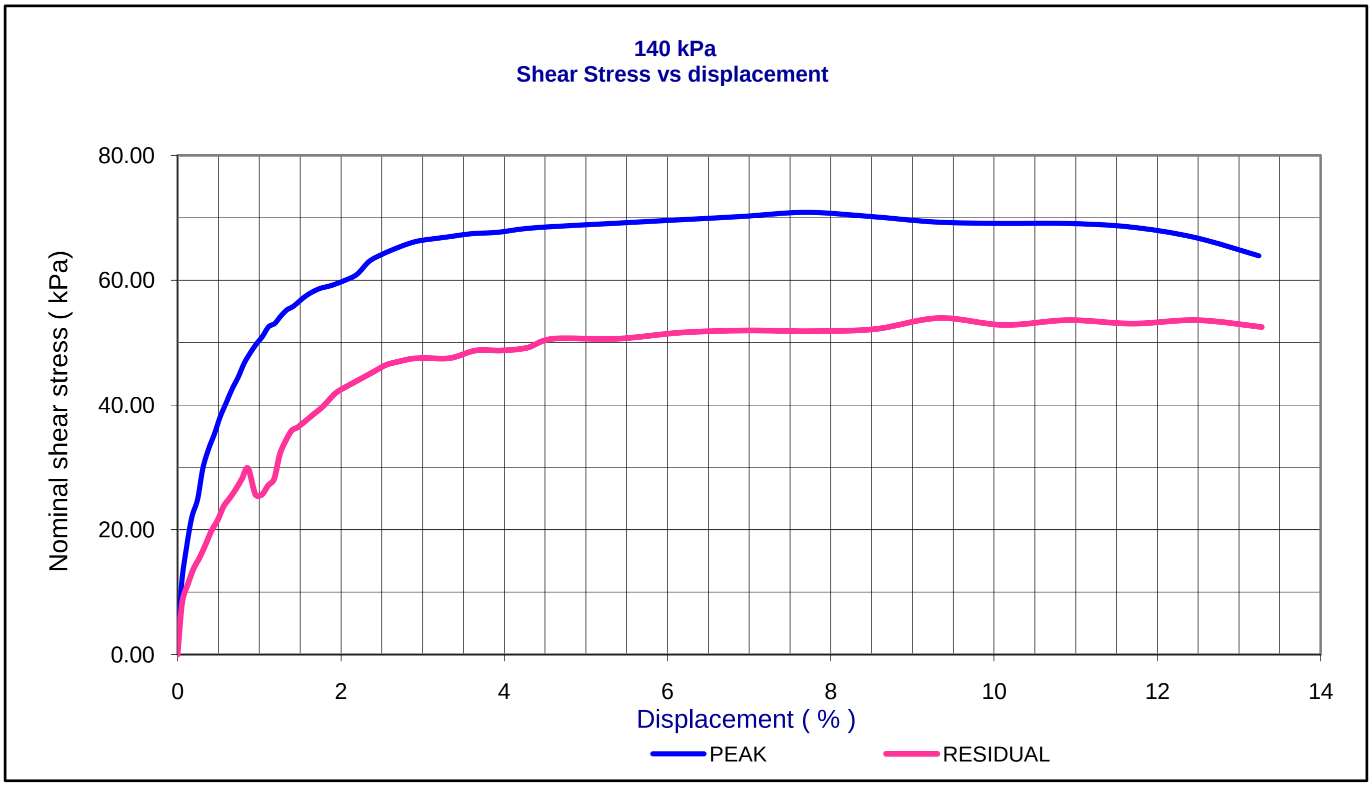
Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**19-October-2011**
Sample ID.:	**B9-1_TW13**	**Tested By:**	**CZ/SB**
Lab No.:	**AdS077_2011**	**Checked By:**	**SB**

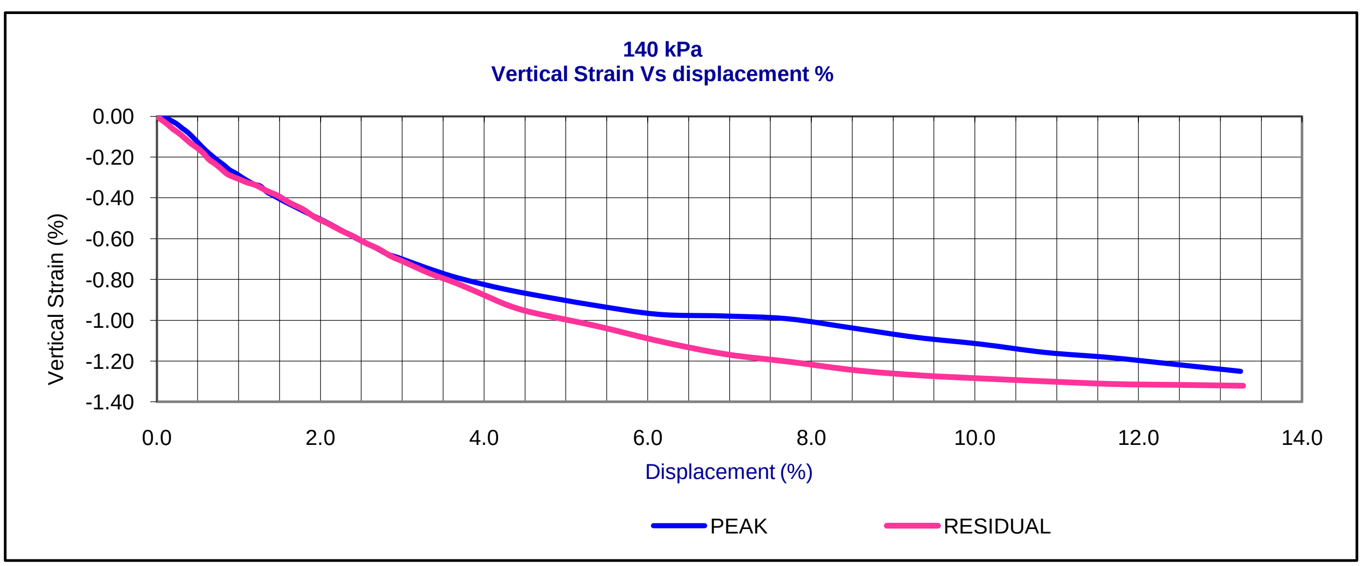




DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

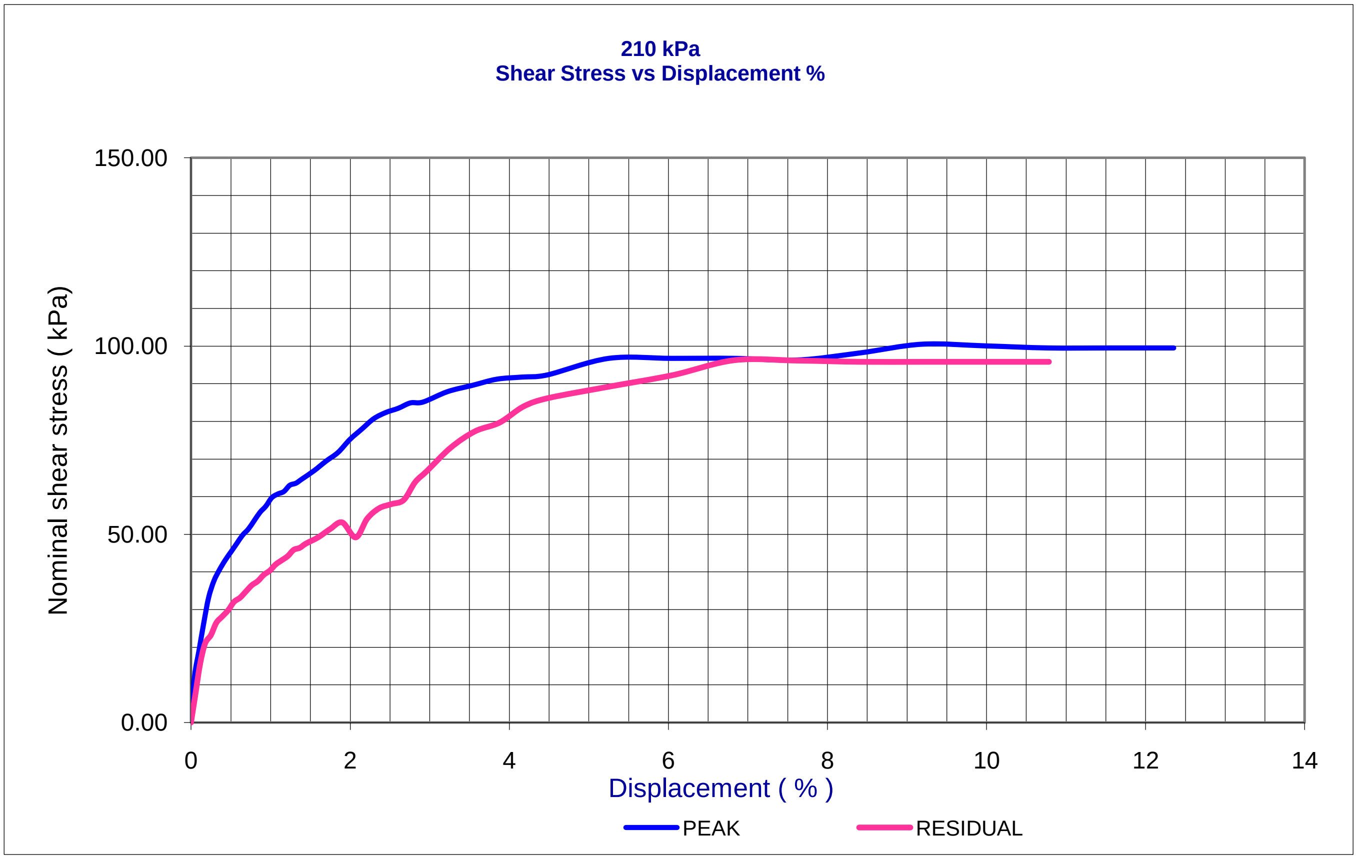
Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**19-October-2011**
Sample ID.:	**B9-1_TW13**	**Tested By:**	**CZ/SB**
Lab No.:	**AdS077_2011**	**Checked By:**	**SB**

