

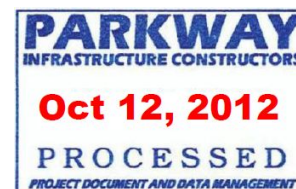
The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report



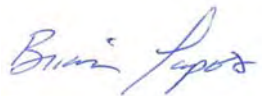
Tunnel T-9 (Cousineau Tunnel)

(Sta. 12+130L to 12+300L)

Geocres No. 40J3-19



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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 upon Permit AY-D-001-09 which was approved in February 2010.

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

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

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

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

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

On December 17, 2010 Infrastructure Ontario and MTO announced that the Windsor Essex Mobility Group (WEMG) reached financial close and signed a fixed-price contract with the Province to design, build, finance and maintain the Windsor-Essex Parkway. To build the initial works, WEMG has formed a Design-Build Joint Venture – Parkway Infrastructure Constructors. This team includes Dragados Canada, Inc., Acciona Infrastructure Canada Inc., and Fluor Canada Ltd. This combination brings a wide range of local and international experience to the project.

1.2 Report Introduction

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

This report presents the 90% geotechnical design of Tunnel T- 9 (Highway 401 Cousineau Tunnel, between Stations 12+130L and 12+300L), located in the LaSalle sector of the Windsor-Essex Parkway (WEP) project. 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. This design report addresses review comments from MTO.

The proposed 170 m long, 2 span Tunnel T-9 structure will carry parkland landscape and local traffic along Cousineau Road and Sandwich West Parkway over Highway 401 between Sta. 12+130L and Sta. 12+300L. A trail and associated trail bridge is located on the north side and parallel to the tunnel. As for all other tunnels at this project, Tunnel T-9 will be a cut-and-cover construction. The proposed structural solution incorporates structural deck on concrete girders supported on semi-integral abutments and centre pier on piles.

The design presented in this report was generally advanced from the preliminary geotechnical design developed for the WEMG (Windsor-Essex Mobility Group) proposal in June 2010 (ref. R-43)¹ and recognized as 30% design. The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines as well as the Parkway Infrastructure Constructors (PIC).

¹ References are listed in Section 9.

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

The design Tunnel T-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-16, R-18, R-19 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 a clayey silt till, silty clay till and glaciolacustrine clay. Hudec (ref. R-26) summarized the overburden geology in Windsor as consisting of the following strata: desiccated lacustrine clay, normally consolidated lacustrine clay, silty Tavistock till, glaciolacustrine clay and coarse Catfish Creek till. A distinct change in overburden deposits occurs in the east-west direction along a boundary located generally along the Huron-Church Road. Whereas, the eastern part of Windsor is underlain by firm to stiff glaciolacustrine silts and clays with upper deposits of stiff sandy to silty weathered clay and hard to stiff lacustrine clay-silt crust, the western part of Windsor is characterized by a thin surficial granular deposit underlain by thin crust layer underlain by soft to firm glaciolacustrine silts and clays.

At the WEP project area, the glacial till like deposit is typically 20 to 35 m thick and consists primarily of silty clay and clayey silt gradation with a random distribution of coarser particles. Random and apparently discontinuous seams / lenses of silt, sand and or gravel are present at various depths within the mass of the silty clay deposit. A firm to hard surficial crust layer has formed due to desiccation. Up to 2 m thick surficial layers of lacustrine silty clay or silt and sand are also encountered in the western sector of the project. A 1 m to 6 m thick very dense or hard basal glacial till or dense silty sand may be found directly overlying the bedrock surface. The bedrock at the project area comprises the Devonian Dundee Formation of the Hamilton group of formation and the underlying Devonian Lucas Formation of the Detroit River group of 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 midcontinent rift south-eastern extension is composed of Paleozoic cover rocks which form the bedrock foundation of the Essex Domain. The bedrock was deposited in the Paleozoic Era during the Middle Devonian period. Within the Essex

Domain the following strata were deposited the Hamilton Group, Dundee Formation, and Detroit River Group Onondaga Formation all consisting of Limestone, Dolostone, and Shale.

2.2 Site Seismic Background

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

In accordance with the CHBDC, the soil profile at the site of the project generally meets 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-21) at locations distributed strategically along is the project alignment between Howard Avenue (east end) and Matchette Road (west end). The measured velocities of the shear waves were consistently over 200 m/s, with the bulk of results ranging between 200 and 300 m/s.

2.3 Existing Site Conditions and Proposed Tunnel Layout

Tunnel T-9 site is situated near the center of the LaSalle segment of the Parkway. The topography of the lands immediately adjacent Tunnel T-9 is sloped with elevations ranging from approximately 182² in the area of the Wolfe Drain at north side of the tunnel to 184 at the surrounding area. Adjacent land use is residential to the north and a mix of residential and commercial to the south.

The tunnel structure will be constructed under WEP Phase I development and will be used to carry Cousineau Road traffic and parkland over Highway 401 to connect with Sandwich West Parkway and Highway 3 on the south side of the proposed depressed Highway 401. The Wolfe Drain (and Culvert CV-3) and Trail Bridge TB-7 are located on the north side of Highway 401. Highway 401 at this location will be constructed within permanent cut. The finished grades along the tunnel walls will be raised by about 1 m and 3 m above the existing grades on the north and south sides, respectively.

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 to 1.0 m³. This estimate is considered applicable for natural soils and/or conventional pavement materials where the ground surface is usually cleaned from the snow cover.

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

² Elevations are in metres and are referred to geodetic datum.

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

3 Geotechnical Investigations

3.1 Scope and Procedures of Geotechnical Investigations

Geotechnical investigations involving a number of boreholes, cone penetration tests (CPT) and Nilcon vane tests had been carried out in 2007-09 by Golder Associates (ref. R-16 to R-23) to develop the conceptual design and serve as background information for development of the WEP proposal designs. Additional geotechnical investigation was carried out 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 Tunnel T-9 comprised a total of 11 boreholes, 2 Nilcon vane tests, 2 CPTs and 1 DMT (flat blade dilatometer probe). Table 3-1 lists the test holes put down at or in close proximity of the tunnel site during both the previous and the current geotechnical investigations.

Table 3-1: Test Holes At and Around Tunnel T-9 Site

Reference	Boreholes	Nilcon Vane Tests	CPT	DMT
Additional Investigation (2011)	BH T9-1	NIL T9-1		
	BH CV3-1			
	BH TB7-1			
	BH TB7-2			
	BH TB7-3			
	BH TB7-4			
	BH 15-RW			
	BH/NIL T9-2	NIL T9-2		
	BH/DMT T9-1			DMT T9-1
	BH/CPT 45-RW		CPT 45-RW	
	BH/CPT 46-RW		CPT 46-RW	
Previous Studies (2007-09)	BH 115/115A			
	BH 116/116A			
	BH/CPT 114		CPT 114	
			CPT 6	

Drawing 285380-04-090-WIP1-2901 shows the locations of the test holes and an interpreted soil stratigraphic profile along the WEP centreline for the general area from Sta. 12+000L to Sta. 12+800L. The test hole locations and stratigraphic sections at the tunnel location and immediate vicinity are illustrated on Drawings 285380-04-090-WIP1-2903, 285380-04-090-WIP1-2902 and 285380-04-091-WIP1-2904.

3.2 Fieldwork for Additional Investigation

The boreholes were advanced using track-mounted CME55 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. Thin-walled Shelby tube (70 mm diameter x 600 mm long) samples were also recovered in the cohesive soil deposits below the upper crust layer. Soil sampling was carried out generally at 0.75 m depth interval in the top 7 to 8 m and at 1.5 m depth intervals thereafter. All samples were identified and placed in airtight containers and transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing⁴. Rock coring of the bedrock was carried out using 1.5 m long NQ or HQ sized core barrels.

Standard Penetration Tests (SPT, ASTM D1586⁵) were carried out in conjunction with split spoon sampling. Field vane tests (using conventional vanes) were carried out in between sampling at selected depths. The Nilcon vane tests listed in Table 3-1 were carried out typically adjacent the boreholes. 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.

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 tests were carried out on rock core samples selected from across the WEP length.

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

The CPT cone was pushed at a constant rate into the ground using hydraulic ram system of the drill rig (ASTM D5778). A pore pressure dissipation test was carried out at CPT 46-RW at 21.1 m below ground surface.

Nilcon blade was pushed in ground using the hydraulic ram of the drill rig. The shear vane tests were conducted in accordance with ASTM D2573-01.

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 the boreholes, Nilcon tests, and CPT executed at and around Tunnel T-9 during the previous pre-bid and additional investigations, and inferred soil profile are shown on Drawing 285380-04-090-WIP1-2903. 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.

⁴ Advanced lab tests (consolidation and consolidated undrained triaxial tests) were carried out in AMEC's Scarborough lab

⁵ American Society for Testing and Materials

Table 3-2: Overburden Thickness and Instrumentation in Boreholes

Borehole	Location	Overburden Thickness, m	Test Name & Elevation					IN
			Rock Coring	Nilcon Vane	S-Piez.	VWP	MSG	
BH T9-1	N4678634.8, E333768.4	32.3	151.7 to 149.1	177.0 to 162.5		174.9, 151.4	174.9, 161.0	149.1*
BH/DMT T9-1	N4678544.5, E333900.9	4.3 (BTWO)						
BH/NIL T9-2	N4678593.7, E333893.5	6.6 (BTWO)		177.0 to 162.0				
BH CV3-1	N4678630.0, E333861.1	9.8 (BTWO)						
BH TB7-1	N4678671.8, E333831.4	10.1 (BTWO)						
BH TB7-2	N4678662.3, E333859.6	10.1 (BTWO)						
BH TB7-3	N4678644.6, E333911.0	10.1 (BTWO)						
BH TB7-4	N4678619.4, E333980.0	10.1 (BTWO)						
BH 15-RW	N4678559.2, E333806.1	6.6 (BTWO)						
BH/CPT 45-RW	N4678688.3, E333708.0	3.0 (BTWO)						
BH/CPT 46-RW	N4678505.0, E333977.6	3.0 (BTWO)						
BH/CPT 114	N4678526.7, E334018.6	1.8 (BTWO)						
BH 115/115A	N4678585.3, E333911.1	32.3	151.5 to 146.2		146.2	173.0		
BH 116/116A	N4678634.3, E333722.5	32.0	151.7 to 147.6		152.0	174.6		

Legend: S-Piez. Standpipe Piezometer (Screen elevations)
VWP Vibrating Wire Piezometer (Sensor elevations)
MSG Spider Magnet Heave/Settlement Gauge
IN Inclinator Casing
BTWO Borehole Terminated within the Overburden
* Inclinator bottom elevation

Note: Location coordinates and elevations are in UTM-NAD 83 (Zone 17) and geodetic datum

3.3 Geotechnical and Analytical Laboratory Testing

All recovered soil samples and rock cores were examined in the field and the 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, 2 representative soil samples were selected for advanced tests (1 consolidated-undrained triaxial compression test and 1 one-dimensional consolidation test).

Selected samples of the silty clay and silt samples obtained from boreholes were sent to the ALS Environmental Analytical Laboratory in London, Ontario to determine the pH, redox potential, resistivity, sulphide and sulphate content of the soil to assess corrosion potential.

The results of geotechnical and analytical laboratory tests are included in Appendices C and D, respectively. Some of the laboratory test results (e.g., geotechnical index properties) are indicated on the borehole logs.

3.4 Instrumentation

Geotechnical instruments (standpipe piezometers – S-Piez., vibrating wire piezometers – VWP, spider magnets heave/settlement gauges – MHSG and inclinometer casings – IN) 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.

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

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

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

Inclinometers: An inclinometer casing was installed in Borehole T9-1. The purpose of this device is to measure the lateral ground movement at the installed location. The bottom end of the casing was anchored approximately 2.6 m into bedrock (elevation 151.7 to 149.1), and the annular space around the casing was filled with bentonite-cement grout. The inclinometer comprised 70 mm diameter RST “Snap Seal Inclinometer Casing”, and probe is IC32005 MEMS digital inclinometer system (0.5 m long).

Proper future decommissioning of the instrumentation holes is responsibility of WEMG/PIC.

3.5 Data Interpretation

Field Vane Test Data Correction: The chart (Figure 3.1⁶) developed initially by Bjerrum (1972) and updated subsequently by Ladd et al (1977) based on circular arc failure analyses of embankment failures

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

suggest correction by multiplying the field vane data by 1.05 to 1.10 for soils with plasticity index of about 15 (ref. R-6 and R-32). 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, and Figure 3.2). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI. Interestingly, at this particular site the undrained shear strength (S_u) profiles inferred from the DMT and the S_u values obtained from the conventional field vane tests in boreholes were consistently higher than the Nilcon vane test values.

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

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

Where:

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

Q_t is the corrected total cone tip resistance;

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

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

The CPT based S_u profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles. In this regard, the N_{kt} factor values used to calibrate the CPT strength profiles varied slightly for different segments of the WEP and the soil strata. 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, the undrained shear strength was estimated from the excess pore pressures (using the N_u method). Figure 3.3 presents the undrained shear strength profiles for WEP segment between Sta. 12+000L and Sta. 12+800L, and shows that the estimated undrained shear strength profile using the CPT data and measured shear strength profile from Nilcon vane tests are in good agreement.

Pre-Consolidation Pressures from Cone Penetration Tests: The approach used for estimating the pre-consolidation pressures from the estimated S_u profiles follows the Stress History and Normalized Soil Engineering Properties (SHANSEP) method developed at MIT (Ladd and Foott, 1974, ref. R-31). The following relationship was used to compute the pre-consolidation pressures:

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

Where:

S_u is the undrained shear strength;

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

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

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

OCR is the overconsolidation ratio; and

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

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

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

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 $S = 0.18$ and K_d is the horizontal stress index represented by:

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

Where:

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

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

The constant 0.18 for S_u/σ'_{vo} for $OCR=1$ is based on average plasticity index of the silty clay to clayey silt stratum and the 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, CPT and DMT carried out between Sta. 12+000L and 12+800L are presented in Figure 3.3. Also included on these figures are $0.18 \times \sigma'_{vo}$ curve (representing undrained strength profile 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; an extensive cohesive clayey silt to silty clay (referred to hereafter as “silty clay”) deposit below about elevation 184, and a lower granular deposit overlying limestone and dolostone bedrock below about elevation 151.7. The thickness of the Clayey Silt to Silty Clay deposit varies between about 27.2 m and 31.7 m. The lower granular deposit (sandy silt / silty sand / sand and gravel) was encountered at two of the three boreholes advanced to sufficient depth and was found to be 0.3 m and 4.7 m thick at BH T9-1 and BH 115, respectively. The bedrock was encountered at depths ranging from about 32.0 m to 32.3 m below the ground surface.

4.1 Surficial Fills, Topsoil and Upper Granular Deposit

All boreholes, except Boreholes T9-1, CV3-1, TB7-3, and 15-RW encountered up to 0.6 m thick layer of brown to black topsoil. The thickness of the topsoil is expected to vary through the project area.

Borehole T9-1 was advanced through about 1.0 m of pavement materials (280 mm of asphaltic concrete over about 0.7 m of grey silty sand and gravel). Boreholes CV3-1, TB7-3, and 15-RW encountered 0.9 to 1.5 m of clayey silt and topsoil fill.

No distinct native granular deposits were observed in the boreholes at the site of this particular structure. However, based on the experience in the general project alignment, local occurrence of native silts and fine sands may be possible.

4.2 Silty Clay Stratum

The cohesive silty clay stratum was encountered directly underlying the surficial topsoil or fill/granular deposit. The encountered depth below existing ground surface was from 0.2 to 1.5 m. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into 5 layers as follows: brown desiccated stiff to very stiff clay crust, transition zone, upper grey silty clay deposit, a mid silty clay deposit, and then a lower coarser grey silty clay deposit. In the vicinity of Tunnel T-9, the silty clay deposit has numerous sand and silt layers of variable thickness occurring at random depths. The natural water content, Atterberg limits and bulk unit weights determined on the samples of the clay sub-strata recovered during the pre-bid and additional geotechnical investigation are summarized in Table 4-1.

Table 4-1: Summary of Index Properties of the Silty Clay Stratum

Property	Clay Crust	Clay Transition	Upper Silty Clay	Mid Silty Clay	Lower Silty Clay
Elevation Range, m	184 ¹ to 178	178 to 175	175 to 166	166 to 163	163 to 154
Natural Water Content, w_N , %	2.3 – 22.9	10.0 – 18.0	11.3 – 37.5	14.3 – 34	9.9 – 26.0
Liquid Limit, w_L	22.6 – 25.9	23.0 – 25.0	24.5 ² – 36.6	23.2	27.4 – 33.2
Plastic Limit, w_P	13.4 – 15.0	13.0	12.0 ² – 19.4	14.0	15.4 – 17.2
Plasticity Index, PI	9.2 – 10.9	10.0 – 12.0	11.1 ² – 17.2	9.2	12.0 – 16.0
Liquidity Index, LI	(-) 0.34 – 0.01	0.20 – 0.23	0.28 – 1.05	0.03	(-) 0.04 – 0.44
Unit Weight, γ , kN/m ³	N/A	21.6	21.0 – 21.5	N/A	21.8

1 - Elevation varies

2 – Out of range measured property on a single sample has been excluded.

The undrained shear strength (S_u) profiles of the stratum between Sta. 12+000L and Sta. 12+800L and at the Tunnel T-9 site are illustrated on Figures 3.3 and 5.1, respectively.

As illustrated on Figure 5.1, the measured undrained shear strength of the clay stratum varied with depth generally as follows:

- Crust layer: > 100 kPa
- Transition layer: 80±20 kPa to 60±10 kPa
- Upper silty clay: 60±10 kPa
- Mid silty clay: 60±10 kPa to 70±10 kPa
- Lower clayey silt: >80±10 kPa.

The stress-strain properties and the effective shear strength properties of the silty clay deposit were based on test results from the pre-bid geotechnical investigations (ref. R-16, R-17, R-18 and R-19) and the one-dimensional consolidation tests, triaxial shear tests and direct shear tests performed during the additional geotechnical investigation described in Section 3.1. These interpreted trends are supported by published correlations in the literature (Kulhawy and Mayne, 1990, ref. R-30, Leroueil et al., 2001, ref. R-34 and Terzaghi et al., ref. R-42).

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 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 Tunnel T-9 site are summarized in Table 4.2.

Table 4-2: Summary of Interpreted Compressibility Properties

Property	Clay Crust	Clay Transition	Upper Silty Clay	Mid Silty Clay	Lower Silty Clay
Average Natural Water Content, w_N , %	14	16	20	24	20
Virgin Compression Index, C_c	0.112	0.129	0.163	0.198	0.163
Recompression Index, C_r	0.0123	0.0142	0.0180	0.0218	0.0180
Swelling Index, C_s	0.0280	0.0322	0.0409	0.0495	0.0409
Secondary Compression Index, C_α	0.0031	0.0036	0.0046	0.0055	0.0046

An oedometer test was carried out on a grey clayey silt sample obtained from Borehole T9-1 at a depth of 15.2 m below ground surface (sample at elevation 168.8 with w_N , w_L and PI values of 20.0%, 29 and 17, respectively) indicated the following compressibility indices: $C_c = 0.116$, $C_r = 0.036$ and $C_s = 0.023$.

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 geotechnical investigation and supported by published PI versus ϕ' relationships (ref. R-34 and R-42). These strengths parameters are summarized as follows (Figures 4.3 and 4.4):

Apparent cohesion, c'	0 kPa
Angle of internal friction, ϕ	30°
Friction angle at critical state ⁷ , Φ_c ,	25°-26°

A Consolidated Isotropic Undrained Triaxial Compression (CIUC) test was carried out on a clayey silt sample from Borehole T9-1 at a depth of 15.2 m below ground surface indicates an effective friction angle of 32 degrees.

The modulus of elasticity has been correlated with the undrained shear strength of the material, published information (ref. R-42) and local experience (ref. R-19) as follows:

$$\text{Elastic Modulus (Undrained), } E_u = 300 S_u$$

$$\text{Elastic Modulus (Drained), } E' = 0.9E_u$$

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.

⁷ Based on triaxial tests (ref. R-18).

Table 4-3: Summary of Interpreted Elastic Properties of the Soils

Soils Stratigraphy	Elastic Modulus - Undrained, MPa	Poisson's Ratio – Undrained (*)	Elastic Modulus - Drained, MPa	Poisson's Ratio – Drained (*)
Clay Crust	35	0.49	31	0.35
Clay Transition	20		18	
Upper Grey Silty Clay	16		14	
Mid Grey Silty Clay	17		15	
Lower Grey Silty Clay	35		31	

(*) Assumed values (ref. R-42).

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, a discontinuous and heterogeneous non-cohesive material deposit (varying from silty sand, to sand and gravel, and clayey silts with sand) was encountered. Based on the Standard Penetration Test (SPT) “N” values ranging from 30 to 48, this material is considered to be in a compact state of compactness. This layer was approximately 0 to 4.7 m thick. The thickness of the layer and state of compactness may vary significantly throughout the project area.

4.4 Bedrock

Where rock coring was undertaken, a white to grey, limestone bedrock was encountered. The bedrock was generally fresh, medium strong, laminated to thinly laminated, fine grained, faintly to highly porous and highly fractured. Bedrock was encountered at elevations ranging from 151.5 to 151.7 in the vicinity of Tunnel T-9. The Rock Quality Designation (RQD) of the recovered rock cores varied on average between 70 to 100 per cent, indicating a fair to excellent quality. Rock quality generally improves with depth. Two samples of the rock core obtained from the Borehole T9-1 were tested in compression to failure. The samples had unconfined compressive strengths of 61.0 and 63.3 MPa. Unconfined compressive strength of two rock cores taken from Boreholes 115 at a depth of 37.5 m and Borehole 116 at 33.0 m was 26.5 MPa and 24.8 MPa, respectively. Photograph of rock core recovered from the additional investigation is provided in Appendix E.

Based on this core logging the rock mass classification was estimated to range from 3.8 to 5.5 for the Q-System (Barton *et. al.*, 1974, ref. R-3) and 51 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.

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. A total of 12 samples were included

for density and unit weight, while 16 were included for unconfined compressive strength. The average strength of the limestone is determined to be 85.5 MPa and is 'strong rock' based on the ISRM (1978, ref. R-28). Additionally, based on the coefficient of variation, enough tests have been performed to characterise the compressive strength.

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

	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

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

4.5 Groundwater Conditions

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

The piezometric water levels within the overburden and the bedrock were observed at about elevations 182.2 to 184.1 and 177.4 to 177.8, respectively (Table 4-5). These observations suggest a slight downward gradient between the overburden and the bedrock. However, based on experience in the project area, localized occurrences of artesian conditions in bedrock cannot be ruled out. The water levels inferred from the shallow piezometer at Borehole T9-1 are noted to be about equal to the ground surface.

Table 4-5: Summary of Measured Water Levels

Borehole	Surface El., m	Piezo. Type	Screen / Sensor El., m	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	El., m
BH T9-1	184.0	VWP	174.9	Silty Clay	Aug. 6, 2011	184.0
		VWP	151.4	Limestone	Aug. 29, 2011	184.1
		VWP	151.4	Limestone	Aug. 6, 2011	177.6
		VWP	151.4	Limestone	Aug. 29, 2011	177.8
BH 115/115A	183.8	S-Piez	146.2	Limestone	Feb. 21, 2008	178.0
		S-Piez	146.2	Limestone	Mar. 20, 2008	178.1
		S-Piez	146.2	Limestone	July 24, 2008	177.7
		S-Piez	146.2	Limestone	Sept. 19, 2008	176.0
		S-Piez	146.2	Limestone	Nov. 14, 2008	177.2
		S-Piez	146.2	Limestone	Jan. 28, 2009	177.4
		VWP	173.0	Sand lens in Upper Silty Clay	Mar. 20, 2008	182.4
		VWP	173.0	Sand lens in Upper Silty Clay	July 24, 2008	182.3
		VWP	173.0	Sand lens in Upper Silty Clay	Sept. 19, 2008	182.3
		VWP	173.0	Sand lens in Upper Silty Clay	Jan. 28, 2009	182.2
BH 116/116A	183.6	S-Piez	152.0	Just above Limestone	July 22, 2008	178.0
		S-Piez	152.0	Just above Limestone	Aug. 11, 2008	176.7
		S-Piez	152.0	Just above Limestone	Sept. 19, 2008	176.1
		S-Piez	152.0	Just above Limestone	Nov. 14, 2008	177.3
		S-Piez	152.0	Just above Limestone	Jan. 28, 2009	177.5
		VWP	174.6	Silty Clay	Mar. 20, 2008	182.6
		VWP	174.6	Silty Clay	July 22, 2008	182.8
		VWP	174.6	Silty Clay	Aug. 11, 2008	182.6
		VWP	174.6	Silty Clay	Sept. 19, 2008	182.6
		VWP	174.6	Silty Clay	Jan. 28, 2009	182.7

Legend: S-Piez. Standpipe Piezometer
VWP Vibrating Wire Piezometer

Perched groundwater is known to accumulate seasonally within the upper deposits of fill, topsoil and granular layers, and within the fissures in the silty clay crust. In 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 on the order of 0.5 mg/L and can be corrosive at concentrations of about 2 mg/L to 3 mg/L in the groundwater. The gas odour was not detected during the drilling at the Tunnel T-9 site.

Although the presence of the H_2S and CH_4 gases was not observed during the 2011 geotechnical investigation at Tunnel T-9 site, their presence cannot be ruled out. Pumping tests (named TOW-1, TOW-2 and TOW-3) were conducted in the field during hydrogeological assessment at three locations across the proposed parkway to determine concentration levels of hydrogen sulphide gas in the groundwater of the area. Of these tests, TOW-3, located northwest of Tunnel T-9, indicated a

concentration of 7.0 mg/L of H₂S gas and at TOW-1, located southeast of Tunnel T-9, the presence of H₂S gas was not detected. As Tunnel T-9 is located between TOW-1 and TOW-2, there is a possibility that H₂S gas may be present in the Tunnel T-9 area.

The understanding of the engineering behaviour (related to the impact on design and construction) of the gassy soils is rather limited. In the case of low permeability cohesive soils it is known that these soils may experience rapid drop in undrained shear strength during unloading. Due to the relatively high compressibility of the pore water fluid in gassy soils, the immediate pore water pressure response (ΔU) to total stress changes can be very low. This phenomena leads to reduction in effective stress and hence shear strength (ref. R-25 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).

5 Development of Geotechnical Design

5.1 Tunnel Configuration

Tunnel T-9 (Cousineau Tunnel) will be constructed along the below-grade section of the WEP between Sta. 12+130L and Sta. 12+300L, and will accommodate the below-grade traffic of Highway 401 (Drawing 285380-03-060-WIP1-2901). The proposed Tunnel T-9 is 170 m long and 40.6 m wide.

As shown on Drawing 285380-03-060-WIP1-2901, Tunnel T-9 is a two-span deck-on-girder structure incorporating semi-integral abutments and centre pier. Deck elevations were estimated using the elevation of WP #2 and calculated for the selected design section locations using the grades shown on Drawing 285380-03-060-WIP1-2901 (dated July 31, 2012). The abutments consist of 1.7 m wide \times 1.5 high pile cap founded on deep end-bearing HP 310 \times 110 steel piles. The centre pier include 3.2 m wide by 1.25 m high pile cap supported on batter piles (1H:6V) as shown on Drawing 285380-03-061-WIP1-2905.

The design incorporates an RSS (Reinforced Soil System) wall (false abutment) with various sections of approved regular backfill, granular backfill and EPS have been developed as illustrated in Appendix I. The false abutments will be founded on reinforced Granular Mat (RGM), which in turn, should be installed over undisturbed native silty clay subgrade. Table 5-1 provides a summary of control elevations at the tunnel abutments used for the geotechnical design development.

Table 5-1: Summary of Interpreted Elevations at Abutments

Location	Existing Ground Surface Elevation*	Top of Finished Grade Elevation**	Top of Deck Elevation	Top of RSS Abutment Wall Elevation	Pavement Subgrade Elevation*
Centerline Tunnel & Hwy # 401 (WP#2)	184.0	186.4	185.4	-	177.0
North Wall – West	184.0	186.6	185.6	182.2	177.5
North Wall – Centre	184.0	186.2	185.2	181.7	177.0
South Wall – West	184.0	187.0	186.0	182.6	177.5
South Wall – Centre	184.0	186.6	185.6	182.1	177.0

(*) Indicates elevations as interpreted from highways drawing sections.

(**) Top of finished grade assumed to be 1 m above deck elevation.

Notes: 1-Top of deck elevations at the North Abutment (West and Centre) and South Abutment (West and Centre) were calculated based on the top of deck elevation of 185.4 at WP #2 and grades of 0.5% parallel and 1.0% perpendicular to the tunnel alignment as shown in Drawing 285380-03-060-WIP1-2901.

2-Top of RSS abutment wall elevations were interpreted from structural drawings to be 1.5 m below top of pile cap.

3-The wing walls will comprise RSS return walls and RSS portal walls extending beyond the tunnel portals

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 have been developed as per the principles of Limit States Design (LS Method) based on Load and Resistance Factors (CFEM, ref. R-8 and CHBDC, R-9).

WS Method was employed for global stability of the earthworks and the soil mass containing earth retaining structures and the external stability (bearing, sliding, and overturning) of the RSS structures. The stability of the soil mass containing the false abutments and return walls is checked for all potential surfaces of sliding. It is anticipated that the existing Wolfe Drain on the north side of the tunnel will appropriately be filled up and realigned before the construction of the tunnel.

Tunnel T-9 construction is expected to involve the following sequence of earthwork, design elements and loading stages:

- Temporary excavation to about 8.0 and 8.5 m depths below existing grade;
- Installation of a 1.5 m thick Reinforced Granular Mats (RGM) foundation at the north and south abutments (void forms may be incorporated within the RGM to accommodate later pile installation through RGM);
- Temporary trenches along the pier;
- Installation of piles (HP310×110) for all tunnel supports;
- Installation of 500 mm diameter CSP around the abutment pile stickup filled up with concrete (the CSP concreting assumed to take place after completion of the RSS);
- Construction of the false abutments comprising RSS structures including associated permanent subdrainage works, and approved backfill behind the RSS structure;
- Construction of the pile caps, abutment stubs, piers and tunnel deck;
- Completion of final stage of backfill behind the semi-integral abutments (specific requirements for construction staging involving partial completion of subbase and base materials over Highway 401 subgrade are being considered);
- Completion of the final topsoil placement and road materials where applicable; and
- Completion of the pavement for Highway 401.

5.3 Design Soil Properties

The design soil properties for the silty clay to clayey silt deposit were interpreted from the CPT and Nilcon vane test and DMT profiles (listed in Table 3-1) and the laboratory test results. The undrained shear strength (S_u) and preconsolidation pressure (σ'_p) profiles were estimated from the Nilcon vane tests and the CPT based on the calibration described in Section 3.5. The S_u and σ'_p profiles inferred from the CPT, DMT and Nilcon tests advanced at and around Tunnel T-9 and the design values obtained from these profiles are shown in Figure 5.1 and summarized hereafter in Table 5-2. As indicated in Figure 5.1 and Table 5-2, the undrained shear strength of the silty clay stratum in the north part of the tunnel was slightly lower than the general trend in the area.

Table 5-2: Summary of Interpreted Design Clay Strength and Consolidation History

Clay Substratum	Elevation Range	Undrained Shear Strength (Su), kPa		Effective Strength Parameters	Preconsolidation Pressure (σ_p'), kPa	OCR
		North Side	South Side			
Clay Crust	184 ^(*) to 178	75 ^(**)	75 ^(**)	$c' = 0, \phi = 30^\circ$	600	>4
Clay Transition	178 to 175	75 to 50	75 to 55	$c' = 0, \phi = 30^\circ$	600 to 300	3
Upper Grey Silty Clay	175 to 166	50	55 to 50	$c' = 0, \phi = 30^\circ$	300	1.5
Mid Grey Silty Clay	166 to 163	50 to 60	50 to 60	$c' = 0, \phi = 30^\circ$	300 to 335	1.2
Lower Grey Silty Clay	163 to 155	60 to 100	60 to 100	$c' = 0, \phi = 30^\circ$	325 to 500	1.5

(*) Elevation varies

(**) Crust strength values used in global stability only

c' = Cohesion intercept

ϕ° = Effective Angle of Internal Friction (ϕ)°

OCR = Over Consolidation Ratio.

As indicated in the above table and illustrated in Figure 5.1, the undrained shear strength estimated for the upper portion of the Grey Silty Clay layer (from NIL T9-2 and CPT 114) for the north side were lower than the south side of the Tunnel T-9 site, and as a result a different shear strength profile was adopted for designing the north and south side. It should be noted that pockets or zones of the weaker silty clay encountered at the north side of the tunnel could exist elsewhere at and around the tunnel site.

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

Table 5-3: Summary of Other Interpreted Design Parameters

Clay Substratum	Horizontal Permeability, cm/sec	Anisotropy ratio, k_h/k_v	Initial Void Ratio, e_0
Clay Crust	6.8×10^{-7}	1	0.38
Clay Transition	3.9×10^{-7}	2	0.43
Upper Grey Silty Clay	1.1×10^{-7}		0.54
Mid Grey Silty Clay	1.1×10^{-7}		0.65
Lower Grey Silty Clay	1.1×10^{-7}		0.54
Lower Granular	1.2×10^{-5}	1	0.46

For design purposes the long-term groundwater levels in the overburden were considered at elevation 183.5 for north side to simulate pond water in Wolfe drain and 182 for south side.

5.4 Excavation and Temporary Cut Slopes

The discussion of the temporary slopes in this report relates only to the anticipated subsurface conditions to assist the designer of temporary works. 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 the design.

Excavations (including sub-excavation for Highway 401 pavement and RGM) are expected to encounter surficial fills, topsoil and possibly water bearing granular soils and will be extended 8.0 and 8.5 m below existing grade to about elevation 175.5 and 176.0 into the native silty clay for the north and south abutments. Pier excavation will be extended to the elevation ranging from 175.5 to 176.0 in the native silty clay. The approximate excavation profile for this structure is shown in Figure 5.2 which was developed on the basis of the roadway cross sections at Highway 401 Stations 12+130L, 12+215L and 12+300L.

Basal hydrostatic uplift was calculated based on the highest measured water level in the bedrock (178.1), anticipated deepest excavation depth (RGM base and bottom of the pile cap at elevation 175.5), and the weight of the silt-clay layer thickness of 19.3 m below the deepest excavation. The factor of safety (FS) against hydrostatic uplift was 1.9. The water level in the piezometers installed in selected boreholes advanced for this structure should be measured on regular basis and based on the results obtain, the basal uplift hydrostatic pressure should be verified.

As described in Section 4.6, presence of gassy soils near 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. Considering this risk, and the effects of the significant soil stress relief by deep excavations, it is recommended that in the case of excavations deeper than 5 m careful monitoring of basal heave and pore water pressures below of the bottom of the excavations be carried out during construction. Adequate number of heave gauges and low-displacement type piezometers 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 Resistance to Axial Loads

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

The piles are expected to be driven to bedrock as per OPSS 903 and accordingly they will mobilize aUltimate 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 is determined by the elastic characteristics of the pile shaft.

Based on the available borehole data at this structure, the bedrock surface elevation varies between 151.5 and 151.7, 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 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 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.
- Adequate hammers should be used to ensure the mobilization of the design ultimate geotechnical resistance and prevent damages to the piles during driving.
- 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 deformations.

The SLS geotechnical resistance to lateral loads is dependent on the acceptable levels of the lateral pile deflections under the design loads and should be obtained on the basis of field load tests. In the absence of field tests, the preliminary design may be based on a conventional SLS resistance of 70 kN along the strong axis, and 50 kN along the weak axis of the HP310x110. This conventional SLS resistance represents the lateral shear force applied on a free-head pile 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 unstabilised pile displacements due to soil failure or pile structural failure. In the absence of field tests, the ULS lateral resistance may be assumed as 220 kN, and 105 kN along the strong axis and weak axis, respectively.

The above estimates were based on a pile model assumed to be embedded within stiff silty clay below elevation 176. The above resistances were estimated using the “p-y” model (LPile 5.0 model Ensoft 2010). The “p-y” curves were generated using the Reese method described in the Technical manual for LPILE, using the Reese “Stiff-Clay without free water” model in conjunction with the following soil parameters described in Tables 5-4 and 5-5. 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.

Table 5-4: Soil Parameters for Pile Interaction Assessment

Soils Around the Piles	Elevation	Design Bulk Unit Weight (kN/m ³)	Undrained Shear Strength, S_u (kPa)	ϵ_{50}
Clay Crust	184 to 178	22	75	0.005
Clay Transition	178 to 175	22	75 to 50	0.005 to 0.007
Upper Grey Silty Clay	175 to 166	20.5	50	0.010
Mid Grey Silty Clay	166 to 163	20.5	50 to 60	0.010 to 0.007
Lower Grey Silty Clay	163 to 152	20.5	60 to 100	0.007

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} .

Table 5-5: Fill Properties for Pile Interaction Assessment

Material	Soil Model in L-Pile	Effective Unit Weight, kN/m ³	ϕ°	n_h , MPa/m
RSS Granular Fill (*)	Sand (Reese)	21	35	10
RGM Granular(**)	Sand (Reese)	21	30	2

(*)The RSS suppliers should be informed and consulted on the impacts from the anticipated loads transferred to the RSS fill and facing by the deflecting piles.

(**) Assumed loose granular around the pile shaft through RGM to account for anticipated void forms as indicated in Section 5.2.

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 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 significant reduction of the actual SLS and ULS resistances. Where the design relies on the lateral resistance provided by the

soils, “repairs” to the disturbed soils must be undertaken (typically, the voids are grouted using non-shrink fills).

The abutment piles embedded within concrete filled CSP and reinforced RSS fill will develop larger resistances to lateral loads than the conventional SLS and ULS resistances described above.

Significant lateral loads in excess of the preliminary values previously cited should be resisted fully or partially by the use of battered piles. For ease of constructability and to ensure hammer energy sufficient for pile driving, batters are usually limited to no steeper than 1H:5V.

The stress-deformation analysis of the piles to lateral loads may be carried out using one of the following methods.

Horizontal Subgrade Reaction Method:

The coefficient of horizontal subgrade reaction, k_h , may be based on the following equations:

$$k_h = n_h \left(\frac{z}{d} \right) \quad \text{for cohesionless soils, and}$$

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

Where:

k_h (MPa/m)	= Soil modulus of horizontal subgrade reaction
n_h (MPa/m)	= Soil coefficient
S_u (MPa)	= Undrained shear strength
z (m)	= Depth below finished grade
d (m)	= Pile diameter/width

The recommended overburden and fill soil parameters are tabulated in Tables 5-4 and 5-5.

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 may be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor indicated in Table 5-6. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed here.

Table 5-6: Lateral Load Capacity Reduction Factors for Pile Groups using the Horizontal Subgrade Reaction Method

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

d = pile diameter

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

Alternative Nonlinear ‘p-y’ Curve Method:

Alternative pile design methods may 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 Tables 5-4 and 5-5. “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 need 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:6V, the p-y curve modifier will be $B_m = 0.75$ and 1.25 for the batter in the direction of the lateral load, and opposite direction of the lateral load, respectively.

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

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

where :

β_{ki} = the influence factor of pile ‘k’ in the group on pile ‘i’, with $k \neq i$, and is calculated with one of the following expressions depending on the relative position of pile ‘k’ in the group with respect to pile ‘i’ (Table 5-7).

Table 5-7: Lateral Load Capacity Reduction Factor For Pile Groups using Nonlinear ‘p-y’ Curve 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 pile in Line (first pile in line parallel to the load direction)	≤ 4	$0.70(s/d)^{0.26} \leq 1$
Trailing piles in line (piles behind the leading pile)	≤ 7	$0.48(s/d)^{0.38} \leq 1$

The space between the piles under the abutments is approximately 2.38 m (Drawing 285380-03-061-WIP1-2905) appropriate reduction factors should be based on individual abutment pile configurations. At the piers, a closer spacing of approximately 2.2 m is anticipated (Drawing 285380-03-061-WIP1-2905). Group reduction factors will apply for lateral pile loadings.

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

5.5.3 Soil Pile Interaction Assessment

Downdrag Loads (Negative Skin Friction – NSF):

Potential for downdrag loads on piles was considered in conjunction with the anticipated ground movements (rebound and settlements) that are assumed to occur during and following excavation of the overburden of up to 8.5 m to accommodate the future depressed highways, followed by partial replacement of RSS wall and backfills to construct the tunnel abutments.

Soil stress-deformation analyses described later 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 at representative stages, namely, after RSS completion, after completion of the top backfill against the tunnel diaphragm (End of Construction - EC) and in long-term (LT), and the associated vertical total stresses, pore water pressures and vertical effective stresses are presented in Figures G.12 and G.16 in Appendix G. The analyses indicated the following:

- Ground settlements are expected to occur along the pile shaft during construction of the RSS wall, tunnel and completion of the associated backfill; and
- Ground rebound is expected to occur after 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 following unfactored downdrag loads were determined for the abutment piles:

- Maximum transient downdrag of 750 kN is estimated to develop during completion of the backfilling at the abutments; and
- Residual (long-term) downdrag of less than 300 kN is estimated to develop after the completion of the ground settlements below the false abutments.

Considering the staging of the construction whereby the topsoil above the tunnel deck is placed after substantial completion of the abutments and finished grades along the tunnel edges, the pile design with respect to the downdrag loads should consider the following load combinations:

- a) Maximum transient downdrag plus structural dead loads (tunnel deck) only; and
- b) Residual downdrag plus total dead load (tunnel deck plus topsoil).

The completion of the tangible portion of the abutment settlements was estimated to occur after about 300 to 500 days after completion of the abutment backfilling.

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

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

No downdrag is anticipated at the pier piles.

Pile Shaft Bending:

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

- The pile was modelled with a 500 mm diameter collar section (CSP pipe filled with concrete around the pile shaft within the RSS wall). Below the RSS wall, the pile section was HP section. The arrangements of pile cap, pile and RSS wall are shown on Drawings 285380-03-060-WIP1-2901 and 285380-03-060-WIP1-2905, and Figure I.1.
- The ground lateral movement (Figures G.17 and G.18) along the pile shaft anticipated to occur after the installation of the piles was estimated using the stress-deformation analysis described below 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). The “p-y” curves were generated using the Reese method described in the Technical manual for LPILE, using the soil parameters indicated in Tables 5-4 and 5-5.
- Due to the strip reinforcement of the pile cap backfill 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 along the pile shaft were calculated using the LPILE software.

Based on the above approach and the anticipated lateral ground displacement, the estimated maximum unfactored bending moment in the shaft was 77 kN-m for the strong axis pile loadings. The shear force diagram indicated that the maximum shear force transferred by the pile shaft to the surrounding RSS wall was 52 kN. The calculated maximum pile deflection at the underside of the RSS wall base was 14 mm. These results should be considered in the structural design of the piles and in the design of RSS structural components. These bending moments, shear forces and deflections are in addition to those caused by bridge loads applied to the piles.

The maximum computed moment in the pile under assumed pile head load equal to the conventional SLS resistance (70 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 157 kN-m for the strong axis pile loadings, which is less than the yield moment of the pile.

Time Effects on Batter Piles:

The time-effects of the ground movement on batter piles were examined in a similar approach described above for the pile shaft bending due to lateral soil movement. The depth profiles of vertical ground movement along the pile shaft and different time phases were determined using the stress-deformation analysis. The component of the vertical movement acting perpendicular to the pile shaft was determined depending on the batter, and was imposed as a field-deformation load type of on the pile shaft.

The maximum bending moments caused by ground movement on batter piles were calculated to be 19 and 16 kN-m for the strong and weak axis, respectively.

5.5.4 Pile Cap/Abutment Stem Anchoring

It is understood that anchoring of the abutment stem within the backfill above the RSS wall is intended using embedded soils reinforcement connected to the pile cap. The detailed design of the anchoring is to be provided by the supplier of the reinforcement. The following is a brief outline of the geotechnical aspects specific to the abutment stem anchoring presented in this report.

The soil material for the reinforced soil zone for pile cap / abutment stem anchoring should be an approved high quality granular fill compatible with the reinforcing materials and meeting also the PA requirements. In the absence of specifications from the supplier, a well graded free-draining crushed granular material meeting the specifications of Granular B Type II containing less than 5% fines (SP110S13) should be considered. The design properties associated with such material compacted to >98% of Standard Proctor Maximum Dry Density to be considered in the reinforced soil zone are:

Unit weight:	21.5 kN/m ³
Friction Angle (Φ):	35 ⁰
K _a :	0.27

The lateral earth pressure, p_h , against the pile cap may be estimated using the expressions:

$$p_h = K_r K_a \sigma_v + \Delta \sigma_H \quad (\text{ref. R-39})$$

where:

- σ_v vertical stress at the point of calculation including the effects of the dead loads and applicable live loads
- $\Delta \sigma_H$ supplemental horizontal pressures from external lateral forces (if present, such as shear force at the bottom of footings resting on top of reinforced zone)
- K_a active earth pressure coefficient
- K_r correction factor varying from 1.2 to 2.5 depending on the type of reinforcement (extensible like geosynthetics, or inextensible like metal strips or metal bar mats & welded wire grids), and depth of calculation section

The backfill above the reinforced zone could be any approved general fill. For the purpose of calculation of the effective vertical stress, the following unit weights should be used for the fills above the reinforced zone:

Regular Backfill:	21 kN/m ³
EPS:	0.5 kN/m ³

The detailed design of the abutment will vary along the tunnels and as such, significant variations in the makeup of the fill above the reinforced zone should be anticipated. In addition, consideration should be given to the possibility that temporary removal of the upper fills may occur at times, during the life span of the facility.

Based on the above, and in conjunction with the proposed abutment configuration, the following unfactored lateral earth pressure loads were estimated:

ELL = 9 kN/m	(earth pressure from Live Loads (LL=16 kPa) on Highway 3)
EDS = 34 kN/m	(earth pressure from 3 m of Dead Surcharge load including EPS above the pile cap)
EB = 9 kN/m	(earth pressure due to backfill behind the pile cap).

Lateral load from the thermal expansion / shrinkage should also be considered as necessary.

The internal design for the reinforcing strips should be carried out by the supplier of the reinforced soil structures.

5.6 RSS False Abutment Walls

Geotechnical design configurations (typical arrangements) for Tunnel 9 North and South abutments were developed based on the global stability and foundation bearing considerations. The proposed configurations are shown in Appendix I.

The abutments comprise retained soil structure (RSS) founded on the reinforced granular mat (RGM), and conventional engineered backfill as well as expanded polystyrene (EPS). The configurations and preliminary dimensions were developed at representative sections along the tunnels to verify the geotechnical design requirements with respect to (a) the global stability of the soil mass containing the structure (b) the anticipated deformations, and (c) the foundation soil bearing resistances based upon the principles of Working Stress Design.

The design assessments were based on (a) assumed strength and deformation properties of the proprietary components (RSS, RGM and EPS), which will have to be confirmed by proprietary suppliers, and (b) the assumed external loads and backfill properties. The final design of the abutment may require adjustments based on the proprietary components and structural design. In general, the RSS wall is to be designed and constructed in accordance with MTO's RSS Design Guidelines and Special Provisions SP599S22 and SP599S23. All RSS walls will be selected and designed from the MTO Designated Sources for Materials (DSM) list for Retained Soil System. As noted earlier, the RSS suppliers should be informed and consulted on the impacts from the anticipated loads transferred to the RSS fill and facing by the deflecting piles.

The RGM foundation is to be installed on intact subgrade or prepared foundation (avoiding disturbance of the excavations due to construction activities, groundwater inflow, etc., and appropriately protected immediately after excavation to final grade).

The properties of the proprietary products and backfill materials assumed in the geotechnical analyses are summarized in Tables 5-8 and 5-9.

Table 5-8: Assumed Proprietary Product Properties

Material	Unit weight, kN/m^3	Limit Equilibrium Analyses (Slope/W Models)		Stress Deformation Analyses (Sigma/W Models)	
		Friction Angle, $^\circ$	Apparent Cohesion, kPa	Modulus of Elasticity, E, MPa	Poisson's ratio, μ
RSS with Approved Granular Fill	21	35	50	60	0.35
RGM	21	35	50	60	0.35
EPS	0.5	0	10	10	0.20

Table 5-9: Assumed Backfill Material Properties

Backfill Material	Unit weight, kN/m^3	Undrained Shear Strength, kPa	Drained Angle of Internal Friction*, $^\circ$	Modulus of Elasticity, E, MPa	Poisson's ratio, μ
Compacted Clay Fill	21	50	30	20	0.35
Granular Backfill	21	N/A	30	22.5	0.35

Preliminary dimensions of the abutments are listed in Table 5-15.

* $\phi' = 30^\circ$ and $c' = 0$ kPa

5.6.1 Global Stability

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

Figures F.1 to F.12 illustrate the stability models for the north and south abutments. The global stability analyses have been carried out for both short-term (undrained soil properties) and long-term (drained soil properties) loading conditions. The analyses using undrained soil properties were carried out to simulate two loading conditions: (a) the pavement structure over the subgrade at the toe of the slope is not present (temporary undrained condition), and (b) full pavement structure is in place (end of construction condition). The drained analyses assumed that all the components of the system are present. The global stability analyses have been carried out for north and south abutments at representative sections at Stations 12+130L and 12+215L. The presence of the piles was not considered in the stability models (somewhat conservative approach). Live Loads of 12 kPa in the area of highway pavement and 9 kPa in the landscape areas were applied at the top of ground surface for both short-term and long-term models, while tension crack was assumed for short-term only.

As discussed earlier in Section 5.4, the global stability of temporary slopes is the Contractor's responsibilities.

As indicated earlier, the abutment configurations were determined in consideration of the global stability and geotechnical bearing of the false abutments using the applicable soil characteristics and the design undrained strength profiles. The calculated factors of safety (FS) against global instability of the abutments for circular slip surface failure exceed 1.3 and 1.5 for undrained and drained loading conditions, respectively, as shown in Figures F.1 to F.12 and summarized in Table 5-10.

Table 5-10: Summary of the Results of Abutment Slope Stability Analyses

Abutment	Factor of Safety for Loading Condition			Reference Figure
	Temporary Undrained Loading Condition	End of Construction Undrained Loading Condition	Long-term Drained Loading Condition	
North Wall – Sta. 12+130L	1.82 (1.50)	2.09 (1.71)	1.83 (1.72)	F.1, F.2 & F.3
North Wall – Sta. 12+215L	1.44 (1.30)	1.59 (1.40)	1.52 (1.44)	F.4, F.5 & F.6
South Wall – Sta. 12+130L	1.37 (1.22)*	1.49 (1.30)	1.52 (1.44)	F.7, F.8 & F.9
South Wall – Sta. 12+215L	1.36 (1.23)*	1.48 (1.31)	1.53 (1.45)	F.10, F.11 & F.12

Note: Values outside parentheses refer to circular failure surfaces and the values in parentheses refer to non-circular failure surface.

(*) Granular base of Highway 401 must be placed before any backfill is placed above the deck seat level to ensure that FS for all potential slip surfaces meet the PA requirements.

5.6.2 Stress Deformation Analyses

Stress-deformation analyses (SDA) were carried out by finite element modeling using SIGMA/W software Version 2007. The main focus of the SDA was to assess the deformations of the soil mass supporting and surrounding the tunnel structure. As such, the structural elements (deck, girders, pile caps and piles) were not included in the model, albeit their presence was simulated with boundary restraints.

The configuration of the calculation model is presented in Figures G.1 to G.3. The calculation model typically assumed the following loading steps:

- Definition of the initial (in-situ) stress condition for level ground assuming an average bulk unit weight of 21 kN/m³ and an at-rest earth pressure coefficient K_0 of 0.75 for the soil deposit (0 days);
- Bulk excavation to the subgrade level under the highway pavement (30 days duration – day 1 to 30);
- Construction of the RGM and RSS structures, and the associated backfill (20 days duration – day 30 to 50);
- Completion of the remaining fill above the RSS structure (20 days duration – day 50 to 70);
- Completion of the pavement structure for Highway 401 (5 days duration – day 70 to 75); and
- Dissipation of excess pore pressure leading to long-term steady state condition.

The stratigraphy and selection of the soil properties (except for the RSS structure and pavement box) was based on the design soil properties discussed in Section 5.3. The RSS structure, RGM and pavement were assumed to comprise homogeneous elastic materials described in Table 5-9.

The SDA were carried out using an effective stress-based model. The phreatic surface was assumed to correspond to the initial groundwater level at elevation 182.0 and then follow the excavation and subgrade surfaces. Elastic-plastic Mohr-Coulomb models were used for all soil layers except the unweathered firm

to stiff silty clay, which was described by the Modified Cam-Clay model. Hydraulic conductivity properties described in Table 5-3 were assigned to the different soil layers.

The construction stages considered were: excavation, completion of the RGM and RSS, and completion of the entire abutment followed by the placement of the pavement box. The excavation was assumed to occur in 30 days, construction of RGM, RSS structure and associated backfills was assumed to occur in 20 days, completion of backfill above the RSS structure was assumed to occur in 20 days and completion of pavement structure for Highway 401 was assumed to occur rapidly in 5 days. This scenario suggests practically no dissipation of any tangible proportion of the excess pore water pressures generated by the soil unloading/reloading of the listed construction stages. Hence, the state of stress and deformations at the end of the first “75 days” largely correspond to undrained conditions. After the completion of the entire construction, the model is allowed to dissipate the excess pore-pressures over a period of time until a steady-state pore pressure condition is achieved.

The SIGMA model was developed for the south abutment (central segment) where the height of the retained soils measured from the top of finished grade to the bottom of the RSS is 9.5 m high and the Highway 3 section is in the closest proximity to the RSS wall. The south abutment (central segment) model will provide the upper limits for the deformation estimates.

Figures G.1, G.2 and G.3 show the cumulative settlement/heave for the end of excavation (30 days), end of construction (“70 days”) undrained conditions for the tunnel and the long-term (“11,075 days”) drained loading conditions. Figures G.4 and G.5 show the cumulative lateral deformation at the end of excavation and long-term drained loading conditions. Figure G.6 illustrates the stabilized pore water pressure contours at the end of dissipation (long-term) period.

5.6.3 Serviceability Limit States (SLS) Assessment

The SLS performance was assessed on the basis of the SDA described above in Section 5.6.2. The cumulative deformations are summarized in Table 5-12.

The ground movements generated by the construction loads are anticipated to stabilize within approximately 10 to 15 years following completion of construction. Figures G.12 and G.17 show cumulative soil settlement and lateral soil displacement along the pile line. These deformations were estimated from SDA, which were used in pile calculation in Section 5.5. Due to the relatively smooth changes in the geometry of the tunnel, the above settlement changes along Highway 401 are expected to be gradual in the longitudinal profile.

All ground movement and deformations discussed above are estimates based on soil deformation / compressibility properties from laboratory tests and empirical correlations. Therefore, the reported values are approximate and should be considered only as an indication of the magnitude of the soil response. These estimates should be verified and refined with respect to the actual performance monitoring in the field.

The settlements discussed above do not include deformations caused by seasonal temperature and moisture variations and due to the effects of the long-term compression of the backfill materials that are

expected to be nominal. In this regard, stringent compaction control must be exercised to minimize the magnitude of backfill compression.

Table 5-11: Summary of Calculated Cumulative Deformations

Parameter	End of RSS Construction	End of Construction (Undrained)	Long-term (Drained)	Figures
Settlements on Top of Ground at Distances (m) from the Edge of Deck				G.7
0 m†	N/A	-35 mm (*)	-40 mm	
5 m	N/A	-40 mm (*)	-45 mm	
North Edge of Highway 3	N/A	-40 mm (*)	-50 mm	
Center of Highway 3	N/A	-25 mm (*)	-35 mm	
South Edge of Highway 3	N/A	-15 mm (*)	-25 mm	
Settlement at the top of RSS facing	-40 mm (*)	-55 mm (*)	-60 mm	G.8
Lateral displacement at the base of RSS facing (mm)	>5 mm	15 mm	5 mm	G.9
Rotation of the RSS facing	>0.001	0.003	0.001	
Maximum Heave (rebound) at Highway 401 Centre Line	45 mm	55 mm	60 mm	G.10 and G.11

N/A Not Applicable – Area located within the temporary excavation.

(-)ve Denotes settlements

(†) Distances measured perpendicular to the tunnel abutment.

(*) Indicates calculated settlement at top of wall / abutment backfill that will be compensated during constructions

The cumulative deformations are rounded up to closest 5 mm.

5.6.4 RSS Wall External Stability

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

Bearing Capacity:

The following net ultimate geotechnical bearing capacity values (q_u) were determined for the native subgrade soils at the two abutments for short-term (undrained) and long-term (drained) loading conditions.

Table 5-12: Subgrade Ultimate Bearing Capacity

Abutment	Assumed Lowest Subgrade Elevation	Loading Condition	q_u (kPa)
North Wall	175.5 ⁽¹⁾	Short-Term (Undrained)	280 ⁽²⁾
		Long-Term (Drained)	630 ⁽³⁾
South Wall	175.5 ⁽¹⁾	Short-Term (Undrained)	310 ⁽⁴⁾
		Long-Term (Drained)	630 ⁽³⁾

(1) Below 1.5 m thick RGM at Abutment

(2) Based on an average cohesion of 55 kPa within the assumed zone of influence of the RSS wall foundation

(3) Based on an assumed soil friction angle $\phi = 30^\circ$

(4) Based on an average cohesion of 60 kPa within the assumed zone of influence of the RSS wall foundation.

Sliding Resistance:

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

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

Where:

A' (m²) = effective contact area of the base;

c' (kPa) = cohesion/adhesion at sliding interface;

δ (°) = friction angle at sliding interface;

V (kN) = vertical force; and

H_f (kN) = design horizontal load.

Based on the estimated elevation of 177.45 for the 100-year flooding event and 177.89 for the regional storm event from Pump Station 6 in the vicinity of Tunnel T-9, flooding of the roadway in Tunnel T-9 is not expected to occur. As the EPS and LWF incorporated in Tunnel T-9 abutments and return walls are located above the base of the pile cap at elevations between 183.3 and 186.0, which is about 5.4 to 8.1 m higher than the flood level, submergence of the material is not anticipated to occur in the area of Tunnel T-9.

The following soil properties (Table 5-13) at the interfaces between the RSS, RGM and silty clay subgrade can be used in the design:

Table 5-13: Soil Properties for use at Sliding Resistance

Interface	Undrained (Short-Term)		Drained (Long-Term)	
	δ , deg	c , kPa	δ' , deg	c' , kPa
RSS to RGM	30	0	30	0
RGM to Silty Clay	0	55		

5.6.5 RGM Foundation Loads

A 1.5 m thick reinforced granular mat (RGM) foundation comprising Granular B Type II was considered under the RSS false abutment walls to improve the load distribution to the bearing soils and satisfy the bearing capacity requirements for undrained conditions at the north and south abutments. A simplified load distribution model was used considering that the RGM foundation distributes the vertical pressures at the base of the RSS walls to the subgrade below the RGM at a 45° angle. The following loads (Table 5-14) were estimated to act on top of the RGM on the basis of conventional calculation of the bearing pressures under gravity retaining walls.

Table 5-14: Estimated Load on RGM

Abutment Location	Maximum Unfactored Bearing Pressure, kPa	Average Unfactored Bearing Pressure, kPa
North Wall	160	155
South Wall	185	165

Based on the above load on RGM, estimated unfactored horizontal tensile loads of 55 and 65 kN per meter of RGM for north and south abutment walls, respectively, were estimated across the entire height of 1.5 m. For cost estimates, the tensile loads can be accommodated by 3 layers of geogrids which have long-term load capacities of at-least 22 kN/m.

The above loads are for the use by the RGM suppliers to assist in the RGM's internal design. The associated bearing capacities at the underside of the RGM are provided in Table 5-12.

5.6.6 Abutment Configurations

Based on geotechnical analyses discussed in Section 5.6.1 to 5.6.5, abutment configurations and dimensions were determined (Table 5-15). The abutment configurations and dimensions indicated in these analyses are preliminary (e.g., the indicated width of the RSS is the minimum width) and are to be finalized by proprietary suppliers. The final design of the abutments is to be developed in consultation with the suppliers of the proprietary component.

Table 5-15: Tentative Abutment Dimensions

Abutment Location	Width of RSS Structure, m	RGM (Thickness × Min. Width at Base), m	EPS Volume, m ³ /m
North Wall	7.0	1.5 × 8.5	3.5 to 5.0
South Wall	7.0	1.5 x 8.5	5.0

The RSS supplier may require wider structures to meet the internal design requirement. The proposed abutment configurations are shown in Appendix I.

5.7 Wingwalls (Return Walls and Portal Walls)

An RSS return wall flared at 90^0 to the tunnel diaphragm is indicated at each corner of the structure. Tapered RSS portal walls are extended beyond the tunnel portals. Similar to the RSS walls at the north and south abutments, the RSS wing walls have been checked for bearing and sliding resistances. Light weight fill (LWF) consisting of water cooled iron blast furnace slag meeting MTO specifications was required in the top 2.5 m height of RSS return walls from the bearing resistance consideration. The proposed RSS return wall configurations are shown on Appendix I. The global stability analyses have been carried out on RSS return walls. The calculated factors of safety are in excess of 1.5 against global instability. The calculated factors of safety are summarized in Table 5-16. Figures F.13 to F.16 in Appendix F illustrate the stability models for the return walls.

Table 5-16: Calculated Factors of Safety for Return Walls against Global Instability

Location	Type of Return Wall	Tentative Width of RSS Return Wall, m	Calculated Factor of Safety against Global Instability	
			Undrained Condition	Drained Condition
North Side	RSS (LWF and RWF)	4.0	1.61	1.73
South Side	RSS (LWF and RWF)	4.0	1.54	1.61

Table 5-17 summarizes the net ultimate geotechnical bearing capacity values (q_u) determined for the cohesionless RSS materials supporting the return walls and native silty clay supporting the portal walls.

Table 5-17: Ultimate Bearing Capacity at Return Walls and Portal Walls

Type of Wing Walls	Location	Tentative Width of RSS Wall, m	Assumed Lowest Base Elevation	q_u (kPa)
Return Walls	North Side	4.0	181.3	390 ⁽¹⁾
	South Side	4.0	181.8	390 ⁽¹⁾
Portal Walls	North Side	4.5 and 7 ⁽⁴⁾	175.5	280 ⁽²⁾
	South Side	4.5 and 7 ⁽⁴⁾	175.5	310 ⁽³⁾

(1) Based on an assumed friction angle, $\phi = 35^0$ and unit weight = 21 kN/m³ of sloped RSS wall.

(2) Based on an average cohesion of 55 kPa within the assumed zone of influence of RSS portal wall.

(3) Based on an average cohesion of 60 kPa within the assumed zone of influence of RSS portal wall.

(4) Width of the Tapered Portal Wall = 7 m when Height of the Wall is >3.5 m, otherwise 4.5 m.

5.8 Backfilling

Behind the concrete abutment and wing walls, non-frost susceptible free draining granular fill should be placed in accordance with the CHBDC (ref. R-9). Construction notes for backfill, light weight fill (LWF) and expanded polystyrene (EPS) are provided on Drawings 285380-04-094-WIP1-2938, 285380-04-094-WIP1-2939 and 285380-04-094-WIP1-2940.

The 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 immediately adjacent the walls of the structure. The backfill adjacent the structure walls should be placed in thin (maximum 100 mm thick) loose lifts and compacted using light rollers or other compactors approved by the Engineer. 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.

Heavy compaction equipment should not be used immediately adjacent the walls of the structure. The backfill adjacent the structure walls should be placed in thin (maximum 100 mm thick) loose lifts and compacted using light rollers or other compactors approved by the Engineer. Effects of backfill compaction activities should be simulated as live load over and above the static lateral earth pressure for structural design in accordance with the CHBDC.

For retained backfill that is placed and compacted in layers, the lateral force caused by compaction should be considered. In the absence of detailed analysis, the additional lateral pressure due to the effects of light compaction, a lateral pressure varying linearly from 12 kPa at the fill surface to 0 kPa at a depth of 1.7 m below the surface should be added to the base lateral earth pressure.

Earth pressures on abutments and wing walls may be calculated on the basis of the parameters listed in Table 5-18. Compactable Group III soils may be used as general backfill within approved areas.

Table 5-18: Soil Parameters for Earth Pressure Calculations

Soil Parameter	Group I Soils	Group II Soils	Group III Soils
Fill Unit Weight, kN/m^3	22	21	20.5
Friction angle, ϕ (degrees)	33 to 35	29 to 32	22 to 30
Coefficients of Static Lateral Earth Pressure:			
'Active' or Unrestrained, $K_a^{(*)}$	0.27 to 0.30	0.30 to 0.35	0.33 to 0.45
'At Rest' or Restrained, $K_o^{(*)}$	0.43 to 0.46	0.47 to 0.52	0.50 to 0.62
'Passive', $K_p^{(*)}$	3.3 to 3.7	2.9 to 3.2	2.2 to 3.0

(*)Values are given for level backfill and ground surface behind the wall. The coefficients of lateral earth pressure should be adjusted if there is sloping ground at the back of the wall.

Note: Compacted to > 95% Standard Proctor maximum dry density.

Legend:

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

5.9 Permanent Subdrainage System

A permanent subdrainage system should be provided behind the abutments and connected to the roadway drainage system.

Use of free draining granular soils within the reinforced soil mass within the RSS structures and the RGM will ensure that these structures will act as a “natural” drain conveying the seepage resulted from the phreatic groundwater and infiltrations from surface precipitations toward the toe of the wall facing and base of the RGM. In order to prevent accumulation and stagnation of groundwater within the RGM, the subgrade should be graded to direct the collected groundwater to subdrains discharging to manholes or sumps.

Depending on the grain size of the RGM materials, a filter layer may be required at the interface between the native soils and the imported granular materials.

Two simplified steady-state models (Figures H.1 and H.2 in Appendix H) were used to estimate seepage rates associated with the long-term drawdown of the groundwater along a typical cross-section of Tunnel T-9 for north and south sides. SEEP/W 2007 software was used for this analysis. The initial groundwater table was assumed at elevation 183.5 for north side to simulate pond water in Wolfe drain. The initial groundwater table was assumed at elevation 182.0 for south side. Groundwater recharge from infiltrations from ground surface sources was also considered. The rates of recharge were estimated on the basis of saturated hydraulic conductivity of the soils in conjunction with the assumption that no mounding of the long-term groundwater should occur. A ground surface infiltration rate of 2×10^{-5} to 4×10^{-5} m/days was accommodated by trial-and-error approach to ensure a sustained groundwater level without excessive mounding.

Based on the above, the flow rate from groundwater seepage across the entire tunnel cross section was estimated to be 4.1 to 7.4 litre/days per meter length of the tunnel. This is an approximate estimate and the actual quantities could differ significantly from this magnitude. The above flow rate does not include additional seepage that may occur from other external sources, like runoff from ground surface, perched groundwater, or accidental water main breaks.

6 Other Geotechnical Recommendations

6.1 Construction Dewatering

The design of the dewatering system should comply with the Ontario Provincial Standard Specification (OPSS) 517 and 518 provisions.

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

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

Effective drainage is an important aspect in the life expectancy and performance of any abutment wall, wing wall, or pavement structure associated with the tunnel. Permanent sub-drainage should be installed behind abutment and wing walls. The subdrains should be surrounded by approved granular material and discharged by gravity 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.

6.2 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 tunnel. 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 and performance (stability, deformability and deterioration) 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.
- In recognition of potential for soil gasses as described in Section 4.6, air quality and subgrade pore pressure monitoring should be carried out during construction. The equipment operating in confined spaces should be selected to safely operate in a potentially gaseous environment. Excavation lifts should be decided in consideration of the pore pressure monitoring data and the potential ground softening.
- The realigned Wolfe drain surfaces should be covered with clay/synthetic liners to control the long-term seepage from drain to the tunnel abutment through the upper granular layer and/or sand/silt seams embedded in clay layer. The recommendations for liners are summarized in Table 6-1.

Table 6-1: Summary of Recommendations for Clay / Synthetic Liners

Available Width between Edge of Wolfe Drain and Edge of Granular Materials associated with the Abutment Structure, m	Type of Liner	Thickness of Liner, m	Thickness of Clay Backfill to be placed below/behind the Liner, m	Compaction Requirements for Liner	Compaction Requirements for Clay Backfill to be placed below/behind the Liner
>2.0	Silty Clay (*)	1.0	1.0	Place in 200 mm thick lifts and compact to >90% of SPMDD at Optimum Moisture Content+3%	Place in 200 mm thick lifts and compact to >95% of SPMDD at Optimum Moisture Content±2%
<2.0	Synthetic	-	1.0	-	Place in 200 mm thick lifts and compact to >95% of SPMDD at Optimum Moisture Content±2%

(*) Horizontal permeability of silty clay liner should be less than 1×10^{-10} m/sec.
SPMDD = Standard Proctor Maximum Dry Density.

6.3 Instrumentation and Monitoring during Construction

As mentioned earlier in Section 5.4, 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 (Table 3-2).

Recommendations for additional instrumentations and monitoring programme as well as guidelines for interpretation, alert levels and contingencies are provided in a separate report (Document No. 285380-04-118-0001).

The Contractor is responsible for planning, installation and maintenance of instrumentation as well as the completion of monitoring of the response of the excavations (ground movement) during construction. Detailed plans and procedures should be submitted to HMQ for approval at least 3 month prior to commencement of the monitoring of the works.

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

6.4 Corrosion Potential

Analytical testing was carried out on a sample of the clayey silt stratum obtained in Borehole BH T9-1 (Sample 8). Table 6-2 summarizes the results of various analyses carried out on the soil sample to assess the potential for corrosion on concrete and metallic elements.

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 T9-1 (Sample 8)	178.7	7.75	102	4130	<0.2	58

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 tested soil would be considered noncorrosive to buried metallic elements (ref. R-2).

Sulphate attack on concrete and steel corrosion should be further reviewed by a 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 construction quality control program, including geotechnical inspection, instrumentation, testing and instrument monitoring, should be developed and implemented throughout the construction phase. In addition, related laboratory testing should be carried out in conjunction with the fieldwork to monitor compliance with the various materials and project specifications.

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

7 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 structures mentioned in the report. The report was prepared with the condition that the structural and other designs of the WEP will be in accordance with applicable standards and codes, regulations of authorities having jurisdiction, and good engineering practices. Further, the recommendations and opinions expressed in this report are only applicable to the proposed project as described within AMEC's report.

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

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

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

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

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

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

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

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

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

8 Closure

The geotechnical report for Tunnel T-9 was prepared by Mr. Nazmur Rahman, P.Eng. and checked by Dr. Dan Dimitriu, P.Eng. The project was executed under the technical direction of Dr. Narendra S. Verma, P.Eng., 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.

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Yours truly,

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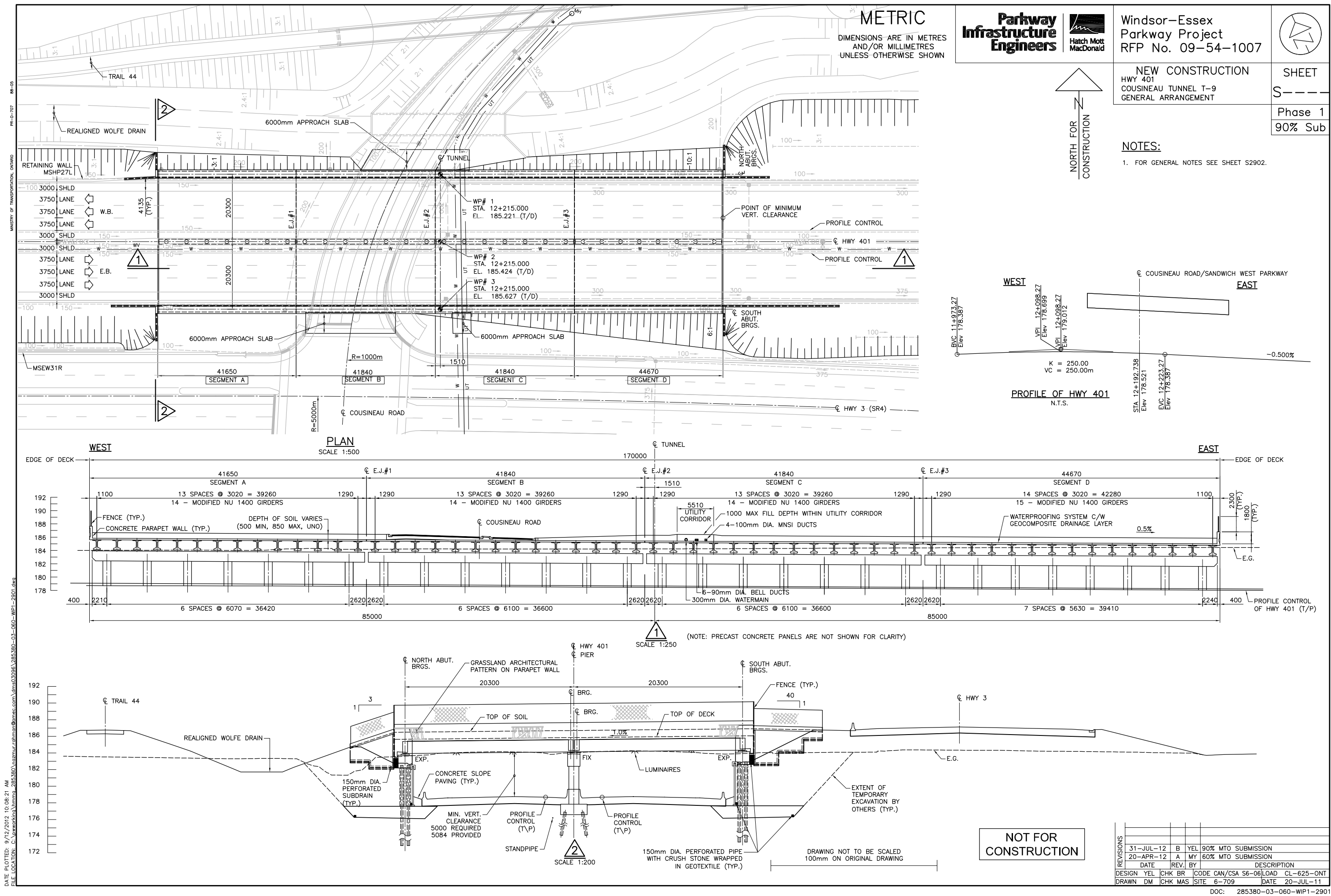
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Drawings

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-9 (Cousineau Tunnel) (Sta. 12+130L to 12+300L)
Doc No.: 285380-04-119-0050 (Geocres No. 40J3-19)

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



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

Parkway
Infrastructure
Engineers



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



NEW CONSTRUCTION
HWY 401
COUSINEAU TUNNEL T-9
FOUNDATION LAYOUT

SHEET

Phase 1

90% Sub

NOTES:

- FOR GENERAL NOTES SEE SHEET S2902.
- THIS DRAWING TO BE READ IN CONJUNCTION WITH THE RETAINED SOIL SYSTEM WALLS AND ABUTMENT LAYOUT DRAWINGS.

PILE NOTES:

- PILE LENGTHS SHOWN ARE ESTIMATED LENGTHS FROM THE CUT-OFF TO THE ESTIMATED BEDROCK / REFUSAL SURFACE.
- ALL PILES ARE HP 310X110 STEEL H PILES.
- ALL PILES SHALL BE FITTED WITH TYPE I DRIVING SHOE PER OPSP 3000.100 OR APPROVED EQUIVALENT.
- PILE SPLICES SHALL BE BUTT WELDED AS PER OPSP 3000.150 AND OPSS 903. SPLICE PLATES ARE NOT PERMITTED.
- 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.
- 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.
- 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.
- 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.
- 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 ENGINEER.
- DURING AND AFTER PILE DRIVING THE CONTRACTOR SHALL IMPLEMENT APPROPRIATE MITIGATION MEASURES AGAINST ANY SEEPAGE OF NATURAL GAS AND GROUNDWATER THAT MIGHT CAUSE LOSS OF BEARING RESISTANCE AND ADEQUATELY PLUG THE GAPS BETWEEN THE PILE SHAFT AND SURROUNDING SOILS.
- 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.

CONSTRUCTION SEQUENCE - ABUTMENTS:

- EXCAVATE FOR TUNNEL OPENING.
- PROVIDE SUITABLE PROTECTION DURING CONSTRUCTION.
- EXCAVATE TO UNDERSIDE OF RGM.
- INSTALL RGM.
- DRIVE PILES.
- PLACE 500mm DIA. GALVANIZED CSP PIPES AND PROVIDE BLOCKING OF PILES IN CSP PIPES.
- CONSTRUCT RETAINED SOIL SYSTEM WALLS AND BACKFILL TO UNDERSIDE OF ABUTMENTS. BACKFILL AS PER SHEET S2907.
- FILL 500mm DIA. CSP PIPES WITH CONCRETE.
- CONSTRUCT ABUTMENTS TO UNDERSIDE OF BEARING PEDESTALS.
- CONTRACTOR TO PROVIDE SUITABLE STABILITY DURING CONSTRUCTION.
- SEE ABUTMENT DETAILS DRAWING FOR CONTINUATION.

CONSTRUCTION SEQUENCE - PIER:

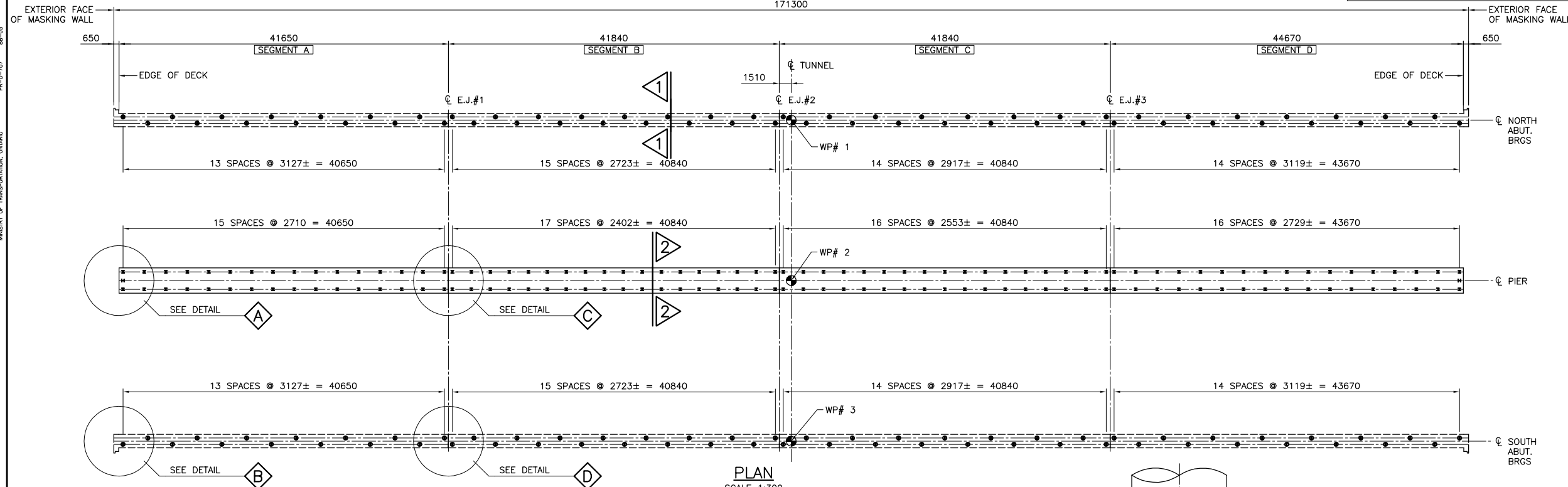
- EXCAVATE TO UNDERSIDE OF FOOTING.
- DRIVE PILES.
- PLACE FOOTING PAD.
- CONTRACTOR TO PROVIDE SUITABLE STABILITY DURING CONSTRUCTION.
- CONSTRUCT PIER FOOTING TO TOP OF FOOTING.
- SEE PIER DETAILS DRAWING FOR CONTINUATION.

APPLICABLE STANDARD DRAWINGS:

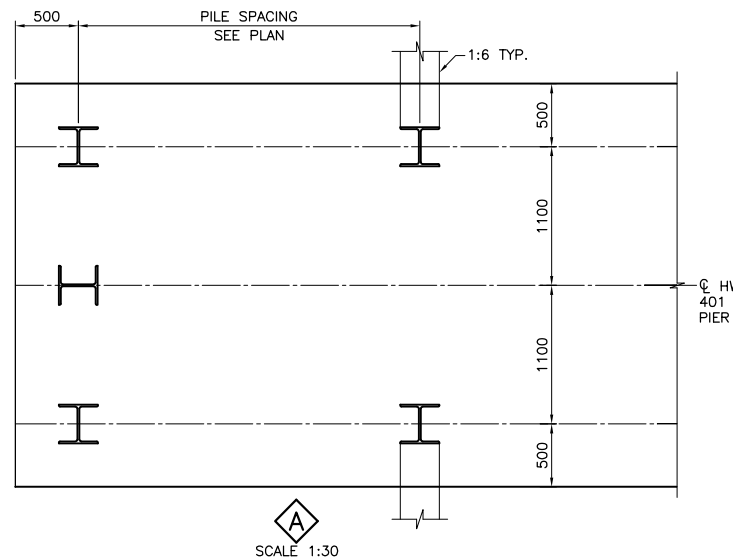
OPSP-3000.100 FOUNDATION PILES - STEEL H-PILE
OPSP-3000.150 FOUNDATION PILES - STEEL H-PILE
FOUNDATION PILES - STEEL H-PILE
DRIVING SHOE
FOUNDATION PILES - STEEL H-PILE
SPLICE

REVISIONS	DATE	REV.	BY	DESCRIPTION
31-JUL-12	B	YEL	90% MTO SUBMISSION	
20-APR-12	A	MY	60% MTO SUBMISSION	
DESIGN	YEL	CHK	MAS	CODE CAN/CSA S6-06 LOAD CL-625-ONT
DRAWN	DM	CHK	BR	SITE 6-709 DATE 20-JUL-11

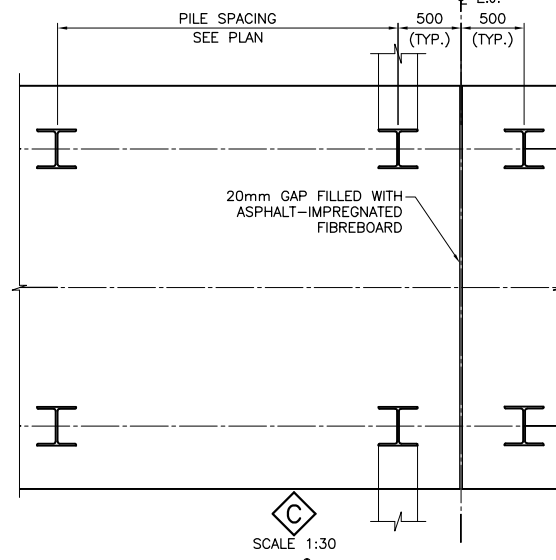
DOC: 285380-03-061-WP1-2905



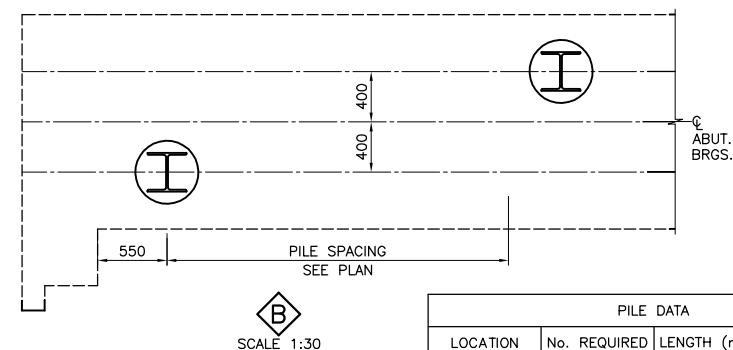
PLAN
SCALE 1:300



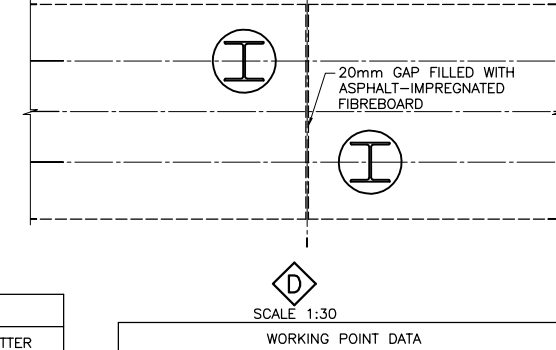
SCALE 1:30



SCALE 1:30



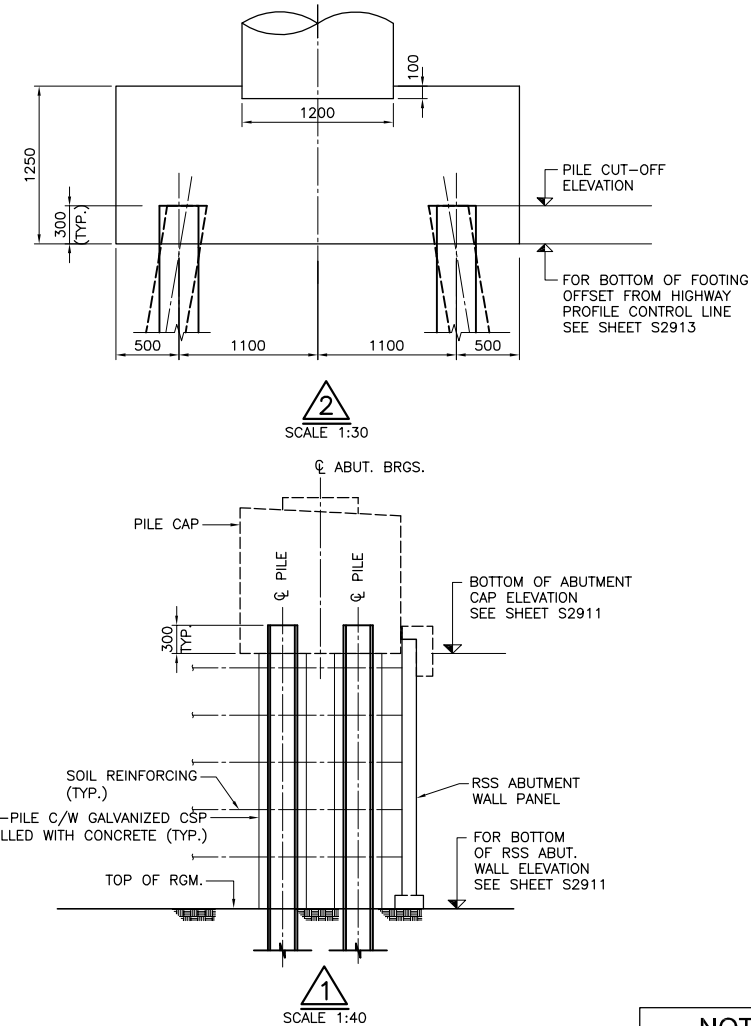
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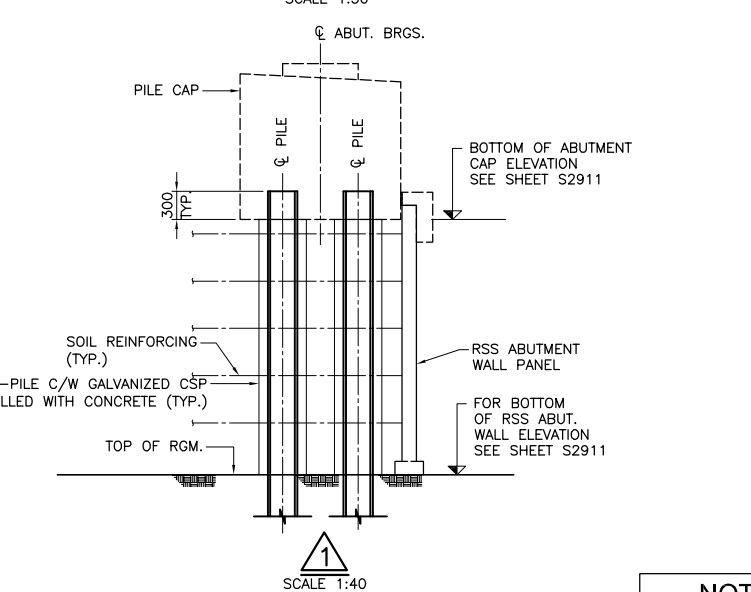
SCALE 1:30

PILE DATA			
LOCATION	No. REQUIRED	LENGTH (m) *	BATTER
N. ABUTMENT	60	30.5	VERTICAL
PIER	70	25.1	VERTICAL
	68	25.5	1:6
S. ABUTMENT	60	31.0	VERTICAL

WORKING POINT DATA		
LOCATION	NORTHING	EASTING
WP #1	4 678 620.714	333 846.466
WP #2	4 678 602.281	333 837.963
WP #3	4 678 583.848	333 829.460



SCALE 1:30



SCALE 1:40

NOT FOR
CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

* ESTIMATED LENGTHS ARE TAKEN AT THE TUNNEL CENTRE LINE. A SPLICE CAN NOT BE LOCATED WITHIN 3m OF THE PILE CUT-OFF ELEVATION. LENGTHS OF PILES SHOULD BE ORDERED ACCORDINGLY.

DATE PLOTTED: 9/12/2012 10:17:03 AM
FILE LOCATION: C:\pwworking\hmm\285380-03-061-WP1-2905.dwg

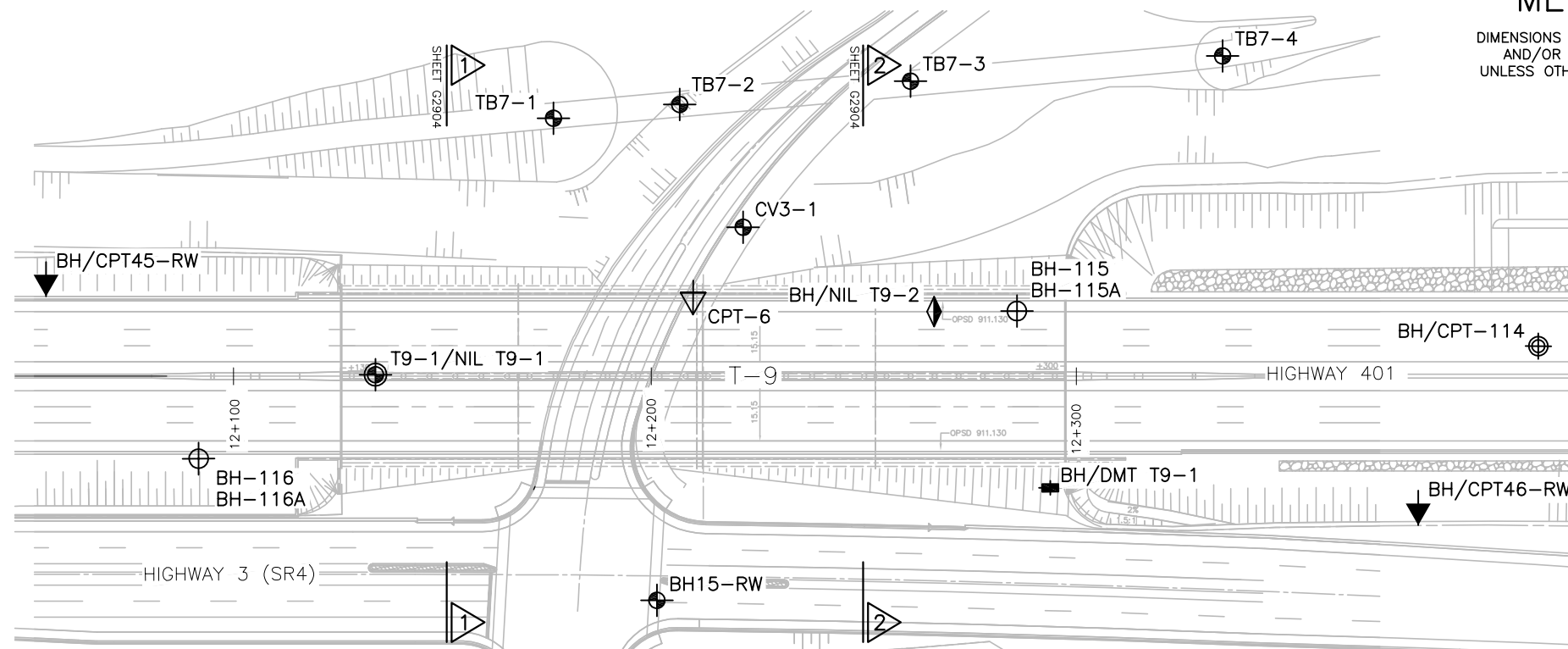
METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN
Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

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

NEW CONSTRUCTION
HWY 401
COUSINEAU TUNNEL T-9
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
G2903

Phase 1
IFC

PLAN
HORIZONTAL SCALE 1:750

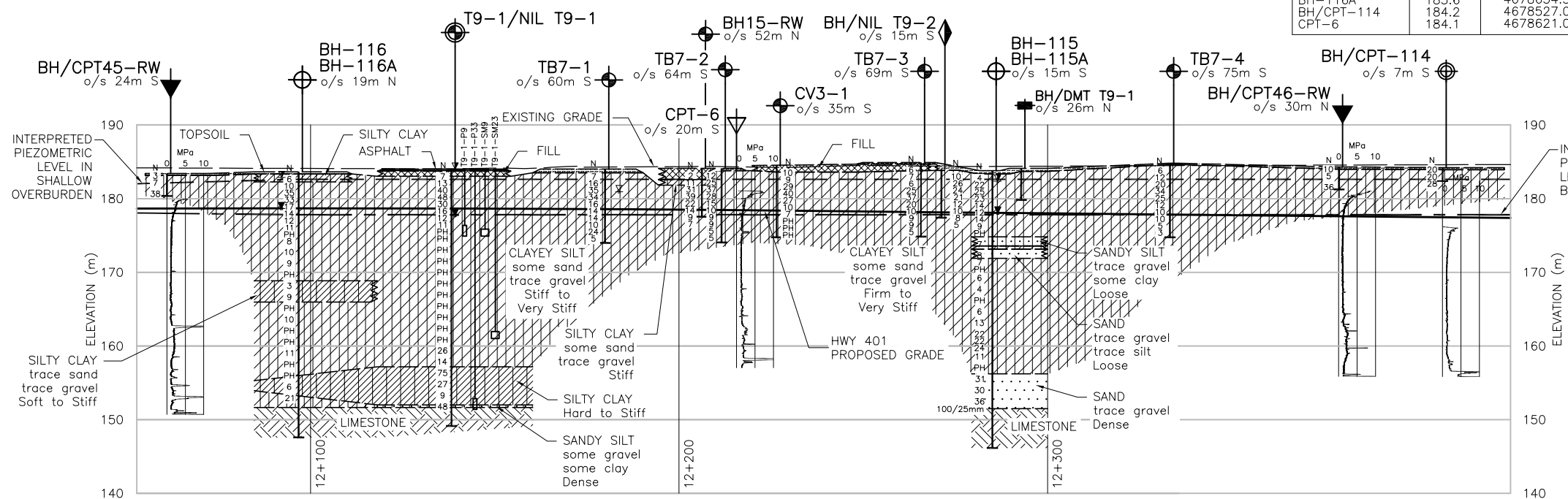
MATERIAL LEGEND

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

LIST OF ABBREVIATIONS

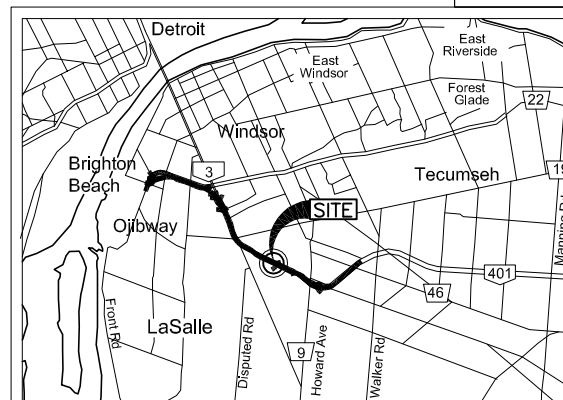
PH	-	SAMPLER ADVANCED BY HYDRAULIC PRESSURE
PM	-	SAMPLER ADVANCED BY MANUAL PRESSURE
WH	-	SAMPLER ADVANCED BY STATIC WEIGHT OF HAMMER
WR	-	SAMPLER ADVANCED BY WEIGHT OF SAMPLER RODS

No.	ELEVATION	CO-ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC TESTHOLES			
BH15-RW	184.1	4678559.2	333806.1
BH/CPT45-RW	183.4	4678688.3	333708.0
BH/CPT46-RW	184.3	4678505.0	333977.6
CV3-1	184.5	4678630.0	333861.1
BH/DMT T9-1	184.1	4678544.5	333900.9
T9-1/NIL T9-1	184.0	4678634.9	333766.7
BH/NIL T9-2	184.0	4678636.5	333765.3
TB7-1	184.0	4678671.8	333831.4
TB7-2	184.1	4678662.3	333859.6
TB7-3	184.9	4678644.6	333911.0
TB7-4	184.8	4678619.4	333980.0
PREVIOUS TESTHOLES			
BH-115	183.8	4678585.3	333911.1
BH-115A	183.8	4678585.3	333911.1
BH-116	183.6	4678634.3	333722.5
BH-116A	183.6	4678634.3	333722.5
BH/CPT-114	184.2	4678527.0	334019.0
CPT-6	184.1	4678621.0	333844.0