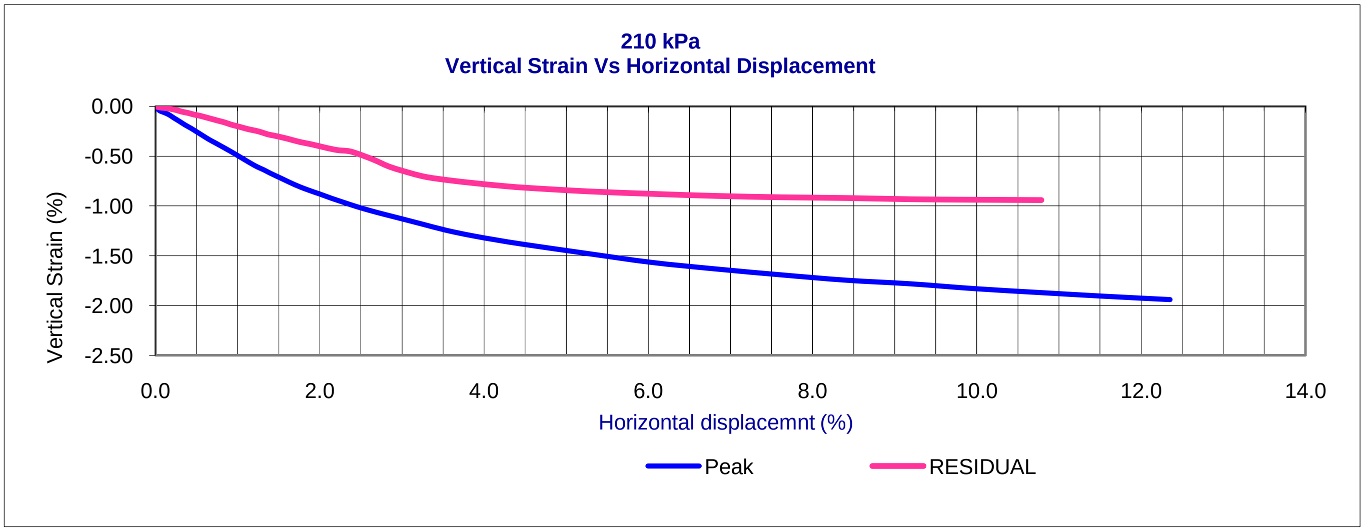




DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**19 October 2011**
Sample ID.:	**B9-1_TW13**	**Tested By:**	**CZ/SB**
Lab No.:	**AdS077_2011**	**Checked By:**	**SB**





Appendix D: Analytical Laboratory Test Results

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

Date Received: 18-JUL-11
Report Date: 25-JUL-11 15:09 (MT)
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1032510
Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 092959-A

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1032510-1	L1032510-2	L1032510-3		
	Description	SOIL	SOIL	SOIL		
	Sampled Date	15-JUL-11	15-JUL-11	15-JUL-11		
	Sampled Time					
	Client ID	B9-1, TW10@30-31.5' GREY SILTY CLAY	B9-2, SS25@105', GREY SILTY CLAY	B9-3, SS12@40', GREY SILTY CLAY		
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	18.3	18.7	19.5		
	pH (pH units)	8.06	7.77	8.07		
	Redox Potential (mV)	110	134	105		
	Resistivity (ohm cm)	3070	1920	4170		
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20	<0.20	<0.20		
Anions and Nutrients	Sulphate (mg/kg)	144	429	34		

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
MOISTURE-WT	Soil	% Moisture	Gravimetric: Oven Dried
PH-WT	Soil	pH	MOEE E3137A

Soil samples are mixed in the deionized water and the supernatant is analyzed directly by the pH meter.

ALS Test Code	Matrix	Test Description	Method Reference**
REDOX-POTENTIAL-WT	Soil	Redox Potential	APHA 2580
RESISTIVITY-WT	Soil	Resistivity	MOEE E3137A
SO4-WT	Soil	Sulphate	EPA 300.0
SULPHIDE-WT	Soil	Sulphide	APHA 4500S2D

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

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

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

Chain of Custody Numbers:

092959-A

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

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

Contact: SHANE MACLEOD

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT	**Soil**							
Batch R2220531								
WG1314260-2 LCS								
% Moisture			84		%		70-130	18-JUL-11
WG1314260-1 MB								
% Moisture			<0.10		%		0.1	18-JUL-11
PH-WT	**Soil**							
Batch R2223567								
WG1318107-1 CVS								
pH			100		%		80-120	22-JUL-11
RESISTIVITY-WT	**Soil**							
Batch R2223537								
WG1318094-1 CVS								
Resistivity			98		%		70-130	22-JUL-11
SO4-WT	**Soil**							
Batch R2222247								
WG1315561-2 DUP		**L1032510-1**						
Sulphate		144	142		mg/kg	1.1	30	20-JUL-11
WG1315561-3 LCS								
Sulphate			101		%		60-140	20-JUL-11
WG1315561-1 MB								
Sulphate			<20		mg/kg		20	20-JUL-11
SULPHIDE-WT	**Soil**							
Batch R2222299								
WG1316784-1 CVS								
Sulphide			107		%		50-120	21-JUL-11
WG1316782-2 DUP		**L1032510-1**						
Sulphide		<0.20	<0.20	RPD-NA	mg/kg	N/A	20	21-JUL-11
WG1316782-1 MB								
Sulphide			<0.20		mg/kg		0.2	21-JUL-11

Quality Control Report

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential							
	1	15-JUL-11	22-JUL-11 13:43	24	170	hours	EHTR
	2	15-JUL-11	22-JUL-11 13:44	24	170	hours	EHTR
	3	15-JUL-11	22-JUL-11 13:45	24	170	hours	EHTR

Legend & Qualifier Definitions:

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

Notes*:
Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1032510 were received on 18-JUL-11 10:35.

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: Slope Stability Analyses

Figure E-1: Slope Stability Result – East Abutment – Short-term Loading (Undrained properties)

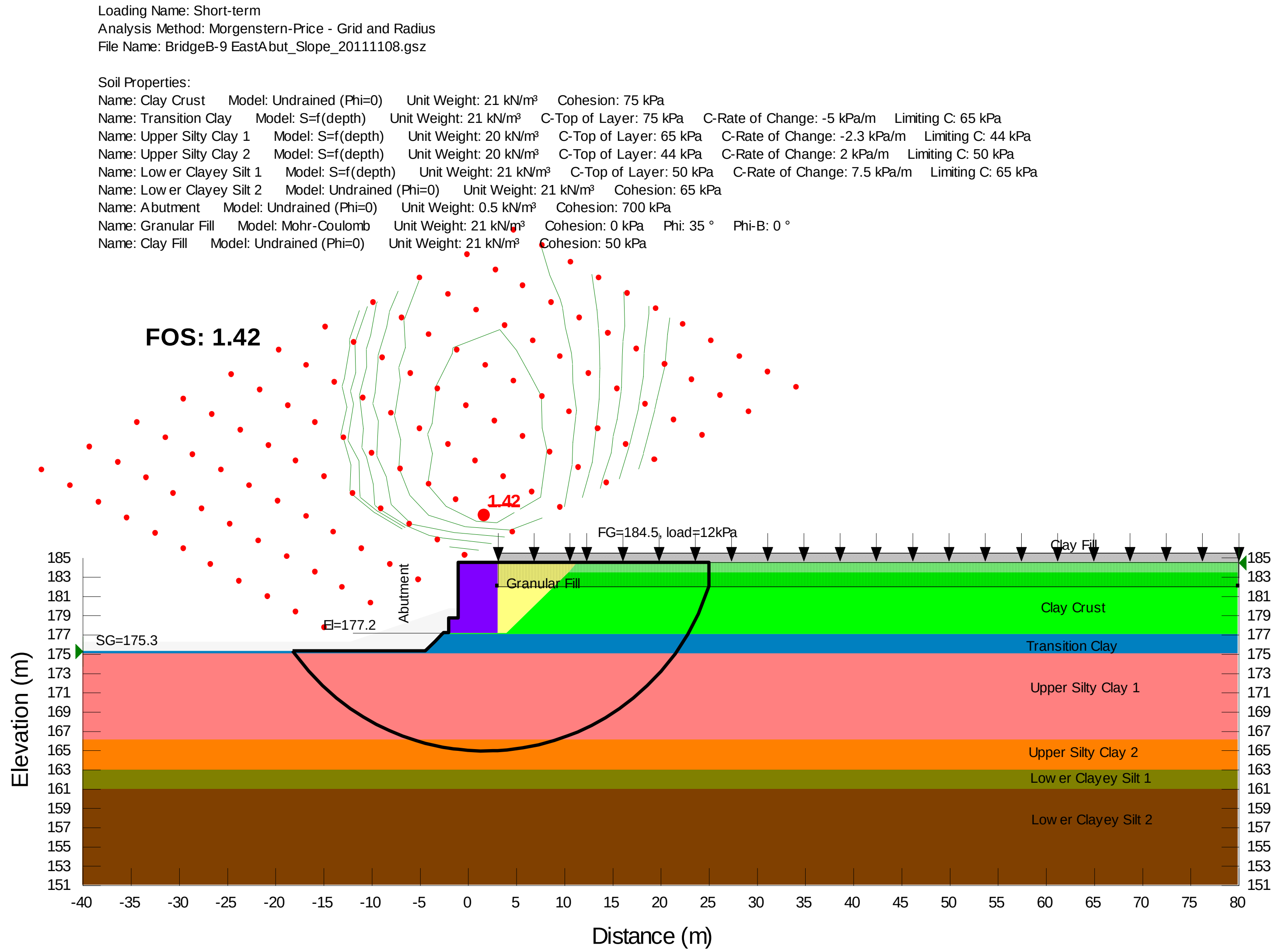


Figure E-2: Slope Stability Result – East Abutment – End of Construction Loading (Undrained properties)

Loading Name: End of Construction
Analysis Method: Morgenstern-Price - Grid and Radius
File Name: BridgeB-9 EastAbut_Slope_20111108.gsz

Soil Properties:
Name: Clay Crust Model: Undrained (Phi=0) Unit Weight: 21 kN/m³ Cohesion: 75 kPa
Name: Transition Clay Model: S=f(depth) Unit Weight: 21 kN/m³ C-Top of Layer: 75 kPa C-Rate of Change: -5 kPa/m Limiting C: 65 kPa
Name: Upper Silty Clay 1 Model: S=f(depth) Unit Weight: 20 kN/m³ C-Top of Layer: 65 kPa C-Rate of Change: -2.3 kPa/m Limiting C: 44 kPa
Name: Upper Silty Clay 2 Model: S=f(depth) Unit Weight: 20 kN/m³ C-Top of Layer: 44 kPa C-Rate of Change: 2 kPa/m Limiting C: 50 kPa
Name: Lower Clayey Silt 1 Model: S=f(depth) Unit Weight: 21 kN/m³ C-Top of Layer: 50 kPa C-Rate of Change: 7.5 kPa/m Limiting C: 65 kPa
Name: Lower Clayey Silt 2 Model: Undrained (Phi=0) Unit Weight: 21 kN/m³ Cohesion: 65 kPa
Name: Abutment Model: Undrained (Phi=0) Unit Weight: 0.5 kN/m³ Cohesion: 700 kPa
Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 °
Name: Clay Fill Model: Undrained (Phi=0) Unit Weight: 21 kN/m³ Cohesion: 50 kPa