PROFILE ALONG CL OF HWY 401

HORIZONTAL SCALE 1:750
VERTICAL SCALE 1:375

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING


KEY PLAN

SCALE
1 0 2 4Km

LEGEND

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

NOTES

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

REVISIONS	13-SEP-12				ISSUED FOR CONSTRUCTION			
	DATE	REV.	BY	DESCRIPTION	DATE	REV.	BY	DESCRIPTION
DESIGN	NR	CHK	NSV	CODE	CAN/CSA	LOAD	CL-625-ONT	
DRAWN	SJL	CHK	NR	SITE	6-709	DATE	07-DEC-11	

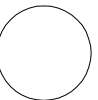
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AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



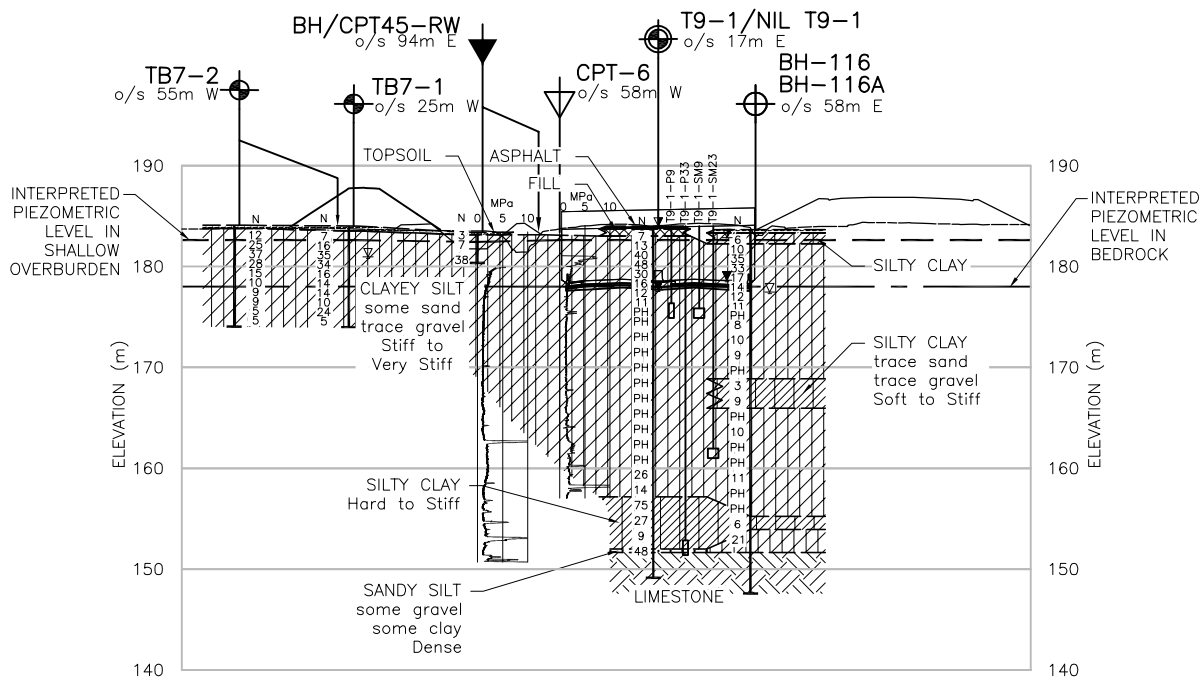
Windsor-Essex
Parkway Project
RFP No. 09-54-1007

NEW CONSTRUCTION
HWY 401
COUSINEAU TUNNEL T-9
SOIL STRATIGRAPHY

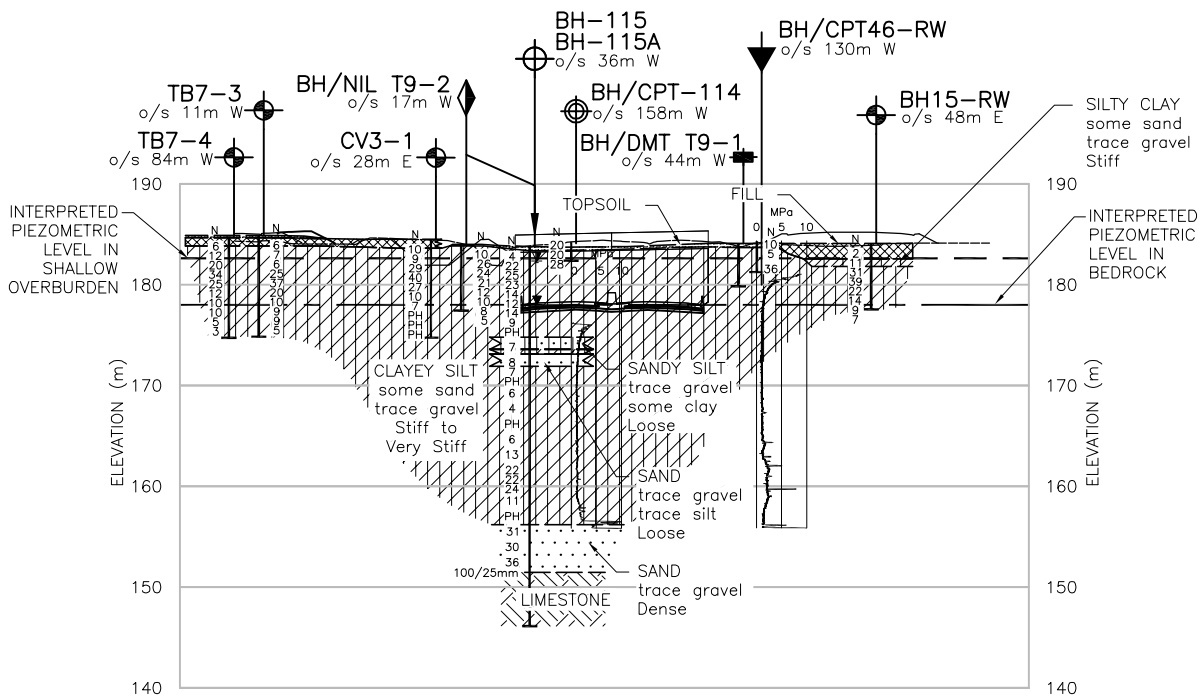


SHEET
G2904

Phase 1
IFC



HORT SCALE 1:750
VERT SCALE 1:375



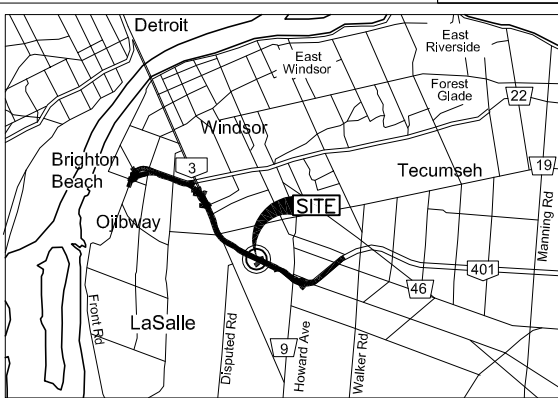
HORT SCALE 1:750
VERT SCALE 1:375

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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



KEY PLAN

SCALE
1 0 2 4Km

LEGEND

	BOREHOLE CURRENT INVESTIGATION
	BOREHOLE AND NILCON VANE CURRENT INVESTIGATION
	SW/SP HOLE (HYDROGEOLOGY) CURRENT INVESTIGATION
	NILCON VANE CURRENT INVESTIGATION
	CPT - CURRENT INVESTIGATION
	DMT - CURRENT INVESTIGATION
	BOREHOLE PREVIOUS INVESTIGATION
	BOREHOLE, CPT AND NILCON VANE PREVIOUS INVESTIGATIONS
	CPT -PREVIOUS INVESTIGATION
	N SPT N-VALUE
	BLOWS/0.3m UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
	MHSG - MAGNETIC HEAVE/SETTLEMENT GAUGE (SM)
	P - VIBRATING WIRE PIEZOMETER (VWP)
	DRY BOREHOLE DRY DURING DRILLING
	WATER LEVEL DURING DRILLING
	WATER LEVEL (SHALLOW PIEZO)
	WATER LEVEL (DEEP PIEZO)

NOTES

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

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS	DATE	REV.	BY	DESCRIPTION
13-SEP-12	0	NR		ISSUED FOR CONSTRUCTION
DESIGN	NR	CHK	NSV	CODE CAN/CSA
DRAWN	SJL	CHK	NR	SITE 6-709
				LOAD CL-625-ONT
				DATE 07-DEC-11

CONSTRUCTION NOTES – BACKFILL AT STRUCTURES

1.0 GENERAL REQUIREMENTS

- 1.1.

THESE CONSTRUCTION NOTES RELATE TO THE SUPPLY AND PLACEMENT OF BACKFILL MATERIALS AT THE STRUCTURES AT THE WINDSOR-ESSEX PARKWAY (WEP) PROJECT AS ILLUSTRATED ON THE ACCOMPANYING DRAWINGS. THE REQUIREMENTS GIVEN HEREFTER ARE THE GENERAL REQUIREMENTS. FOR DETAILED REQUIREMENTS, THE CONTRACTOR SHOULD REFER TO APPROPRIATE ONTARIO PROVINCIAL STANDARD SPECIFICATIONS (OPSS) LISTED IN SECTION 1.6.
- 1.2.

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

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

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

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

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

•

ASTM D2216

•

ASTM D2850

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

•

ASTM D2922

•

ASTM D3017

•

ASTM D5856

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

•

OPSS 201

•

OPSS 206

•

OPSS 212

•

OPSS 401

•

OPSS 501

•

OPSS 517

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

•

OPSS 518

•

OPSS 805

•

OPSS 902

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

•

OPSS 1001

•

OPSS 1004

•

OPSS 1010

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

•

OPSS 1860

•

OPSD 208.010

GEOTEXTILE
BENCHING OF EARTH SLOPES

1.7.

IF THERE IS ANY CONFLICT BETWEEN THE REQUIREMENTS GIVEN ON THIS DRAWING AND THE STANDARDS AND SPECIFICATIONS DOCUMENTS LISTED IN SECTION 1.6, THE DESIGNER SHOULD BE CONSULTED FOR CLARIFICATION AND RECOMMENDATIONS.

1.8.

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

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

2.2

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

2.3

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

2.4

ALL EXCAVATION WORKS SHOULD BE CARRIED OUT IN ACCORDANCE WITH THE GUIDELINES OUTLINED IN OCCUPATIONAL HEALTH AND SAFETY ACT (OHSA) AND ONTARIO PROVINCIAL STANDARD SPECIFICATION (OPSS) 902. NATIVE DEWATERED SOILS AT THE SITE AND COMPACTED FILLS MAY BE CLASSIFIED IN GENERAL AS TYPE 3 SOILS. UNDEWATERED FILLS, NATIVE SAND AND SILTS, AND WATER BEARING BACKFILL WITHIN TRENCHES OF ACTIVE AND/OR ABANDONED UTILITIES MAY DEVELOP TYPE 4 SOIL CONDITIONS AND SHALL BE ADDRESSED ACCORDINGLY.

2.5

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

2.6

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

2.7

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

2.8

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

2.9

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

2.10

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

2.11

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

2.12

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

3.0 REINFORCED GRANULAR MAT (RGM)

3.1

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

3.2

GRANULAR FILL FOR RGM: THE FILL MATERIAL SHALL BE GRANULAR 'A' OR GRANULAR 'B' TYPE II (OPSS 1010) PLACED AS PER NOTE 5.3 AND COMPACTED TO NOT LESS THAN 98%.

3.3

REINFORCEMENT FOR RGM: AS PER CONTRACT DOCUMENTS.

4.0 FILL MATERIALS

4.1

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

4.2

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

4.3

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

4.4

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

4.5

LWF AND EPS: SEE RESPECTIVE CONSTRUCTION NOTES.

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

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

NEW CONSTRUCTION
HWY 401
COUSINEAU TUNNEL T-9
CONSTRUCTION NOTES – BACKFILL AT STRUCTURES

SHEET
G2938

Phase 1

IFC

5.0 FILL PLACEMENT AND COMPACTION

5.1 GENERAL:

•

THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER THEIR QC/QA INSPECTION AND TEST PLAN FOR REVIEW/COMMENT PRIOR TO THE PLACEMENT/COMPACTION OF FILL.

•

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

•

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

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

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

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

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

5.6 COMPACTION AND PLACEMENT OF GRANULAR MATERIALS FOR RSS WALLS SHALL CONFORM TO THE MANUFACTURER’S RECOMMENDATIONS.

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

6.0 DRAINAGE – DEWATERING

6.1

REFER TO OPSS 518 FOR DEWATERING REQUIREMENTS.

6.2

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

6.3

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

7.0 USE

7.1

THIS DRAWING PROVIDES CONSTRUCTION REQUIREMENTS FOR GEOTECHNICAL ASPECTS OF BACKFILLING AT TUNNELS.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS					
	13-SEP-12	0	NR	ISSUED FOR CONSTRUCTION	
	DATE	REV.	BY	DESCRIPTION	
DESIGN	SF	CHK	NSV	CODE CAN/CSA S6-06	LOAD CL-625-ONT
DRAWN	MM	CHK	DD	SITE 6-709	DATE 20-DEC-11

DOC: 285380-04-094-WP1-2938

DATE PLOTTED: 9/13/2012 2:00:08 PM
FILE LOCATION: C:\working\hmmg_285380\alphabet\ibault@ames.com\dms09856\285380_04-094-WIP1-2939.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

PRE-D-707
88-85

CONSTRUCTION NOTES – LIGHTWEIGHT FILL MATERIAL

1.0 GENERAL REQUIREMENTS

- 1.1.

THE CONSTRUCTION NOTES ON THIS DRAWING COVER THE REQUIREMENTS FOR THE SUPPLY AND PLACEMENT OF WATER COOLED ULTRA LIGHTWEIGHT BLAST FURNACE SLAG TO BE USED FOR CONSTRUCTION OF THE STRUCTURES FOR THE WINDSOR–ESSEX PARKWAY (WEP) PROJECT. AT THE WEP PROJECT, THE ULTRA LIGHTWEIGHT BLAST FURNACE SLAG MATERIAL IS GENERALLY REFERRED TO AS THE LIGHT WEIGHT FILL (LWF).
- 1.2.

THESE CONSTRUCTION NOTES ARE TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING DESIGN DRAWING(S), OTHER RELEVANT CONSTRUCTION NOTES AND GEOTECHNICAL REPORT.
- 1.3.

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

- MTO
 - ASTM D422
 - ASTM D2216
 - ASTM D2850
 - ASTM D2922
 - ASTM D3017
 - OPSS 212
 - OPSS 501
 - OPSS 517
 - OPSS 1010
 - OPSS 1860

NSSP ULTRA LIGHTWEIGHT BLAST FURNACE SLAG (WATER COOLED)

PARTICLE–SIZE ANALYSIS OF SOILS

MOISTURE CONTENT OF SOILS

UNCONSOLIDATED–UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS

DENSITY OF SOIL AND SOIL–AGGREGATE IN PLACE BY NUCLEAR METHODS

WATER CONTENT OF SOIL AND ROCK IN PLACE BY NUCLEAR METHODS

BORROW

COMPACTION

DEWATERING

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

GEOTEXTILES

1.4.

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

1.5.

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

THE SITE PREPARATION AND EXCAVATION REQUIREMENTS ON THE CONSTRUCTION NOTES FOR THE BACKFILL AT STRUCTURES ARE APPLICABLE.
- 3.0 SUBMISSION AND DESIGN REQUIREMENTS
- 3.1

THE CONTRACTOR SHALL SUBMIT TO PIC AND THE ENGINEER CERTIFICATES OF CONFORMANCE SEALED AND SIGNED BY THE QUALITY VERIFICATION ENGINEER AS FOLLOWS:

a.

PRIOR TO THE PLACEMENT OF THE LIGHTWEIGHT FILL MATERIAL ON THE PROJECT, THE CONTRACTOR SHALL SUBMIT TO THE CONTRACT ADMINISTRATOR A CERTIFICATE OF CONFORMANCE STATING THAT THE MATERIAL SATISFIES THE MATERIAL PROPERTIES SPECIFIED IN SECTION 4.1.

b.

FOLLOWING FILL PLACEMENT, THE CONTRACTOR SHALL SUBMIT TO THE CONTRACT ADMINISTRATOR A CERTIFICATE OF CONFORMANCE STATING THAT THE MATERIAL SATISFIES THE REQUIREMENTS OF THIS SPECIFICATION AND THAT THE WORK HAS BEEN CARRIED OUT IN GENERAL CONFORMANCE WITH THE CONTRACT DOCUMENTS AND SPECIFICATIONS. THE CONTRACTOR SHALL ALSO SUBMIT ALL QUALITY CONTROL TEST RESULTS FOR INFORMATION ONLY.
- 4.0 MATERIAL
- 4.1

THE LWF SHALL SATISFY THE FOLLOWING PHYSICAL, MECHANICAL AND CHEMICAL PROPERTY REQUIREMENTS:

- ANGLE OF INTERNAL FRICTION
 - HYDRAULIC CONDUCTIVITY
 - CHEMICAL COMPOSITION
 - IN SITU WET UNIT WEIGHT

>35° (ASTM 2850–85)

>8 E–03 CM/S (ASTM 5856–95, METHOD A)

THE MATERIAL SHALL MEET THE LEACHATE CRITERIA ESTABLISHED UNDER ONTARIO REGULATIONS 347

<12.5 kN/m³ (ASTM D2922) (MAXIMUM WHEN PLACED AND COMPACTED IN ACCORDANCE WITH THE SPECIFICATIONS)

5.0 CONSTRUCTION

5.1

THE LWF (BLAST FURNACE SLAG) IS SUSCEPTIBLE TO CRUSHING IF OVERCOMPACTED AND CAREFUL CONSTRUCTION PROCEDURES AND SUPERVISION ARE REQUIRED. THE CONTRACTOR SHALL PLACE THE LWF MATERIAL AND SHALL ACHIEVE COMPACTION WITHOUT CRUSHING THE MATERIAL SINCE CRUSHING INCREASES ITS UNIT WEIGHT. THE CONTRACTOR SHALL PLACE THE LWF MATERIAL WITHOUT EXCEEDING THE SPECIFIED IN SITU UNIT WEIGHT AND MAINTAINING CRUSHING OF THE MATERIAL BELOW 5%.

5.2

TO PREVENT OVER–CRUSHING AND OVER–COMPACTION, THE LWF SHALL BE PLACED AS FOLLOWS:

a.

FOR EMBANKMENTS THE LWF SHALL BE PLACED IN LIFTS OF 300 mm AND COMPACTED BY 3 PASSES OF SINGLE DRUM VIBRATORY EQUIPMENT APPROVED BY THE ENGINEER (E.G., BOMAG 142 OR EQUIVALENT, TABLE 1).

b.

FOR BACKFILL TO STRUCTURES, THE LWF SHALL BE PLACED IN LIFTS OF 300 mm AND COMPACTED WITH 8 PASSES OF MANUALLY GUIDED TAMPER SUCH AS A BOMAG BPR 30/38 D OR EQUIVALENT (TABLE 1).

c.

THE CONTRACTOR SHALL PLACE AND SPREAD THE LOOSE LIFTS USING A RUBBER TIRE FRONT–END LOADER SUCH AS A CATERPILLAR 980 F OR EQUIVALENT.

5.3

COMPACTION EQUIPMENT TECHNICAL DETAILS ARE PROVIDED IN TABLE 1.

5.4

THE LWF ZONES SHALL BE APPROPRIATELY WRAPPED IN GEOTEXTILE TO AVOID LOSS OF FINES FROM THE ADJACENT BACKFILL OR NATIVE MATERIALS IN CONTACT WITH THE LWF ZONES.

6.0 QUALITY CONTROL

6.1

QUALITY CONTROL (QC) TESTING SHALL BE CARRIED OUT BY THE CONTRACTOR TO ENSURE THAT THE LWF MATERIAL IS PLACED AND COMPACTED AS SPECIFIED. FIELD DENSITY AND FIELD MOISTURE DETERMINATION SHALL BE MADE IN ACCORDANCE WITH ASTM D2922 AND ASTM D3017, RESPECTIVELY.

6.2

THE CONTRACTOR SHALL BUILD A CONTROL STRIP TO VERIFY THAT THE PLACEMENT AND COMPACTION PROCEDURE WILL ACHIEVE THE REQUIREMENTS OF THESE SPECIFICATIONS WITHOUT EVIDENCE OF CRUSHING AND WITHOUT EXCEEDING THE SPECIFIED MAXIMUM IN SITU WET UNIT WEIGHT OF 12.5 kN/m³.

6.3

MATERIAL PLACED IN THE CONTROL STRIP SHALL HAVE THE MOISTURE CONTENT THAT WILL YIELD THE SPECIFIED IN–SITU UNIT WEIGHT. FOR THE CONTROL STRIP DETERMINATION, THE NUCLEAR GAUGE METHOD WILL NOT BE CONSIDERED AN ACCEPTABLE METHOD OF DETERMINING THE IN–SITU MOISTURE CONTENT OF THE LWF MATERIAL. MOISTURE CONTENT SHALL BE DETERMINED BY THE OVEN DRY METHOD ON SELECTED COMPACTED EMBANKMENT MATERIAL SAMPLES IN ACCORDANCE WITH ASTM D2216.

6.4

AFTER THE TRIAL AREA IS COMPLETE, SAMPLES FOR MOISTURE CONTROL AND IN SITU UNIT WEIGHT DETERMINATION TESTING SHALL BE AS PER ASTM D2922.

6.5

IN ADDITION, GRADATION AS PER ASTM D422–63 BEFORE AND AFTER COMPACTION EFFORT SHALL BE PERFORMED TO DETERMINE THAT CRUSHING IS KEPT WITHIN 5%.

6.6

THE REQUIREMENTS OF THE CONTROL STRIP MUST BE SATISFIED AS PART OF THE ACCEPTANCE CRITERIA OF ANY PROPOSED CHANGE TO THE SPECIFIED COMPACTION METHOD OF THIS SPECIAL PROVISION.

7.0 USE

7.1

THIS DRAWING PROVIDES CONSTRUCTION REQUIREMENTS FOR GEOTECHNICAL ASPECTS OF BACKFILLING AT TUNNELS.

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

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

NEW CONSTRUCTION
HWY 401
COUSINEAU TUNNEL T–9
CONSTRUCTION NOTES – LIGHTWEIGHT FILL MATERIAL

SHEET
G2939

Phase 1
IFC

TABLE 1: COMPACTION EQUIPMENT TECHNICAL DETAILS

	BOMAG 142 D	BOMAG BPR 30/38 D
WEIGHTS		
• OPERATING WEIGHT (kg)	4690±	175±
• MASS PER SQUARE METRE OF BASE PLATE (kg/m²)	N/A	1439
DIMENSIONS		
• DRUM WIDTH (mm)	1426±	N/A
• DRUM DIAMETER (mm)	1058±	N/A
• WIDTH OF BASE PLATE (mm)	N/A	380
• LENGTH OF BASE PLATE (mm)	N/A	730
DRIVE		
• PERFORMANCE DIN 6271 IFN (kW)	37±	3.7
• PERFORMANCE SAE (kW)	39.5	N/A
• SPEED (RPM)	2300	3600
VIBRATORY SYSTEM		
• FREQUENCY (Hz)	32±	68±
• AMPLITUDE (mm)	1.24±	N/A
• CENTRIFUGAL FORCE (KN)	66±	30±

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS					
	13–SEP–12	0	NR	ISSUED FOR CONSTRUCTION	
	DATE	REV.	BY	DESCRIPTION	
DESIGN	SF	CHK	NSV	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN	MM	CHK	DD	SITE 6–709	DATE 20–DEC–11

DOC: 285380–04–094–WIP1–2939

CONSTRUCTION NOTES – EXPANDED POLYSTYRENE FILL

1.0 GENERAL REQUIREMENTS

- 1.1.THE REQUIREMENTS ON THIS DRAWING RELATE TO THE CONSTRUCTION OF THE EXPANDED POLYSTYRENE (EPS) FILL WITHIN BACKFILL AT THE STRUCTURES AND HIGH EMBANKMENTS TO BE BUILT ALONG THE WINDSOR–ESSEX PARKWAY (WEP) PROJECT AS ILLUSTRATED ON THE DRAWINGS. THE REQUIREMENTS GIVEN HEREFTER ARE THE PRINCIPAL REQUIREMENTS. FOR DETAILED REQUIREMENTS, THE CONTRACTOR SHOULD REFER TO MTO MATERIAL SPECIFICATION REQUIREMENTS STATED IN NSSP EXPANDEDPOLYSTYRENEREQUIREMENT.DOC.
- 1.2.THESE CONSTRUCTION NOTES ARE TO READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN DRAWINGS AND REPORT.
- 1.3.THE CONSTRUCTION WORKS SHALL BE EXECUTED IN ACCORDANCE WITH THE GEOTECHNICAL DESIGN ILLUSTRATED ON THE ACCOMPANYING DRAWINGS, THE SUPPLIER SPECIFICATIONS AND THE REQUIREMENTS SPECIFIED IN THE FOLLOWING STANDARDS, SPECIFICATIONS AND PUBLICATIONS:

- MTO NSSP EXPANDED POLYSTYRENE REQUIREMENT
- CAN/ULC–S701–11 THERMAL INSULATION, POLYSTYRENE BOARDS AND PIPE COVERING
- ASTM D1621 COMPRESSIVE PROPERTIES OF RIGID CELLULAR PLASTICS
- ASTM C203 BREAKING LOAD AND FLEXURAL PROPERTIES OF BLOCK TYPE THERMAL INSULATION
- ASTM C177 STEADY STATE HEAT FLUX MEASUREMENTS AND THERMAL TRANSMISSION PROPERTIES BY MEANS OF THE HEAT FLOW APPARATUS
- ASTM D2842 WATER ABSORPTION BY RIGID CELLULAR PLASTICS
- ASTM D2863 MEASURING THE MINIMUM OXYGEN CONTENT
- ASTM D2126 RESPONSE OF RIGID CELLULAR PLASTICS TO THERMAL AND HUMID AGING
- ASTM D6817 STANDARD SPECIFICATION FOR RIGID CELLULAR POLYSTYRENE GEOFOAM
- OPSS 201 CLEARING, CLOSE CUT CLEARING, GRUBBING, REMOVAL OF SURFACE AND PILES BOULDERS
- OPSS 212 BORROW
- OPSS 501 COMPACTION
- OPSS 518 DEWATERING
- OPSS 904 CONSTRUCTION SPECIFICATION FOR CONCRETE STRUCTURES
- OPSS 905 CONSTRUCTION SPECIFICATION FOR STEEL REINFORCEMENT FOR CONCRETE
- OPSS 1010 AGGREGATES – GRANULAR A, B, M, AND SELECTED SUBGRADE MATERIAL
- OPSS 1440 MATERIAL SPECIFICATION FOR STEEL REINFORCEMENT FOR CONCRETE
- OPSS 1605 EXPANDED EXTRUDED POLYSTYRENE PAVEMENT INSULATION
- OPSS 1860 GEOTEXTILES
- NCHRP REPORT 529 GEOFOAM APPLICATIONS IN HIGHWAY EMBANKMENTS
- CAN/ULC–S102.2–10–EN BURNING CHARACTERISTICS

- 1.4 IF THERE IS ANY CONFLICT BETWEEN THE REQUIREMENTS GIVEN ON THIS DRAWING AND THE STANDARDS AND SPECIFICATIONS DOCUMENTS LISTED IN SECTION 1.3, THE DESIGNER SHOULD BE CONSULTED FOR CLARIFICATION AND RECOMMENDATION.
- 1.5 IN THE FOLLOWING CONSTRUCTION NOTES, THE CONTRACTOR MEANS PIC AND ITS SUB–CONTRACTORS, THE SUPPLIER MEANS THE MANUFACTURER AND PROPRIETARY SUPPLIER OF THE EPS, THE ENGINEER MEANS THE GEOTECHNICAL SITE ENGINEER, AND THE DESIGNER MEANS THE GEOTECHNICAL DESIGNER OF THE PROJECT.

2.0 SITE PREPARATION

- 2.1 CLEAR AND GRUB SITE AND REMOVE ANY SUBGRADE MATERIAL UNSUITABLE FOR EPS BLOCK PLACEMENT AS PER TECHNICAL SPECIFICATIONS FOR CLEARING, GRUBBING AND STRIPPING (OPSS 201).
- 2.2 DEWATERING: THERE SHALL BE NO STANDING WATER OR ACCUMULATED SNOW OR ICE ON THE SUBGRADE WITHIN THE AREA WHERE EPS BLOCKS ARE PLACED. EPS BLOCKS SHALL NOT BE PLACED ON A FROZEN SUBGRADE (OPSS 518).
- 2.3 PLACE GRANULAR LEVELLING PAD AS PER DRAWINGS BUT NOT LESS THAN 150 mm THICK CONSISTING OF GRANULAR 'A' OR GRANULAR 'B' MATERIAL WITH GRADATION AND PHYSICAL REQUIREMENTS AS SPECIFIED IN OPSS 1010. WHERE LEVELLING PAD IS THICKER THAN 100 mm, THE PAD SHALL BE COMPACTED TO 95% STANDARD PROCTOR MAXIMUM DRY DENSITY.
- 2.4 EPS SHALL NOT BE FOUNDED DIRECTLY ON EXISTING ASPHALT PAVEMENT. THE CONSTRUCTOR SHALL REMOVE EXISTING PAVEMENT IN ADDITION TO ANY MATERIAL CONTAINING HYDROCARBONS AND REPLACE WITH CLEAN GRANULAR MATERIAL. WHERE AN EPS EMBANKMENT IS FOUNDED ABOVE A PRE–EXISTING SUBSURFACE PAVEMENT LAYER THERE SHALL BE MINIMUM 200 mm OF FREE DRAINING LEVELING COURSE BELOW THE EPS BLOCKS.

3.0 MATERIALS

- 3.1 THE CONTRACTOR SHALL SUBMIT INFORMATION ON THE EPS MATERIAL, MANUFACTURER, PHYSICAL AND MECHANICAL PROPERTIES OF THE MATERIAL, AND AGING AND DURABILITY CHARACTERISTICS AS PER THE MTO–NSSP REQUIREMENTS.
- 3.2 THE CONTRACTOR SHALL PROVIDE CERTIFICATE OF COMPLIANCE OF PHYSICAL AND MECHANICAL PROPERTIES AND THE IDENTIFICATION OF THE LABORATORY ACCREDITED BY THE STANDARDS COUNCIL OF CANADA TO TEST THE EPS. THE PHYSICAL AND MECHANICAL PROPERTIES INCLUDE GEOMETRY, NOMINAL DENSITY, COMPRESSIVE STRENGTH, FLEXURAL STRENGTH, THERMAL RESISTANCE, DIMENSIONAL STABILITY, FLAMMABILITY AND WATER ABSORPTION.
- 3.3 THE PRODUCT SHALL BE SUITABLY MARKED TO IDENTIFY ITS TYPE, NUMBER AND THE MANUFACTURER’S NAME OR TRADEMARK.
- 3.4 EPS BLOCKS SHALL MEET ASTM D6817 STANDARD SPECIFICATION FOR RIGID CELLULAR POLYSTYRENE GEOFOAM AS PER THE FOLLOWING:

ASTM DESIGNATION	DENSITY, kg/m ³	COMPRESSIVE RESISTANCE, kPa		MAXIMUM WATER ABSORPTION, %
		AT 1% DEFORMATION	AT 5% DEFORMATION	
EPS 22	22	50	115	4
EPS 24	24	65	140	3
EPS 29	29	75	170	2

- 3.5 TESTING OF EPS SAMPLES SHALL BE UNDERTAKEN ACCORDING TO ASTM D1621 (PROCEDURE A). FOR EACH EPS GRADE PRODUCED BY THE SUPPLIER, A MINIMUM OF ONE SAMPLE SHALL BE TESTED PER 500 m³ FOR THE FIRST 2000 m³. A MINIMUM OF ONE SAMPLE PER 2000 m³ SHALL BE TESTED THEREAFTER.
- 3.6 THE CONTRACTOR SHALL SUBMIT THE METHOD OF DELIVERY, STORAGE, HANDLING AND PROTECTION FROM DAMAGE BY WEATHER, TRAFFIC, CONSTRUCTION STAGING AND OTHER CAUSES AS PER THE RIGID EXPANDED POLYSTYRENE MANUFACTURER’S REQUIREMENTS.
- 3.7 THE CONTRACTOR SHALL PROTECT THE EXPANDED POLYSTYRENE FROM EXPOSURE TO SUNLIGHT TO AVOID ULTRAVIOLET DEGRADATION AS PER MANUFACTURER’S RECOMMENDATION. PROTECTION OF MATERIALS AND WORKS FROM DAMAGE BY WEATHER, TRAFFIC, CONSTRUCTION STAGING, FIRE OR VANDALISM AND OTHER CAUSES SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR.
- 3.8 CONCRETE AND CONCRETE MATERIALS SHALL CONFORM TO OPSS 1350 WITH THE FOLLOWING EXCEPTIONS AND/OR ADDITIONS: CLASS OF CONCRETE 36 MPa AT 28 DAYS, COARSE AGGREGATE 19 mm NOMINAL MAXIMUM SIZE, AIR CONTENT 7% ± 1.5%, AND MAXIMUM SLUMP 60 mm. THE STEEL REINFORCEMENT SHALL CONFORM TO THE REQUIREMENT OF OPSS 1440 AND SHALL BE PLACED IN ACCORDANCE WITH OPSS 905.

4.0 CONSTRUCTION

- 4.1 THE CONTRACTOR SHALL SUBMIT FULL DETAILS OF THE METHOD OF FOUNDATION EXCAVATION AND PREPARATION, CONSTRUCTION OF LEVELLING PAD, METHOD OF PLACEMENT OF THE EPS BLOCKS, AND THE METHODS OF PLACEMENT OF MINIMUM 125 mm THICK REINFORCED CONCRETE BASE PAD, SUBBASE MATERIAL AND SIDE SLOPE COVER.
- 4.2 FOUNDATION EXCAVATION SHALL BE CARRIED OUT TO THE DESIGN ELEVATION SHOWN ON THE DRAWINGS. ANY SOFTENED, LOOSENED OR DELETERIOUS MATERIALS AT THE FOUNDATION FOOTING ELEVATION SHALL BE SUBEXCAVATED AND REPLACED WITH GRANULAR 'A' OR GRANULAR 'B' MATERIAL.
- 4.3 PLACE, LEVEL AND COMPLETE A LAYER OF GRANULAR 'A' OR GRANULAR 'B' MATERIAL IN ACCORDANCE WITH OPSS 501 TO WITHIN ±30 mm OF THE DESIGN ELEVATION. THE LEVELLING PAD SHALL NOT DEVIATE BY MORE THAN 10 mm AT ANY PLACE ON A 3 m STRAIGHT EDGE OVER THE LIMITS OF THE BOTTOM COURSE OF BLOCKS. THE LEVELLING PAD SHALL NOT BE PLACED ON FROZEN GROUND.
- 4.4 THE EPS SHALL BE INSTALLED IN ACCORDANCE WITH THE MANUFACTURER’S INSTRUCTIONS AND GOOD CONSTRUCTION PRACTICE. THE INDIVIDUALLY MARKED BLOCKS SHALL BE PLACED ON THE PREPARED LEVELLING PAD. THE TOP SURFACE OF THE FIRST LAYER OF BLOCKS IS TO BE SET PLANE AND LEVEL. LOCAL TRIMMING OF THE BLOCKS MAY BE NECESSARY. SUBSEQUENT SUCCESSIVE LAYERS SHALL BE ORIENTED WITH THE LONG AXIS OF BLOCKS POSITIONED AT 90° TO THE PREVIOUS LAYER IN ORDER TO AVOID CONTINUOUS JOINTS. BLOCK JOINTS SHALL BE OFFSET AND STAGGERED BETWEEN LAYERS AS ILLUSTRATED ON THE DRAWINGS OR RECOMMENDED BY THE SUPPLIER.
- 4.5 SLOPING END ADJUSTMENTS AT THE ABUTMENTS SHALL BE ACCOMPLISHED BY LEVELLING TERRACES IN THE SUBSOIL IN ACCORDANCE WITH THE BLOCK THICKNESS.

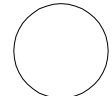
METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



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

NEW CONSTRUCTION
HWY 401
COUSINEAU TUNNEL T–9
CONSTRUCTION NOTES – EXPANDED POLYSTYRENE



SHEET
G2940

Phase 1

IFC

- 4.6 TEMPORARY BALLAST SHALL BE PROVIDED AS NECESSARY TO PREVENT MOVEMENT OF EXPANDED POLYSTYRENE BOTH IN STORAGE AND AS PLACED DUE TO WINDY CONDITIONS. TIMBER FASTENERS OR EQUIVALENT SHALL BE USED AS NECESSARY.
- 4.7 THE EXPANDED POLYSTYRENE FILL/EMBANKMENTS SHALL BE PROTECTED FROM ACCIDENTAL IGNITION DUE TO WELDING, SMOKING, GRINDING OR CUTTING TOOLS, ETC. THE CONTRACTOR SHALL TAKE ALL NECESSARY PRECAUTIONS TO PREVENT IGNITION OF THE EXPANDED POLYSTYRENE.
- 4.8 THE EXPANDED POLYSTYRENE SHALL BE PROTECTED FROM ORGANIC SOLVENTS AND OTHER AGGRESSIVE, HARMFUL CHEMICALS DURING CONSTRUCTION. THE PROPOSED METHOD OF PROTECTION DURING CONSTRUCTION SHALL BE SUBMITTED TO THE CONTRACTOR’S QUALITY VERIFICATION ENGINEER FOR REVIEW AND TO THE CONTRACT ADMINISTRATOR FOR INFORMATION PURPOSES.
- 4.9 EXPOSED BLOCKS SHALL BE COVERED IMMEDIATELY TO AVOID POSSIBLE BURROWING BY ANIMALS.
- 4.10 INDIVIDUALLY MARKED BLOCKS SHALL BE FABRICATED AND PLACED TO ENSURE THE TOP SURFACE MATCHES THE ELEVATION AND CROSSFALL SHOWN ON THE DRAWINGS.
- 4.11 THE TOP SURFACE AND SIDE SURFACES OF THE EXPANDED POLYSTYRENE SHALL BE COVERED WITH 10 MIL POLYETHYLENE SHEETING EXTENDING ONTO ADJACENT WORK AT THE LONGITUDINAL ENDS OF THE EMBANKMENT/ABUTMENTS. ALL JOINTS SHALL BE LAPPED A MINIMUM 300 mm TO PROVIDE A FULLY SEALED ENCLOSURE. THE JOINTS IN THE LONGITUDINAL AND TRANSVERSE DIRECTIONS SHALL BE ARRANGED TO OVERLAP THE BLOCKS IN THE LOWER LAYER OF THE EPS.
- 4.12 THE CONTRACTOR SHALL INSTALL THE CONCRETE PAD COVER AS DESCRIBED IN SECTIONS 3.8 AND 4.1 ABOVE. THE STEEL REINFORCEMENT SHALL BE PLACED IN ACCORDANCE WITH OPSS 905. THE TOP OF THE EPS SHOULD BE SLOTTED TO PREVENT RELATIVE DISPLACEMENT BETWEEN THE EPS AND CONCRETE PAD.
- 4.13 THE CONTRACTOR SHALL SUBMIT DETAILS OF THE SEQUENCE AND METHOD OF INSTALLATION TO THE ENGINEER FOR REVIEW AT LEAST 3 WEEKS PRIOR TO THE INSTALLATION OF THE EPS. THE SUBMITTAL SHALL SATISFY ALL SPECIFICATIONS.
- 4.14 TRAFFIC: EQUIPMENT OTHER THAN RUBBER–TIRE SAWING EQUIPMENT SHALL NOT BE PERMITTED ON THE CONCRETE UNTIL IT HAS ATTAINED A MINIMUM COMPRESSIVE STRENGTH OF 2 MPa. A LIFT OF GRANULAR NO LESS THAN 600 mm IN THICKNESS SHALL BE PLACED ON THE CONCRETE PAD BEFORE TRAFFIC IS PERMITTED. EQUIPMENT SHALL BE LIMITED IN WEIGHT AND SIZE AND RESTRICTED IN OPERATION TO AVOID DAMAGING THE EPS AS PER THE SUPPLIER’S REQUIREMENT.

5.0 DRAINAGE

- 5.1 TOP SURFACE OF EPS BLOCKS SHALL BE STEPPED OR SLOPED TO MATCH SUPER ELEVATION OR CROSSFALL. DRAINAGE CHANNELS COMPRISING 19 mm CLEAR CRUSH STONE WRAPPED WITH NON–WOVEN GEOTEXTILE (AMOCO 4545 OR APPROVED EQUIV.) SHALL BE PROVIDED UNLESS REQUIRED OTHERWISE BY THE DESIGNER OR NOTED ON THE DESIGN DRAWINGS. SUBDRAINS SHALL BE PERFORATED PVC DRAIN PIPE WITHIN 19 mm CLEAR CRUSH BEDDING WRAPPED IN NON–WOVEN GEOTEXTILE AS PER DESIGN DRAWINGS.
- 5.2 APPROPRIATE DRAINAGE SHALL BE PROVIDED IN EPS EMBANKMENT/FILL FOUNDATION TO ENSURE EFFECTIVE DRAINAGE AND PREVENT PRESENCE OF STANDING WATER OR ACCUMULATED SNOW OR ICE ON THE SUBGRADE WITHIN THE AREA WHERE EPS BLOCKS ARE PLACED.

6.0 USE

- 6.1 THIS DRAWING PROVIDES CONSTRUCTION REQUIREMENTS FOR GEOTECHNICAL ASPECTS OF BACKFILLING AT TUNNELS.

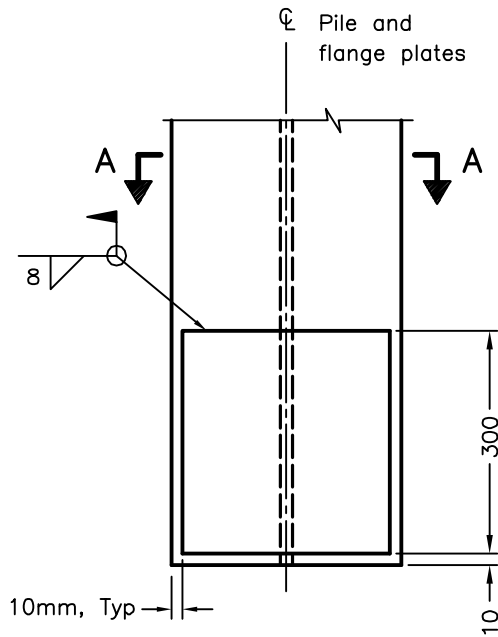
DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS					
	13–SEP–12	0	NR	ISSUED FOR CONSTRUCTION	
	DATE	REV.	BY	DESCRIPTION	
DESIGN	SF	CHK	NSV	CODE CAN/CSA S6-06	LOAD CL–625–ONT
DRAWN	MM	CHK	DD	SITE 6–709	DATE 20–DEC–11

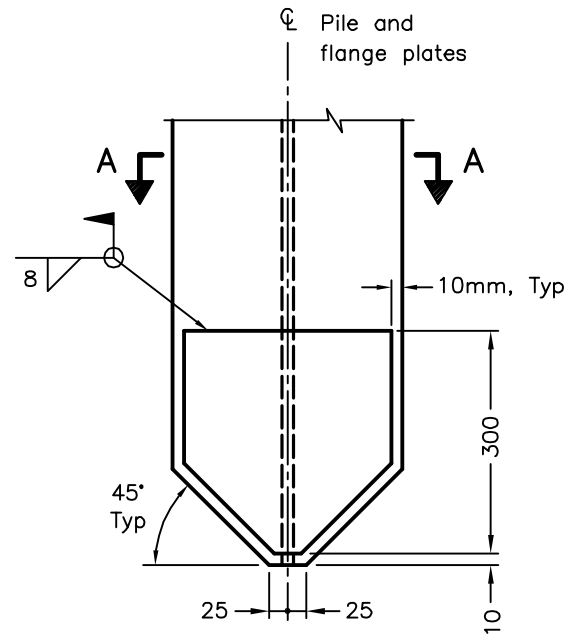
Applicable OPSDs

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-9 (Cousineau Tunnel) (Sta. 12+130L to 12+300L)
Doc No.: 285380-04-119-0050 (Geocres No. 40J3-19)

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

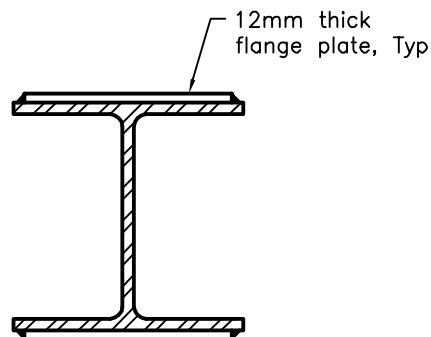


TYPE I



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

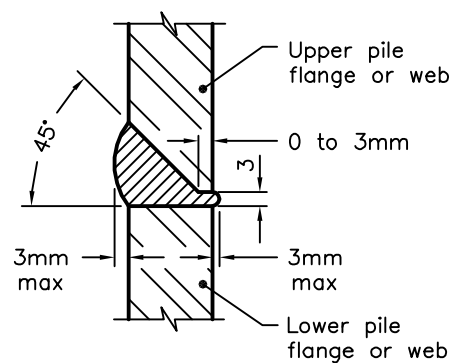
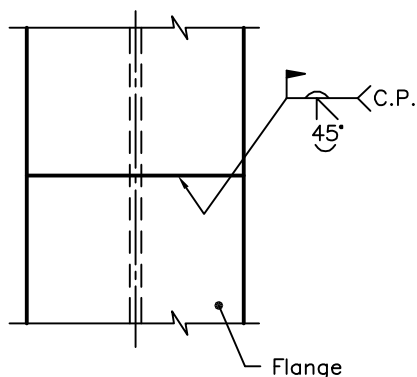
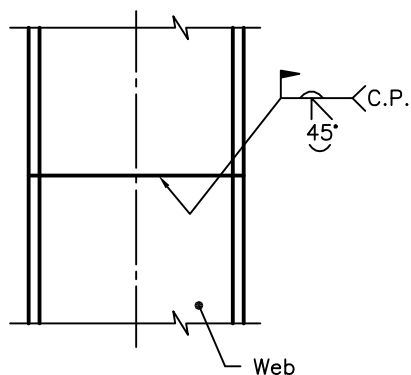
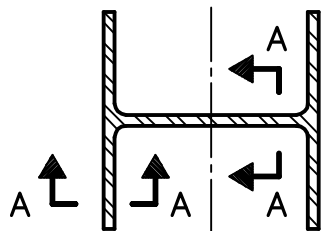
Rev 2

FOUNDATION
PILES

STEEL H-PILE DRIVING SHOE

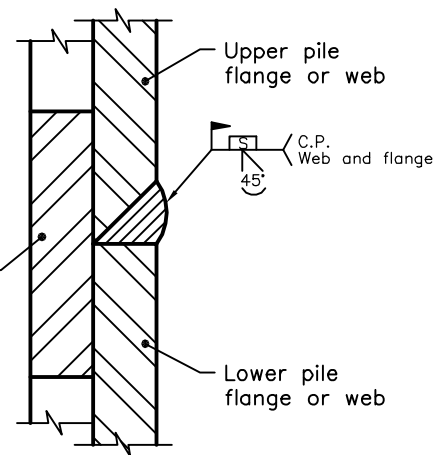
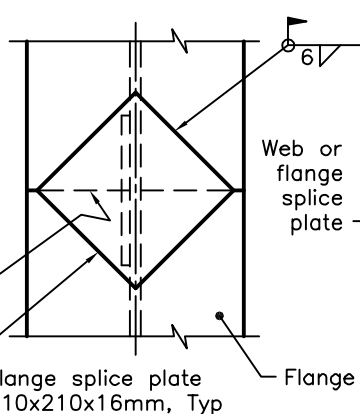
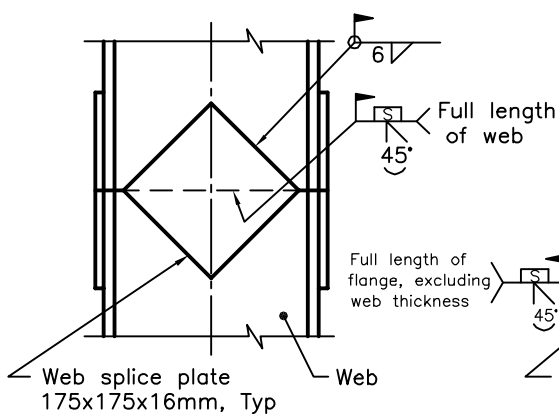
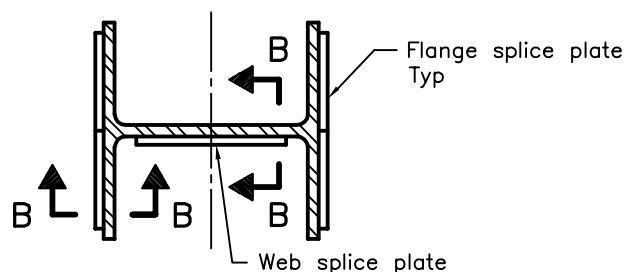
OPSD 3000.100





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

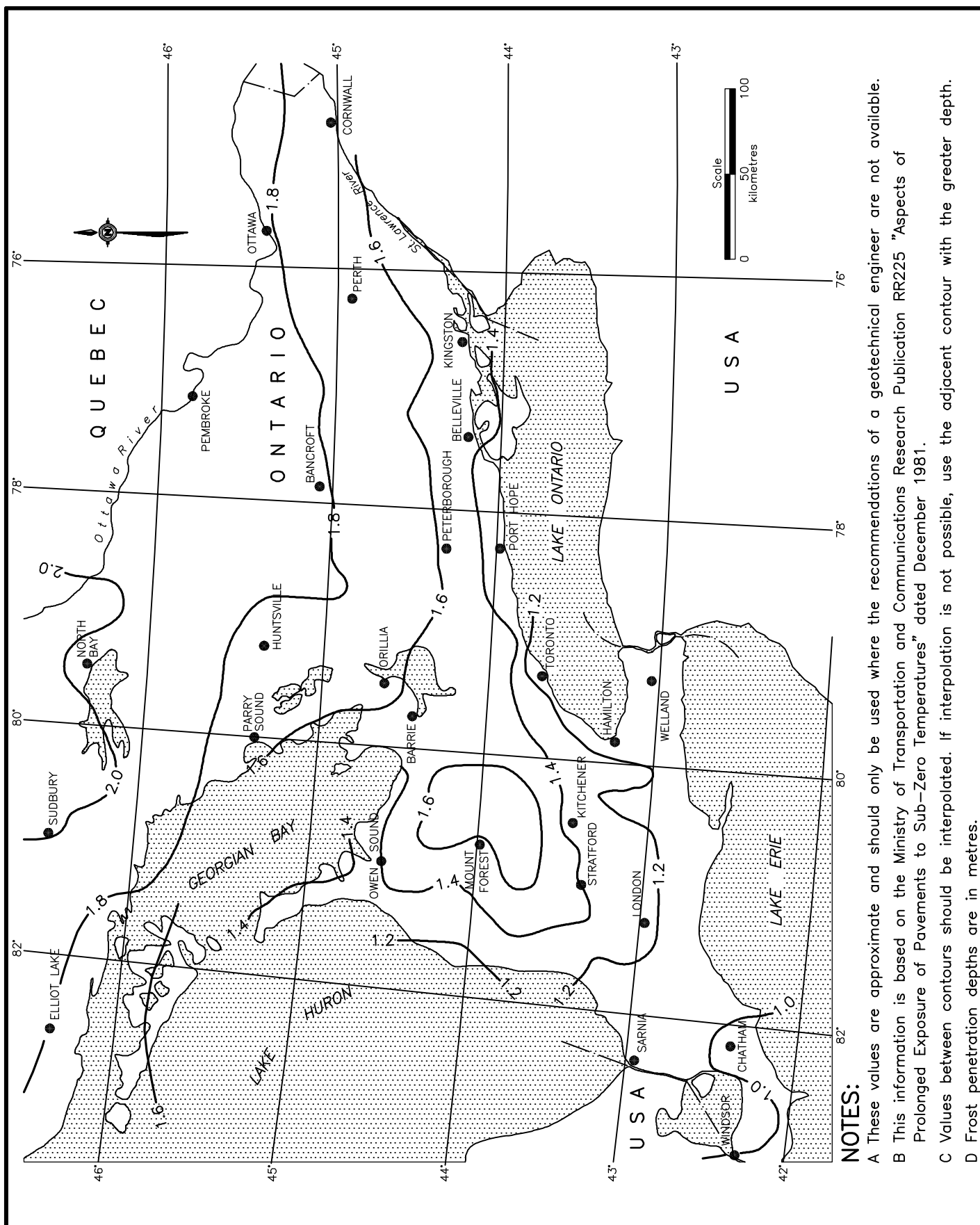
Rev

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

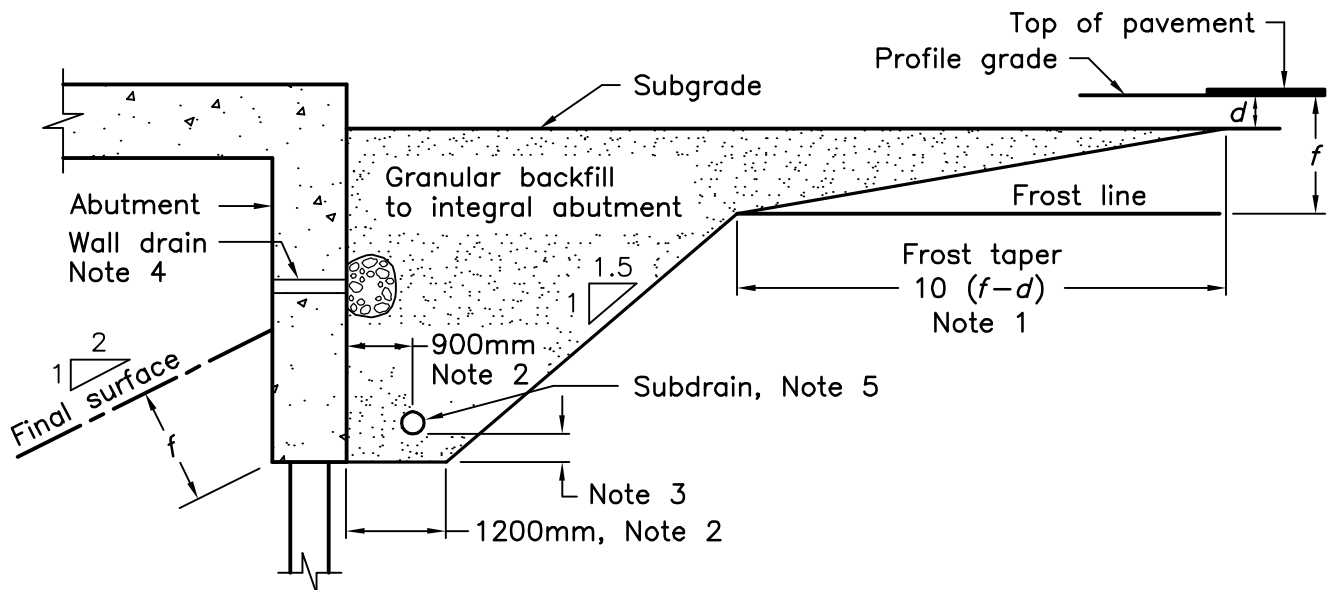
Nov 2010

Rev 1

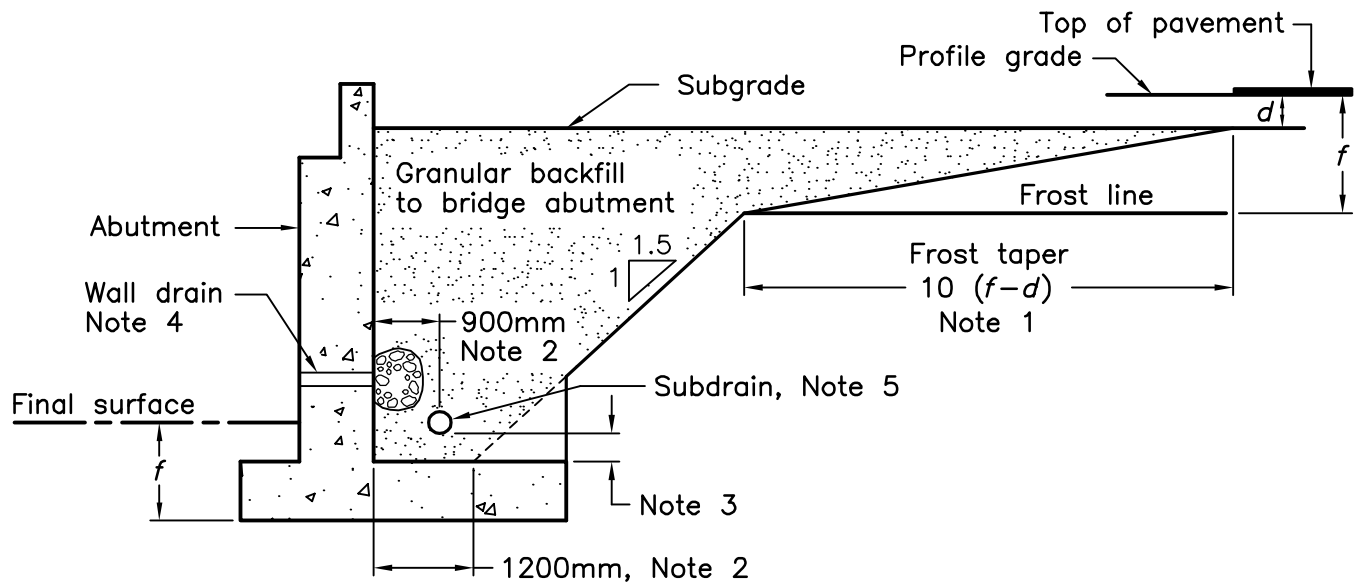
FOUNDATION FROST PENETRATION DEPTHS FOR SOUTHERN ONTARIO



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

OPSD 3101.150

Figures

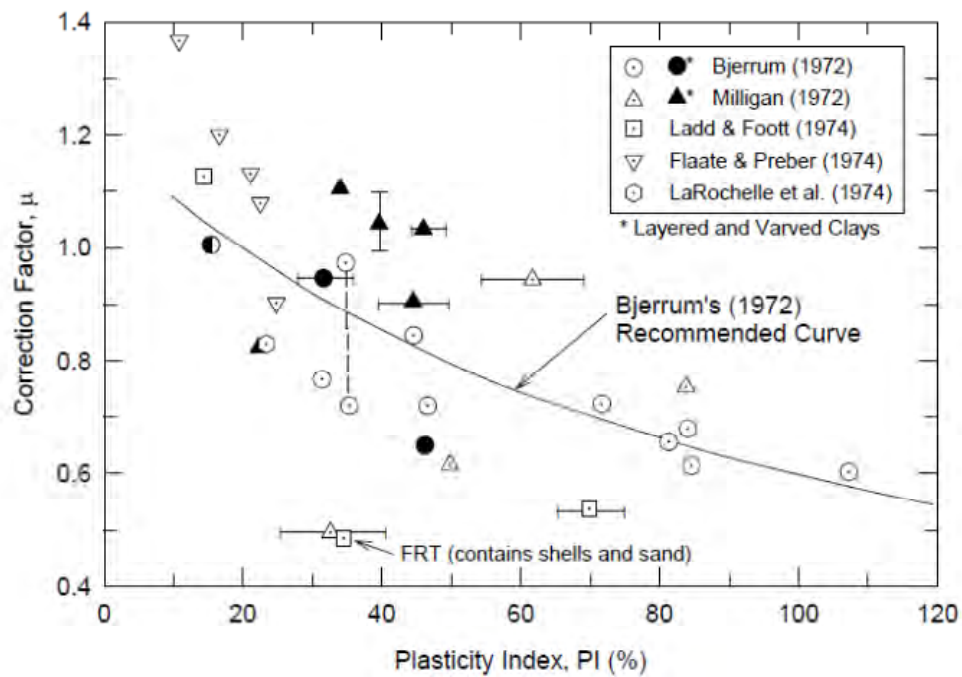


Figure 3.1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures (Ladd & DeGroot, 2004)

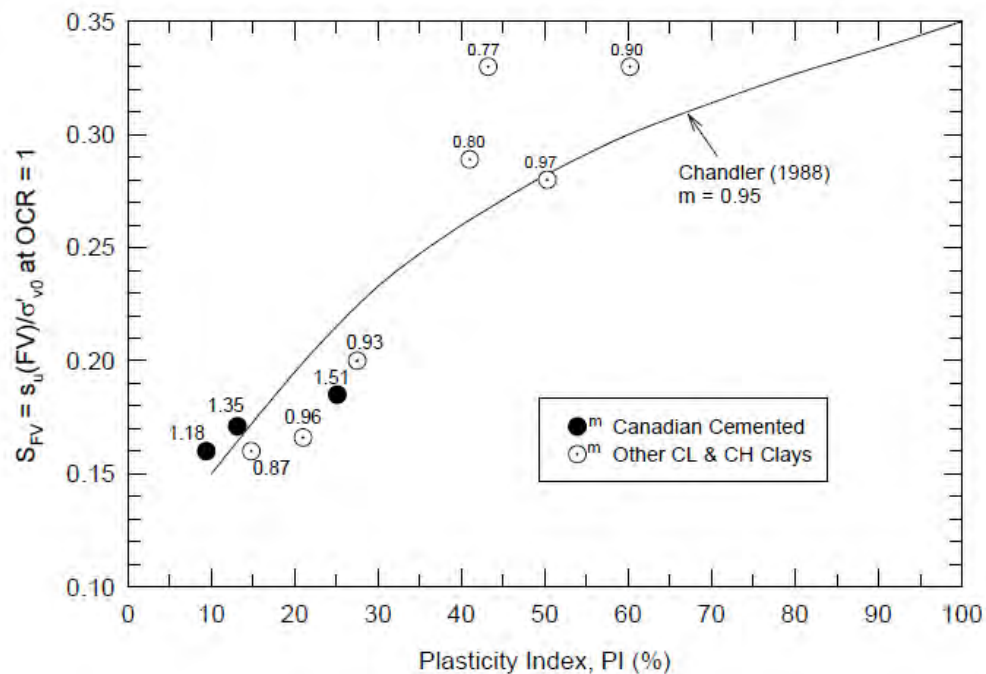
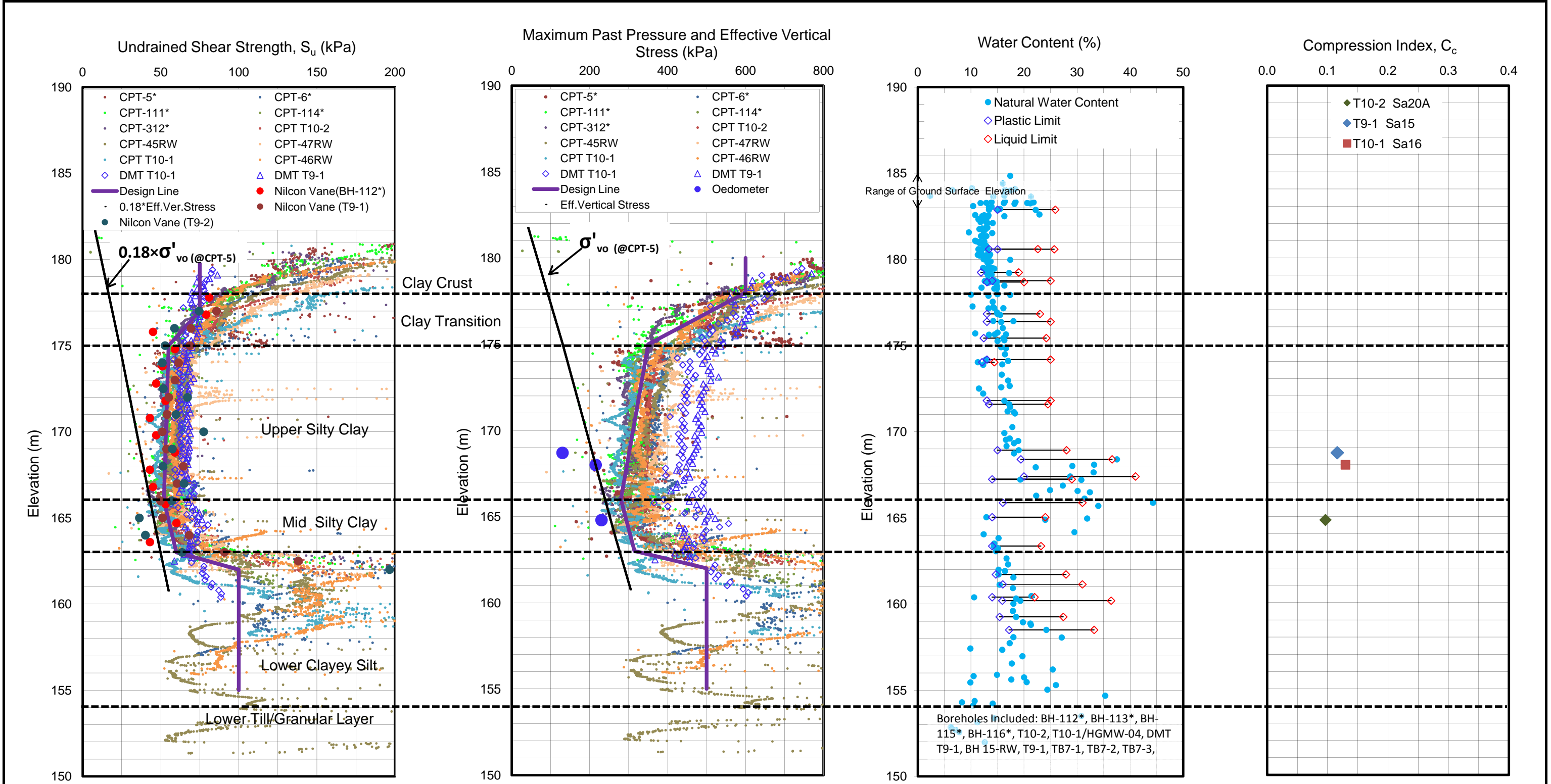


Figure 3.2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays (Ladd & DeGroot, 2004)




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 (ref. R-16 to R-23).



Environment & Infrastructure

CLIENT :

PROJECT: WINDSOR ESSEX PARKWAY	
TITLE: SOIL PROPERTIES PROFILES STA.12+000L TO 12+800L	
DATE: Sep 2011	JOB NO.: SW8801.1002
CAD FILE:	FIGURE NO.: 3.3
REV.	

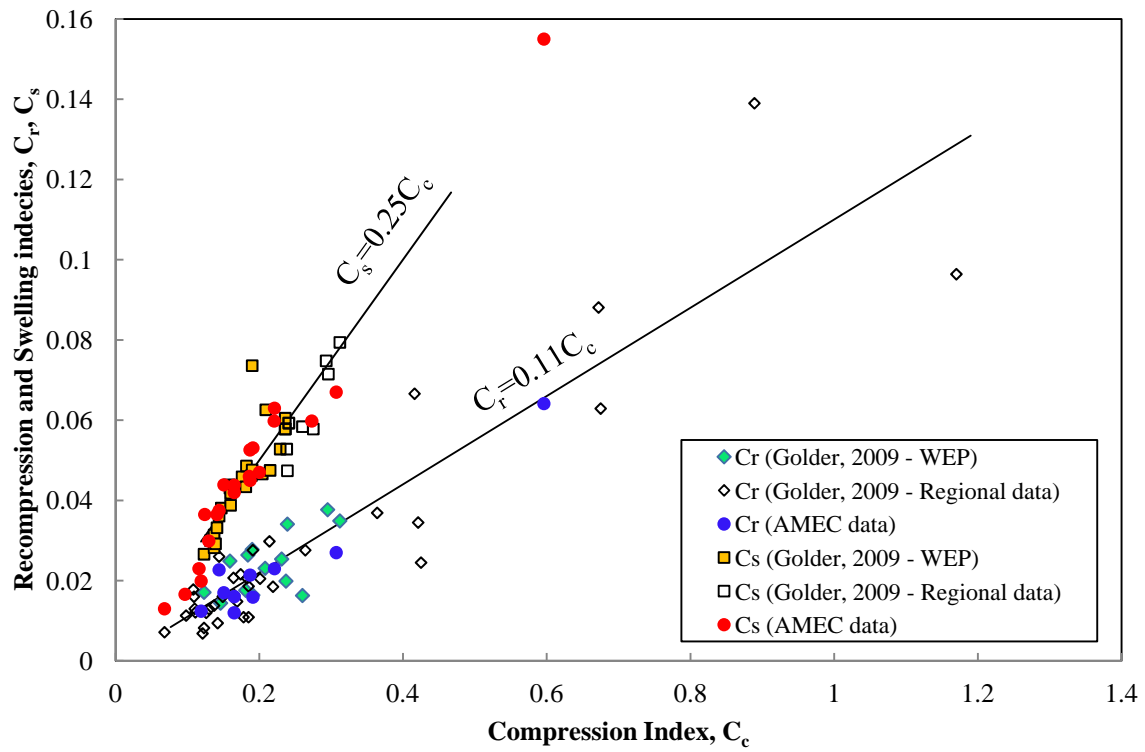
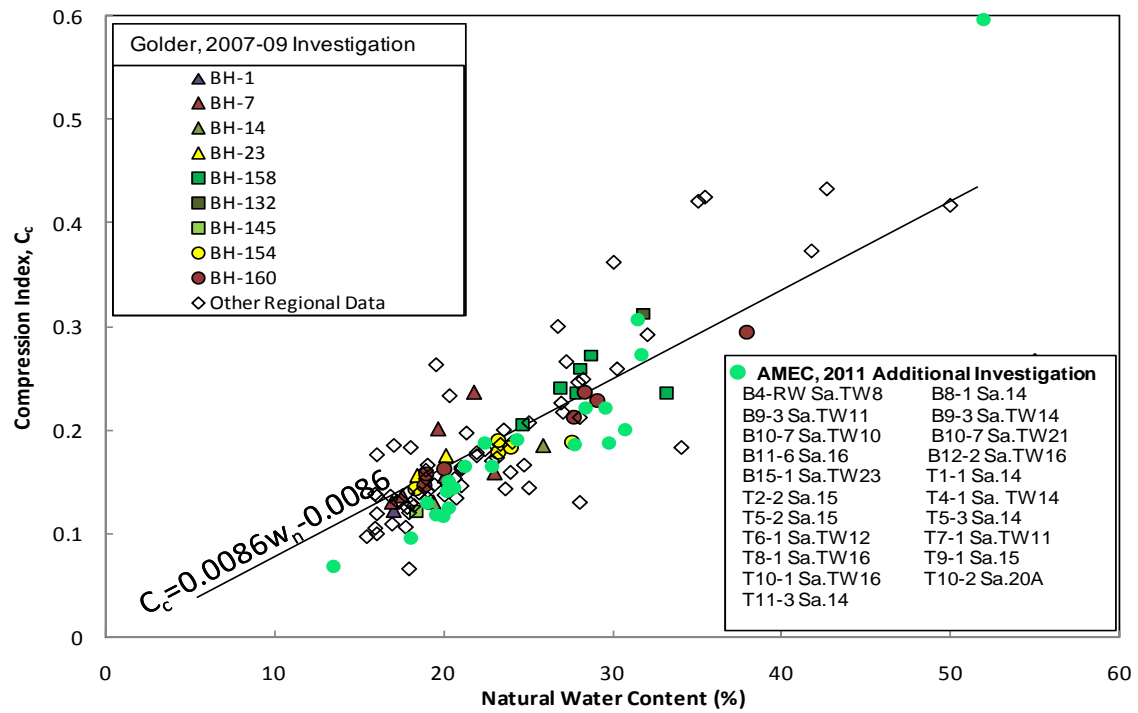


Figure 4.1: Compressibility Parameters at WEP

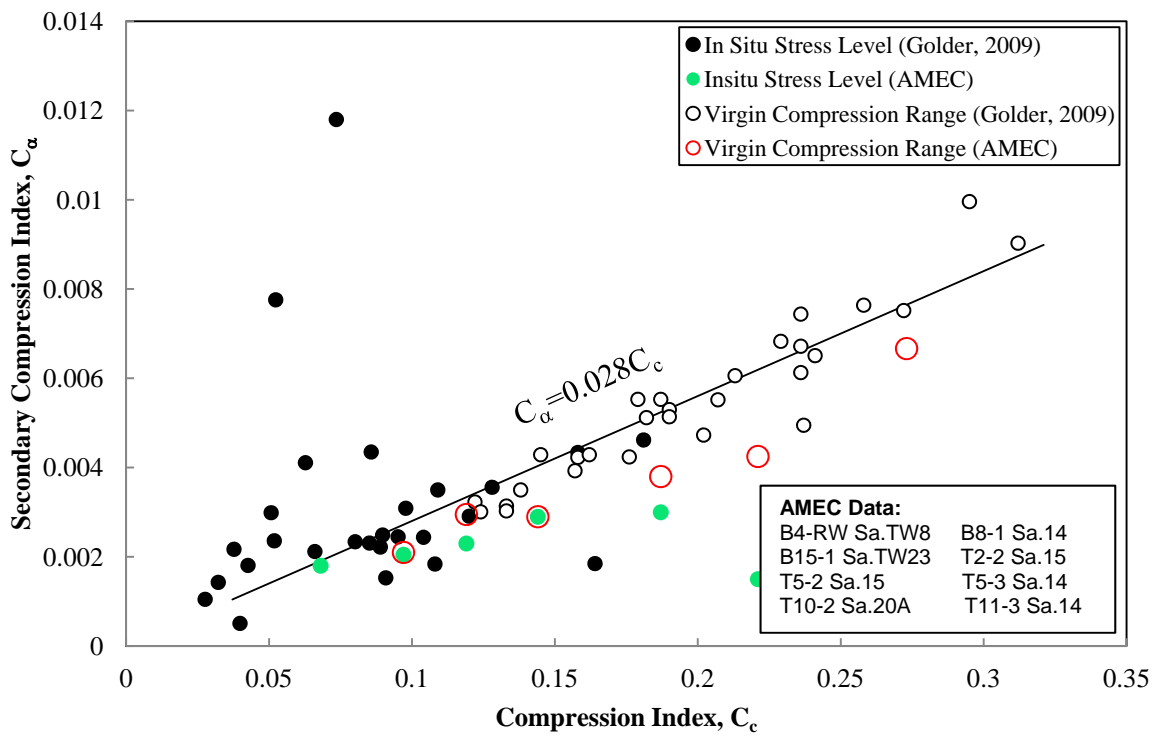


Figure 4.2: C_c versus C_α Relationship at WEP

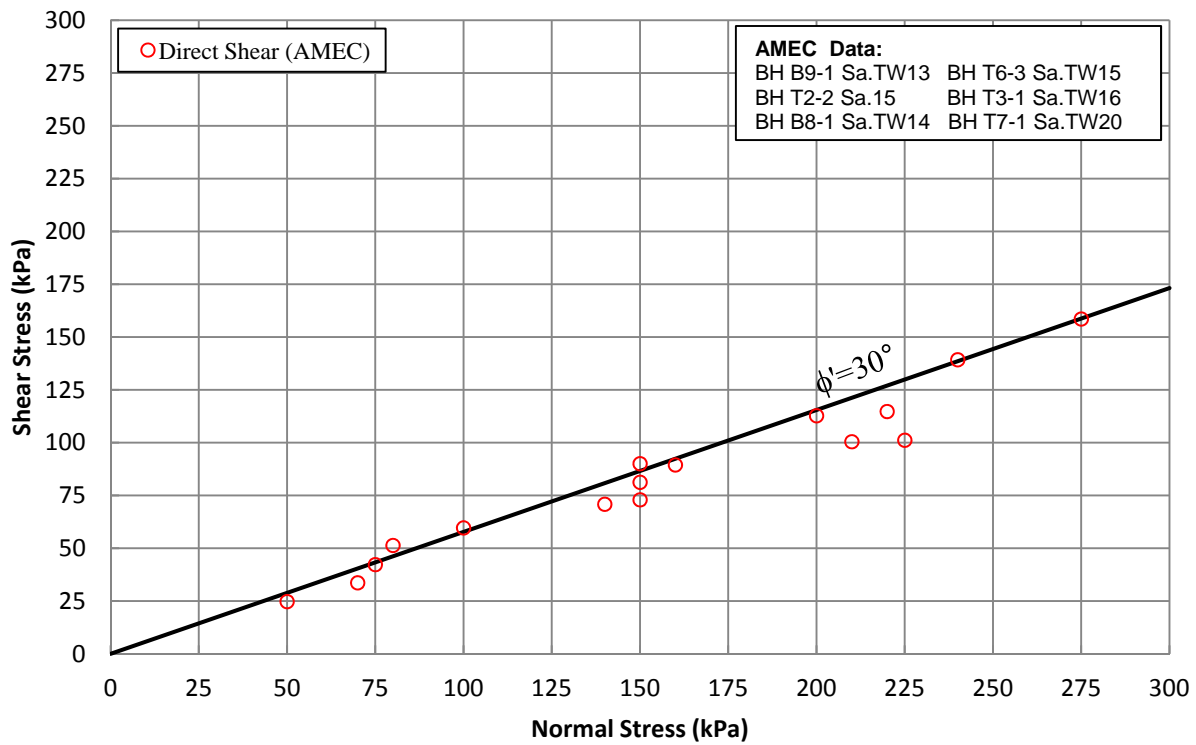
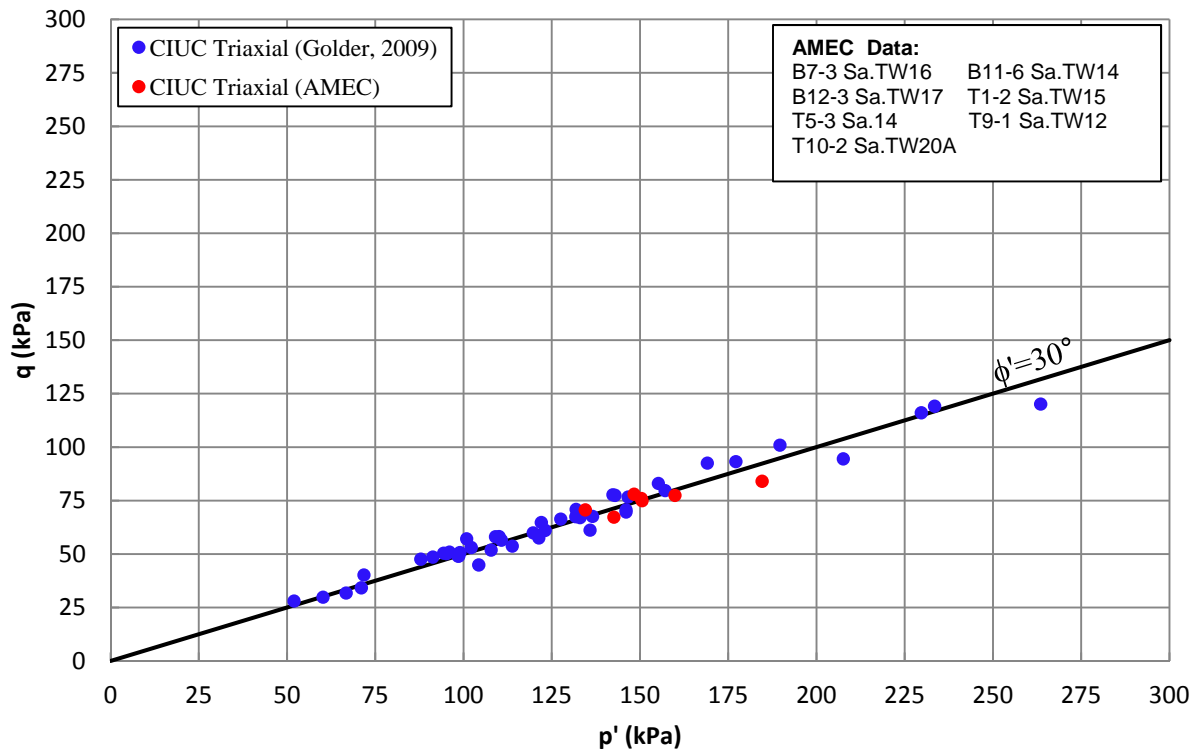


Figure 4.3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP

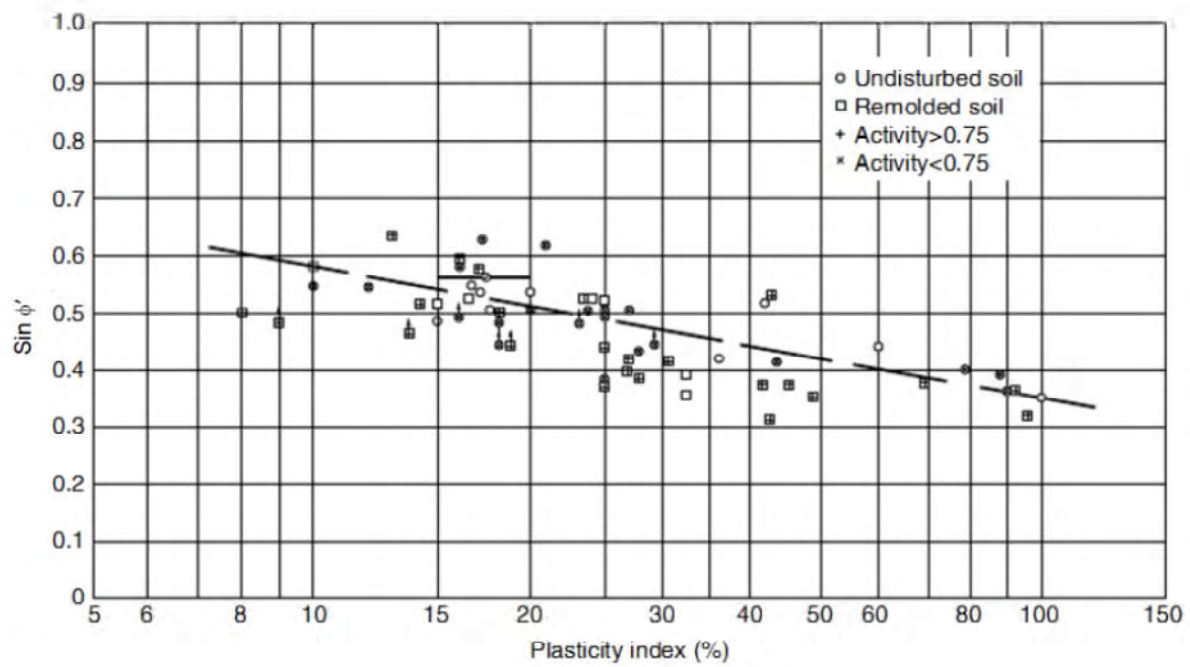


Figure 4.4: Relationship between $\sin \phi'$ and Plasticity Index for Normally Consolidated Soils
(Kenney, 1959)

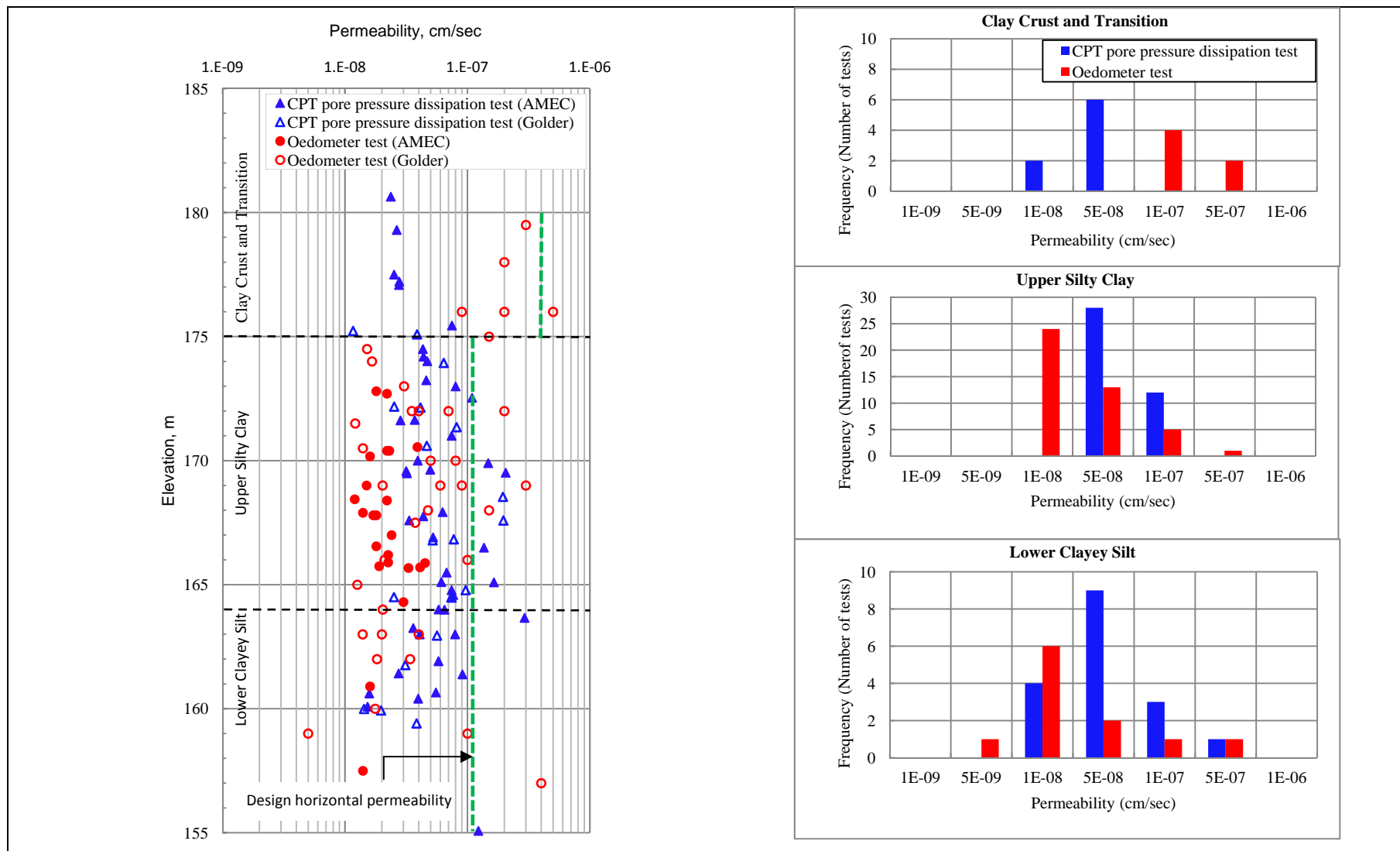
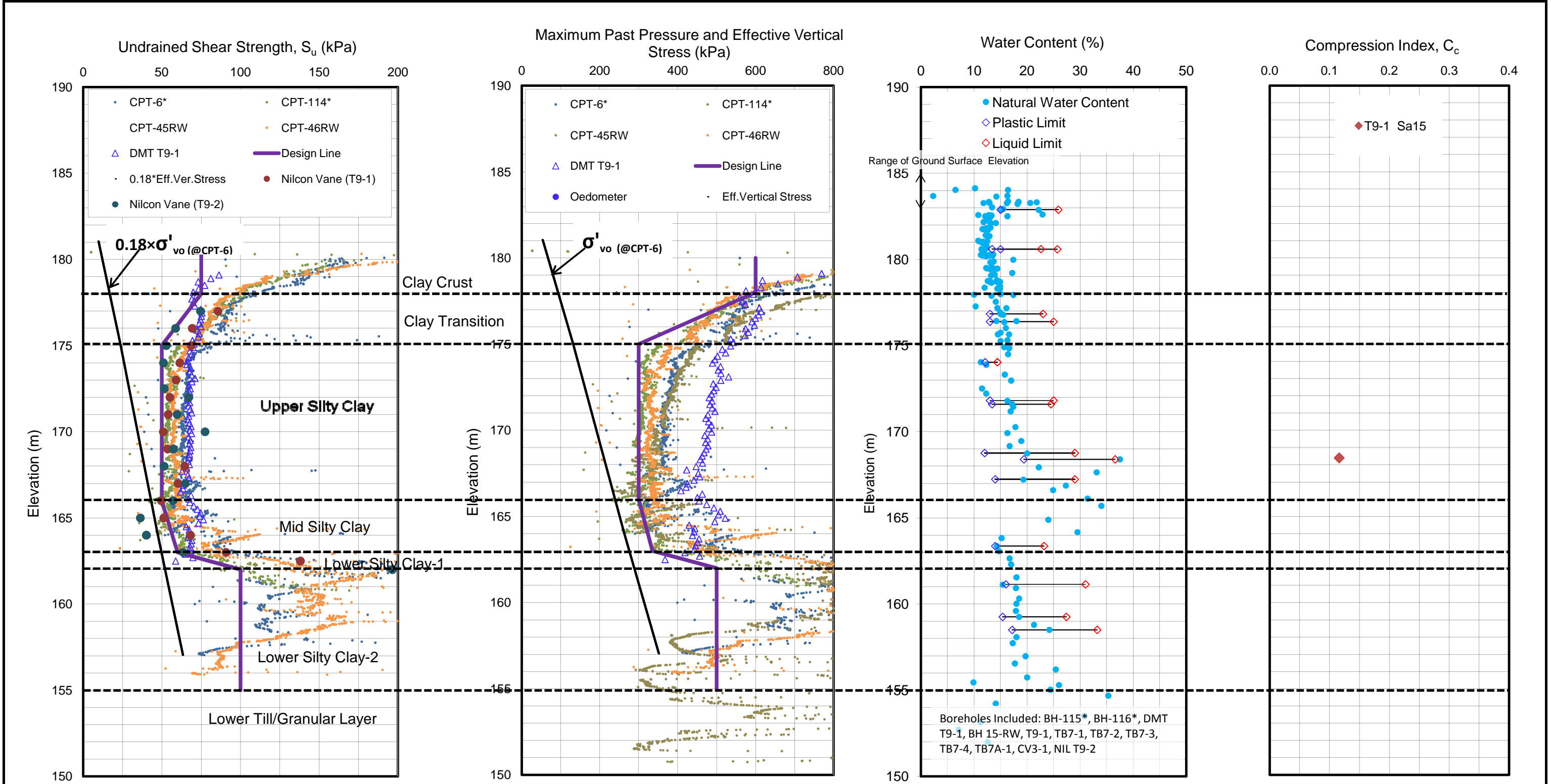


Figure 4.5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests




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

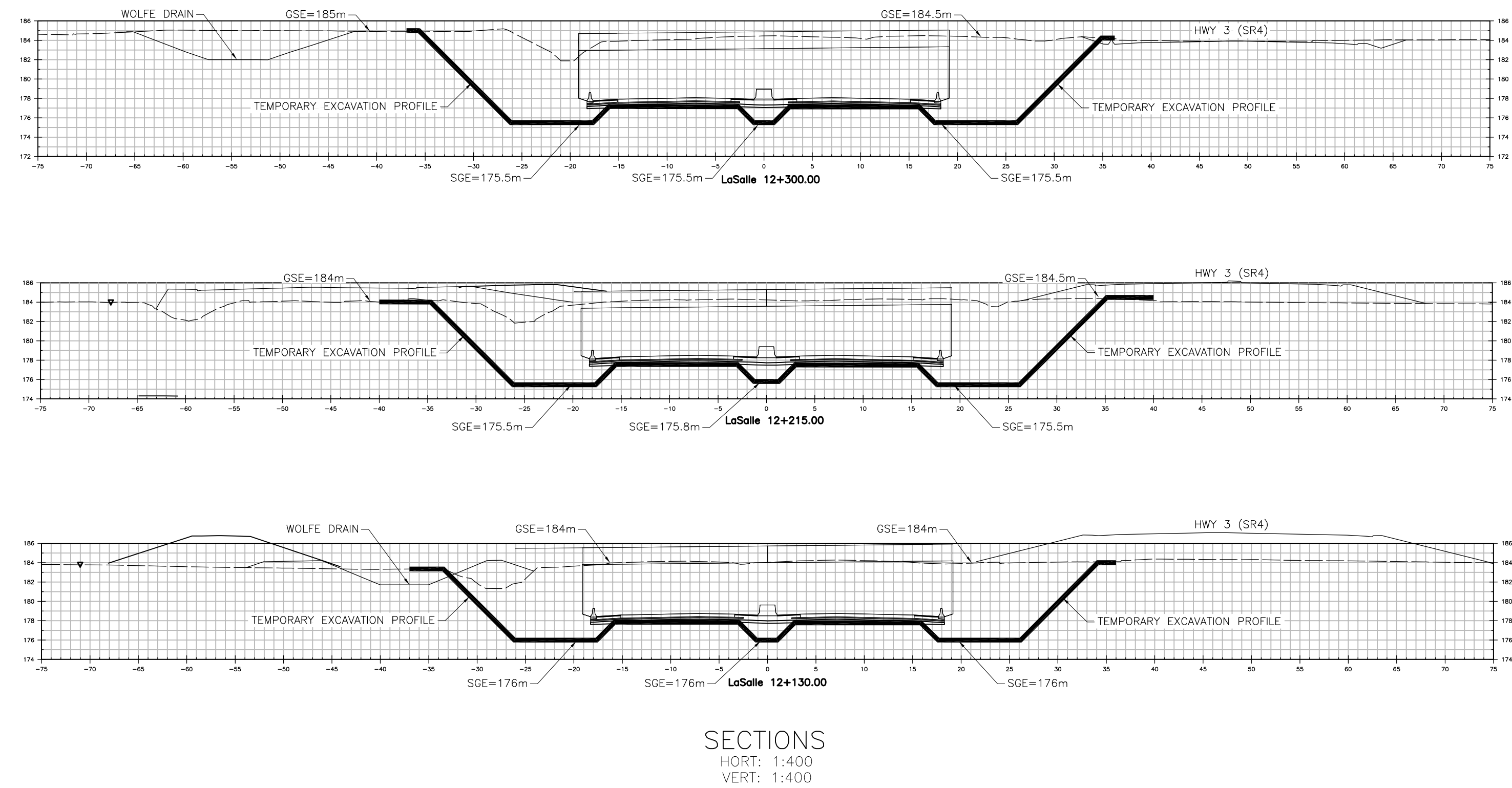
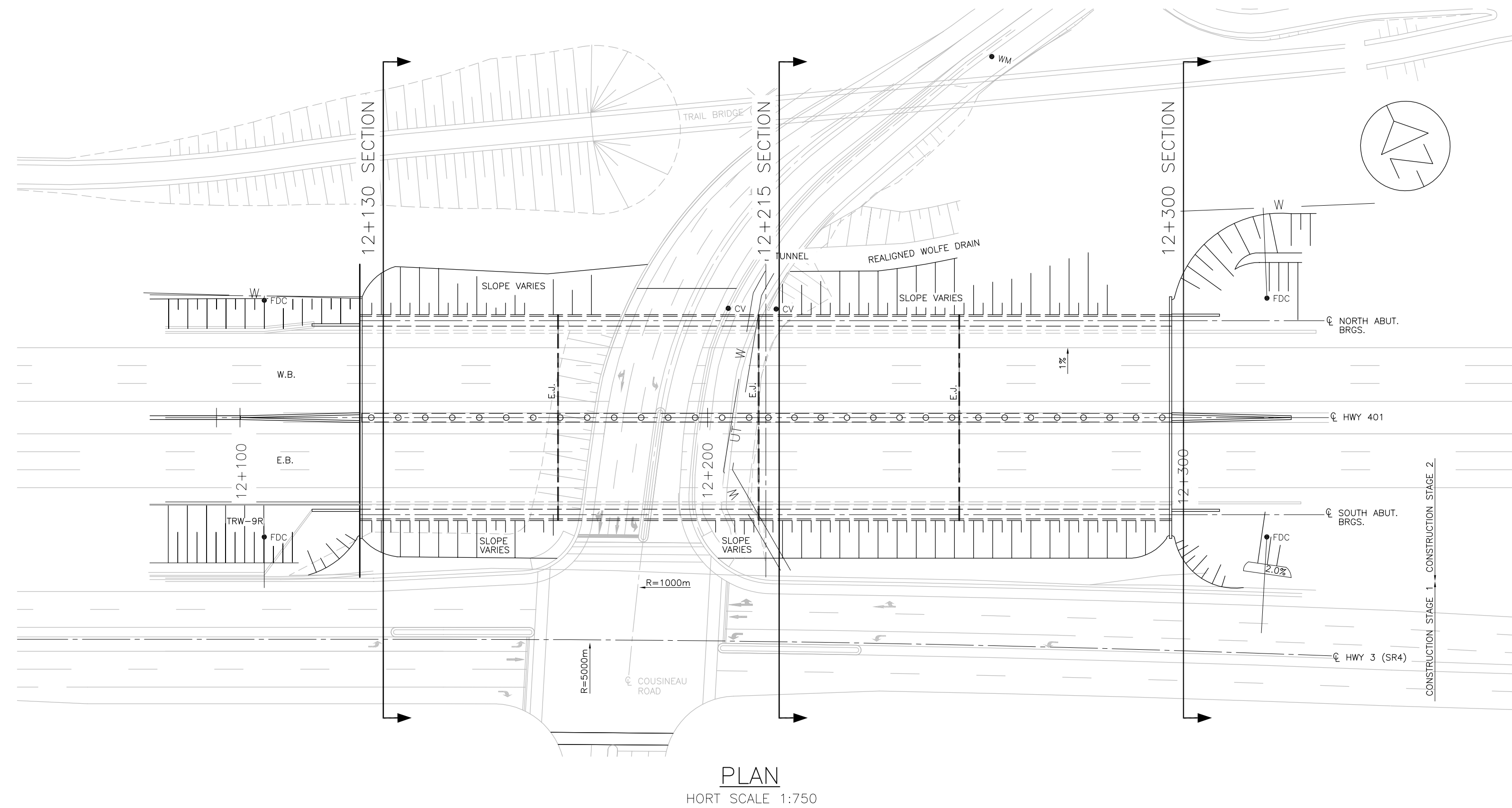
*From previous investigations (ref. R-16 to R-23).