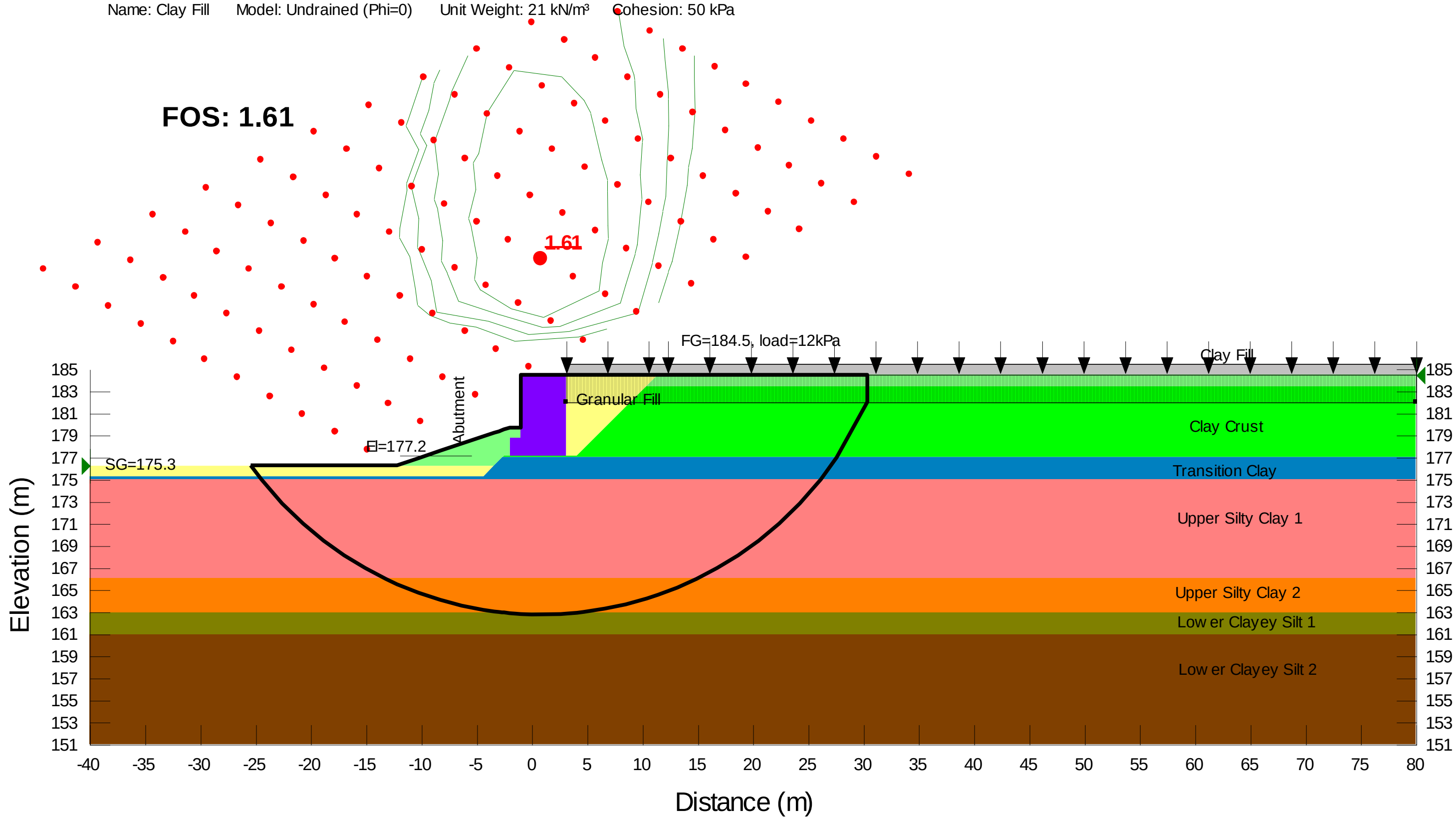
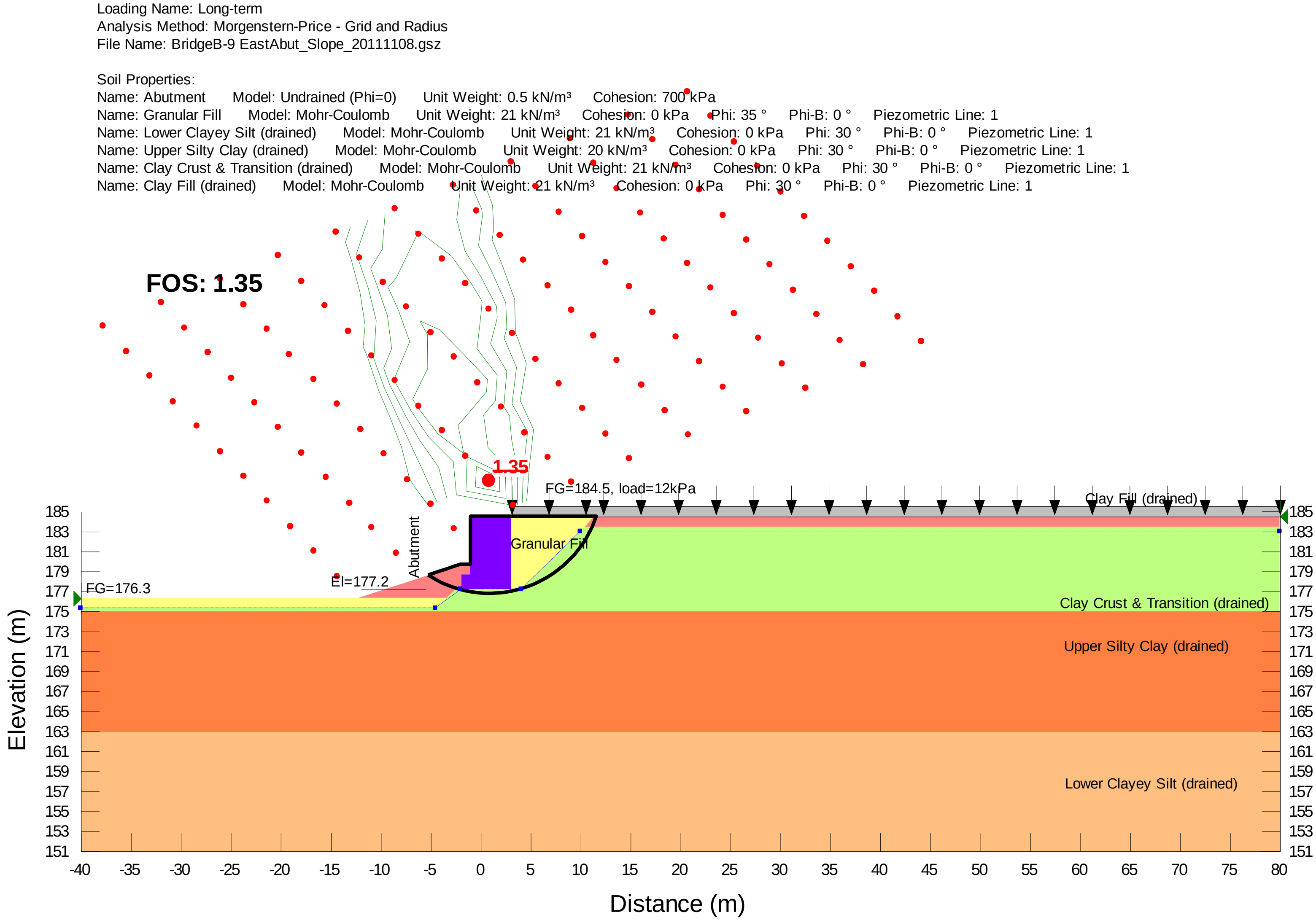


Figure E-3: Slope Stability Result – East Abutment – Long-term Loading (Drained properties)

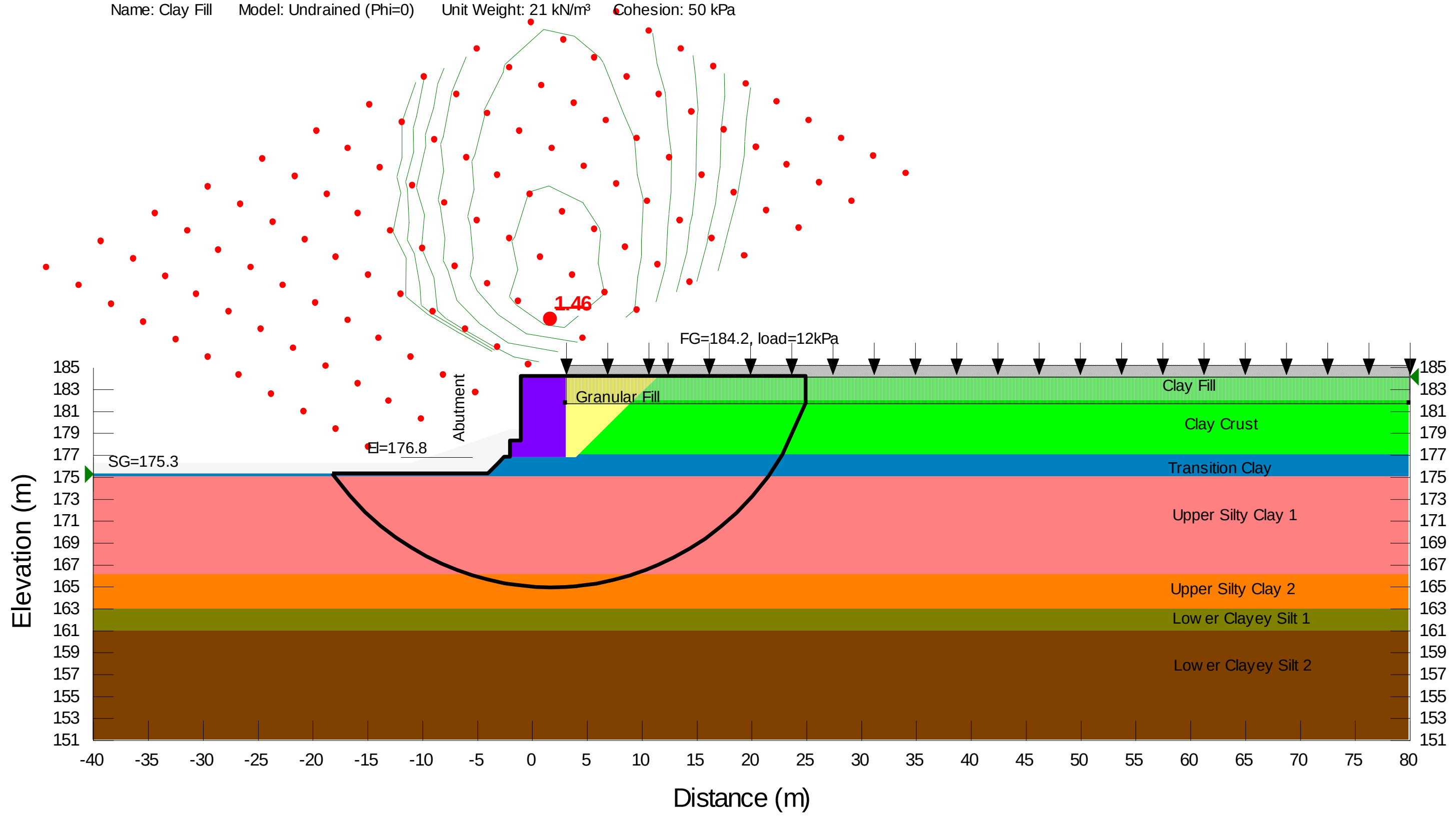


Project:	Windsor-Essex Parkway	**Date:**	March / 2012
Document:	Geotechnical Investigation and Design Report – Bridge B-9 (Hwy. 3 East Ramp Underpass at Hwy. 401 Sta. 9+549L to 9+676L, LaSalle)	**Rev:**	0
Doc No.:	285380-04-119-0025 (Geocres No. 40J6-31)	**Page No.:**	App E- 3 of 6

Figure E-4: Slope Stability Result – West Abutment – Short-term Loading (Undrained properties)

Loading Name: Short-term
Analysis Method: Morgenstern-Price - Grid and Radius
File Name: BridgeB-9 WestAbut_Slope_20111108.gsz

Soil Properties:
Name: Clay Crust Model: Undrained (Phi=0) Unit Weight: 21 kN/m³ Cohesion: 75 kPa
Name: Transition Clay Model: S=f(depth) Unit Weight: 21 kN/m³ C-Top of Layer: 75 kPa C-Rate of Change: -5 kPa/m Limiting C: 65 kPa
Name: Upper Silty Clay 1 Model: S=f(depth) Unit Weight: 20 kN/m³ C-Top of Layer: 65 kPa C-Rate of Change: -2.3 kPa/m Limiting C: 44 kPa
Name: Upper Silty Clay 2 Model: S=f(depth) Unit Weight: 20 kN/m³ C-Top of Layer: 44 kPa C-Rate of Change: 2 kPa/m Limiting C: 50 kPa
Name: Lower Clayey Silt 1 Model: S=f(depth) Unit Weight: 21 kN/m³ C-Top of Layer: 50 kPa C-Rate of Change: 7.5 kPa/m Limiting C: 65 kPa
Name: Lower Clayey Silt 2 Model: Undrained (Phi=0) Unit Weight: 21 kN/m³ Cohesion: 65 kPa
Name: Abutment Model: Undrained (Phi=0) Unit Weight: 0.5 kN/m³ Cohesion: 700 kPa
Name: Granular Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 ° Phi-B: 0 °
Name: Clay Fill Model: Undrained (Phi=0) Unit Weight: 21 kN/m³ Cohesion: 50 kPa



Project:	Windsor-Essex Parkway	**Date:**	March / 2012
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Figure E-5: Slope Stability Result – West Abutment – End of Construction Loading (Undrained properties)

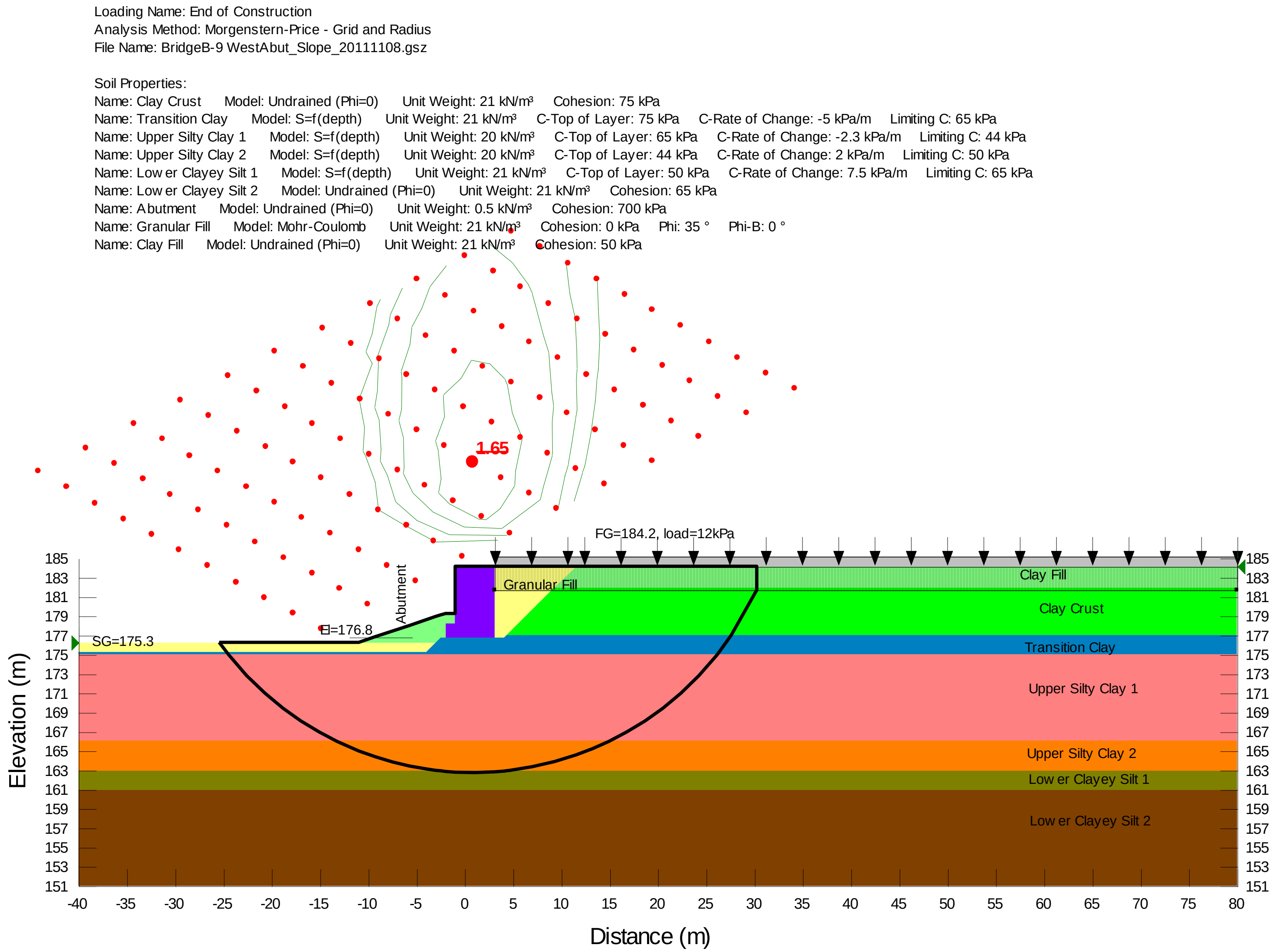
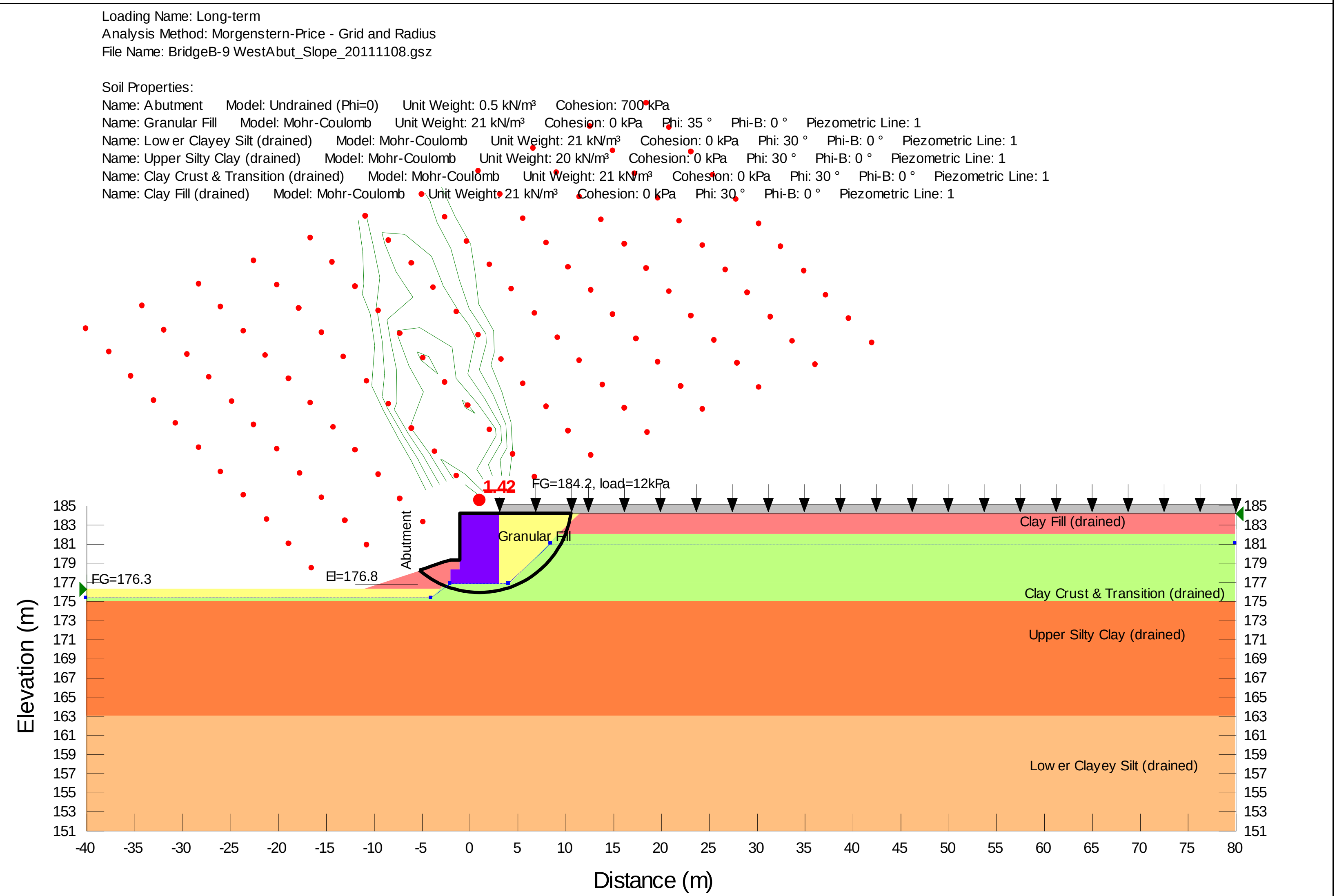


Figure E-6: Slope Stability Result – West Abutment – Long-term Loading (Drained properties)



Project:	Windsor-Essex Parkway	**Date:**	March / 2012
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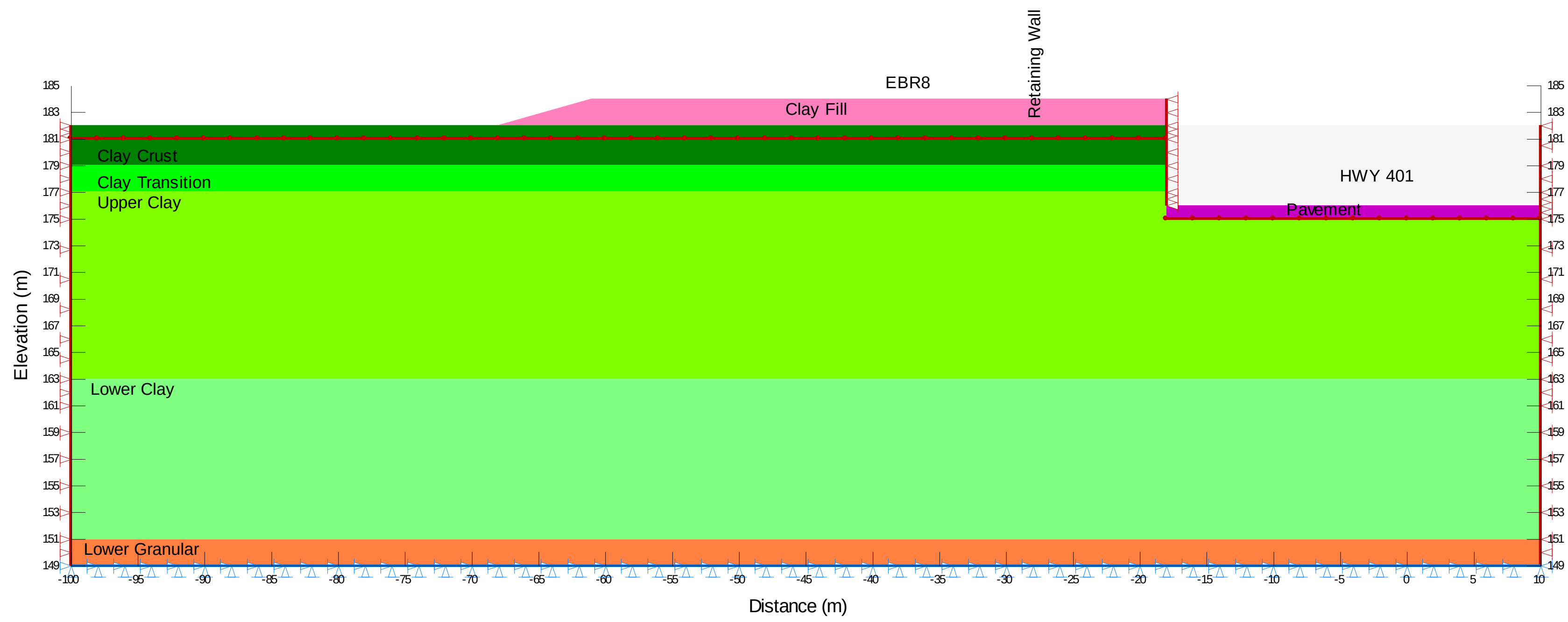
Doc No.:	285380-04-119-0025 (Geocres No. 40J6-31)	**Page No.:**	App E- 6 of 6