Environment & Infrastructure

CLIENT :

PROJECT: WINDSOR ESSEX PARKWAY	
TITLE: SOIL PROPERTIES PROFILES AT AND AROUND TUNNEL T-9	
DATE: Sep 2011	JOB NO.: SW8801.1002
CAD FILE:	FIGURE NO.: 5.1
REV.	



DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR
CONSTRUCTION

Appendix A Borehole, Nilcone Vane, CPT and DMT Logs from Additional 2011 Geotechnical Investigation

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-9 (Cousineau Tunnel) (Sta. 12+130L to 12+300L)
Doc No.: 285380-04-119-0050 (Geocres No. 40J3-19)

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

RECORD OF BOREHOLE No T9-1

1 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678634.9, E333766.7 ORIGINATED BY NB
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Jul 15, 11 - Jul 16, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								20 40 60 80 100								
								20 40 60 80 100								
184.0	Pavement Surface															GR SA SI CL
0.0	280mm ASPHALT															
183.7	FILL		1	SS												
0.3	Crushed Limestone															
183.0	Silty sand and gravel		2	SS	7											
1.0	Grey															
	CLAYEY SILT															
	Some sand, trace gravel															
	Stiff to hard															
	Brown		3	SS	13											
			4	SS	40											
			5	SS	48											
			6	SS	30											
	Grey		7	SS	16											
			8	SS	12											
			9	SS	11											
			VT													
	Numerous Sand Layers between Elevations 176.4m and 167.2m		10	TW	PH											
			11	TW	PH											
			VT													
			12	TW	PH											
			13	TW	PH											
			VT													
			14	TW	PH											

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 19/12/11

-Slope
Inclinometer
casing installed in
sampled
borehole;
Vibrating Wire
Piezometers
(VWP) installed
in adjacent
boring at
N4678635.6,
E333769.0
-Spider Magnets
(MG) installed in
adjacent boring
Nilcon vane
advanced
adjacent to
sampled
borehole from 7
m to 21.5 m (El.
177.0 m to El.
162.5 m)

-end of drilling
July 15;
continued July 16

-VWP T9-1-P9
and MG
T9-1-SM9
installed at 9.1m
below ground
surface (El. 174.9
m)

-Switched to
wash boring at
14.6m below
ground surface
(El. 169.4m)

METRIC

ELEV DEPTH	SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	WATER CONTENT (%) W _p W W _L 10 20 30	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
								○ UNCONFINED ● POCKET PEN.					+ FIELD VANE × LAB VANE
							20 40 60 80 100 						
							20 40 60 80 100 						

[illegible]

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 19/12/11

METRIC

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


ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 19/12/11

RECORD OF BOREHOLE No NIL T9-2

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678636.5, E333765.3 ORIGINATED BY SD
 DIST HWY WEP BOREHOLE TYPE CME 850 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Aug 15, 11 - Aug 15, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED	+	FIELD VANE									
								● POCKET PEN.	×	LAB VANE									
184.0																			
0.0	TOPSOIL																		
0.2	CLAYEY SILT Some sand, trace gravel Stiff to very stiff Mottled brown and grey																		
				1	SS	10													
	Brown			2	SS	26													
				3	SS	24													
				4	SS	21													
	Grey		5	SS	12														
			6	SS	10														
			7	SS	8														
			8	SS	5														
177.4	END OF BOREHOLE Continued with Nilcon vane from 7.0 m to refusal at 22.0 m (El. 177.0 to El. 162.0 m)																		
6.6	Borehole dry on completion																		
							177												
							176												
							175												
							174												
							173												
							172												
							171												
							170												



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

RECORD OF BOREHOLE No DMT T9-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678544.5, E333900.9 ORIGINATED BY LC
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Jul 19, 11 - Jul 19, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
								○ UNCONFINED	○ POCKET PEN.	+ FIELD VANE	× LAB VANE							
184.1	Ground Surface																	
0.0	Clayey TOPSOIL																	
183.7																		
0.4	CLAYEY SILT Some sand, trace gravel Stiff to hard Mottled brown and grey		1	AS														
	-Weathered fissures -Some sand, trace gravel with topsoil/organics in fissures		2	SS														
	Brown fissures		3	SS														
	Oxidized		4	SS														
	Silty fissures Grey		5	SS														
179.8	END OF SAMPLED BOREHOLE DMT advanced from 0.2 m to refusal at 21.6 m (El. 183.9 m to El. 162.5 m)																	
4.3	No groundwater observed on July 19, 2011																	
							179											
							178											
							177											
							176											
							175											
							174											
							173											
							172											
							171											
							170											

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

RECORD OF BOREHOLE No BH15-RW

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678559.2, E333806.1 ORIGINATED BY LC
 DIST HWY WEP BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Jul 15, 11 - Jul 15, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)			
								○ UNCONFINED	○ POCKET PEN.	+ FIELD VANE						× LAB VANE			
184.1	Fill Surface																		
0.0	<div><div><div>FILL</div><div>Silty Topsoil to Mixed Clay/Silt/Sand/Roots/concrete</div><div>FILL</div><div>Soft Clay/Topsoil</div></div></div>																		
0.1			1	SS	2														
182.6																			
1.5	<div><div><div>SILTY CLAY</div><div>Weathered, fissures Some sand, trace gravel Stiff Brown</div></div></div>		2	SS	11														
181.8																			
2.3	<div><div><div>CLAYEY SILT</div><div>Hard to firm Brown</div><div>Some sand, trace gravel Moist Fissured occasionally</div><div>Grey</div></div></div>		3	SS	31														
			4	SS	39														
			5	SS	22														
			6	SS	14														
			7	SS	9														
			8	SS	7														
177.5	<div><div><div>END OF BOREHOLE</div><div>Borehole dry on completion</div></div></div>																		
6.6																			

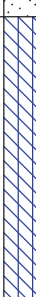
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CPT45-RW

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678688.3, E333708.0 ORIGINATED BY TA
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Aug 9, 11 - Aug 9, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE												
183.4	Ground Surface						20	40	60	80	100									
0.0	TOPSOIL																			
183.1	CLAYEY SILT Some sand, trace gravel, trace fissures Firm to hard Mottled brown and grey		1	SS	3		183													
0.3			2	SS	7		182													
							181													
180.4	Brown		3	SS	38															
3.0	END OF SAMPLED BOREHOLE Continued with CPT from 3.5 m to refusal at 32.8 m (El. 179.9 m to El. 150.6 m) Borehole dry on completion						180													
							179													
							178													
							177													
							176													
							175													
							174													
							173													
							172													
							171													
							170													
							169													

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

RECORD OF BOREHOLE No CPT46-RW

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678505.0, E333977.6 ORIGINATED BY TA
DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE Aug 5, 11 - Aug 5, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE									
184.3	Ground Surface						20	40	60	80	100						
0.0	TOPSOIL		1	SS	10												
184.0	CLAYEY SILT Some sand, trace gravel Firm to hard Mottled brown and grey		2	SS	5												
0.3																	
	Brown Trace fissures		3	SS	36												
181.3	END OF SAMPLED BOREHOLE (continued with CPT to refusal)																
3.0	Borehole dry on completion																
							181										
							180										
							179										
							178										
							177										
							176										
							175										
							174										
							173										
							172										
							171										
							170										

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

RECORD OF BOREHOLE No CV3-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678630.0, E333861.1 ORIGINATED BY TA
 DIST HWY WEP BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Jul 12, 11 - Jul 12, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
184.5	Fill Surface							○ UNCONFINED ● POCKET PEN.	+ FIELD VANE × LAB VANE								
								20 40 60 80 100	20 40 60 80 100	10 20 30							
0.0	FILL Silty clay Some sand, trace gravel Trace to some topsoil Brown						184										
183.6	CLAYEY SILT Some sand, trace gravel Stiff to hard Mottled brown and grey		1	SS	10												
0.9							183										
			2	SS	9												
							182										
	Trace fissures, trace silt seams Brown		3	SS	29												
							181										
			4	SS	40												
							180										
	Grey		5	SS	27												
							179										
			6	SS	10												
							178										
			7	SS	7												
							177										
			8	TW	PH												
							176										
	Numerous Sand Layers at Elevation 176.9 m		9	TW	PH												
							175										
			10	TW	PH												
174.7	END OF BOREHOLE Borehole dry on completion						174										
9.8							173										
							172										
							171										
							170										

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 19/12/11

RECORD OF BOREHOLE No TB7-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678671.8, E333831.4 ORIGINATED BY TA
 DIST HWY WEP BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Jul 9, 11 - Jul 10, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)				
								○ UNCONFINED	+	FIELD VANE	● POCKET PEN.						×	LAB VANE			
184.0	Ground Surface																				
0.0	TOPSOIL																				
183.5																					
0.5	CLAYEY SILT Some sand, trace gravel, trace cobbles Firm to hard Mottled brown and Grey		1	SS	7		183														
	Brown -Trace fissures		2	SS	16		182														
			3	SS	35		181														
			4	SS	34		180														
	Grey		5	SS	16		179														
			6	SS	14		178														
			7	SS	14		177														
			8	SS	10		176														

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 16/01/12

RECORD OF BOREHOLE No TB7-2

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678662.3, E333859.6 ORIGINATED BY TA
 DIST HWY WEP BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Jul 10, 11 - Jul 10, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa													
184.1	Ground Surface							○ UNCONFINED	○ FIELD VANE	● POCKET PEN.	× LAB VANE	WATER CONTENT (%)									
0.0 183.8 0.3	TOPSOIL							20 40 60 80 100	20 40 60 80 100	10 20 30											
	CLAYEY SILT Some sand, trace gravel Firm to hard Mottled brown and grey		1	SS	12																
	Trace fissures Brown		2	SS	25																
			3	SS	37																
	Trace to some oxidized fissures		4	SS	28																
	Grey		5	SS	15																
			6	SS	10																
			7	SS	9																
			8	SS	9																
			9	SS	5																
			VT																		
		10	SS	5																	
		VT																			
174.0 10.1	END OF BOREHOLE Borehole dry on completion																				

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 16/01/12

RECORD OF BOREHOLE No TB7-3

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678644.6, E333911.0 ORIGINATED BY TA
 DIST HWY WEP BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE Jul 12, 11 - Jul 12, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa													
								○ UNCONFINED	+	FIELD VANE											
								● POCKET PEN.	×	LAB VANE											
184.9	Fill Surface						20	40	60	80	100										
0.0	FILL																				
0.2	Topsoil																				
	FILL																				
	Silty clay and topsoil																				
	Brown																				
183.8			1	SS	6																
1.1	CLAYEY SILT																				
	Firm to hard																				
	Mottled brown and grey		2	SS	7																
	Brown		3A, B	SS	6																
	Moist to wet																				
	Trace fissures																				
			4	SS	25																
			5	SS	37																
	Grey		6	SS	20																
			7	SS	10																
			8	SS	9																
			9	SS	9																
			10	SS	5																

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 19/12/11

METRIC

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

RECORD OF NILCON VANE TEST NIL T9-1

Project : Windsor-Essex Parkway

Test Date: 8/16/2001

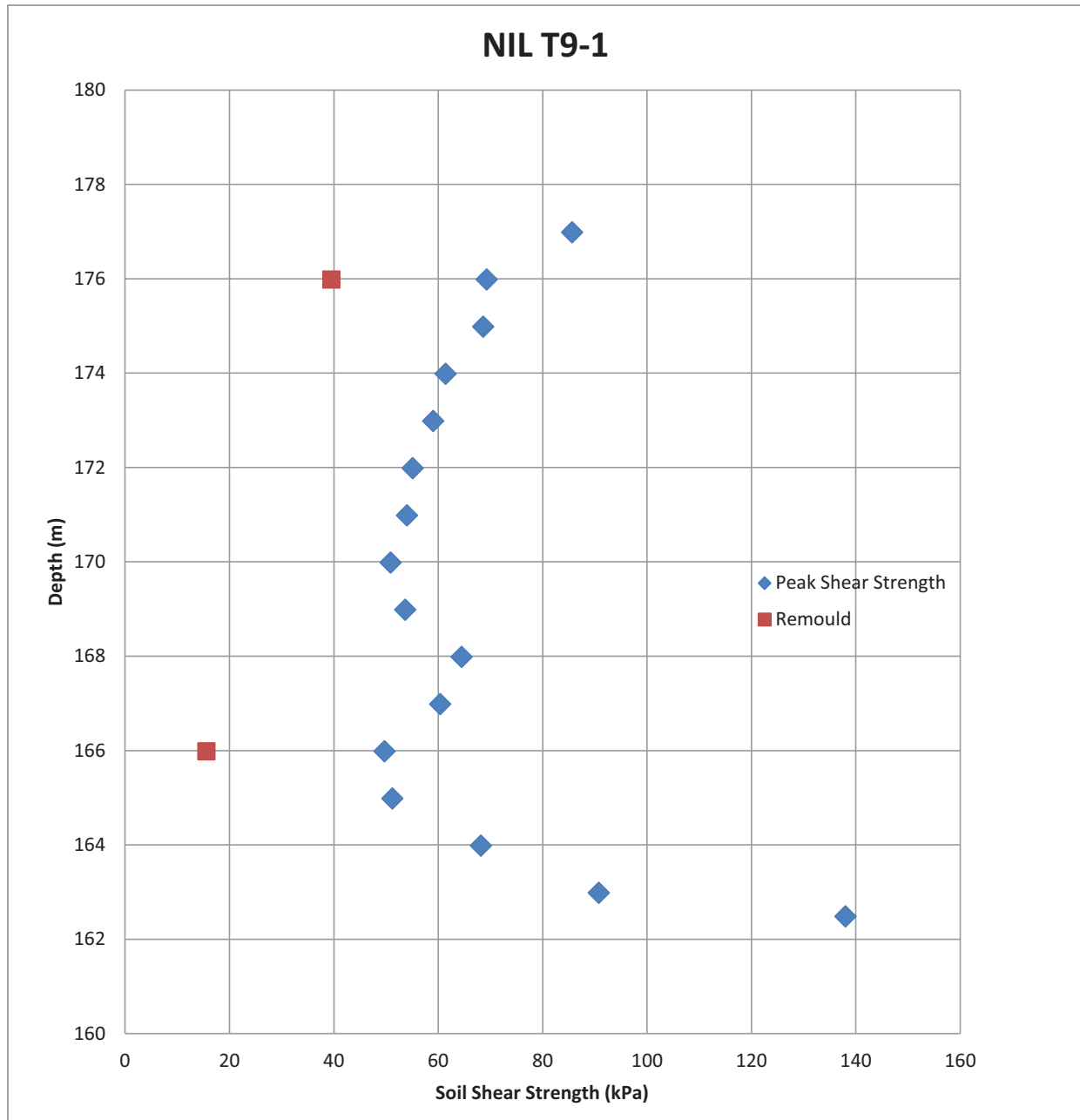
Sheet 1 of 1

Location: N4678636.5; E333765.3

Predrill Depth : 6.1 m

Datum Geodetic

Ground Surface Elevation: 184.0 m



Operator: SD

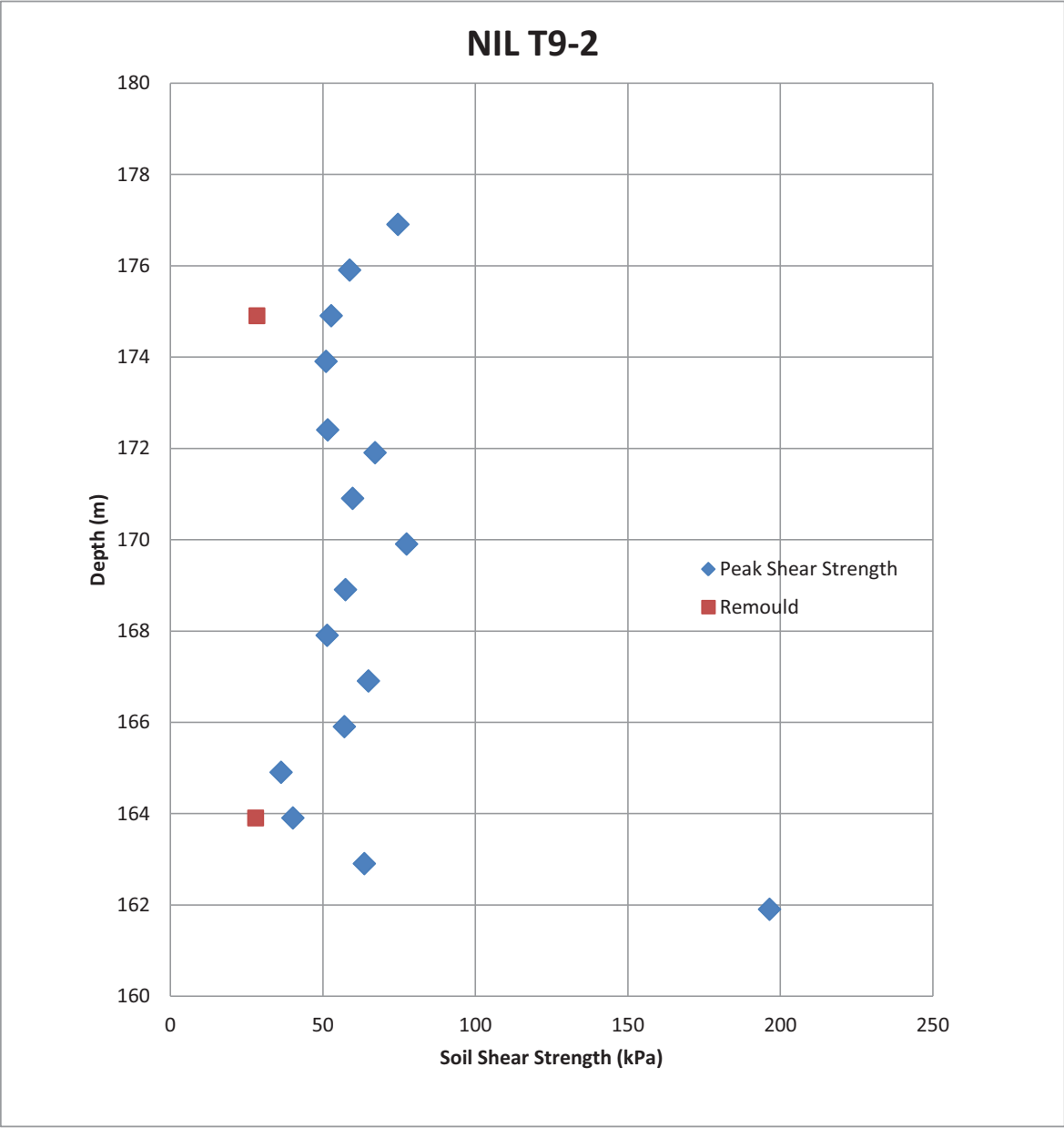
Checked: DD

RECORD OF NILCON VANE TEST NIL T9-2

Project : Windsor-Essex Parkway
Location: N4678593.7; E333893.5
Ground Surface Elevation: 183.9 m

Test Date: 8/15/2001
Predrill Depth : 6.6 m

Sheet 1 of 1
Datum Geodetic



RECORD OF CONE PENETRATION TEST CPT 45-RW

METRIC

PROJECT Windsor-Essex Parkway

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

SHEET 1 OF 3

LOCATION N4678688.3; E333708

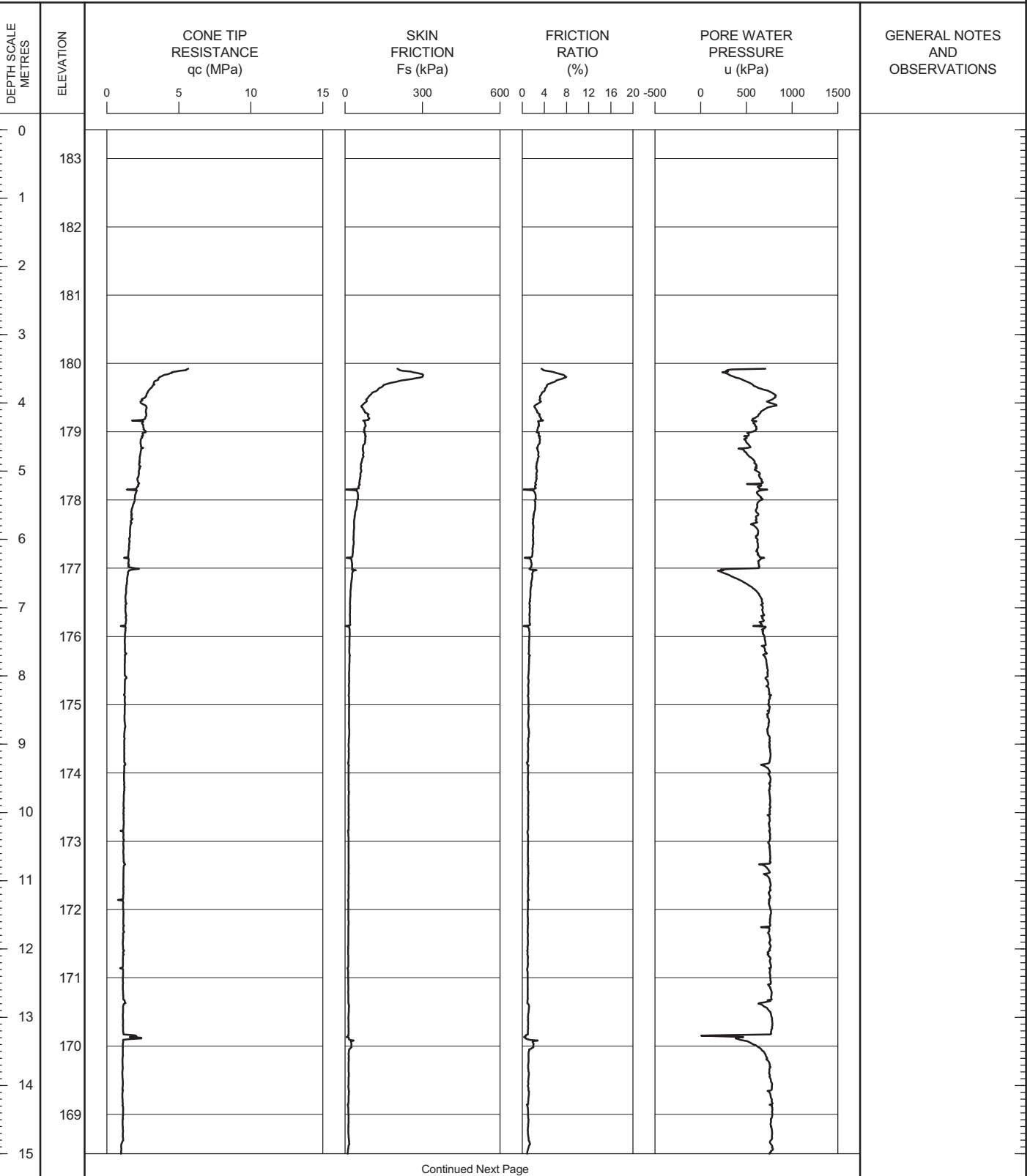
DATUM Geodetic

GROUND SURFACE ELEVATION: 183.4

PREDRILL DEPTH: 3.05

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 45-RW

METRIC

PROJECT Windsor-Essex Parkway

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

SHEET 2 OF 3

LOCATION N4678688.3; E333708

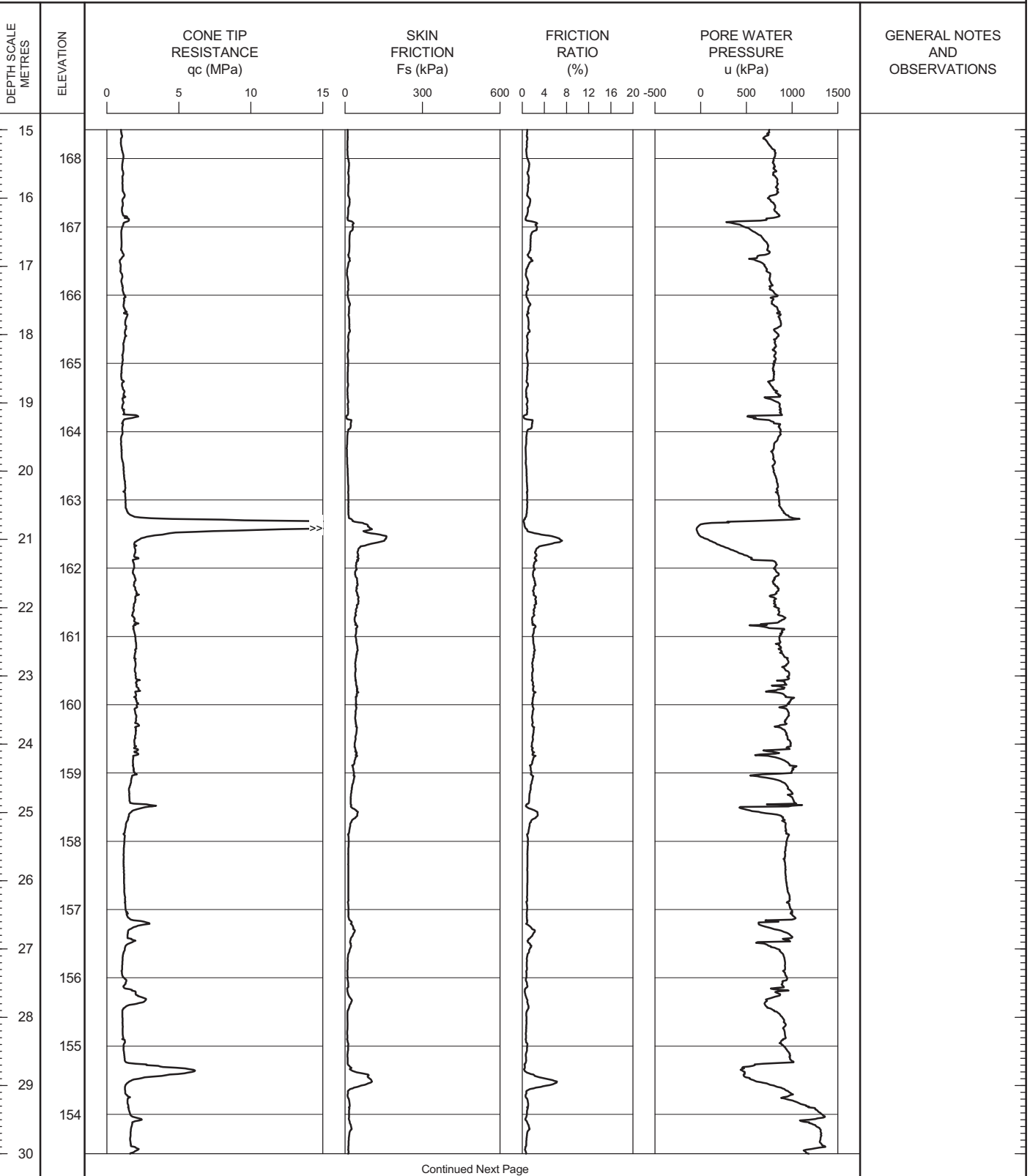
DATUM Geodetic

GROUND SURFACE ELEVATION: 183.4

PREDRILL DEPTH: 3.05

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 45-RW

METRIC

PROJECT Windsor-Essex Parkway

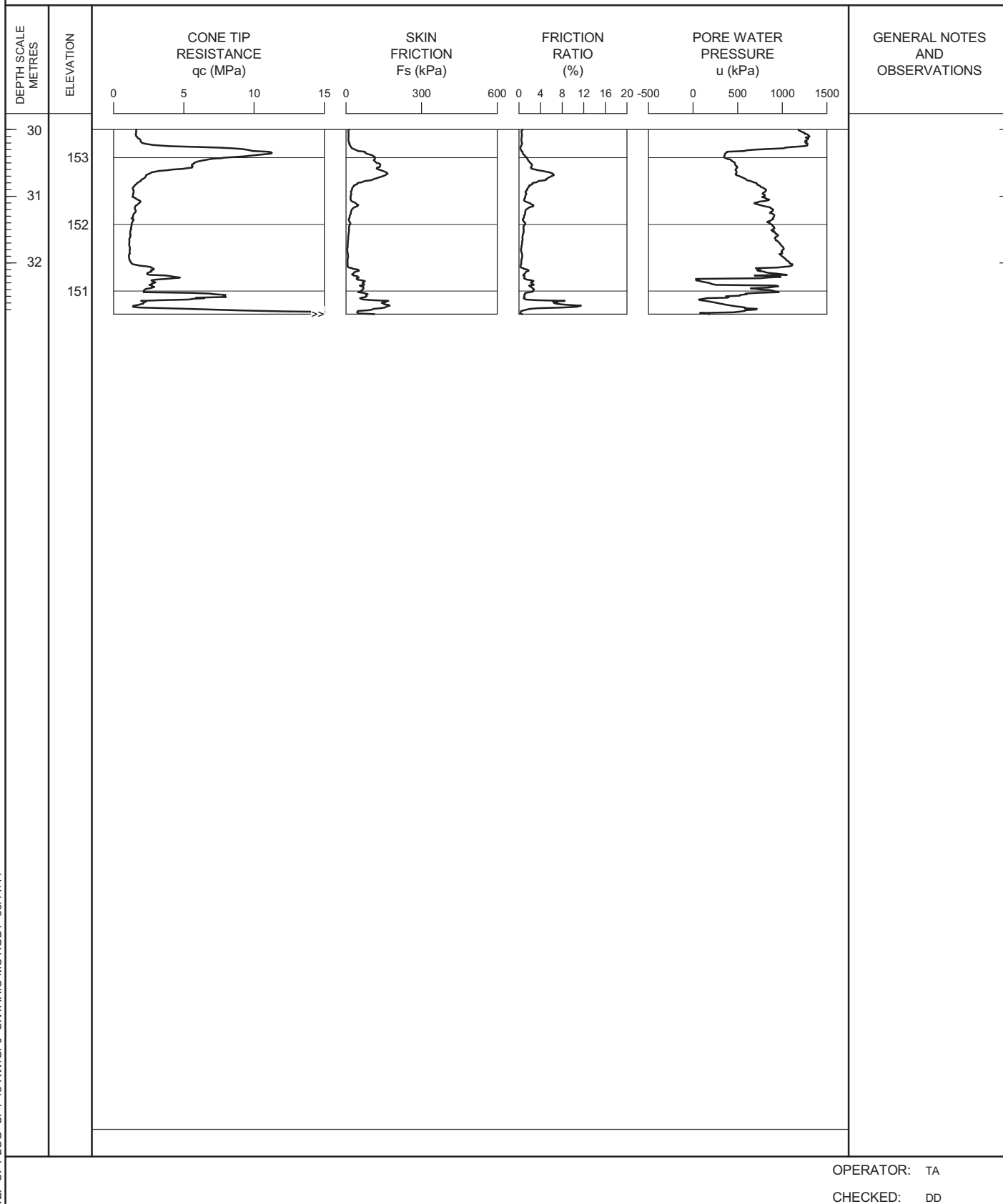
TEST DATE 8/9/2011 - 8/9/2011

SHEET 3 OF 3

LOCATION N4678688.3; E333708

DATUM Geodetic

GROUND SURFACE ELEVATION: 183.4 PREDRILL DEPTH: 3.05 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



RECORD OF CONE PENETRATION TEST CPT 46-RW

METRIC

PROJECT Windsor-Essex Parkway

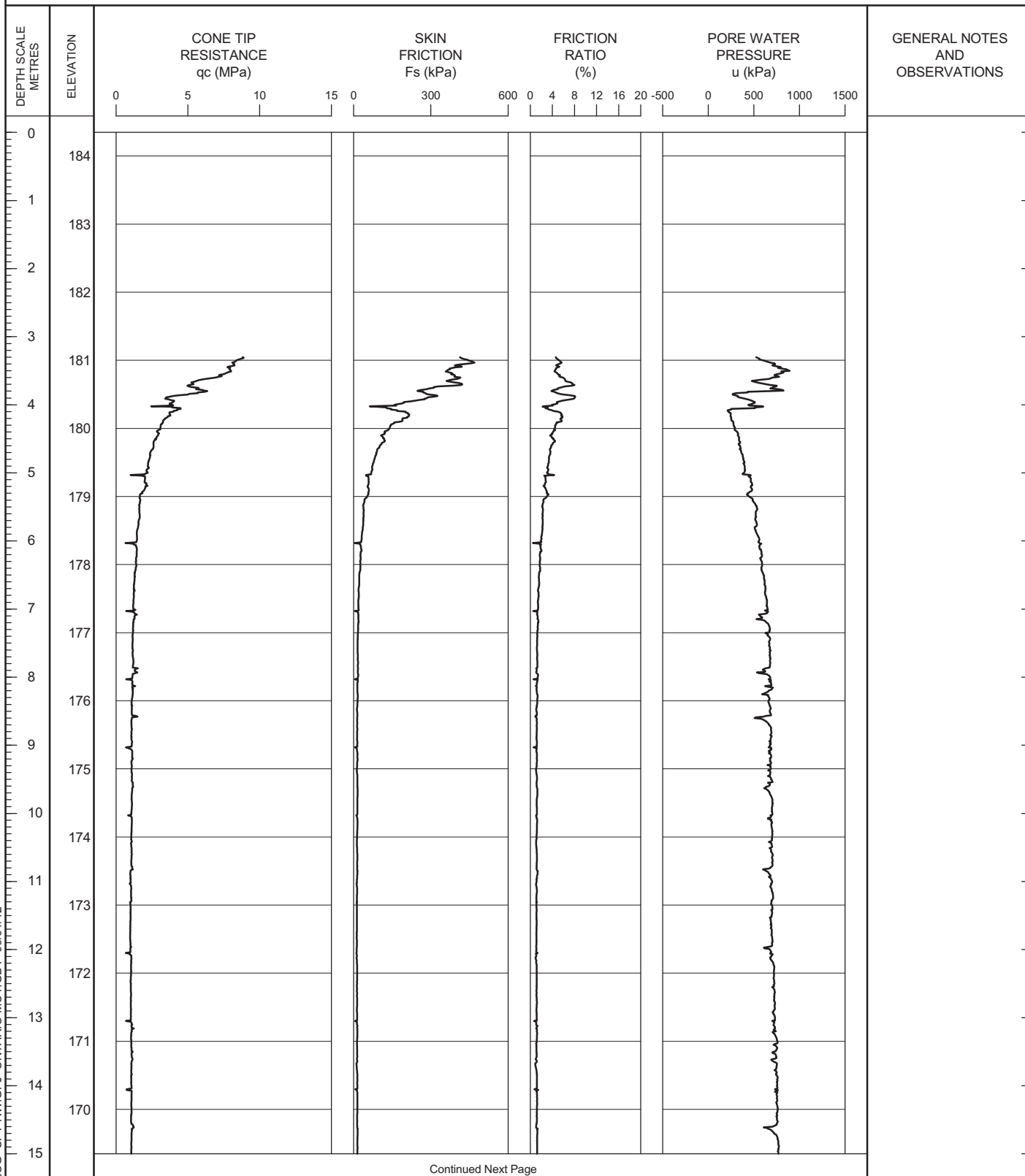
TEST DATE 8/5/2011 - 8/5/2011

SHEET 1 OF 2

LOCATION N4678505.0; E333977.6

DATUM Geodetic

GROUND SURFACE ELEVATION: 184.3 PREDRILL DEPTH: 3 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



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

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 46-RW

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 8/5/2011 - 8/5/2011

SHEET 2 OF 2

LOCATION N4678505.0; E333977.6

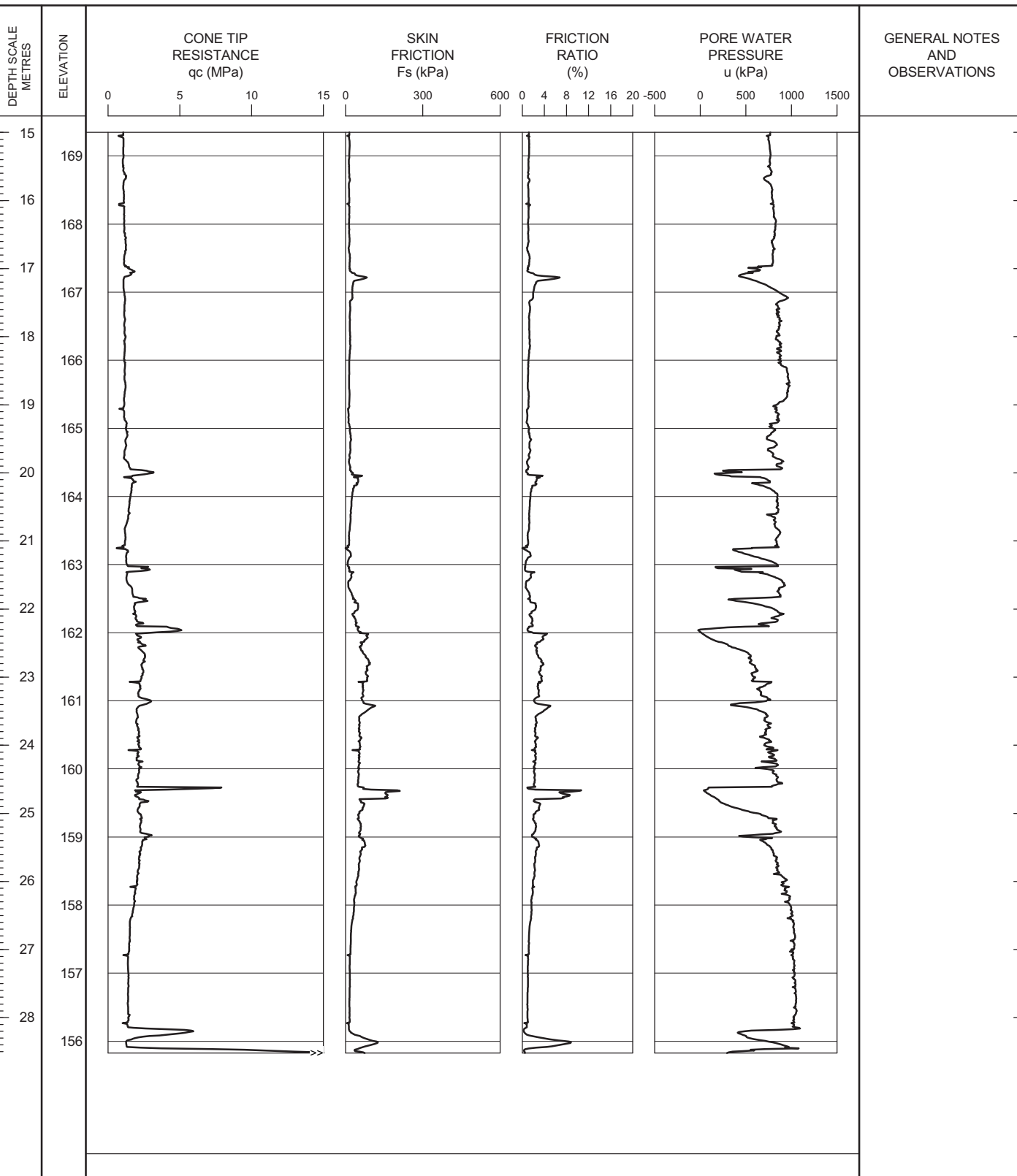
DATUM Geodetic

GROUND SURFACE ELEVATION: 184.3

PREDRILL DEPTH: 3

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



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

OPERATOR: TA

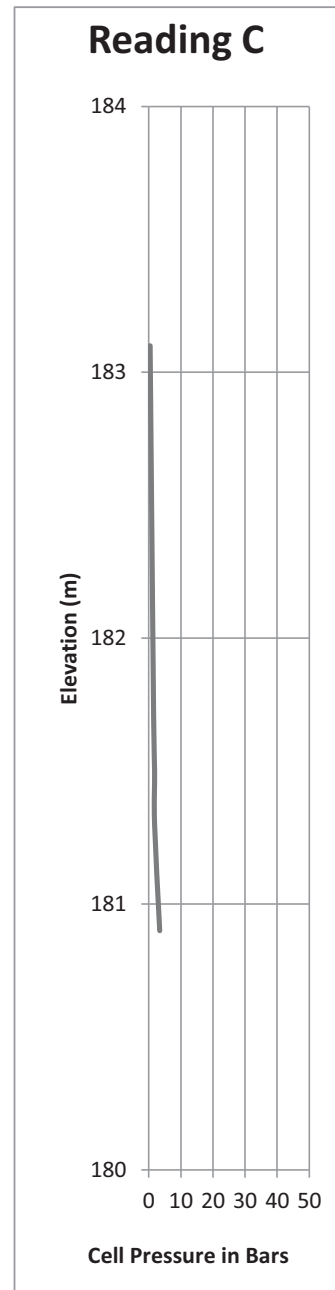
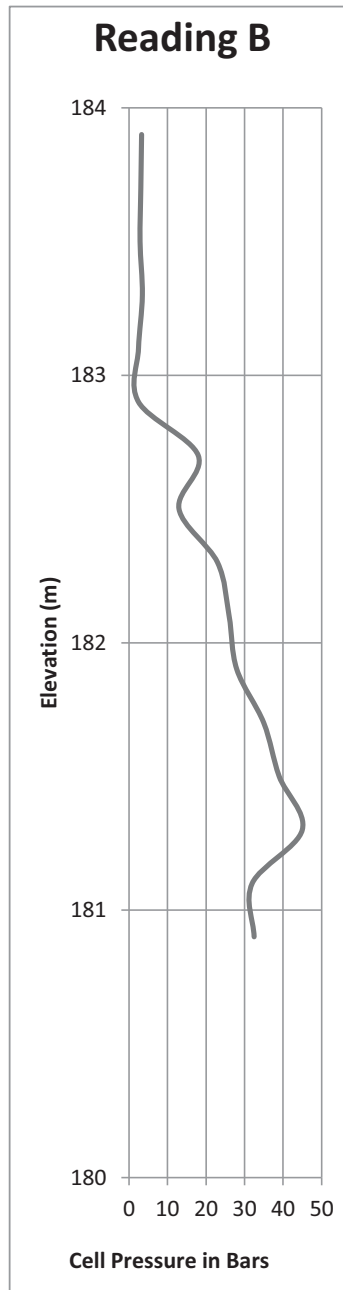
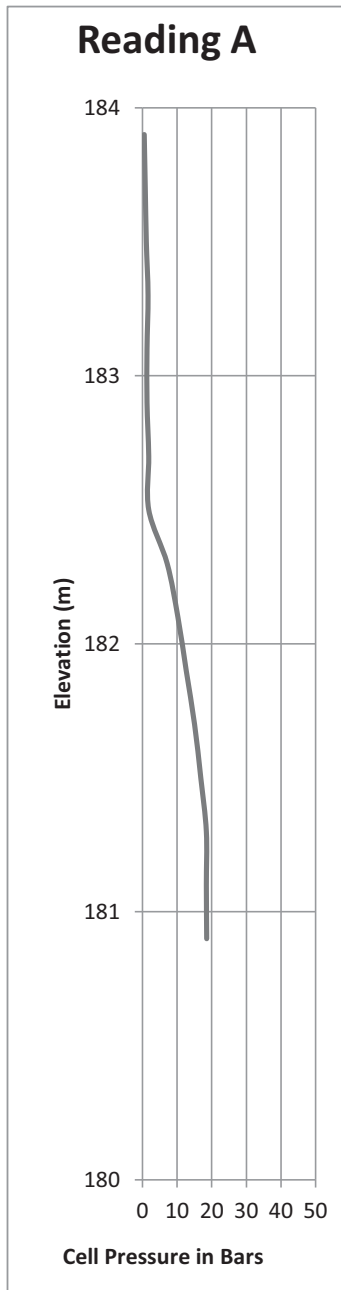
CHECKED: DD

RECORD OF DILATOMETER TEST DMT T9-1-SHALLOW

Project : Windsor-Essex Parkway
Location: N 4678544.5; E 333900.9
Ground Surface Elevation : 184.1

Test Date: 7/19/2011
Predrill Depth : 0.2 m
Delta A: 0.14 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.22 Bar



Note: DMT refusal at elevation 180.9m .Redrill to elevation 179.5m
Resumed DMT to elevation 162.5m

Operator: LC

Checked: DD

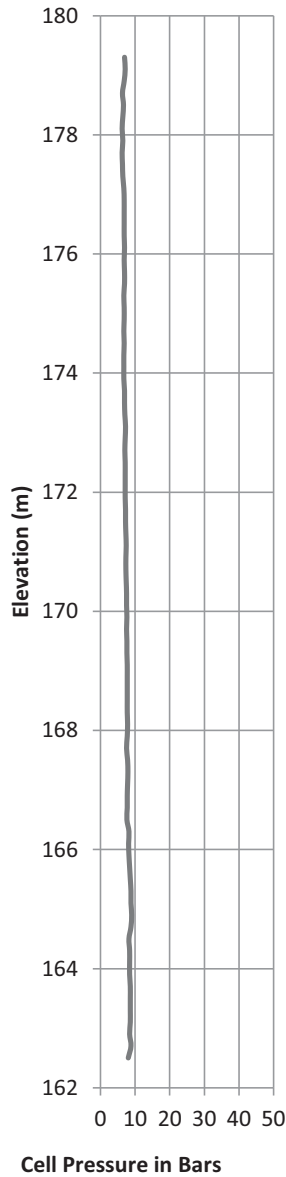
RECORD OF DILATOMETER TEST DMT T9-1-DEEP

Project : Windsor-Essex Parkway
Location: N 4678544.5; E 333900.9
Ground Surface Elevation : 184.1

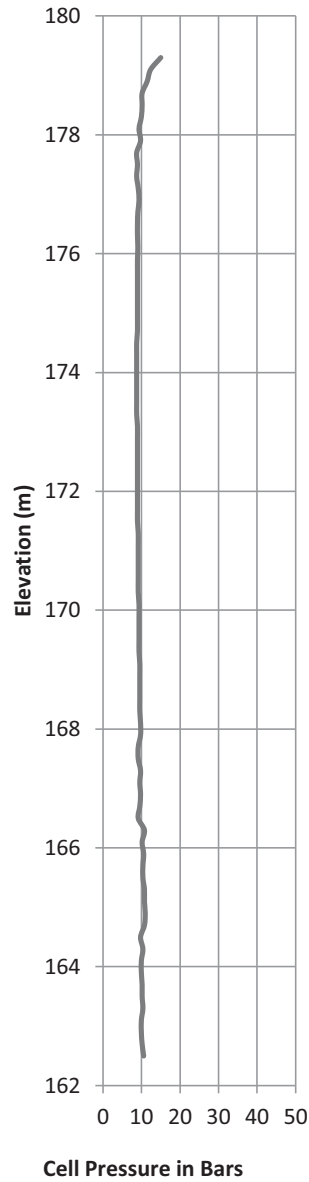
Test Date: 7/19/2011
Predrill Depth : 4.6 m
Delta A: 0.10 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.37 Bar

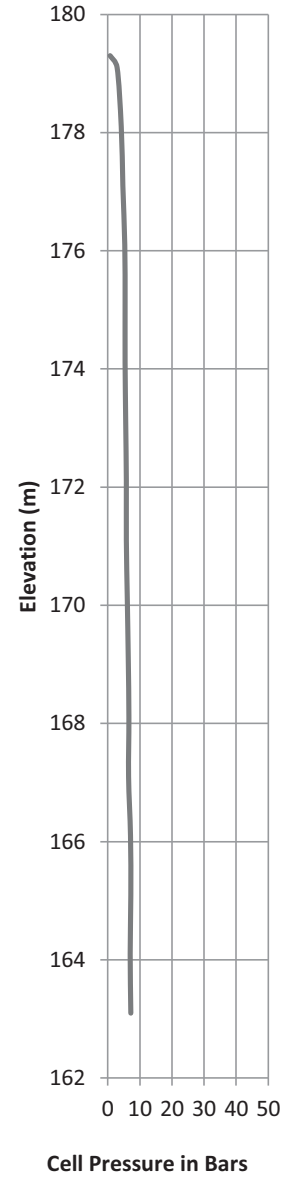
Reading A



Reading B



Reading C



Operator: LC

Checked: DD

Appendix B Borehole and CPT Logs from Previous Investigations

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-9 (Cousineau Tunnel) (Sta. 12+130L to 12+300L)
Doc No.: 285380-04-119-0050 (Geocres No. 40J3-19)

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

PROJECT 07-1130-207-0		RECORD OF BOREHOLE No 115		1 OF 4	METRIC
W.P.	LOCATION	N 4678585.3 E 333911.1		ORIGINATED BY MA	
DIST WEST HWY 401/3	BOREHOLE TYPE	POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY BRS	
DATUM GEODETIC	DATE	February 15, 2008 - February 21, 2008		CHECKED BY <i>SJB</i>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60
183.79	GROUND SURFACE														
0.00	TOPSOIL, silty Brown						Concrete								
183.36							Bentonite								
0.43	CLAYEY SILT, some sand, trace gravel Soft to very stiff Brown		1	SS	4		183					o			
			2	SS	22		182					o			
			3	SS	25							o			
			4	SS	23		181					o			
180.44															
3.35	CLAYEY SILT, some sand, trace gravel Stiff Grey		5	SS	14		180					o			
			6	SS	12		179					o			
			7	SS	14							o			
							178								
			8	SS	9		177					o			
			9	TO	PH		176								
							Grout								
							175								
174.80															
8.99	SANDY SILT, some clay, trace gravel Loose Grey														
			10	SS	7		174					o			3 43 39 14
173.58															
10.21	CLAYEY SILT, some sand, trace gravel Firm Grey						173								
173.12															
10.67	SAND, trace gravel, trace silt Loose Grey														
			11	SS	8		172					o			(9)
171.90															
11.89	CLAYEY SILT, some sand, trace gravel Soft to very stiff Grey		12	SS	7		171					o			2 30 40 28
			13	TO	PH										
							170								
			14	SS	6		169					o			

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 8/29/09

Continued Next Page

+3, x3. Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4678585.3 E 333911.1

ORIGINATED BY MA

DIST WEST HWY 401/3

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

COMPILED BY BRS

DATUM GEODETIC

DATE February 15, 2008 - February 21, 2008

CHECKED BY **SLB**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	SHEAR STRENGTH kPa				WATER CONTENT (%) W _p W W _L	
								○ UNCONFINED	+ FIELD VANE				● QUICK TRIAXIAL
	CLAYEY SILT, some sand, trace gravel Soft to very stiff Grey												
			15	SS	4		168						
							167						
			16	TO	PH		166						
							165						
			17	SS	6		164						
							163						
			18	SS	13		162						
							161						
			19	SS	22		160						
							159						
			20	SS	22		158						
							157						
			21	SS	24		156						
							155						
			22	SS	11		154						
			23	TO	PH								
156.21 27.58	SAND, trace sand, trace gravel, trace clay Dense Grey												
			24	SS	31								
154.83 28.96	SAND, trace gravel Compact to dense Grey												
			25	SS	30								

Continued Next Page

 $+ 3 \times 3.$

Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 115

3 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4678585.3 : E 333911.1

ORIGINATED BY MA

DIST WEST HWY 401/3

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

COMPILED BY BRS

DATUM GEODETIC

DATE

February 15, 2008 - February 21, 2008

CHECKED BY **SJB**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							20 40 60 80 100							
							20 40 60 80 100							
153.31														
30.48	SAND AND GRAVEL, trace silt Dense Grey		26	SS	36		153							25 66 6 3
							152							
151.48														
32.31	LIMESTONE, fresh, medium strong, laminated, fine grained Light grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		27	SS	100/ 25mm		151							
							150							
			28	NQ RC			149							
			29	NQ RC			148							
			30	NQ RC			147							
146.15														UC
37.64	END OF BOREHOLE													
	Water level in borehole at about elev. 156.19m during drilling on February 21, 2008.													
	Water level measured in deep piezometer at elev. 178.00m on February 21, 2008.													
	Water level measured in deep piezometer at elev. 178.10m on March 20, 2008.													
	Water level measured in deep piezometer at elev. 177.69m on July 24, 2008.													
	Water level measured in deep piezometer at elev. 175.99m on September 19, 2008.													
	Water level measured in deep piezometer at elev. 177.25m on November 14, 2008.													
	Water level measured in deep piezometer at elev. 177.35m on January 28, 2009.													