Appendix F: Stress-Deformation Analyses

Figure F-1: Finite Element Model Configurations

Analysis Name: Dissipation
File Name: BridgeB-9_WAbut_Hwy_Sigma_20110929.gsz
Last Solved Date: 30/09/2011

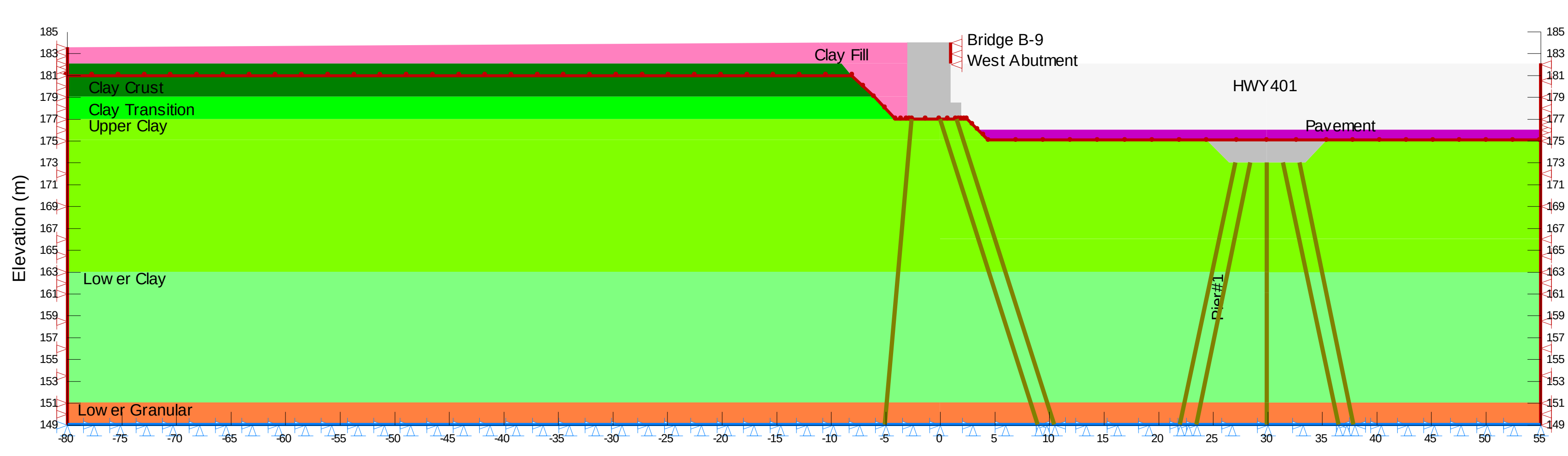
Name: Pavement   Model: Linear Elastic   Young's Modulus (E): 50000 kPa   Unit Weight: 22 kN/m³   Poisson's Ratio: 0.25
Name: Clay Crust   Model: Elastic-Plastic   Effective Young's Modulus (E'): 27000 kPa   Poisson's Ratio: 0.35   Cohesion': 0 kPa   Phi': 30 °   Unit Weight: 21 kN/m³   K-Ratio: 1   K-Function: Conductivity_Crust
Name: Lower Granular   Model: Elastic-Plastic   Effective Young's Modulus (E'): 40000 kPa   Poisson's Ratio: 0.35   Cohesion': 0 kPa   Phi': 32 °   Unit Weight: 22 kN/m³
Name: Clay Transition   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   K-Ratio: 0.5   K-Function: Conductivity_Unweathered
Name: Upper Clay   Model: Soft Clay (MCC)   O.C. Ratio: 1.5   Poisson's Ratio: 0.35   Lambda: 0.08   Kappa: 0.009   Initial Void Ratio: 0.6   Unit Weight: 20 kN/m³   Phi': 26 °   K-Ratio: 0.5   K-Function: Conductivity_Unweathered
Name: Lower Clay   Model: Elastic-Plastic   Effective Young's Modulus (E'): 18000 kPa   Poisson's Ratio: 0.35   Cohesion': 0 kPa   Phi': 30 °   Unit Weight: 20 kN/m³   K-Ratio: 0.5   K-Function: Conductivity_Unweathered
Name: Clay Fill   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



Wall Section – Perpendicular to Highway 401

Analysis Name: Dissipation
File Name: BridgeB-9 WAbut_Abt_Sigma_20111026.gsz
Last Solved Date: 04/11/2011

Name: Pavement   Model: Linear Elastic   Young's Modulus (E): 50000 kPa   Unit Weight: 22 kN/m³   Poisson's Ratio: 0.25
Name: Clay Crust   Model: Elastic-Plastic   Effective Young's Modulus (E'): 27000 kPa   Poisson's Ratio: 0.35   Cohesion': 0 kPa   Phi': 30 °   Unit Weight: 21 kN/m³   K-Ratio: 1   K-Function: Conductivity_Crust
Name: Lower Granular   Model: Elastic-Plastic   Effective Young's Modulus (E'): 40000 kPa   Poisson's Ratio: 0.35   Cohesion': 0 kPa   Phi': 32 °   Unit Weight: 22 kN/m³
Name: Clay Transition   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   K-Ratio: 0.5   K-Function: Conductivity_Unweathered
Name: Upper Clay   Model: Soft Clay (MCC)   O.C. Ratio: 1.5   Poisson's Ratio: 0.35   Lambda: 0.08   Kappa: 0.009   Initial Void Ratio: 0.6   Unit Weight: 20 kN/m³   Phi': 26 °   K-Ratio: 0.5   K-Function: Conductivity_Unweathered
Name: Lower Clay   Model: Elastic-Plastic   Effective Young's Modulus (E'): 18000 kPa   Poisson's Ratio: 0.35   Cohesion': 0 kPa   Phi': 30 °   Unit Weight: 20 kN/m³   K-Ratio: 0.5   K-Function: Conductivity_Unweathered
Name: Clay Fill   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
Name: Concrete   Model: Linear Elastic   Effective Young's Modulus (E'): 27000000 kPa   Poisson's Ratio: 0.334   Unit Weight: 24 kN/m³



Abutment Section – Perpendicular to Abutment Face

Figure F-2: Abutment Section – Cumulative Heave/Settlement – Temporary Excavation

Analysis Name: Excavation
File Name: BridgeB-9 WAbut_Abt_Sigma_20111026.gsz
Last Solved Date: 04/11/2011

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Lower Granular Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition 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 K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Upper Clay Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.08 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Lower Clay Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 20 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered

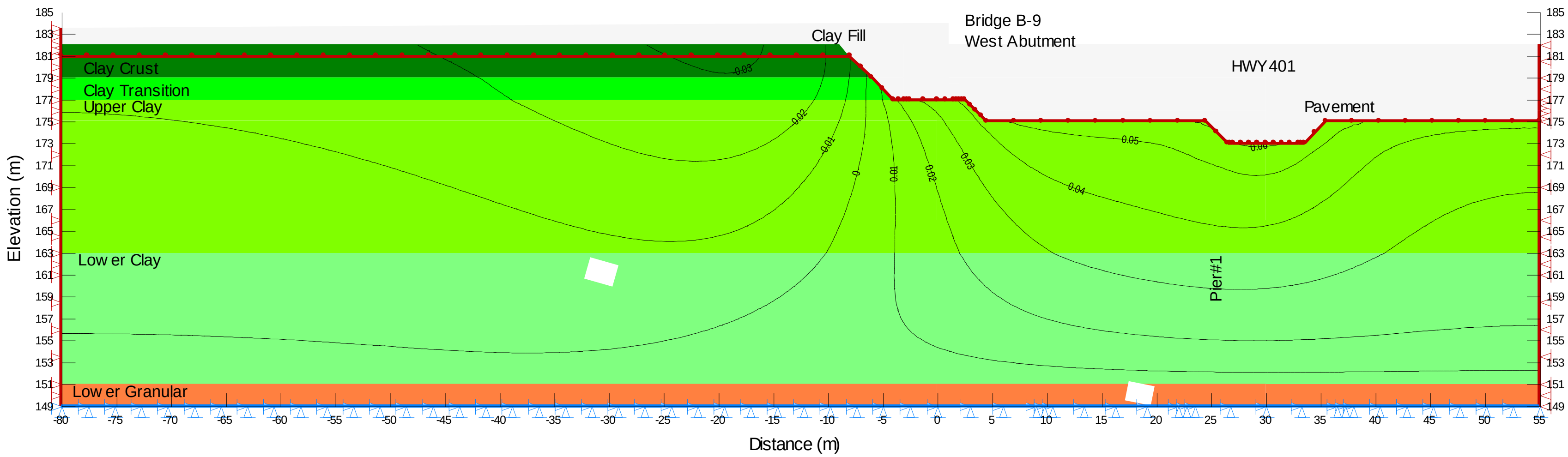


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Figure F-3: Abutment Section – Cumulative Heave/Settlement – End of Construction

Analysis Name: Road Fill
File Name: BridgeB-9 WAbut_Abt_Sigma_20111026.gsz
Last Solved Date: 04/11/2011

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Lower Granular Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition 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 K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Upper Clay Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.08 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Lower Clay Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 20 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Clay Fill 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
Name: Concrete Model: Linear Elastic Effective Young's Modulus (E'): 27000000 kPa Poisson's Ratio: 0.334 Unit Weight: 24 kN/m³