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 8/29/09

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 115

SHEET 4 OF 4

LOCATION: N 4678585.3 ; E 333911.1

DRILLING DATE: February 15, 2008 - February 21, 2008

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (m/min)	COLOUR (m/min)	FLUSH % RETURN	ELEVATION	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth Ro - Rough Br - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols										HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETER POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
				DEPTH (m)	RECOVERY						R.Q.D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁻⁶	10 ⁻⁴	10 ⁻²																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
													TOTAL CORE %	SOLID CORE %						0				15	30	45	60	75	90																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
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		ROCK SURFACE		151.48 32.31																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											

DEPTH SCALE

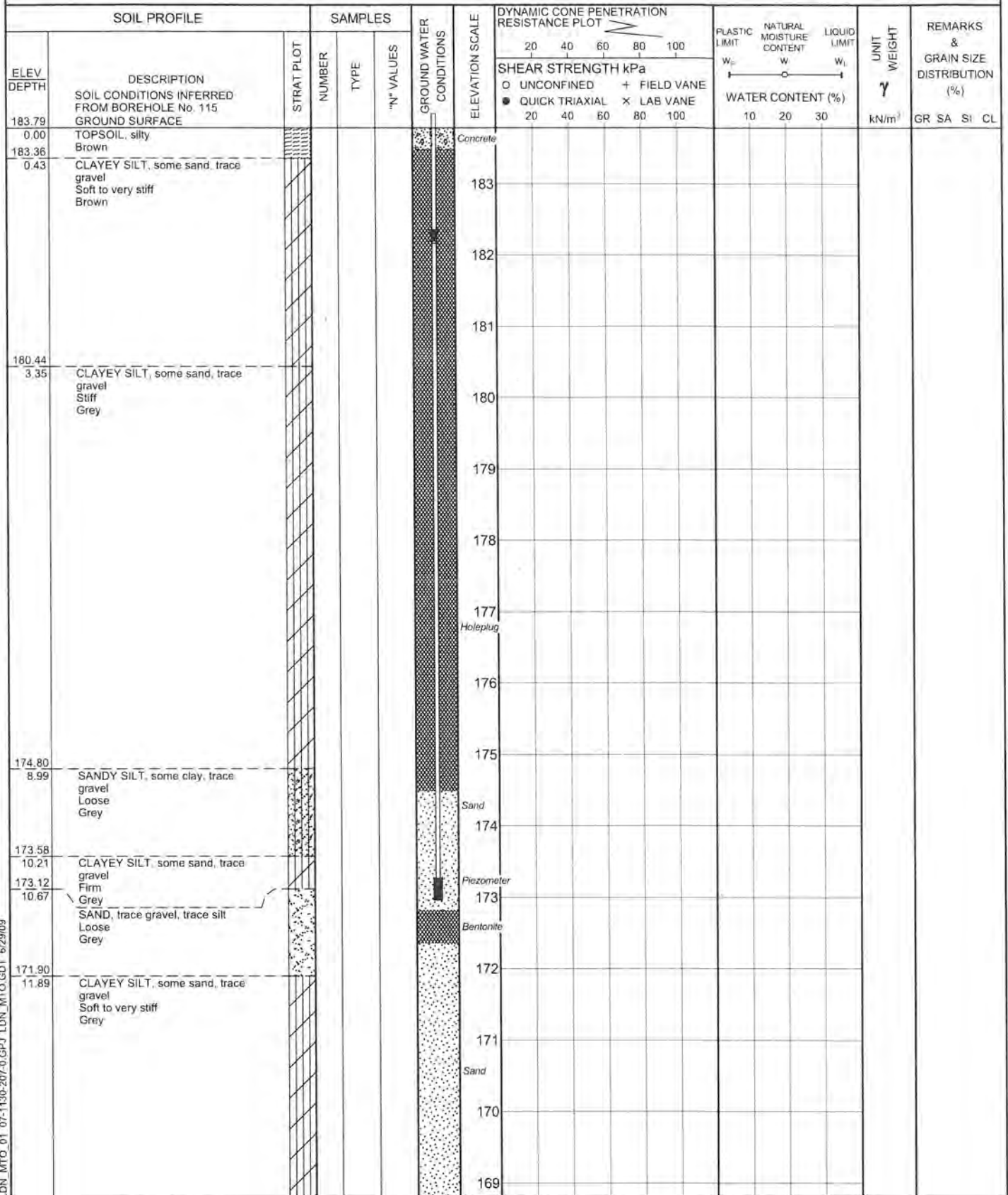
1 : 75



LOGGED: SG

CHECKED: SUB

PROJECT <u>07-1130-207-0</u>		RECORD OF BOREHOLE No 115A		1 OF 2	METRIC
W.P. _____		LOCATION <u>N 4678585.3 ; E 333911.1</u>		ORIGINATED BY <u>MA</u>	
DIST <u>WEST</u> HWY <u>401/3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>BRS</u>	
DATUM <u>GEODETIC</u>		DATE <u>February 20, 2008 - February 21, 2008</u>		CHECKED BY <u>SJB</u>	



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

Continued Next Page

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

PROJECT 07-1130-207-0		RECORD OF BOREHOLE No 115A				2 OF 2		METRIC			
W.P. _____		LOCATION N 4678585.3 E 333911.1				ORIGINATED BY MA					
DIST WEST HWY 401/3		BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC				COMPILED BY BRS					
DATUM GEODETIC		DATE February 20, 2008 - February 21, 2008				CHECKED BY SJB					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	CLAYEY SILT, some sand, trace gravel Soft to very stiff Grey						168				
							167				
							166				
							165				
163.98	END OF BOREHOLE						164				
19.81	Water level measured in shallow piezometer at elev. 182.36m on March 20, 2008. Water level measured in shallow piezometer at elev. 182.34m on July 24, 2008. Water level measured in shallow piezometer at elev. 182.26m on September 19, 2008. Water level measured in shallow piezometer at elev. 182.20m on January 28, 2009.										

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

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4678634.3 ; E 333722.5

ORIGINATED BY SM

DIST

WEST

HWY 401/3

BOREHOLE TYPE

POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY BRS

DATUM GEODETIC

DATE _____

February 20, 2008 - February 25, 2008

CHECKED BY **SJB**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa							WATER CONTENT (%)			
							20 40 60 80 100										
							○ UNCONFINED + FIELD VANE										
							● QUICK TRIAXIAL × LAB VANE										
							20 40 60 80 100										
183.64	GROUND SURFACE																
0.00	TOPSOIL, clayey Black																
0.30	SILTY CLAY, some sand, trace gravel Firm Mottled brown and grey		1	SS	6												
182.27																	
1.37	CLAYEY SILT, some sand, trace gravel Stiff to hard Brown		2	SS	10												
			3	SS	35												
			4	SS	33												
179.98																	
3.66	CLAYEY SILT, some sand, trace gravel Firm to very stiff Grey		5	SS	17												
			6	SS	14												
			7	SS	12												
			8	SS	11												

Continued Next Page

+3, X3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 116

2 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4678634.3 E 333722.5

ORIGINATED BY SM

DIST WEST HWY 401/3

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

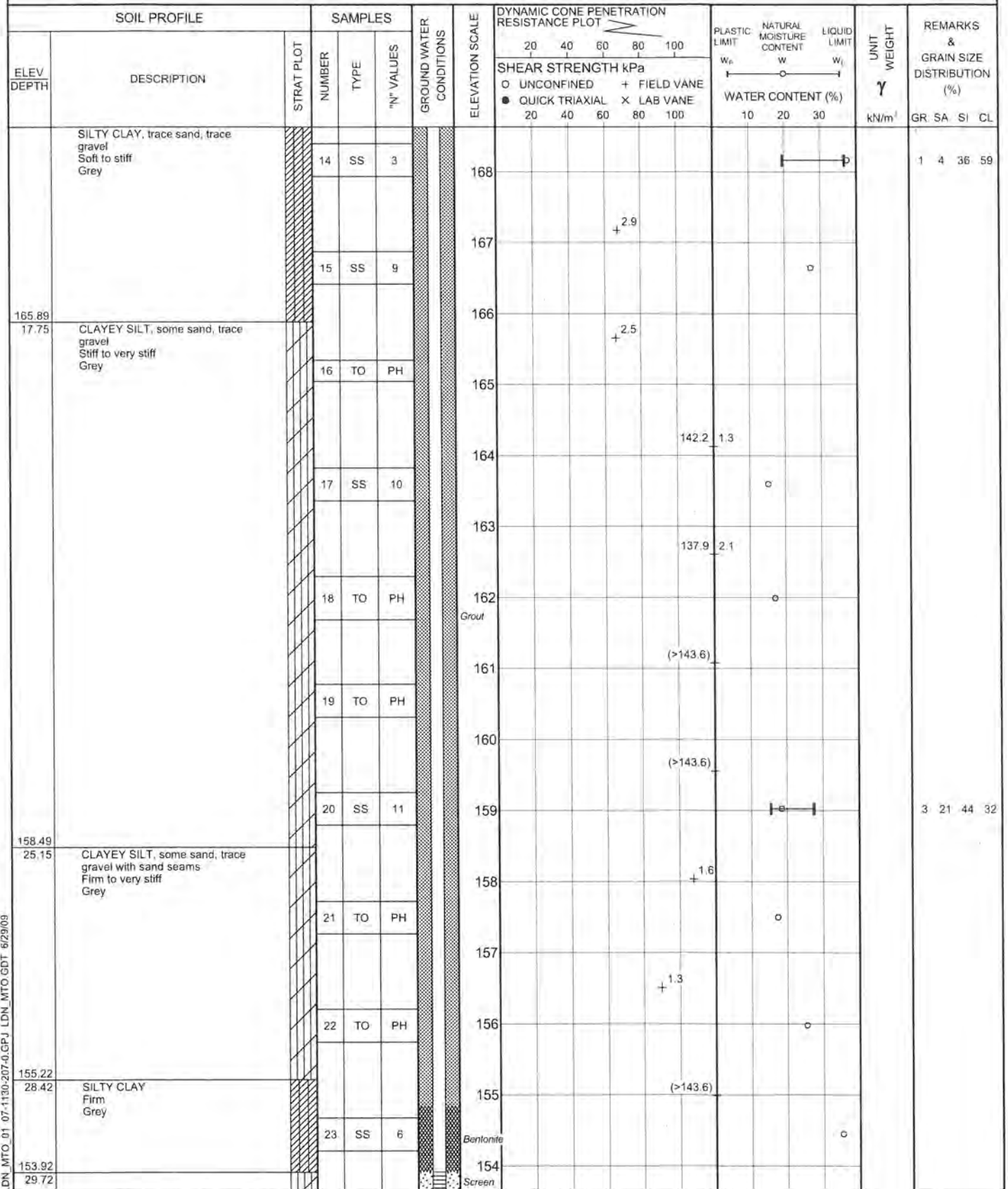
COMPILED BY BRS

DATUM GEODETIC

DATE

February 20, 2008 - February 25, 2008

CHECKED BY **SJB**



Continued Next Page

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

PROJECT 07-1130-207-0 **RECORD OF BOREHOLE No 116** 3 OF 4 **METRIC**
W.P. LOCATION N 4678634.3 E 333722.5 ORIGINATED BY SM
DIST WEST HWY 401/3 BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC COMPILED BY BRS
DATUM GEODETIC DATE February 20, 2008 - February 25, 2008 CHECKED BY *SYB*

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

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

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 116

SHEET 4 OF 4

LOCATION: N 4678634.3 ; E 333722.5

DRILLING DATE: February 20, 2008 - February 25, 2008

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (mm/min)	COLOUR % RETURN	FLUSH	ELEVATION	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate										BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage										PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular										PO - Polished K - Stickensided SM - Smooth Ro - Rough										Br - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols:	HYDRAULIC CONDUCTIVITY k, cm/sec				DIAMETRAL POINT LOAD INDEX (MPa)				NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
				DEPTH (m)						RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		DIP w/1 CORE AXIS	TYPE AND SURFACE DESCRIPTION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
										TOTAL CORE %	SOLID CORE %																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									

LDN ROCK 03 07-1130-207-0-ROCK.GPJ GLDR LDN GDT 8/29/09 DATA INPUT: WDF

DEPTH SCALE

1:75



LOGGED: SG

CHECKED: *SG*

RECORD OF BOREHOLE No 116A

1 OF 1

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4678634.3 E 333722.5

ORIGINATED BY SM

DIST WEST HWY 401/3

BOREHOLE TYPE POWER AUGER, SOLID STEM

COMPILED BY BRS

DATUM GEODETIC

DATE

February 25, 2008

CHECKED BY SJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
183.64	SOIL CONDITIONS INFERRED FROM BOREHOLE No. 116							20 40 60 80 100	20 40 60 80 100	10 20 30				
0.00	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
0.30	TOPSOIL, clayey Black							20 40 60 80 100	20 40 60 80 100	10 20 30				
182.27	SILTY CLAY, some sand, trace gravel Firm Mottled brown and grey							20 40 60 80 100	20 40 60 80 100	10 20 30				
1.37	CLAYEY SILT, some sand, trace gravel Stiff to hard Brown							20 40 60 80 100	20 40 60 80 100	10 20 30				
179.98	CLAYEY SILT, some sand, trace gravel Firm to very stiff Grey							20 40 60 80 100	20 40 60 80 100	10 20 30				
3.66	CLAYEY SILT, some sand, trace gravel Firm to very stiff Grey							20 40 60 80 100	20 40 60 80 100	10 20 30				
174.50	END OF BOREHOLE							20 40 60 80 100	20 40 60 80 100	10 20 30				
9.14	Water level measured in shallow piezometer at elev. 182.55m on March 20, 2008.							20 40 60 80 100	20 40 60 80 100	10 20 30				
	Water level measured in shallow piezometer at elev. 182.80m on July 22, 2008.							20 40 60 80 100	20 40 60 80 100	10 20 30				
	Water level measured in shallow piezometer at elev. 182.59m on August 11, 2008.							20 40 60 80 100	20 40 60 80 100	10 20 30				
	Water level measured in shallow piezometer at elev. 182.57m on September 19, 2008.							20 40 60 80 100	20 40 60 80 100	10 20 30				
	Water level measured in shallow piezometer at elev. 182.72m on January 28, 2009.							20 40 60 80 100	20 40 60 80 100	10 20 30				

LDN_MTO 01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

PROJECT <u>07-1130-207-0</u>		RECORD OF BOREHOLE No CPT-114		1 OF 1	METRIC
W.P. _____		LOCATION <u>N 4678526.7 :E 334018.6</u>		ORIGINATED BY <u>CC</u>	
DIST <u>WEST</u> HWY <u>401/3</u>		BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>SJL</u>	
DATUM <u>GEODETIC</u>		DATE <u>September 10, 2008</u>		CHECKED BY <u>SJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
184.21	GROUND SURFACE												
0.00	TOPSOIL, silty, trace to some sand Compact Black		1	SS	20		184						
0.28	CLAYEY SILT, trace to some sand, trace gravel Very stiff Mottled brown and grey becoming brown at about elev. 183.0m		2	SS	20		183						
182.38			3	SS	28								
1.83	END OF BOREHOLE Borehole dry during drilling on September 10, 2008.												

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

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-6

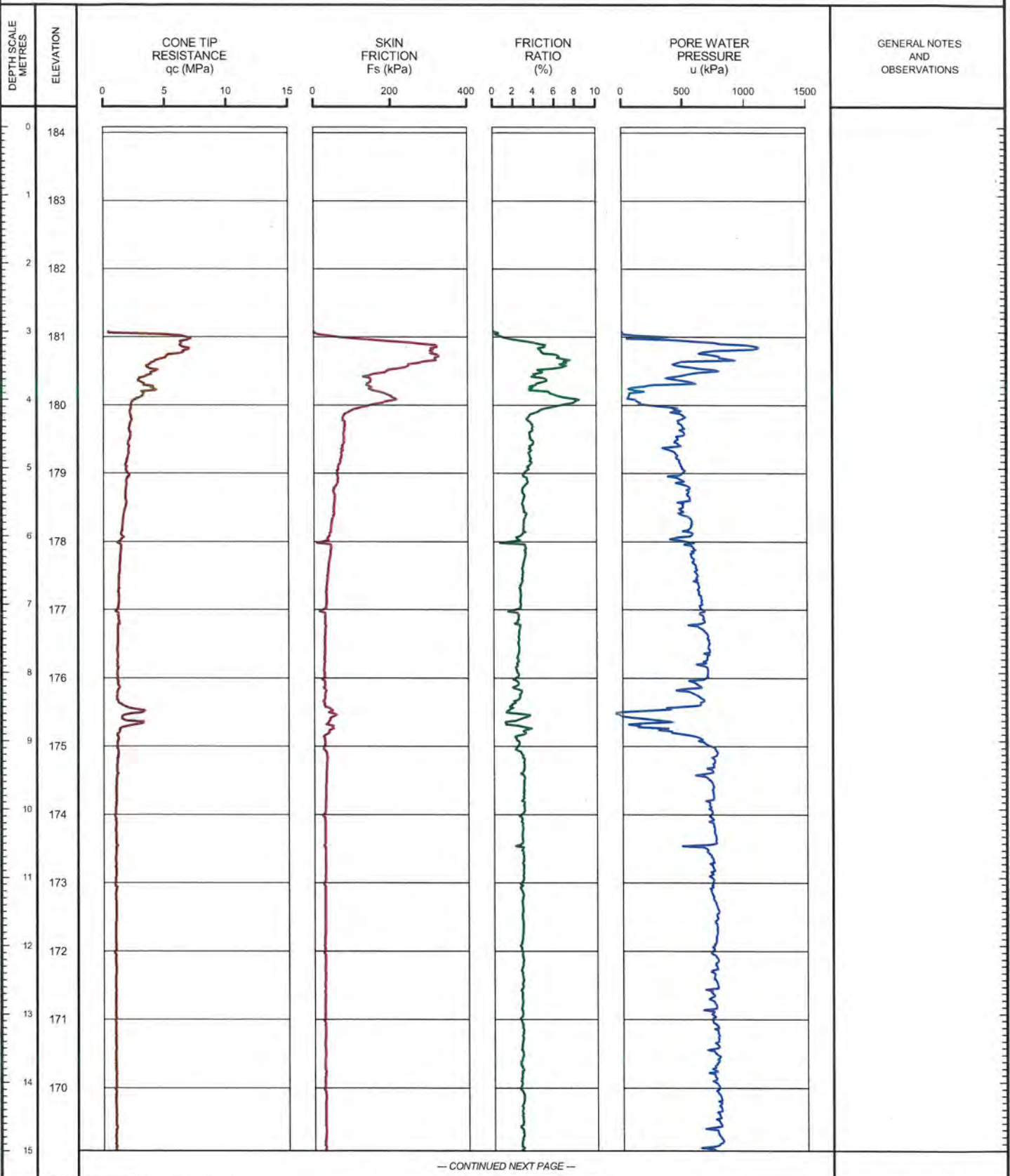
SHEET 1 OF 2

LOCATION: N 4678621.0 ; E 333844.0

TEST DATE: November 13, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 3.00m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



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

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: *536*

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-6

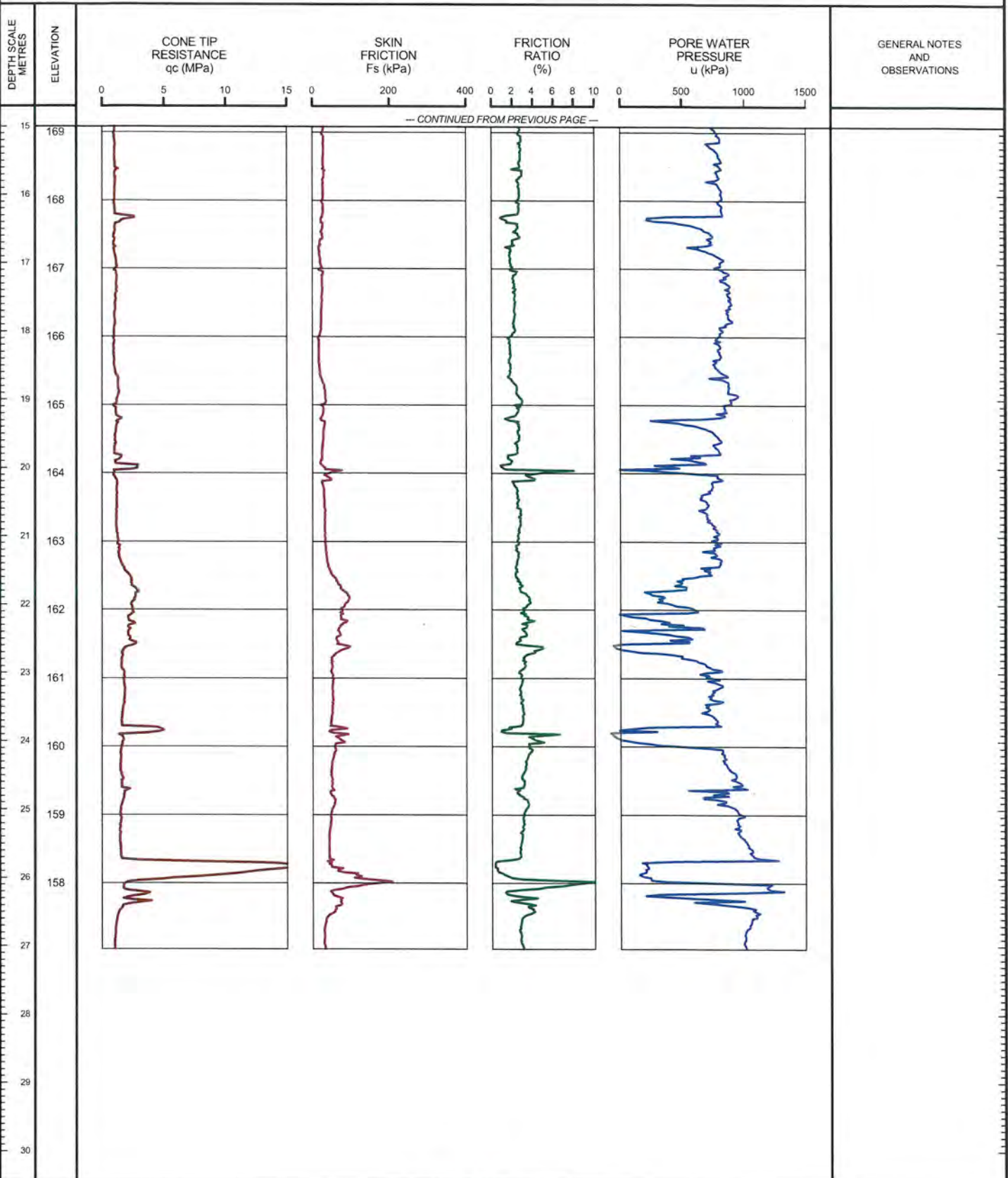
SHEET 2 OF 2

LOCATION: N 4678621.0; E 333844.0

TEST DATE: November 13, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 3.00m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



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

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: *SVB*

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-114

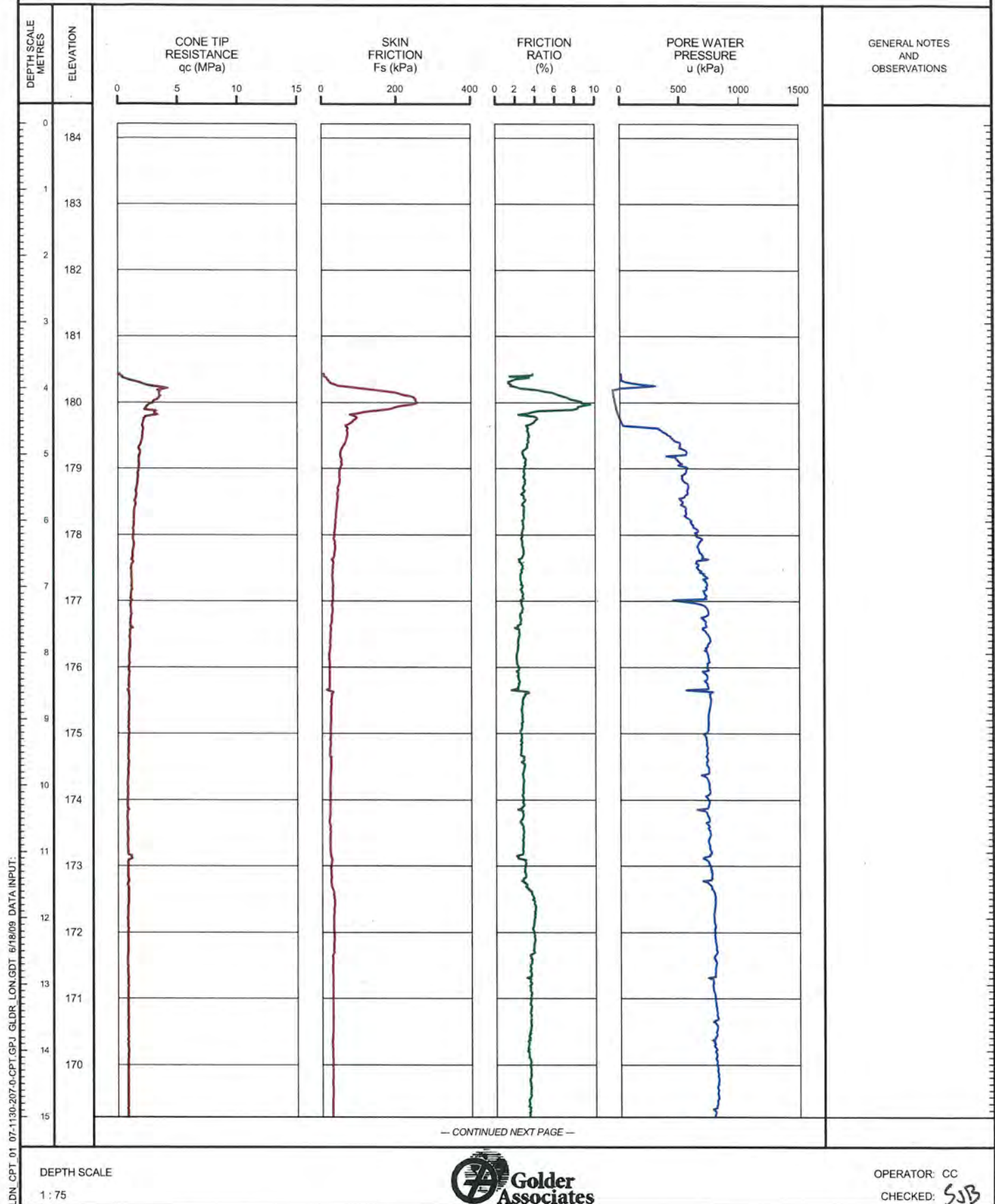
SHEET 1 OF 2

LOCATION: N 4678526.7 ,E 334018.6

TEST DATE: September 10, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 3.80m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-114

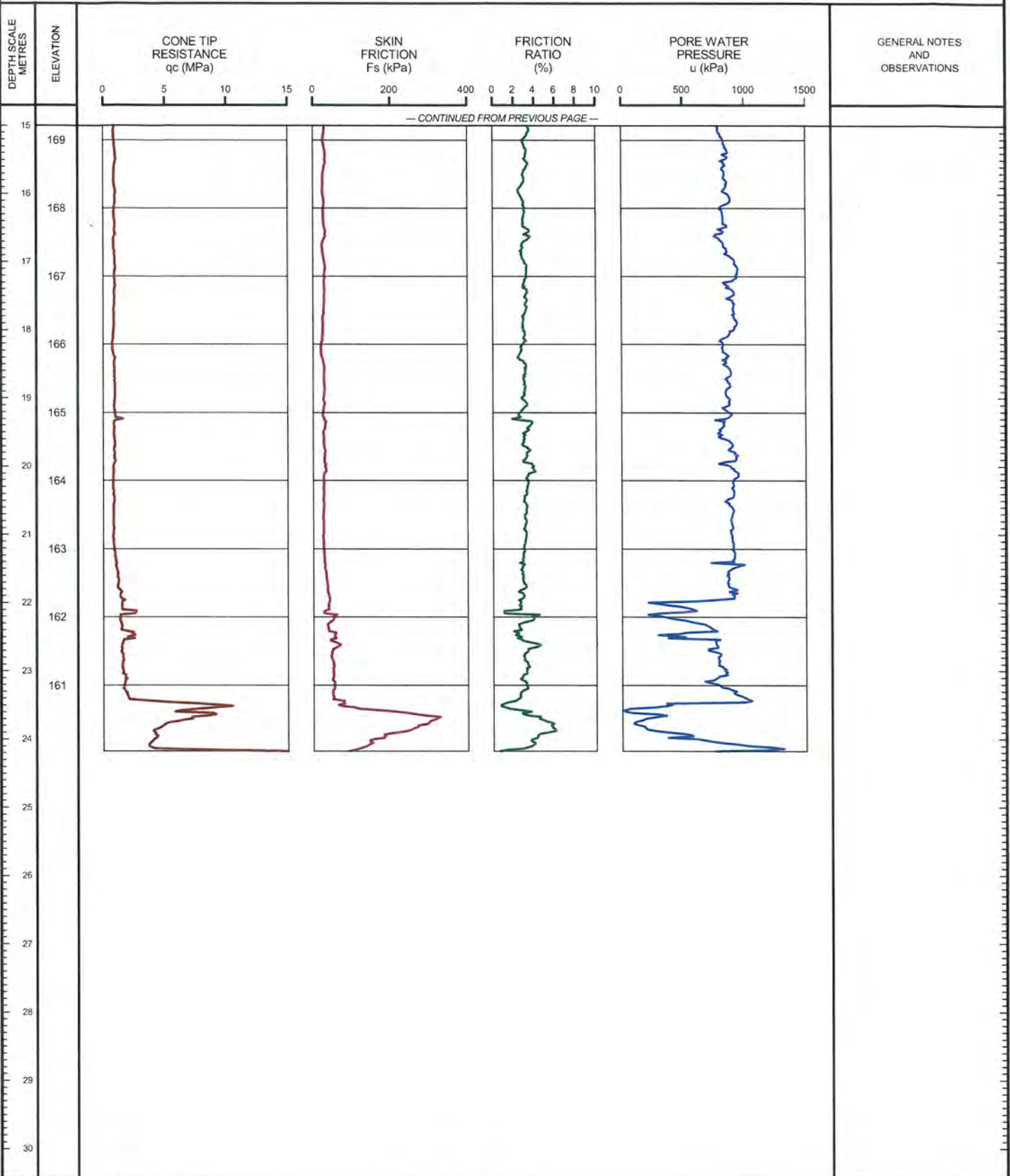
SHEET 2 OF 2

LOCATION: N 4678526.7 ;E 334018.6

TEST DATE: September 10, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 3.80m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



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

DEPTH SCALE

1 : 75



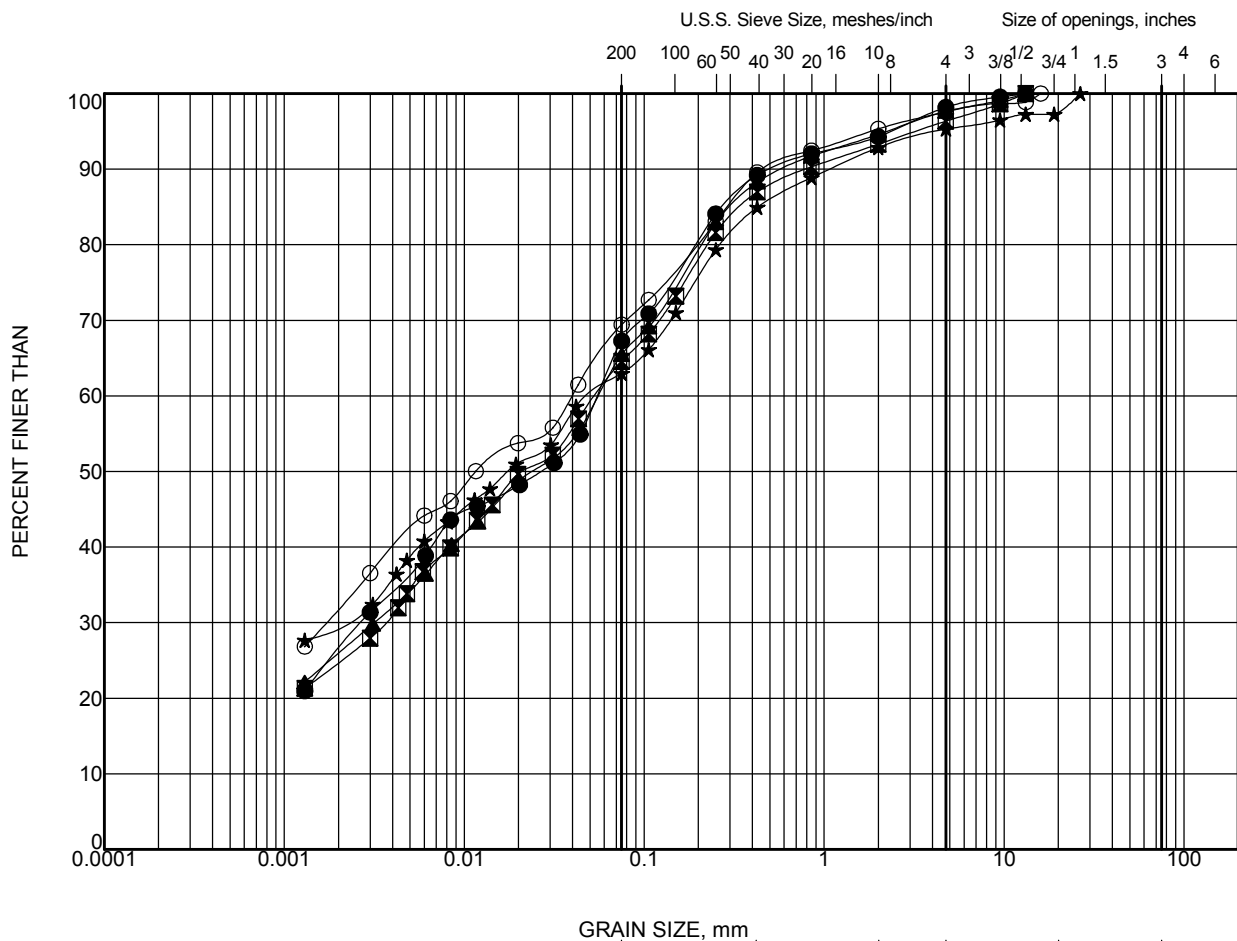
OPERATOR: CC

CHECKED: *SJB*

Appendix C Geotechnical Laboratory Test Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-9 (Cousineau Tunnel) (Sta. 12+130L to 12+300L)
Doc No.: 285380-04-119-0050 (Geocres No. 40J-19)



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

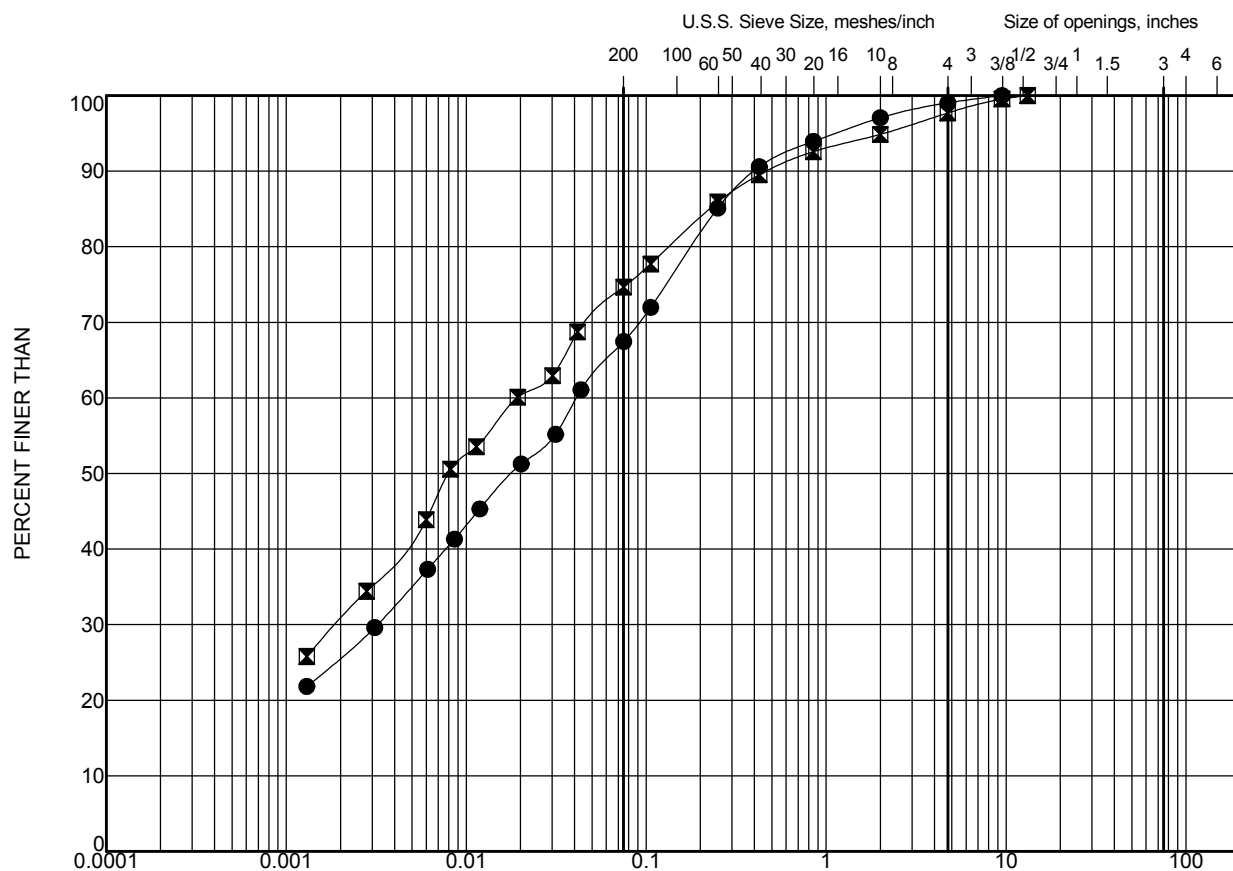


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T9-1	10	7.6
■	T9-1	12	10.7
▲	T9-1	13	12.2
★	T9-1	15	15.2
○	T9-1	16	16.8

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		GRAIN SIZE DISTRIBUTION CLAYEY SILT	
 		PROJECT No. SW8801.1004.101	FILE No.
DRAWN	EA	SCALE	REV.
CHECK	MSO	FIGURE C.1	

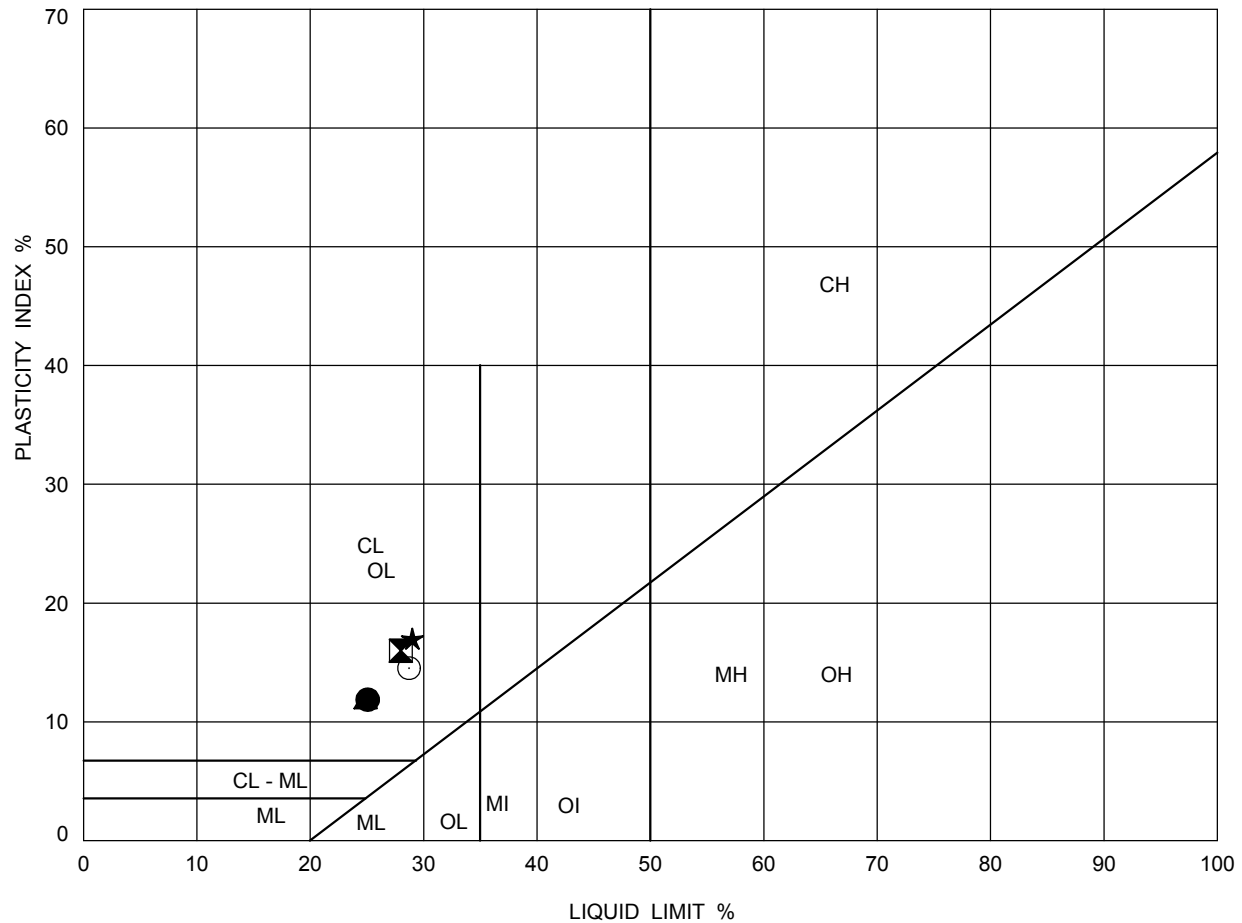


CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

LEGEND:



SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	CV3-1	9	7.6
×	T9-1	20	22.9

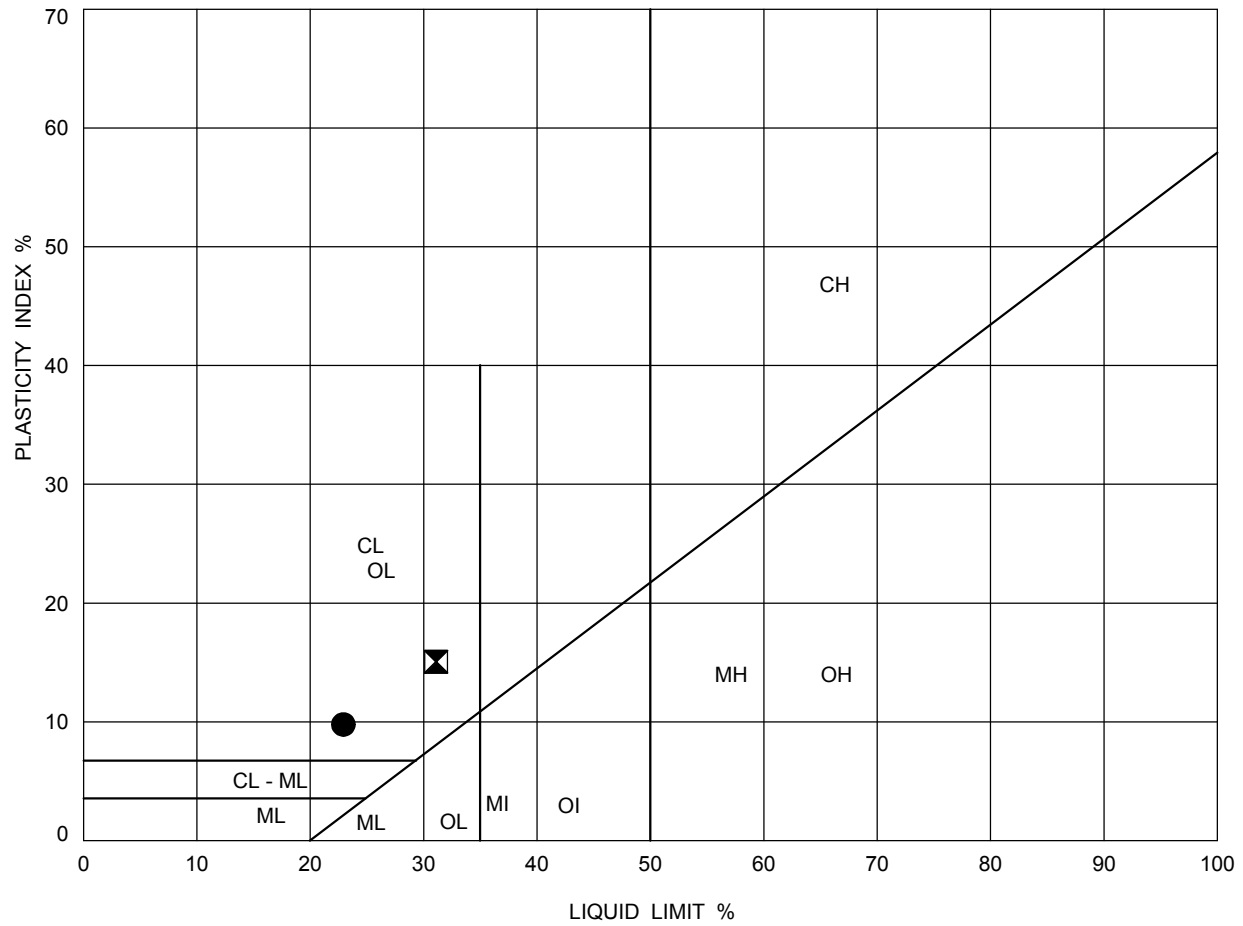
PROJECT		Windsor Essex Parkway (WEP)	
		Windsor, Ontario	
TITLE		GRAIN SIZE DISTRIBUTION	
		CLAYEY SILT	
	PROJECT No. SW8801.1004.101		FILE No.
	DRAWN	EA	SCALE
	CHECK	MSO	REV.
			FIGURE C.2



LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T9-1	10	7.6	25	13	12
⊠	T9-1	12	10.7	28	12	16
▲	T9-1	13	12.2	25	13	12
★	T9-1	15	15.2	29	12	17
○	T9-1	16	16.8	29	14	15

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART CLAYEY SILT	
 		PROJECT No. SW8801.1004.101	FILE No.
DRAWN		EA	SCALE
CHECK		MSO	REV.
		FIGURE C.3	





SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	CV3-1	9	7.6	23	13	10
⊠	T9-1	20	22.9	31	16	15

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART CLAYEY SILT	
 		PROJECT No. SW8801.1004.101	FILE No.
DRAWN	EA	SCALE	REV.
CHECK	MSO	FIGURE C.4	

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP**
 Client: **Hatch Mott MacDonald Limited**
 Date: **21-Oct-11**

Job No.: **SW8801.1004.101**
 Sample ID: **T9-1_Sa15**
 Depth(m): **15.25 to 15.86**

Test Data

Ring # :	A	Ring Height (in) =	0.758	Wt of dry filter paper (g)	0.67
Wet soil + Ring Wt (g)			205.82	Wt of ring (g)	76.59
Wet soil + Wet Paper + Ring (g)			201.90	Wet Paper (g)	2.00
Dry Soil + Dry Paper + Ring (g)			184.96	Ring Dia (in)	2.498
Initial moisture Content (%)			19.99	Final moisture Content (%)	14.49
Area of Ring (in ²)			4.90	Initial Volume (in ³)	3.7149
Initial Bulk Density (kg/m ³)			2123	Initial Dry Density (kg/m ³)	1769
Specific Gravity of Soil			2.74	Equiv. Thick. of solids (mm)	12.441
Final Bulk Density (kg/m ³)			2245	Final Dry Density (kg/m ³)	1871
Initial gauge reading for Load 1			0.2566	Gauge reading for last Loading	0.1824
Initial Voids Ratio			0.548	Final Void Ratio	0.396
Initial Degree of Saturation (%)			100	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	5.0	7.5	11.5	17.0	25.0	37.5	55.0
Load (tsf)	0.052	0.078	0.120	0.177	0.260	0.390	0.572
Gauge Reading (in)	0.2533	0.2511	0.2484	0.2446	0.2418	0.2359	0.2295
(H-Hs) mm	6.730	6.672	6.604	6.508	6.437	6.287	6.124
Voids ratio	0.541	0.536	0.531	0.523	0.517	0.505	0.492
t90 (min)		60.06	51.84	46.24	59.29	36.00	30.25
Cv (m ² /day)		0.002	0.002	0.002	0.002	0.003	0.004
k' (MPa)		0.824	1.136	1.085	2.131	1.580	2.010
Mv (mm ² / N)		1.2137	0.8805	0.9215	0.4692	0.6329	0.4976

Trial #	8	9	10	11	12	13	14
Load (kPa)	85.0	130.0	195.0	130.0	85.0	55.0	37.5
Load (tsf)	0.884	1.352	2.028	1.352	0.884	0.572	0.390
Gauge Reading (in)	0.22115	0.2122	0.2031	0.2035	0.2045	0.2055	0.2069
(H-Hs) mm	5.912	5.684	5.454	5.465	5.490	5.515	5.550
Voids ratio	0.475	0.457	0.438	0.439	0.441	0.443	0.446
t90 (min)	26.52	18.49	16.40				
Cv (m ² /day)	0.004	0.005	0.006				
k' (MPa)	2.626	3.613	5.136				
Mv (mm ² / N)	0.3808	0.2768	0.1947				

Trial #	15	16	17	18	19	20	21
Load (kPa)	25.0	17.0	11.5	7.5	11.5	17.0	25.0
Load (tsf)	0.26	0.177	0.120	0.078	0.120	0.177	0.260
Gauge Reading (in)	0.2072	0.2085	0.2097	0.2113	0.2111	0.2107	0.2098
(H-Hs) mm	5.558	5.590	5.621	5.661	5.657	5.647	5.624
Voids ratio	0.447	0.449	0.452	0.455	0.455	0.454	0.452
t90 (min)							
Cv (m ² /day)							
k' (MPa)							
Mv (mm ² / N)							



Project

WINDSOR ESSEX PARKWAY

TITLE

**CONSOLIDATION TEST
TUNNEL T-9 (T9-1-SA15)**

Date

December 2011

JOB NO

SW8801.1004.101

FIGURE NO.
C.5-A

REV

Trial #	22	23	24	25	26	27	28
Load (kPa)	37.5	55.0	85.0	130.0	195.0	290.0	440.0
Load (tsf)	0.390	0.572	0.884	1.352	2.028	3.016	4.576
Gauge Reading (in)	0.2088	0.2074	0.2058	0.2042	0.2007	0.1937	0.1841
(H-Hs) mm	5.597	5.563	5.522	5.482	5.393	5.214	4.971
Voids ratio	0.450	0.447	0.444	0.441	0.433	0.419	0.400
t90 (min)						9.30	9.30
Cv (m ² /day)						0.010	0.010
k' (MPa)						9.461	10.917
Mv (mm ² / N)						0.1057	0.0916

Trial #	29	30	31	32	33	34	35
Load (kPa)	660.0	990.0	1500	750.0	370.0	185.0	90.0
Load (tsf)	6.864	10.296	15.6	7.800	3.848	1.924	0.936
Gauge Reading (in)	0.1747	0.1652	0.15485	0.1560	0.1571	0.1606	0.1648
(H-Hs) mm	4.732	4.491	4.228	4.257	4.285	4.374	4.481
Voids ratio	0.380	0.361	0.340	0.342	0.344	0.352	0.360
t90 (min)	8.70	6.25	4.84				
Cv (m ² /day)	0.010	0.014	0.018				
k' (MPa)	16.044	23.486	32.847				
Mv (mm ² / N)	0.0623	0.0426	0.0304				

Trial #	36	37	38	39			
Load (kPa)	45.0	22.5	11.5	5.5			
Load (tsf)	0.468	0.234	0.1196	0.0572			
Gauge Reading (in)	0.1679	0.1728	0.1778	0.1824			
(H-Hs) mm	4.560	4.684	4.811	4.928			
Voids ratio	0.367	0.376	0.387	0.396			
t90 (min)							
Cv (m ² /day)							
k' (MPa)							
Mv (mm ² / N)							

Project

WINDSOR ESSEX PARKWAY

TITLE

**CONSOLIDATION TEST
TUNNEL T-9 (T9-1-SA15)**

Date

December 2011

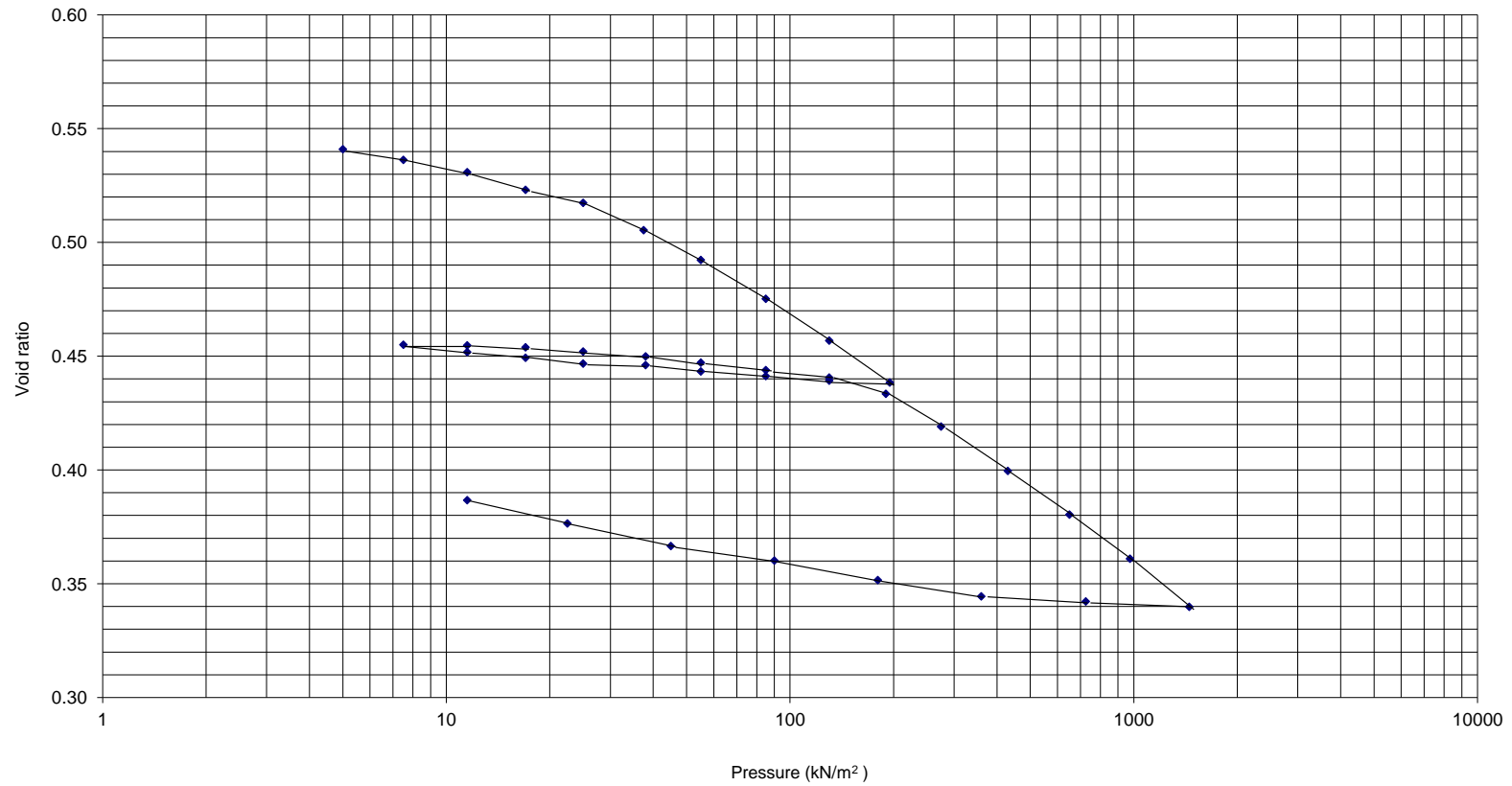
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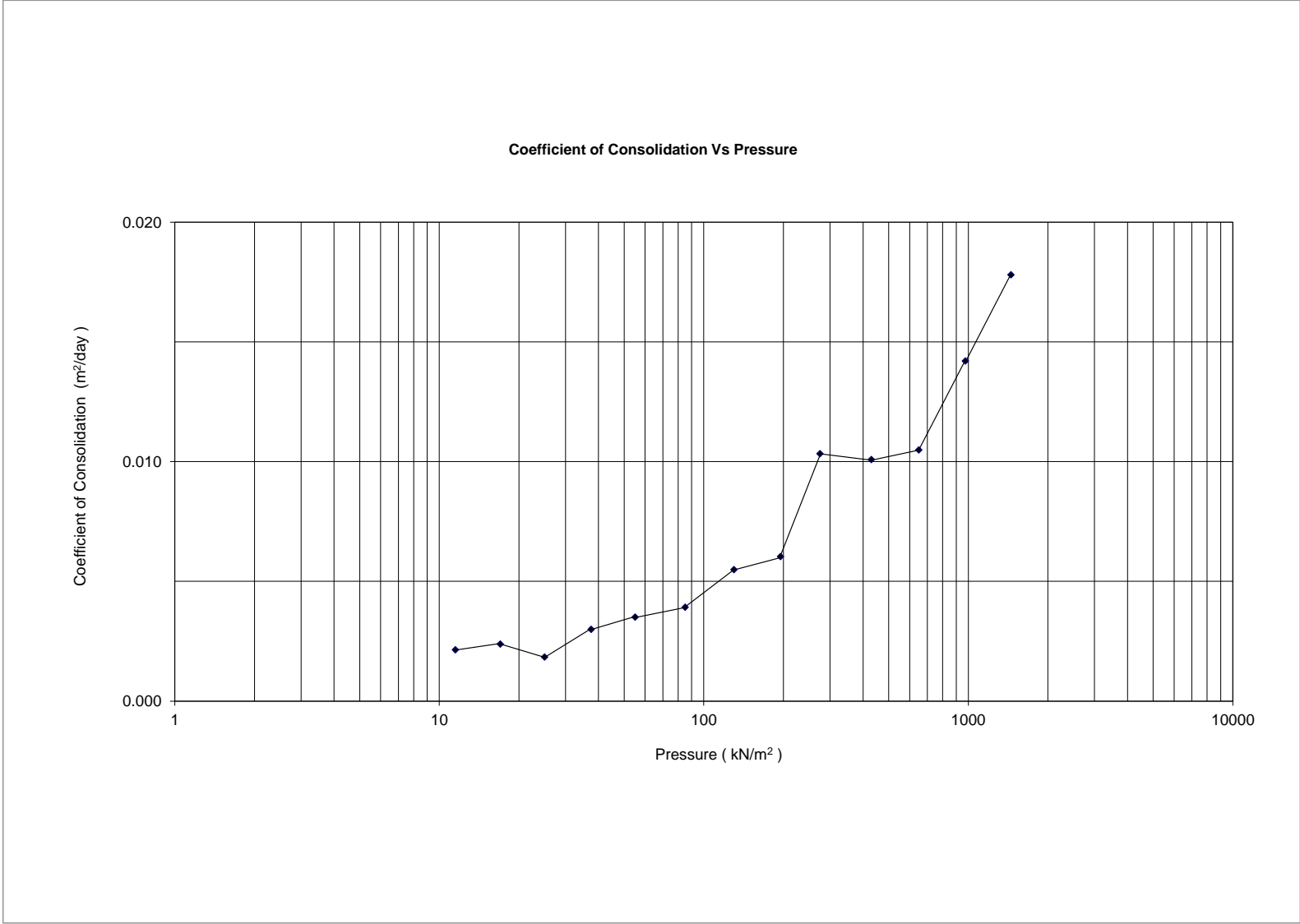
SW8801.1004.101

FIGURE NO.
C.5-B

REV

Void Ratio Vs Pressure



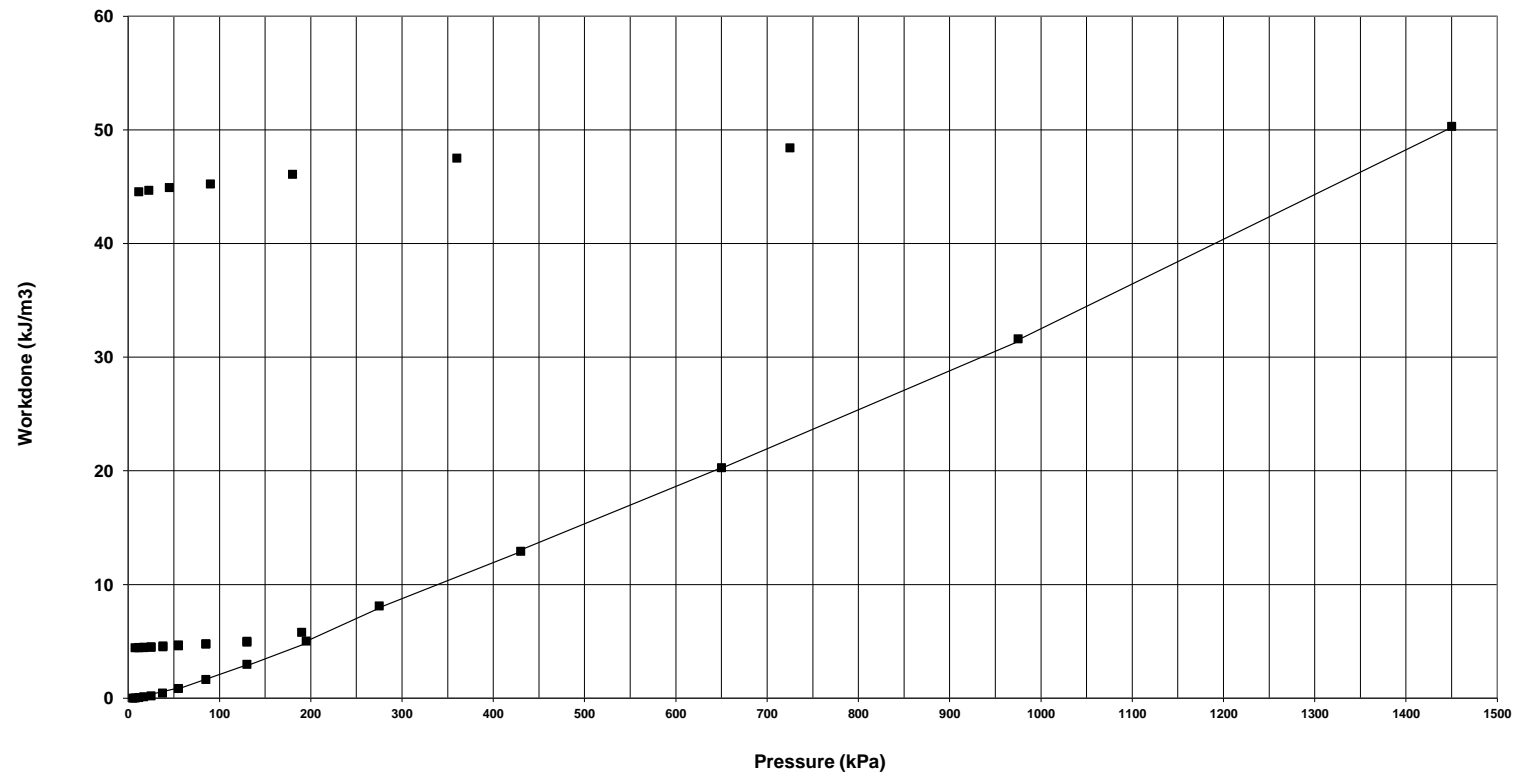


Strain Energy Data

Presssure (kN/m ²)	c _v (m ² /day)	Void ratio
5.0		0.541
7.5		0.536
11.5	0.002	0.531
17.0	0.002	0.523
25.0	0.002	0.517
37.5	0.003	0.505
55.0	0.004	0.492
85.0	0.004	0.475
130.0	0.005	0.457
195.0	0.006	0.438
130.0		0.439
85.0		0.441
55.0		0.443
38.0		0.446
25.0		0.447
17.0		0.449
11.5		0.452
7.5		0.455
11.5		0.455
17.0		0.454
25.0		0.452
38.0		0.450
55.0		0.447
85.0		0.444
130.0		0.441
190.0		0.433
275.0	0.010	0.419
430.0	0.010	0.400
650.0	0.010	0.380
975.0	0.014	0.361
1450.0	0.018	0.340
725.0		0.342
360.0		0.344
180.0		0.352
90.0		0.360
45.0		0.367
22.5		0.376
11.5		0.387
5.5		0.396

Presssure (kN/m ²)	Height mm	Total Work (kJ/m ³)
5.0	19.253	0.000
7.5	19.195	0.019
11.5	19.128	0.052
17.0	19.031	0.124
25.0	18.960	0.203
37.5	18.811	0.449
55.0	18.648	0.850
85.0	18.436	1.646
130.0	18.207	2.979
195.0	17.978	5.026
130.0	17.988	4.932
85.0	18.013	4.783
55.0	18.038	4.684
38.0	18.073	4.595
25.0	18.116	4.520
17.0	18.148	4.483
11.5	18.179	4.458
7.5	18.219	4.437
11.5	18.216	4.439
17.0	18.205	4.448
25.0	18.182	4.474
38.0	18.155	4.520
55.0	18.121	4.608
85.0	18.080	4.765
130.0	18.040	5.007
190.0	17.951	5.795
275.0	17.772	8.114
430.0	17.529	12.926
650.0	17.291	20.281
975.0	17.049	31.620
1450.0	16.786	50.316
725.0	16.816	48.424
360.0	16.844	47.522
180.0	16.932	46.097
90.0	17.039	45.247
45.0	17.119	44.932
22.5	17.242	44.688
11.5	17.369	44.563
5.5	17.486	44.506

Strain Energy Method for Preconsolidation Pressure



**CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST
FOR COHESIVE SOILS (ASTM D-4767)**

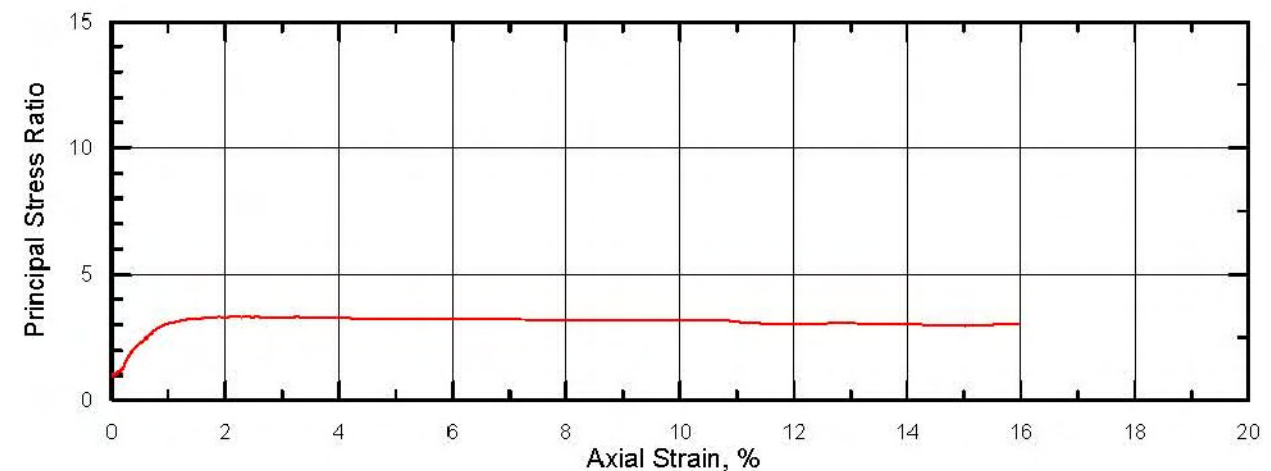
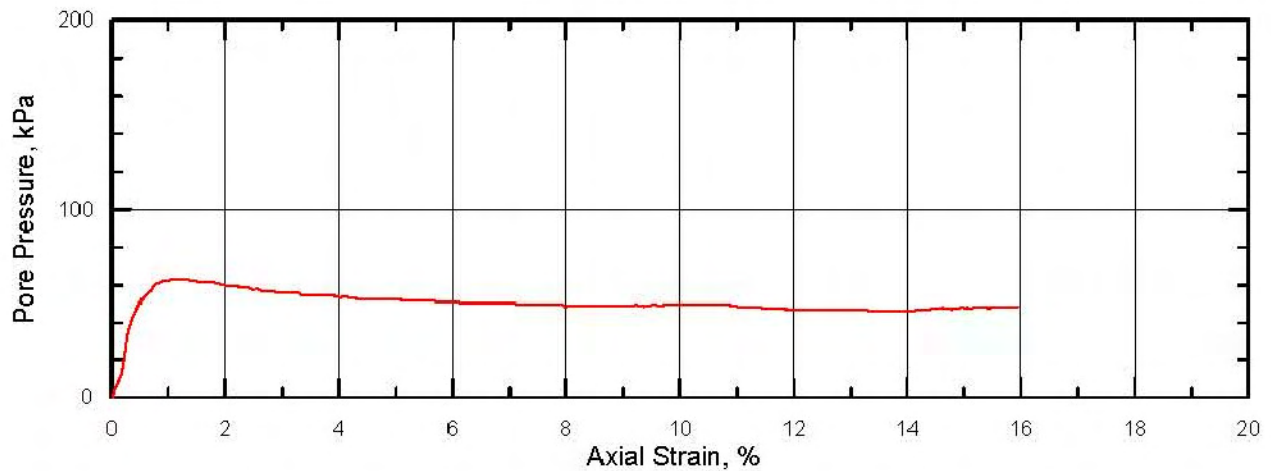
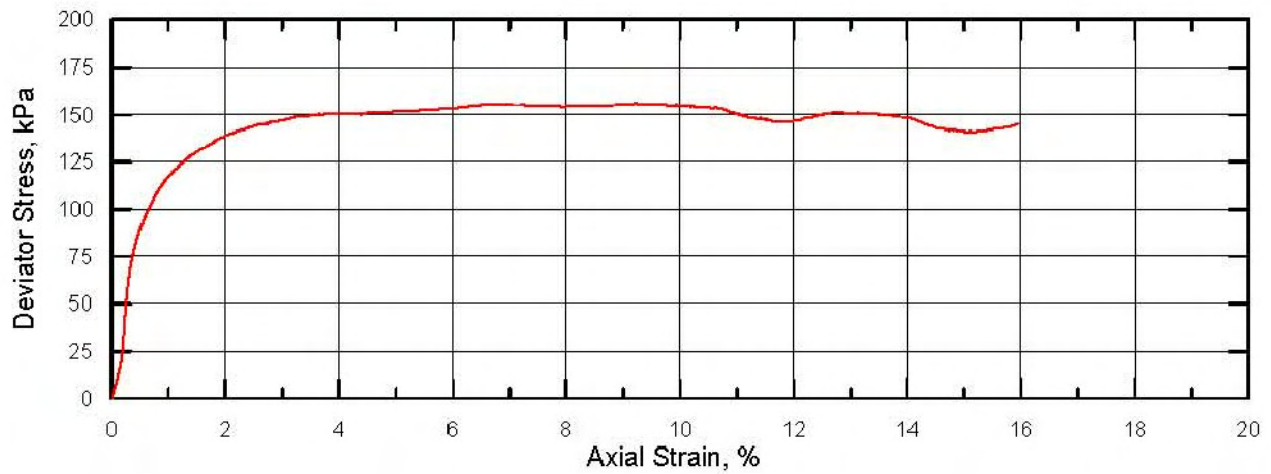
Project: WEP
Client: Hatch Mott MacDonald Limited
Location: Windsor, ON.

Sample ID: T9-1_TW12

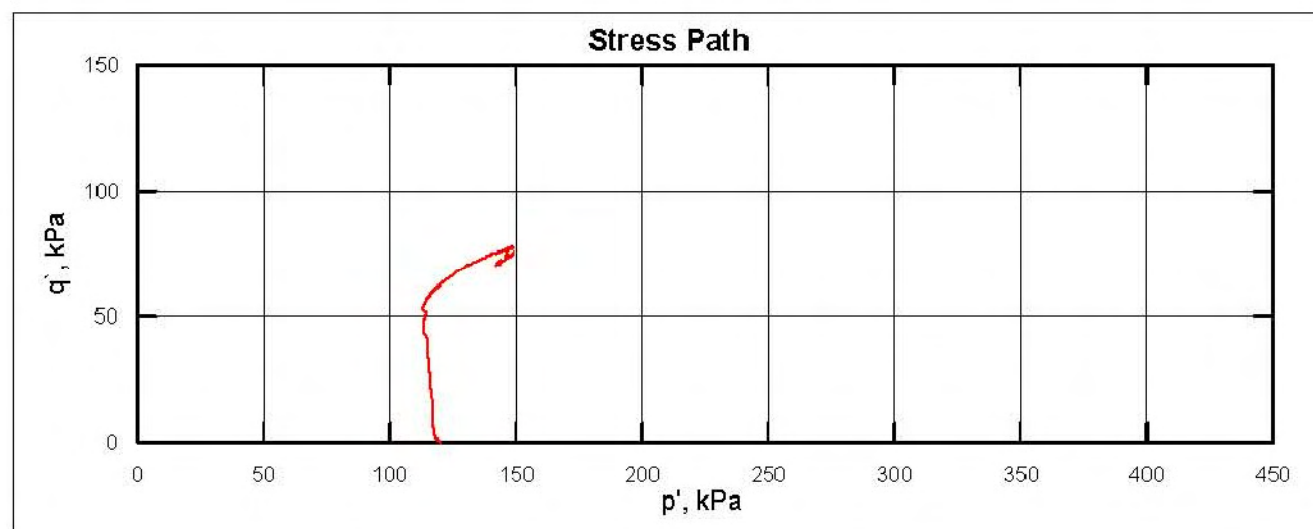
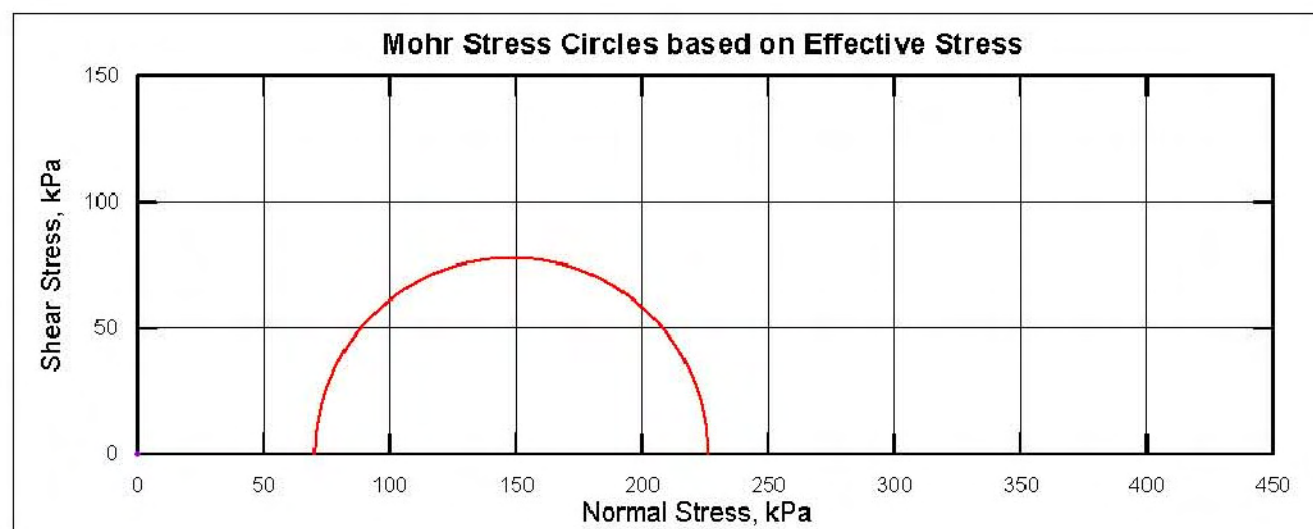
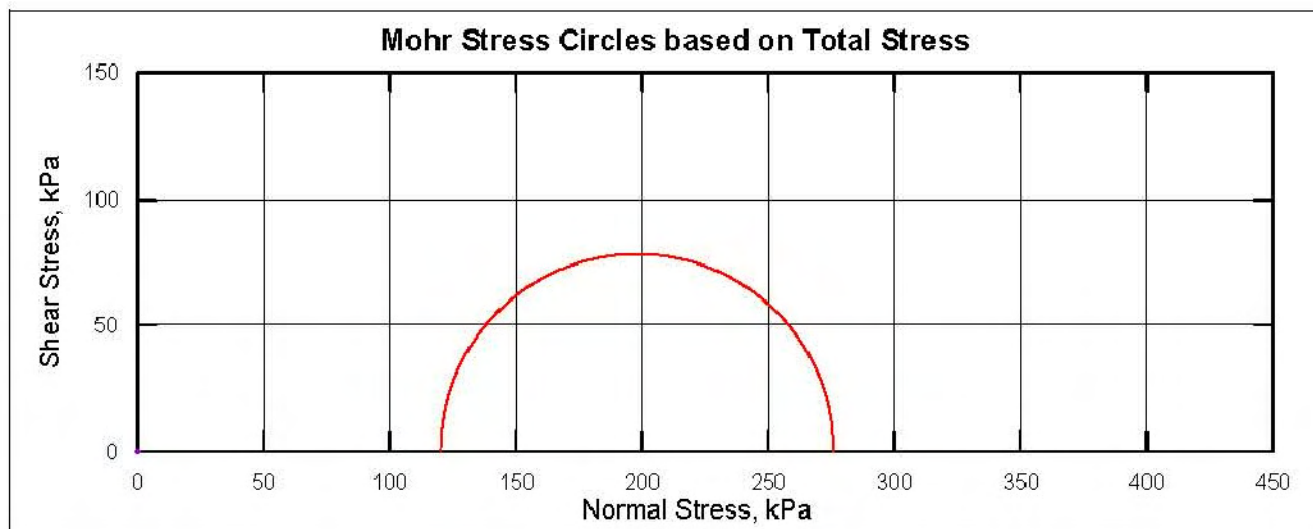
Project No.: SW8801.1004.101
Date: 1-Nov-11
Depth(m): 10.7 to 11.3

Sample Description: Sandy Silty Clay trace gravel

Sample Parameters				
Initial		Specimen 1	Specimen 2	Specimen 3
Diameter	cm	7.256		
Height	cm	14.625		
Volume	cm ³	604.756		
Wet Mass	g	1322.80		
Dry Density	kg/m ³	1874		
Water Content	%	16.7		
Specific Gravity	Actual	2.740		
Void Ratio		0.46		
Degree of Saturation		99.1		
Before Shear (after consolidation)				
Volume	cm ³	589.656		
B - Value		0.98		
After Shear				
Wet Mass	g	1316.64		
Dry Density	kg/m ³	1920		
Water Content	%	16.3		
Void Ratio		0.43		
Degree of Saturation		100.0		
Stress - Strain				
Cell Pressure	kPa	420.00		
Back Pressure	kPa	300.00		
Consolidation Stress	kPa	120.00		
Rate of Strain	mm/min	0.0200		
Vertical Strain at Failure	%	9.22		
Deviator Stress at Failure	kPa	156.00		
Pore Pressure at Failure	kPa	49.70		
Total Stress				
Minor Principal Stress, σ_3	kPa	120.00		
Major Principal Stress, σ_1	kPa	276.00		
Radius, $(\sigma_1 - \sigma_3)/2$	kPa	78.00		
Intersection Point, $(\sigma_1 + \sigma_3)/2$	kPa	198.00		
Effective Stress				
Minor Principal Stress, σ_3'	kPa	70.30		
Major Principal Stress, σ_1'	kPa	226.30		
Radius, $(\sigma_1' - \sigma_3')/2$	kPa	78.00		
Intersection Point, $(\sigma_1' + \sigma_3')/2$	kPa	148.30		



— 120 kPa



— 120 kPa

Note:
Failure based on maximum deviator stress

Appendix D Analytical Laboratory Test Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-9 (Cousineau Tunnel) (Sta. 12+130L to 12+300L)
Doc No.: 285380-04-119-0050 (Geocres No. 40J3-19)

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



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

Date Received: 25-JUL-11
Report Date: 29-JUL-11 20:51 (MT)
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #:	L1035556
Project P.O. #:	NOT SUBMITTED
Job Reference:	SW8801.1004.101
Legal Site Desc:	
C of C Numbers:	112828

Gayle Braun
Senior Account Manager

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

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

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample ID Description Sampled Date Sampled Time Client ID		L1035556-1 SOIL 22-JUL-11 T9- 1,SS8@17.5',SILT Y CLAY, GREY				
Grouping	Analyte					
SOIL						
Physical Tests	% Moisture (%)	13.9				
	pH (pH units)	7.75				
	Redox Potential (mV)	102				
	Resistivity (ohm cm)	4130				
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20				
Anions and Nutrients	Sulphate (mg/kg)	58				

Reference Information

Test Method References:

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

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

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

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

Chain of Custody Numbers:

112828

GLOSSARY OF REPORT TERMS

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

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

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

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

mg/L - milligrams per litre.

< - Less than.

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

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

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

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

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



Quality Control Report

Workorder: L1035556

Report Date: 29-JUL-11

Page 1 of 3

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	R2224277							
WG1318502-2	LCS							
% Moisture			92		%		70-130	25-JUL-11
WG1318502-1	MB							
% Moisture			<0.10		%		0.1	25-JUL-11
PH-WT								
	Soil							
Batch	R2226613							
WG1321682-1	CVS							
pH			100		%		80-120	27-JUL-11
RESISTIVITY-WT								
	Soil							
Batch	R2226581							
WG1319414-2	CVS							
Resistivity			99		%		70-130	27-JUL-11
SO4-WT								
	Soil							
Batch	R2225769							
WG1319770-3	LCS							
Sulphate			101		%		60-140	27-JUL-11
WG1319770-1	MB							
Sulphate			<20		mg/kg		20	27-JUL-11
SULPHIDE-WT								
	Soil							
Batch	R2224730							
WG1319337-1	CVS							
Sulphide			96		%		50-120	26-JUL-11
WG1319332-1	MB							
Sulphide			<0.20		mg/kg		0.2	26-JUL-11

Quality Control Report

Workorder: L1035556

Report Date: 29-JUL-11

Page 2 of 3

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Workorder: L1035556

Report Date: 29-JUL-11

Page 3 of 3

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential	1	22-JUL-11	27-JUL-11 14:11	24	122	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 L1035556 were received on 25-JUL-11 10:00.

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

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

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

Appendix E Rock Core Photographs

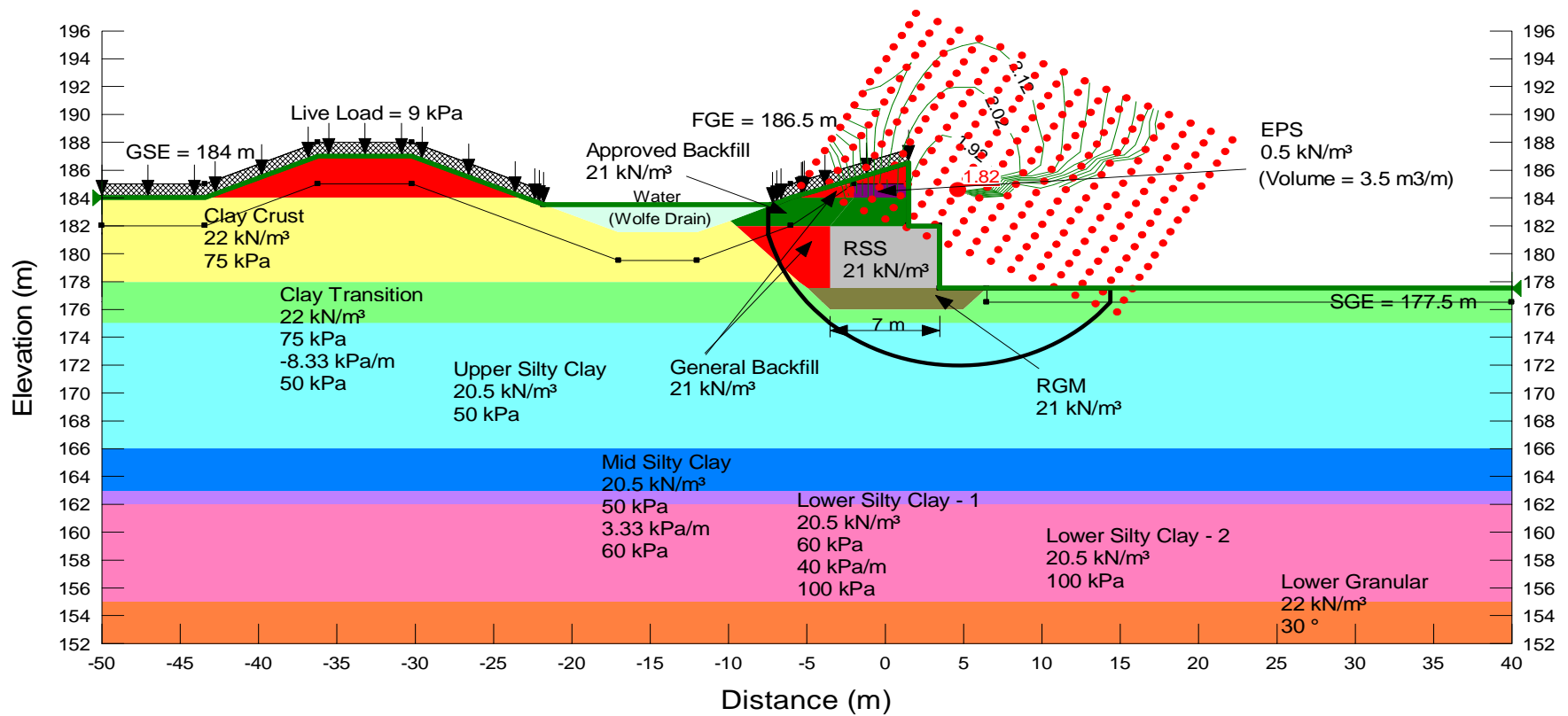


Photo 1 Borehole T9-1 - Rock Core. Elevation 151.7 meters to 149.1 meters

Appendix F Slope Stability Analyses Results

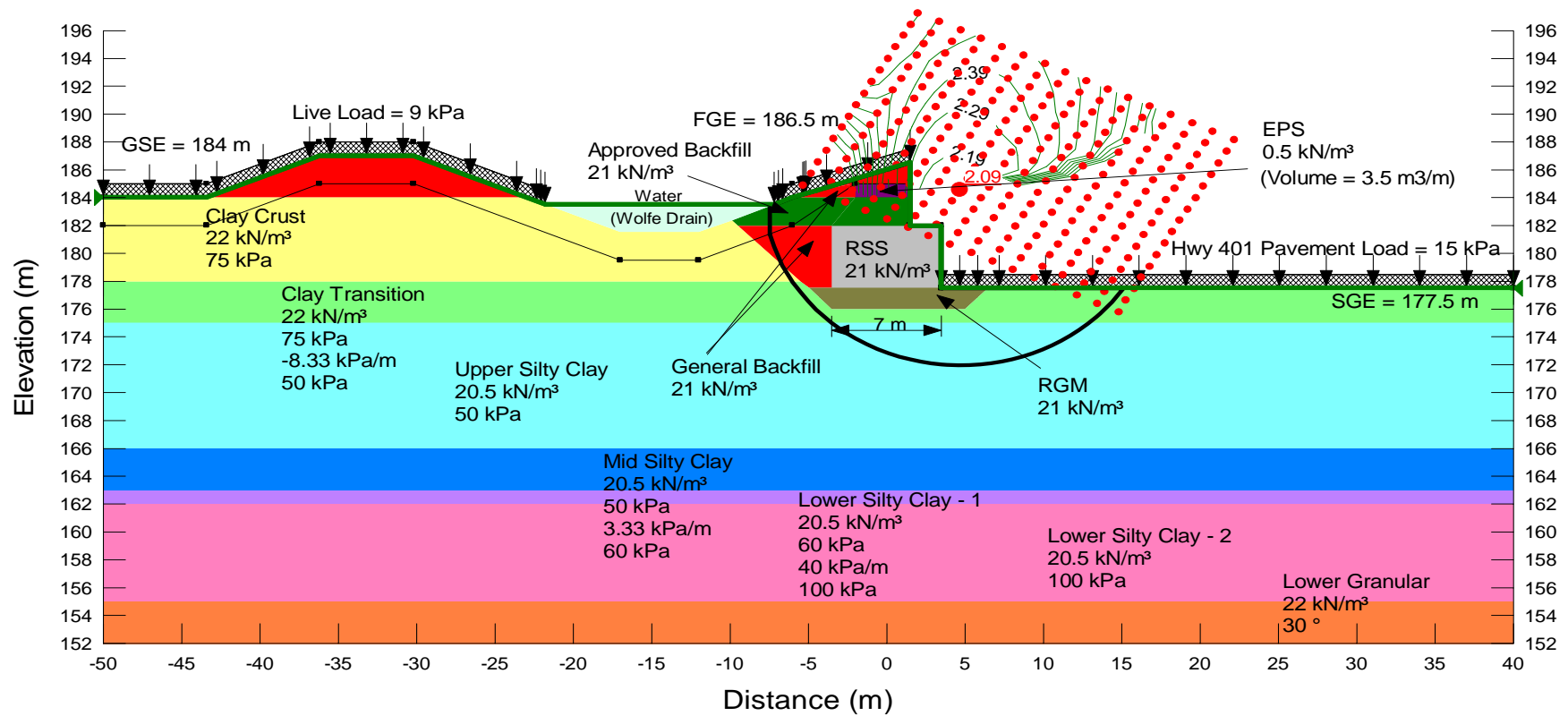
Tunnel T-9-North RSS Wall-Sta. 12+130L-Undrained.gsz

WEP SW8801.1002.101



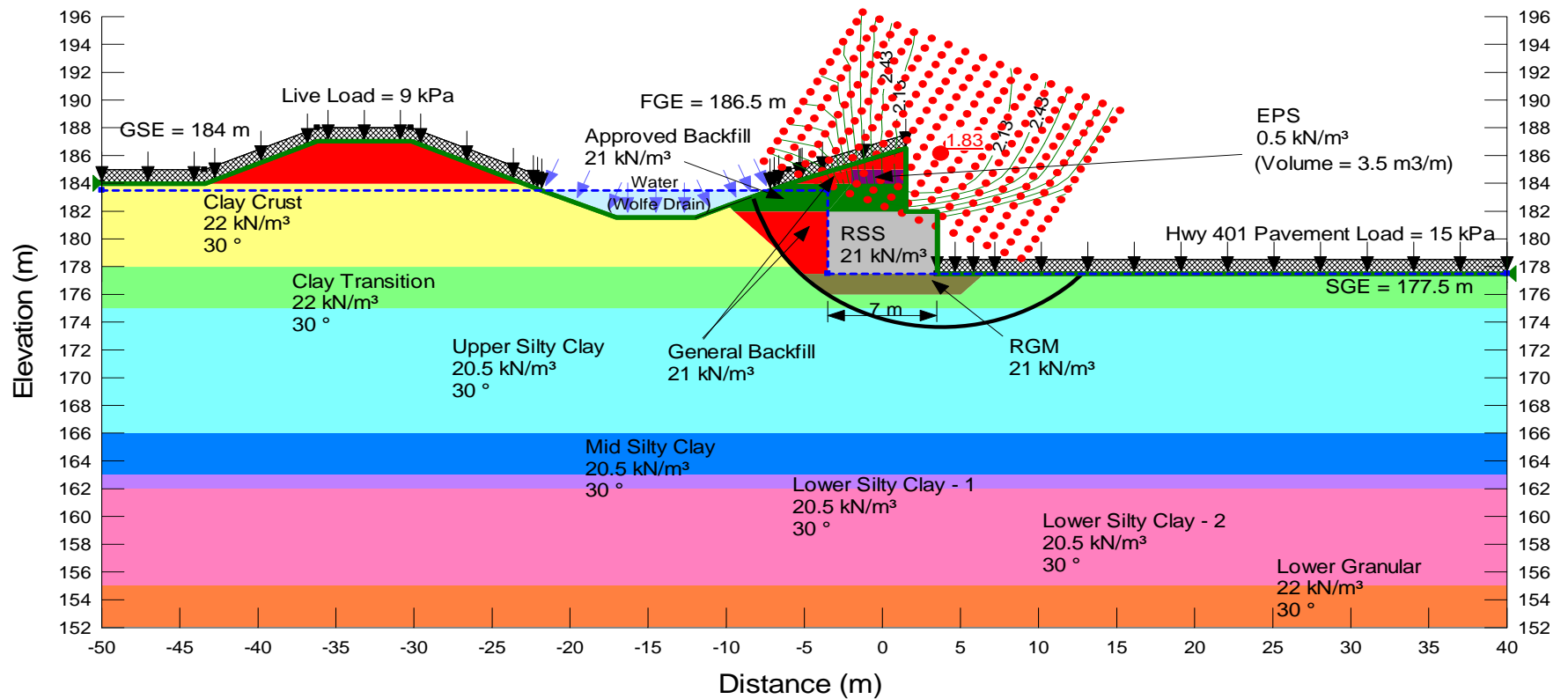
Tunnel T-9-North RSS Wall-Sta. 12+130L-Undrained.gsz