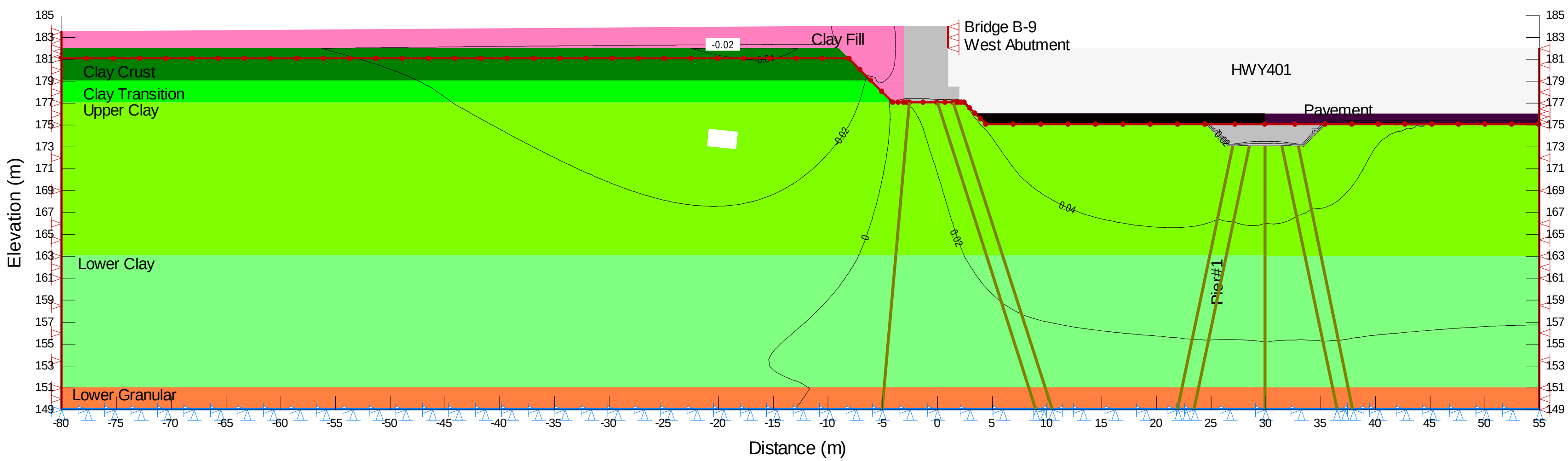

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Figure F-4: Abutment Section – Cumulative Heave/Settlement – Long-term Condition

Analysis Name: Dissipation
File Name: BridgeB-9 WAbut_Abt_Sigma_20111026.gsz
Last Solved Date: 04/11/2011

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Lower Granular Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition 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 K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Upper Clay Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.08 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Lower Clay Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 20 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Clay Fill 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
Name: Concrete Model: Linear Elastic Effective Young's Modulus (E'): 27000000 kPa Poisson's Ratio: 0.334 Unit Weight: 24 kN/m³

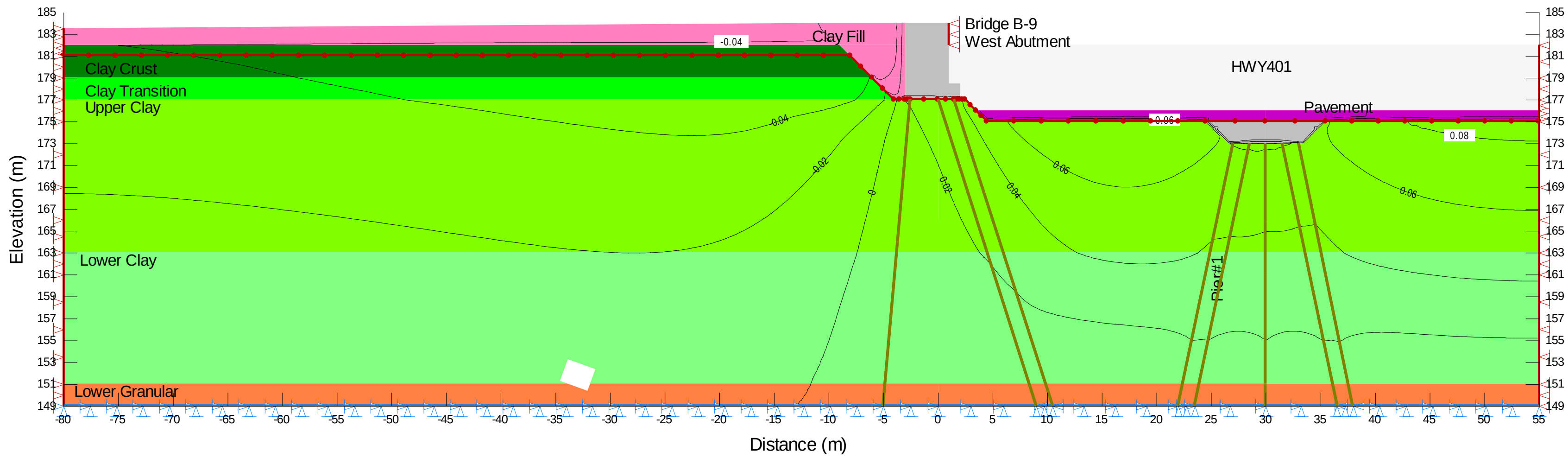


Figure F-5: Abutment Section – Lateral Displacement – End of Construction Condition

Analysis Name: Road Fill
File Name: BridgeB-9 WAbut_Abt_Sigma_20111026.gsz
Last Solved Date: 04/11/2011

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Lower Granular Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition 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 K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Upper Clay Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.08 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Lower Clay Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 20 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Clay Fill 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
Name: Concrete Model: Linear Elastic Effective Young's Modulus (E'): 27000000 kPa Poisson's Ratio: 0.334 Unit Weight: 24 kN/m³

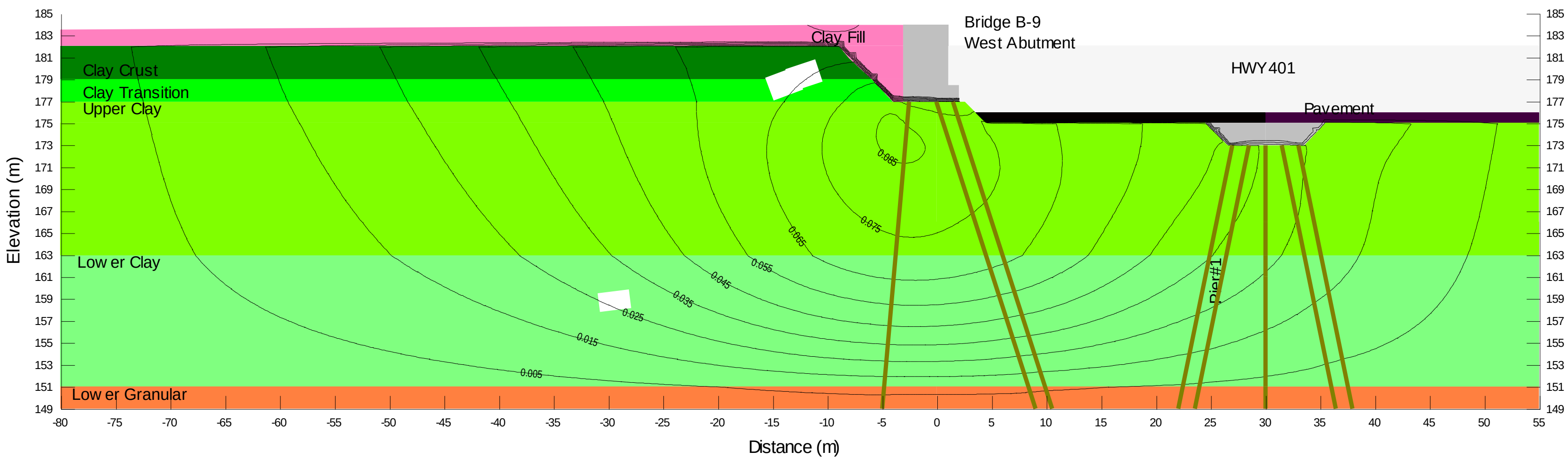


Figure F-6: Abutment Section – Lateral Displacement – Long-term Condition

Analysis Name: Dissipation
File Name: BridgeB-9 WAbut_Abt_Sigma_20111026.gsz
Last Solved Date: 04/11/2011

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Lower Granular Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition 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 K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Upper Clay Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.08 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Lower Clay Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 20 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Clay Fill 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
Name: Concrete Model: Linear Elastic Effective Young's Modulus (E'): 27000000 kPa Poisson's Ratio: 0.334 Unit Weight: 24 kN/m³

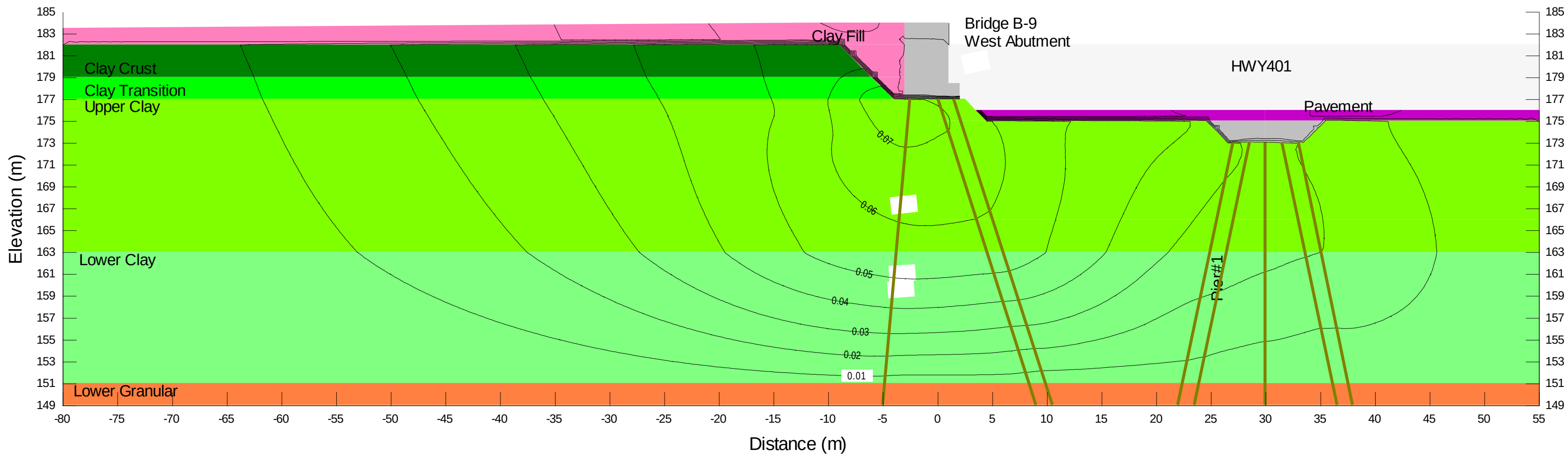


Figure F-7: Abutment Section – Pore Pressure Distribution – Long-term Condition

Analysis Name: Dissipation
File Name: BridgeB-9 WAbut_Abt_Sigma_20111026.gsz
Last Solved Date: 04/11/2011

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Lower Granular Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition 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 K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Upper Clay Model: Soft Clay (MCC) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.08 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 20 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Lower Clay Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 20 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Clay Fill 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
Name: Concrete Model: Linear Elastic Effective Young's Modulus (E'): 27000000 kPa Poisson's Ratio: 0.334 Unit Weight: 24 kN/m³

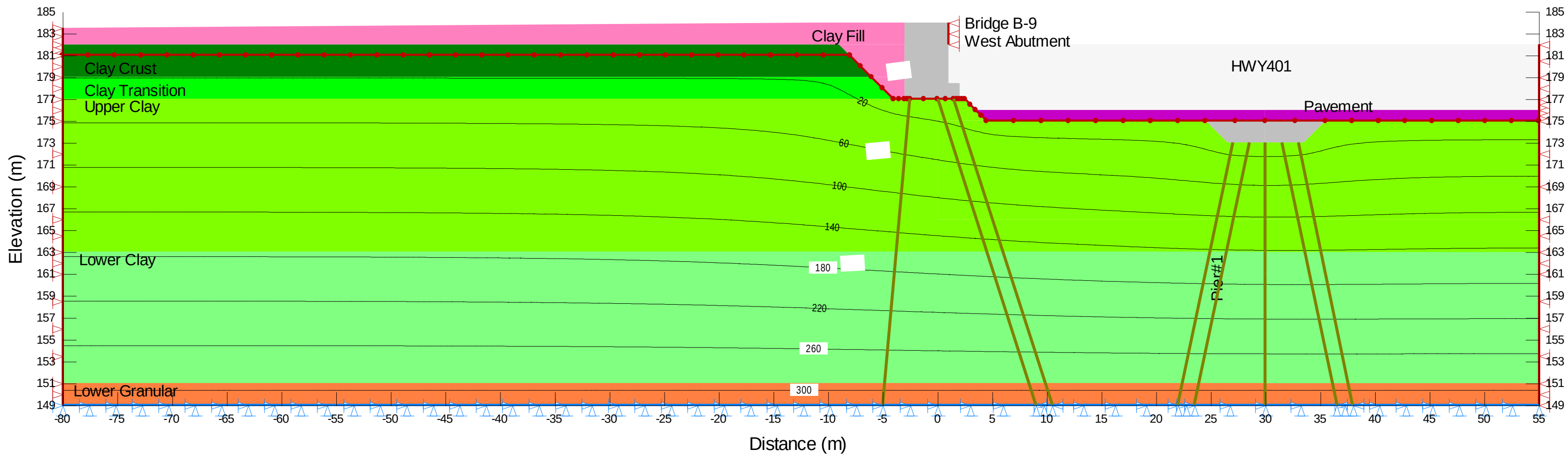
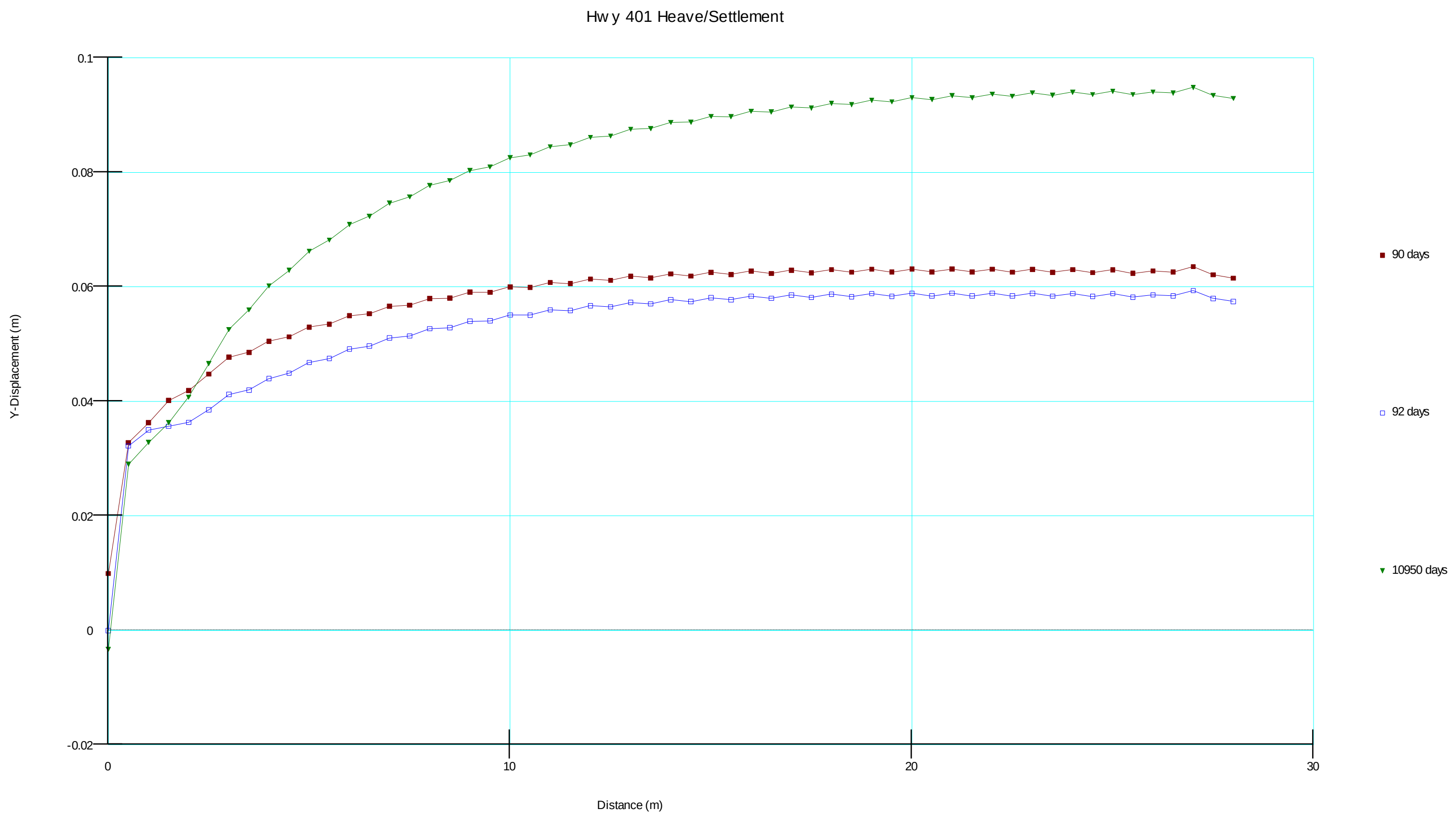
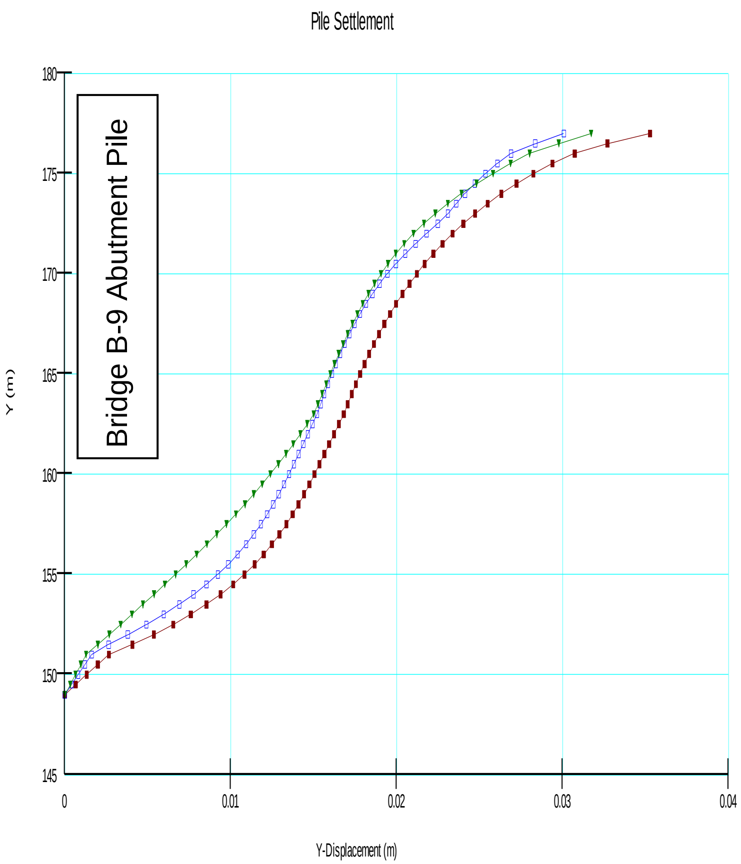


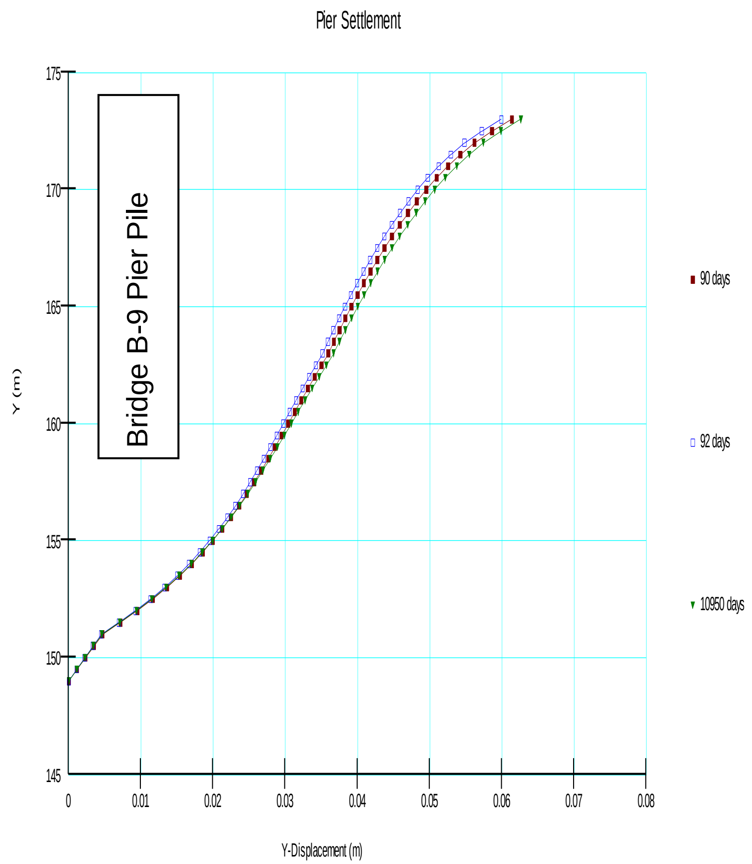
Figure F-8: Wall Section – Cumulative Highway 401 Settlement/Heave



Note: Distance refers from wall
Legend:
90 days = End of Excavation
92 days = End of Construction
10,950 days = 30 years = Long-term

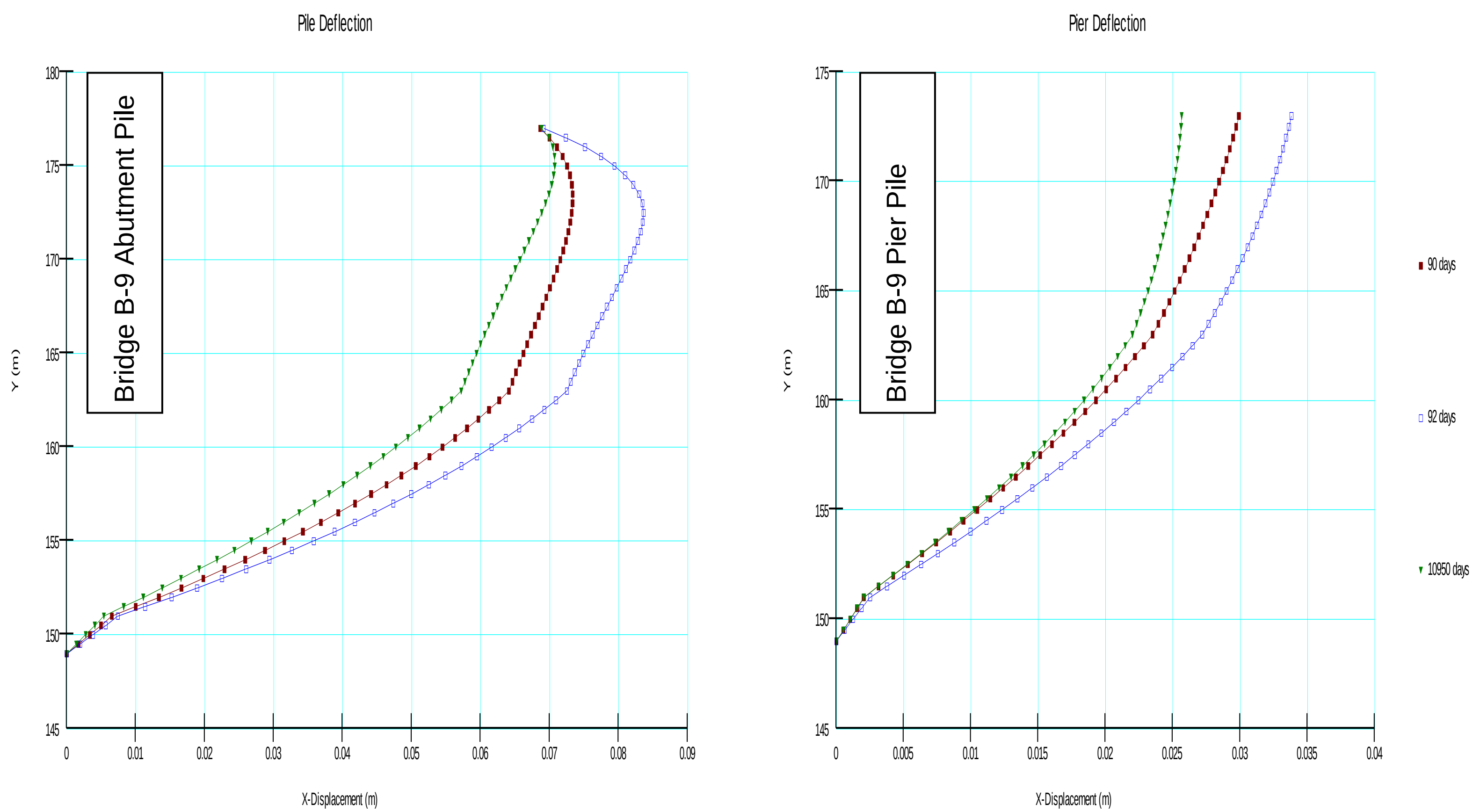
Figure F-9: Abutment Section – Cumulative Soil Settlement Profile along Pile Line





Legend:
90 days = End of Excavation
92 days = End of Construction
10,950 days = 30 years = Long-term

Figure F-10: Abutment Section – Cumulative Lateral Soil Displacement Profile along Pile Line



Legend:
90 days = End of Excavation
92 days = End of Construction
10,950 days = 30 years = Long-term

Project:	Windsor-Essex Parkway	**Date:**	March / 2012
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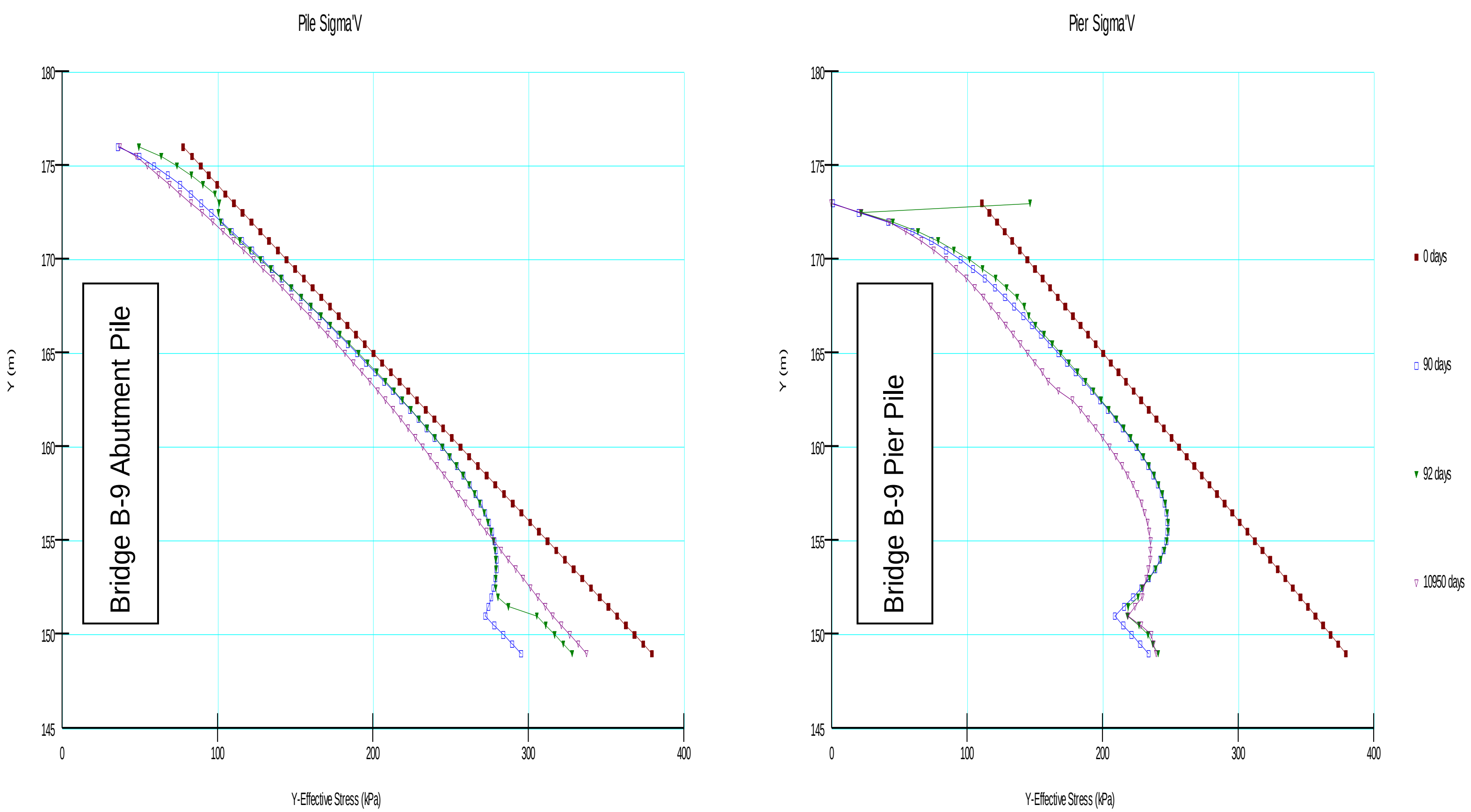
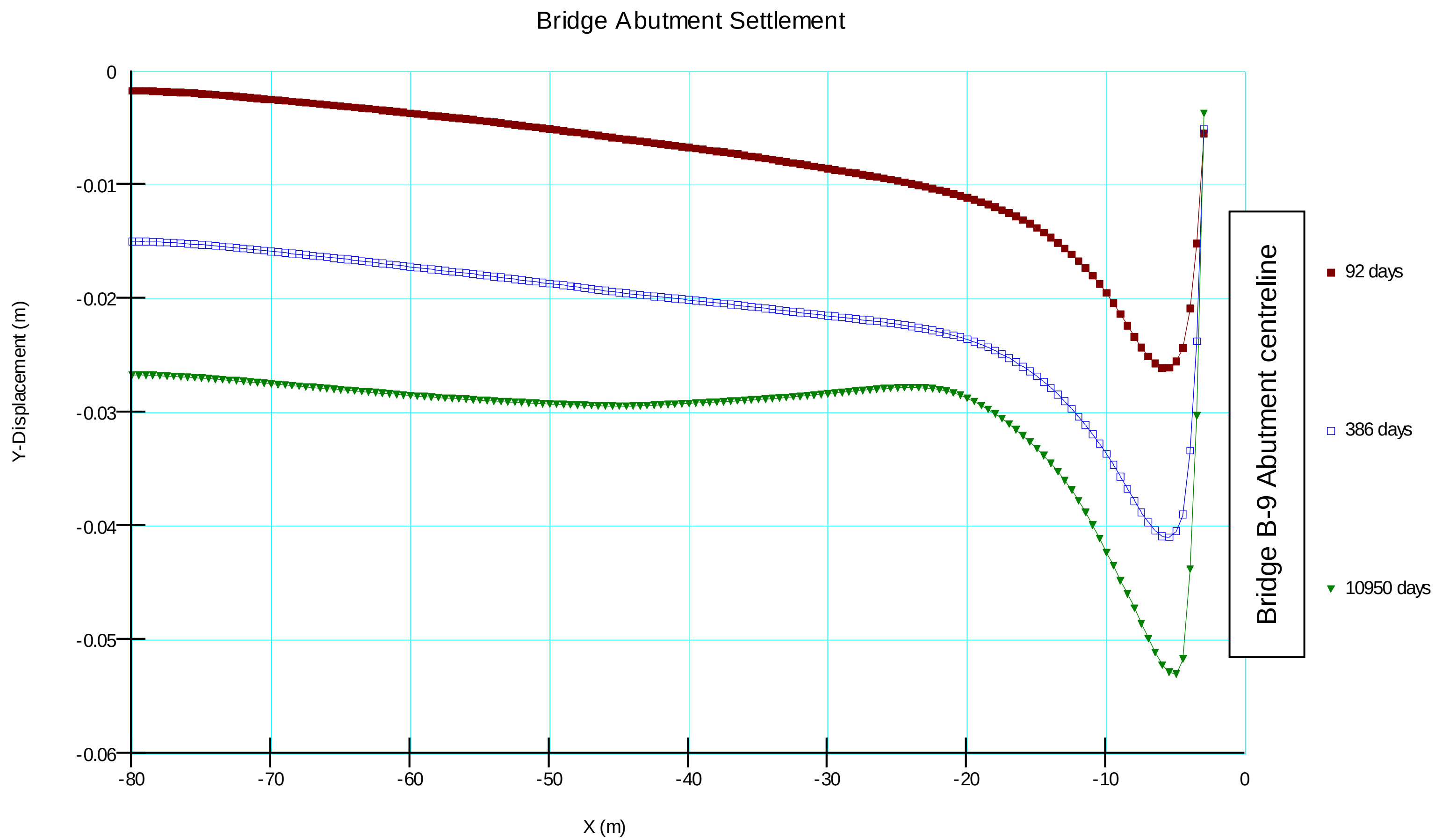
Figure F-11: Abutment Section – Vertical Effective Stress Profile along Pile Line



Legend:
0 days = Initial
90 days = End of Excavation
92 days = End of Construction
10,950 days = 30 years = Long-term

Figure F-12: Abutment Section – Cumulative EBR8 Settlement/Heave



Legend:
92 days = End of Construction
386 days = 1 year
10,950 days = 30 years = Long-term

Appendix G: Selected Site Photographs

Photograph G-1: Site View (Talbot Rd NBL looking North to B9-1 and B9-2)

Photograph G-2: Site View (Talbot Rd SBL looking South to B9-1 and B9-2)

Appendix H: Selected Core Photographs

Photograph H-1: Borehole B9-1 – Rock Core Elevation 146.5 to 144.9 m

Photograph H-2: Borehole B9-2 – Rock Core Elevation 146.5 to 143.2 m

Photograph H-3: Borehole B9-3 – Rock Core Elevation 145.9 to 144.3 m