WEP SW8801.1002.101



Tunnel T-9-North RSS Wall-Sta. 12+130L-Drained.gsz

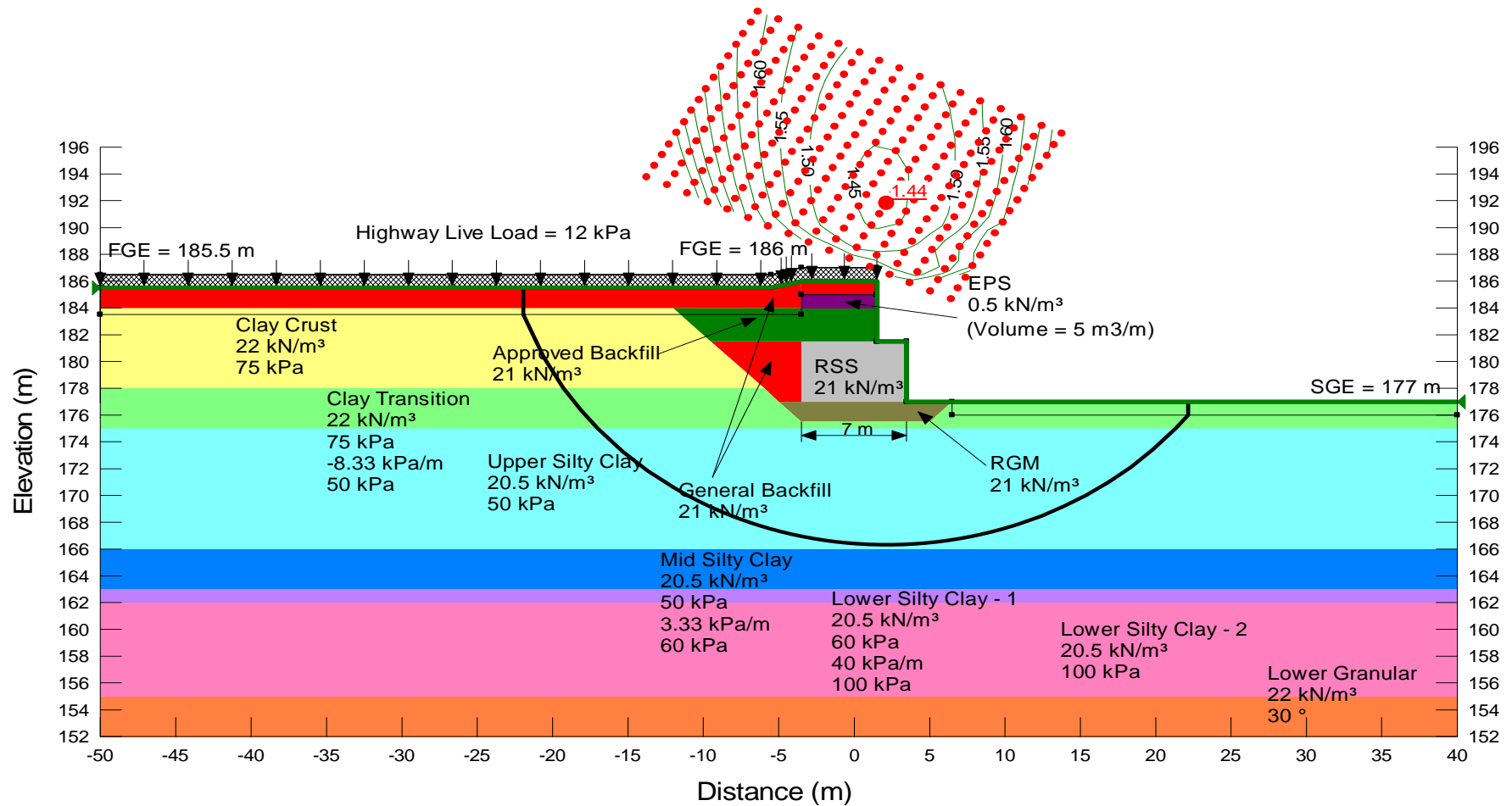
WEP SW8801.1002.101



Tunnel T-9-North RSS Wall-Sta 12+215L-Undrained.gsz

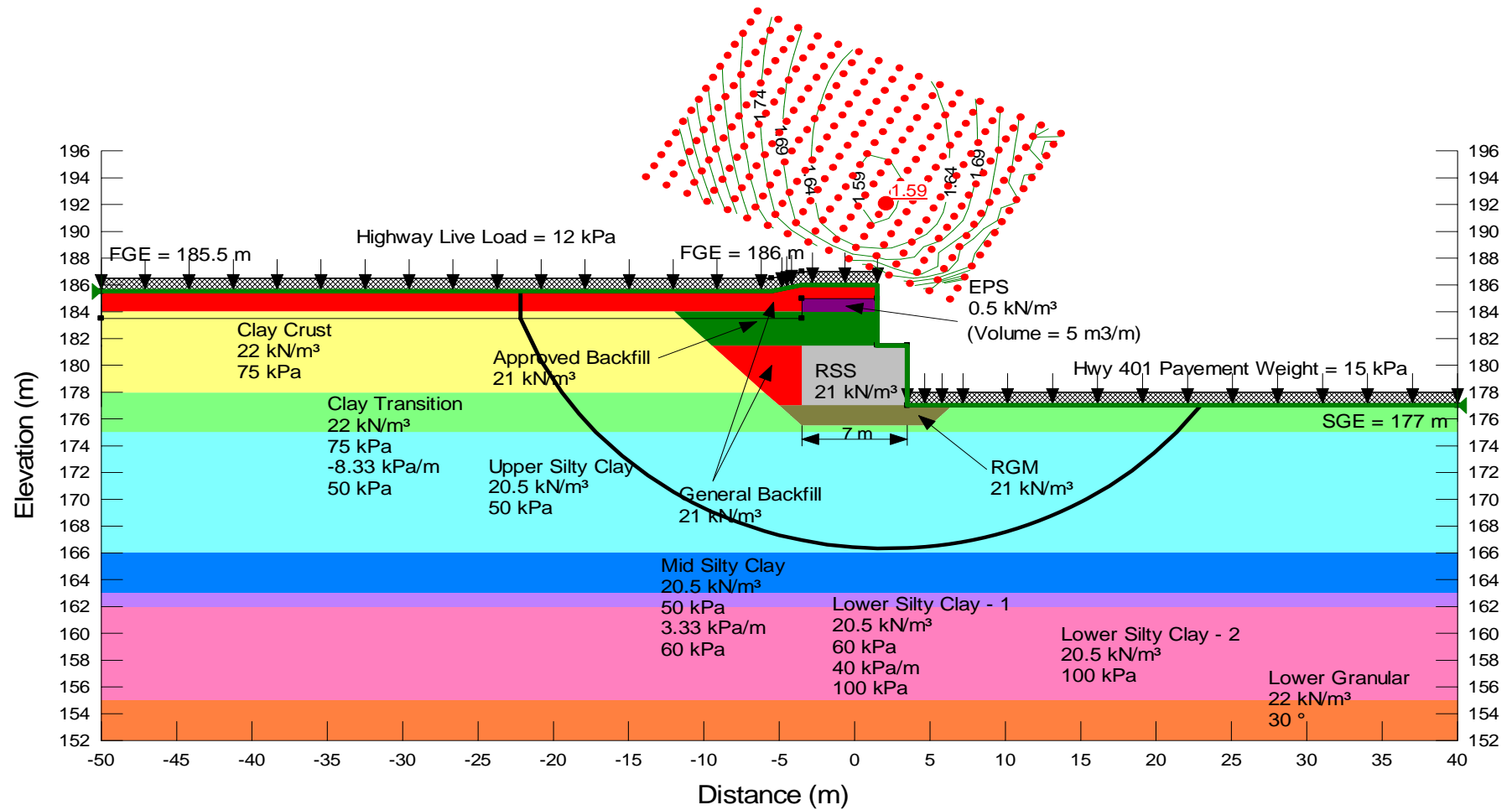
1/06/2012

WEP SW8801.1002.101

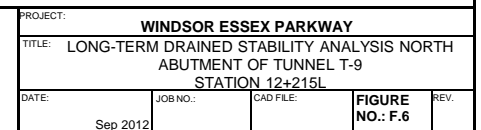


1/06/2012

WEP SW8801.1002.101

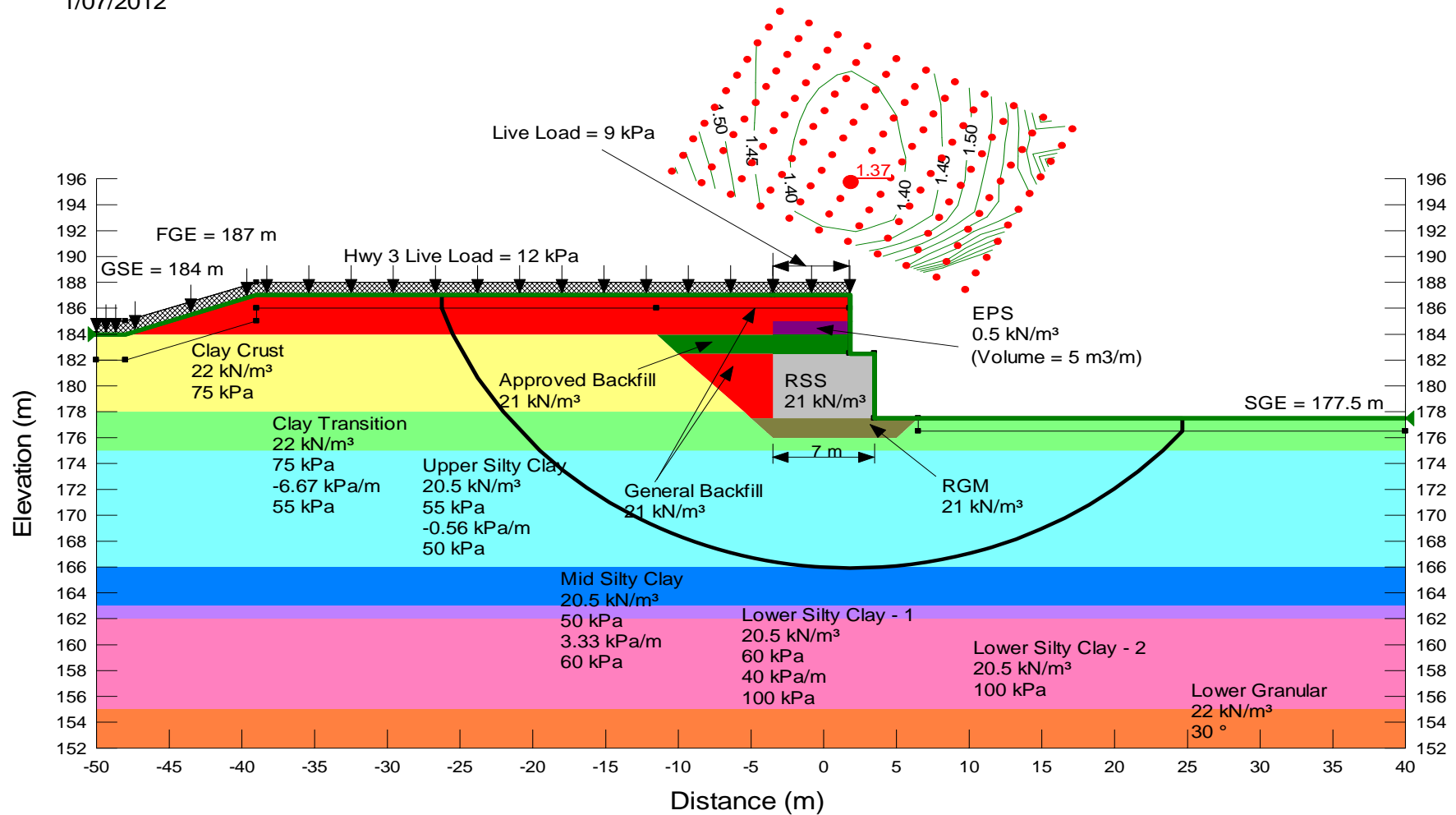


WEP SW8801.1002.101



Tunnel T-9-South RSS Wall-Sta 12+130L-Undrained.gsz
1/07/2012

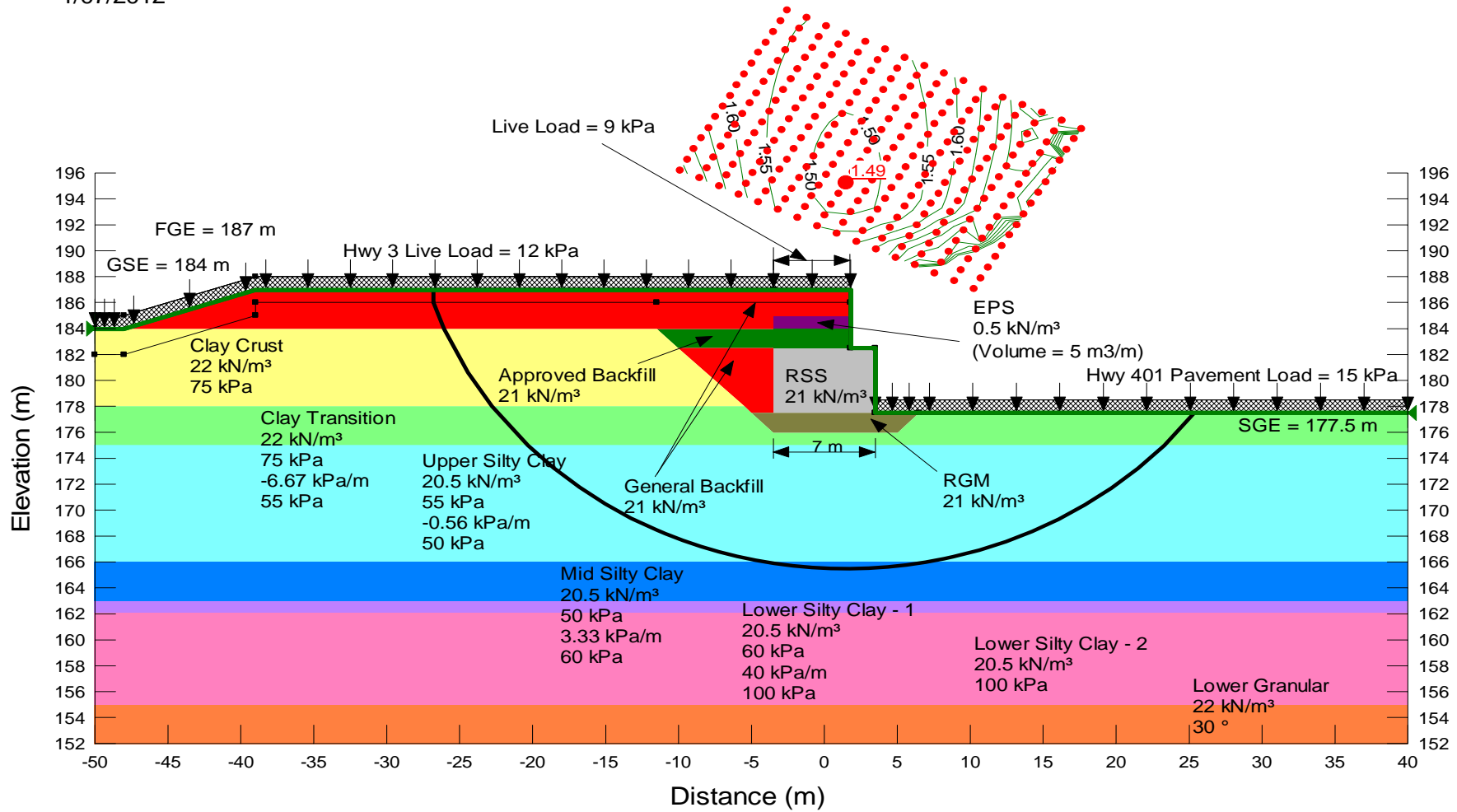
WEP SW8801.1002.101



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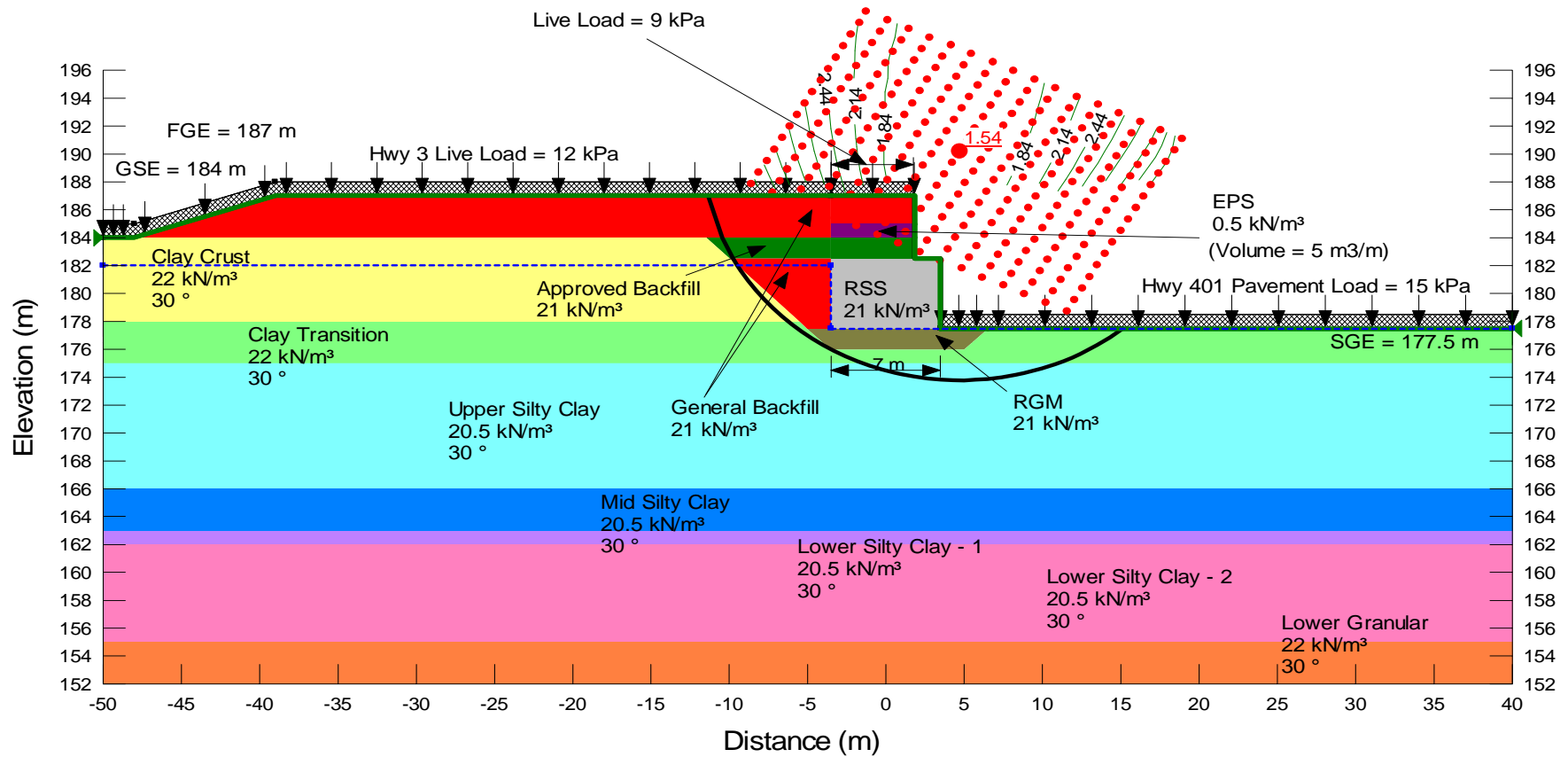
1/07/2012

WEP SW8801.1002.101



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1/07/2012

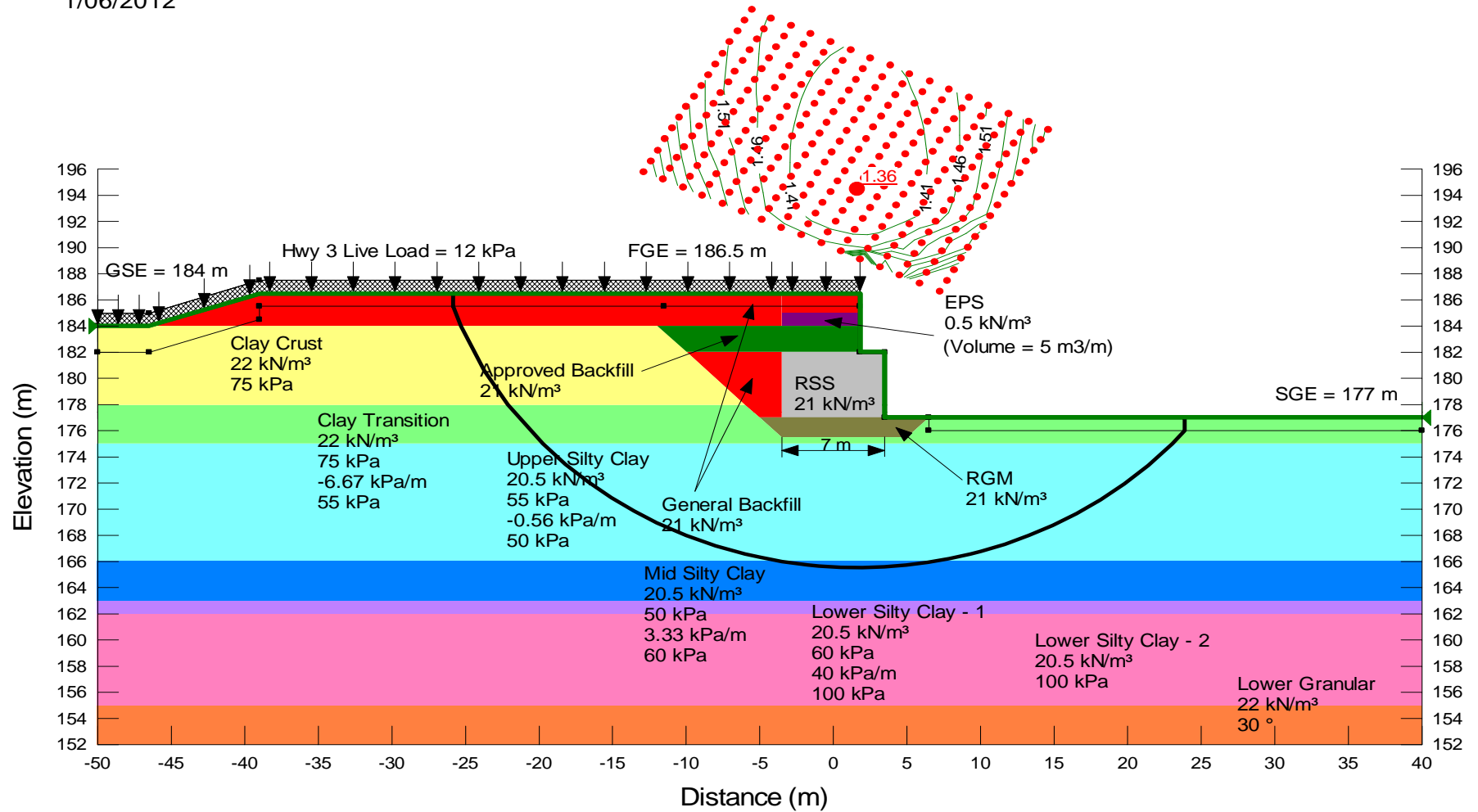
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1/06/2012

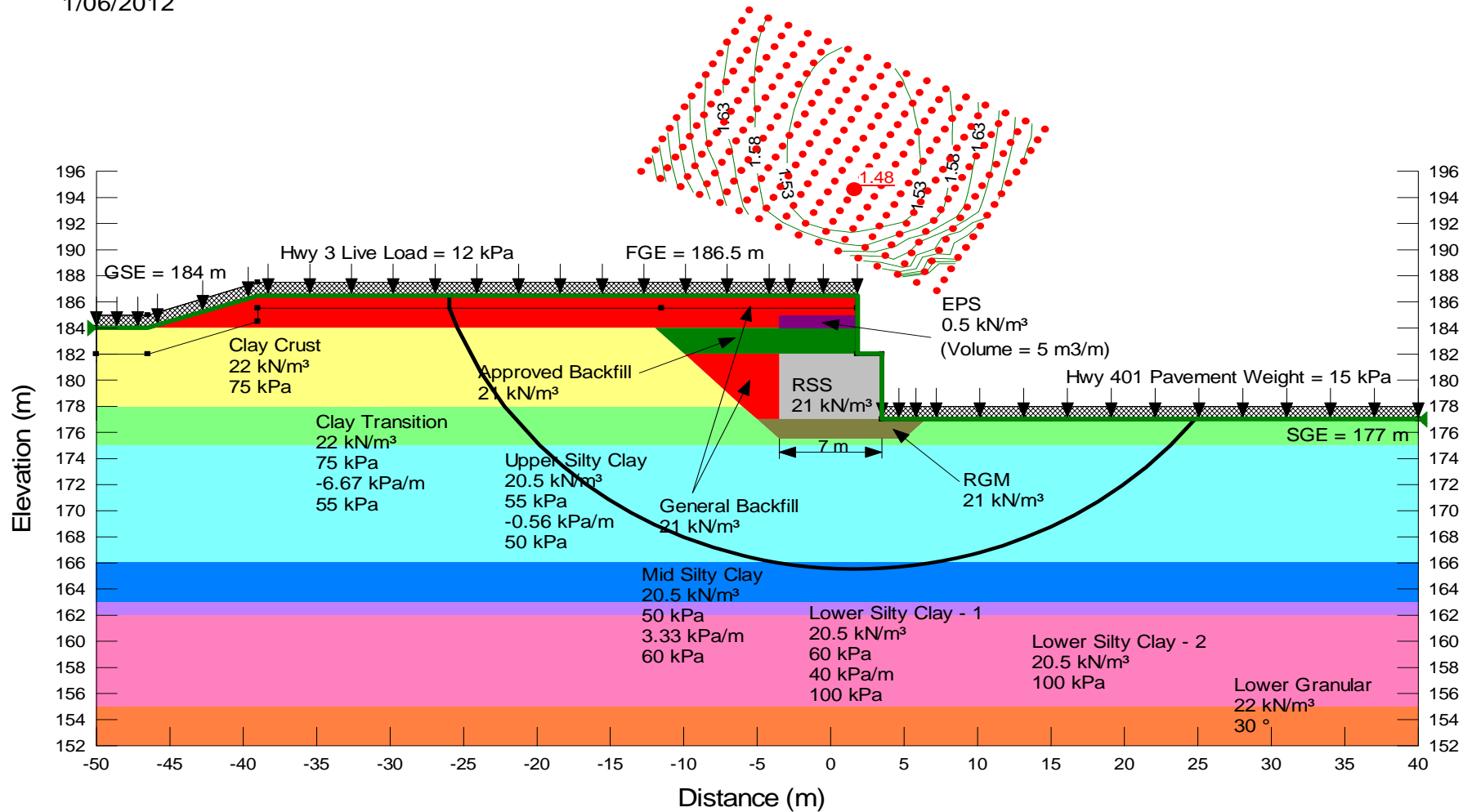
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1/06/2012

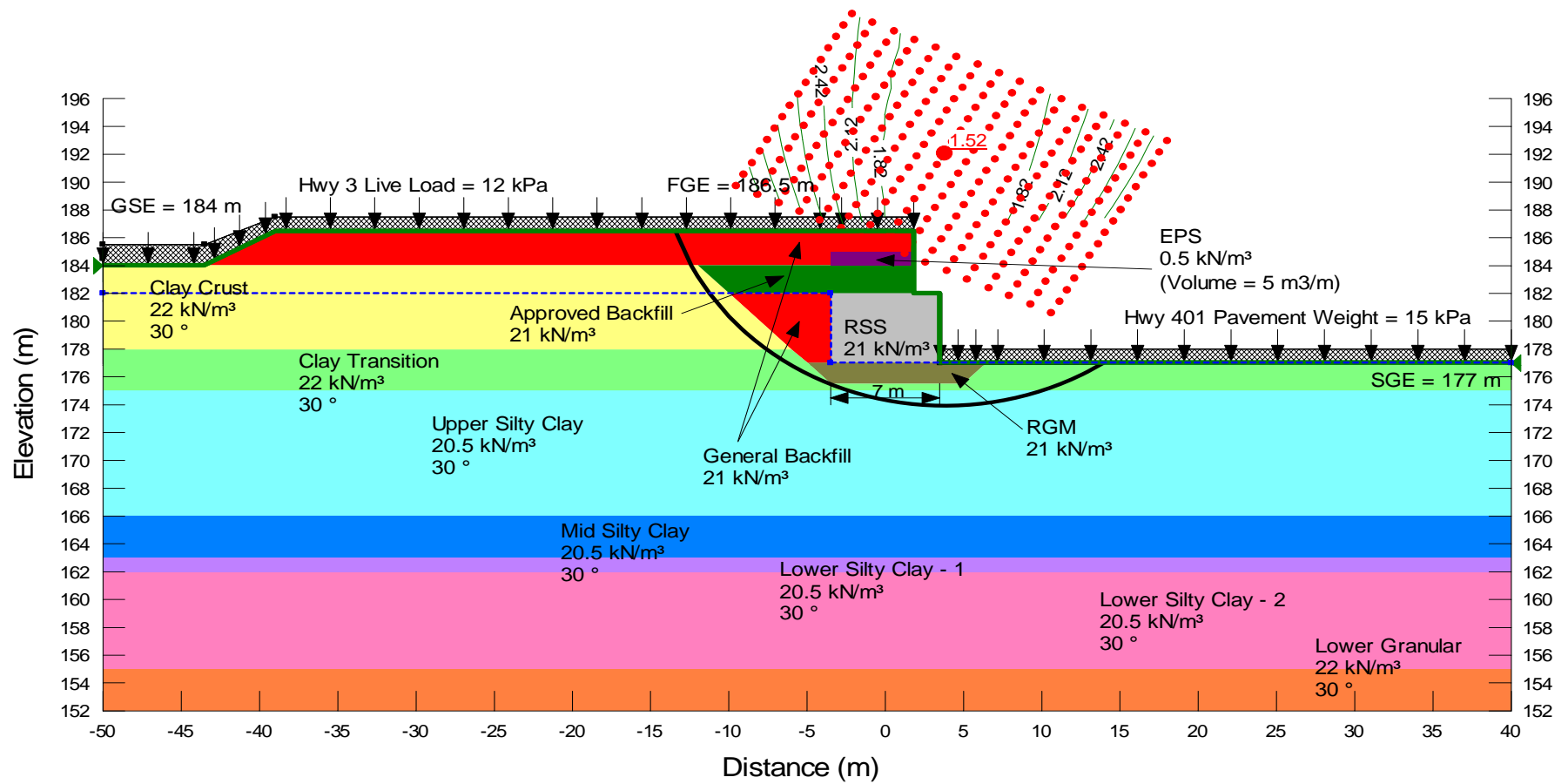
WEP SW8801.1002.101



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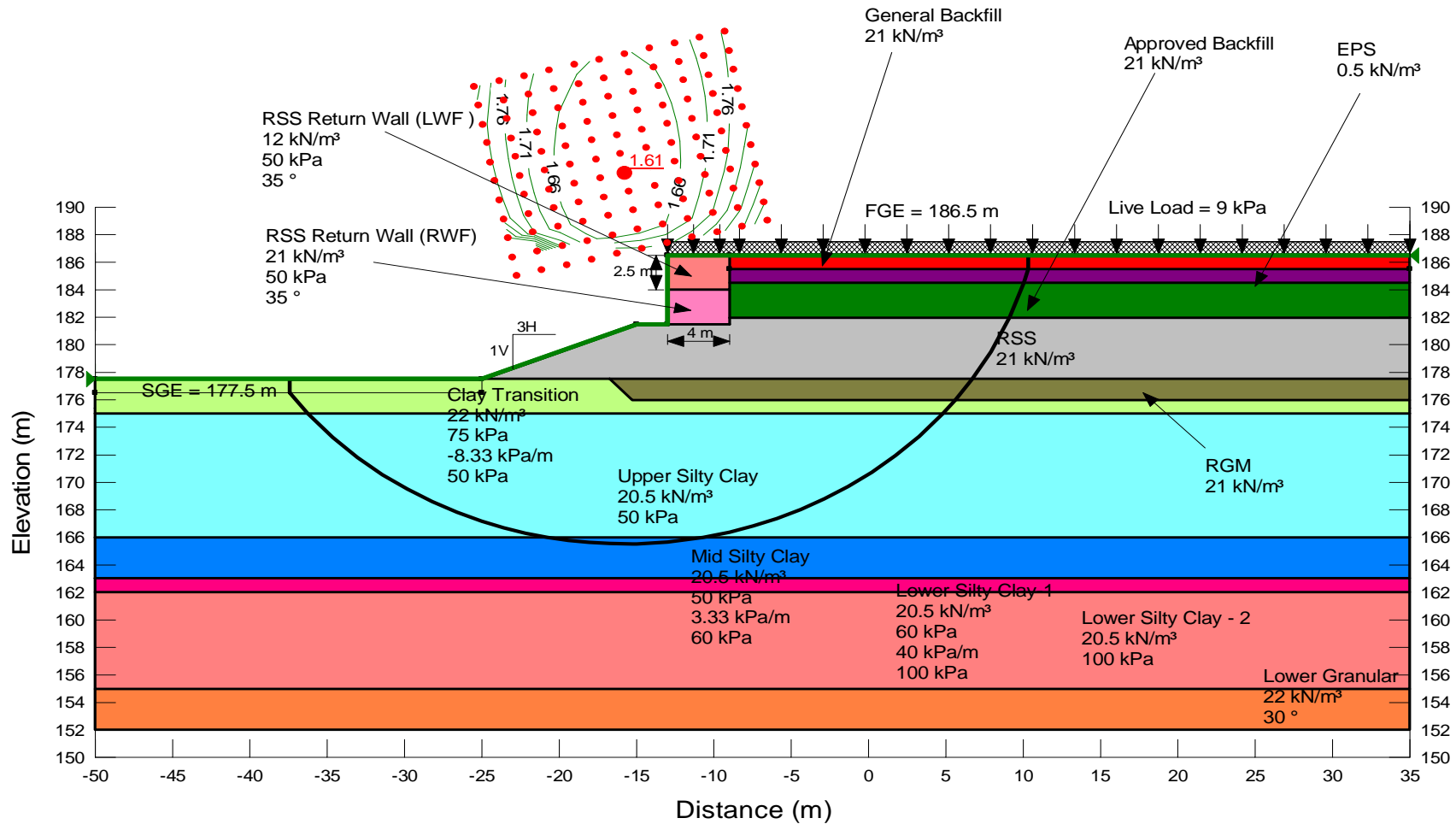
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WEP SW8801.1002.101



WEP SW8801.1002.101

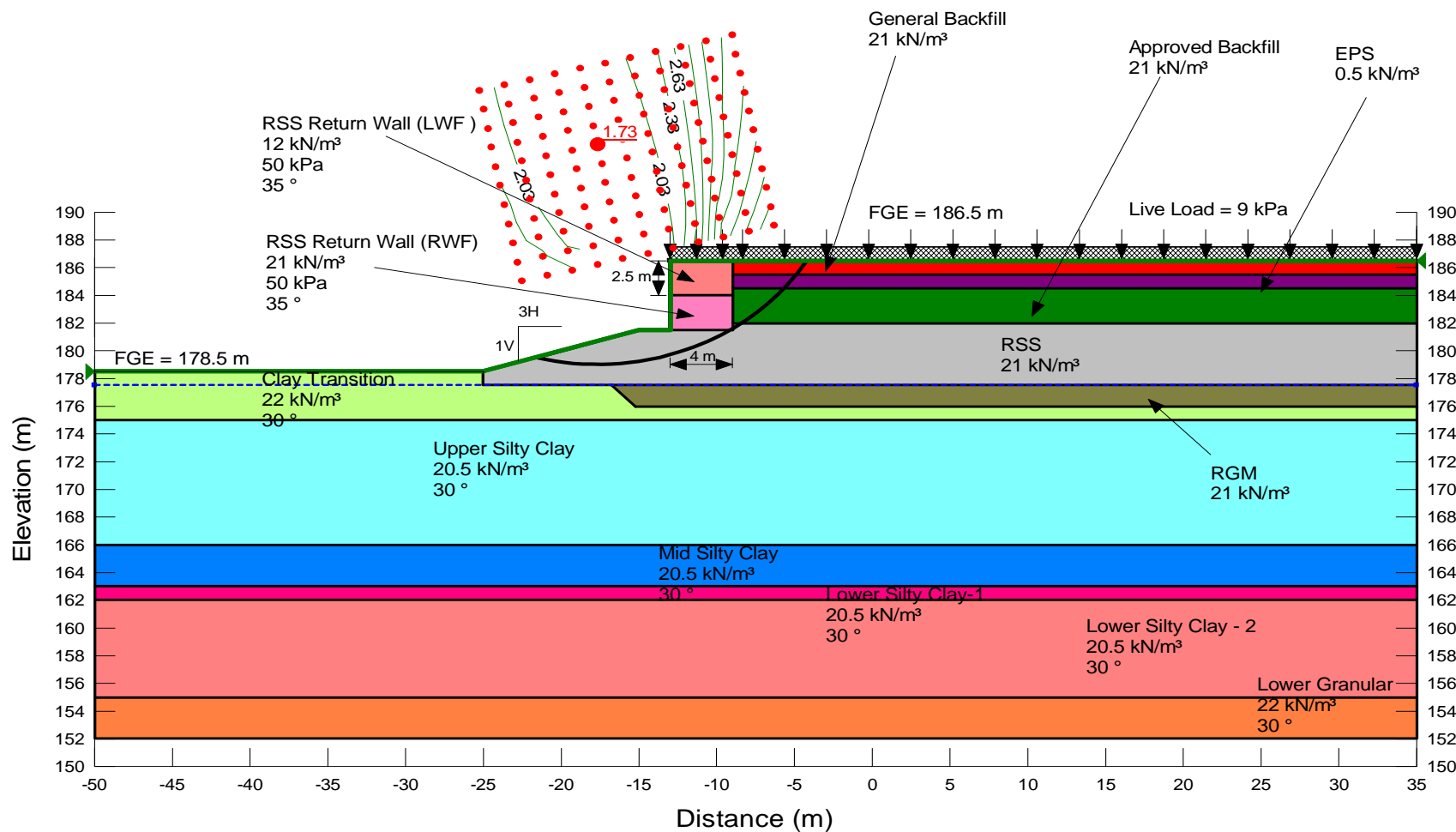
1/12/2012



Tunnel T-9- North RSS Return Wall-Combined LWF and RWF-D.gsz

1/12/2012

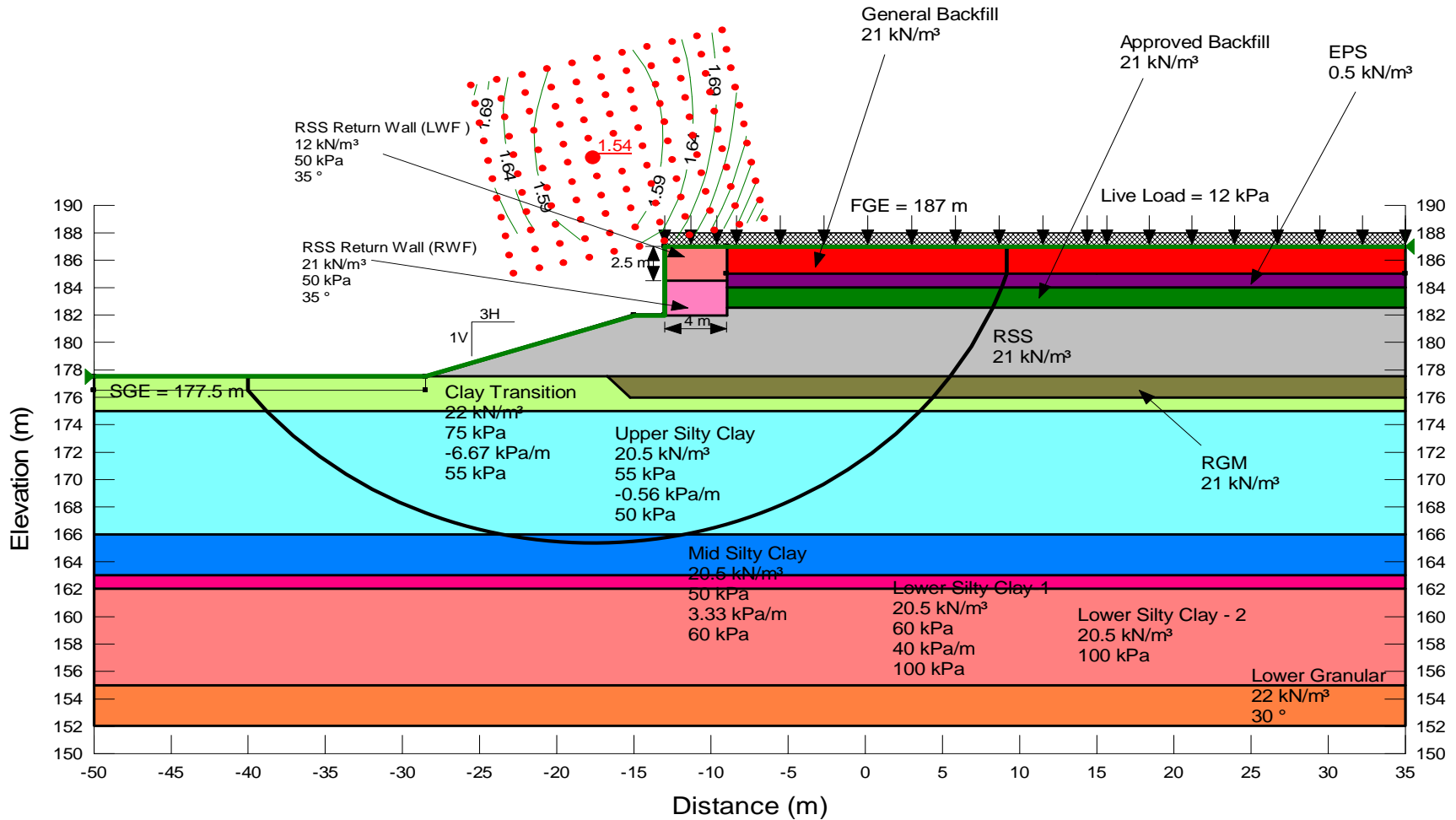
WEP SW8801.1002.101



Tunnel T-9- South RSS Return Wall-Combined LWF and RWF-UD.gsz

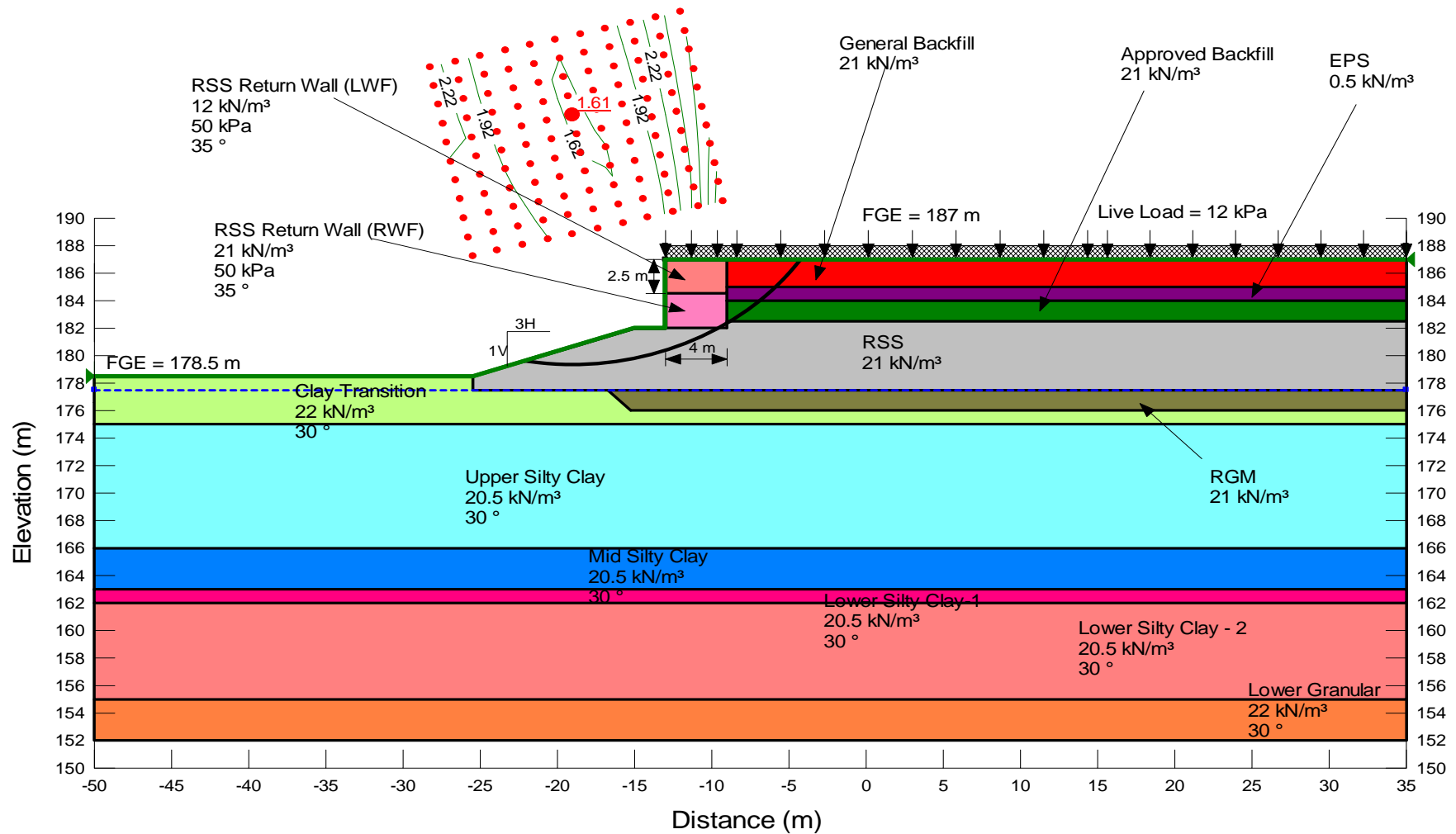
1/12/2012

WEP SW8801.1002.101



Tunnel T-9- South RSS Return Wall-Combined LWF and RWF-D.gsz
1/12/2012

WEP SW8801.1002.101



Appendix G Stress-Deformation Analysis Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-9 (Cousineau Tunnel) (Sta. 12+130L to 12+300L)
Doc No.: 285380-04-119-0050 (Geocres No. 40J3-19)

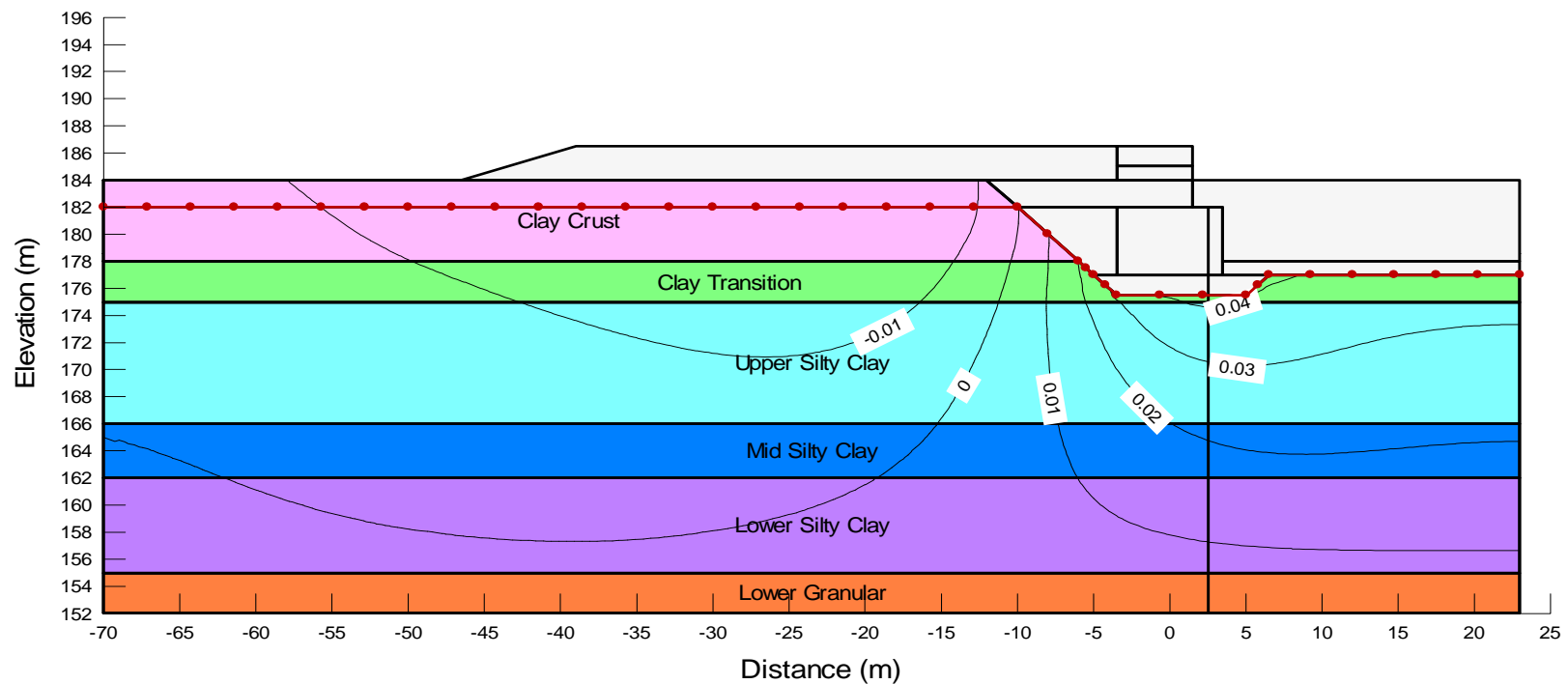
Date: September/2012
Rev: 0
Page No.: Appendix G

Tunnel T-9-RSS Wall-Central Segment-Deformation.gsz

WEP SW8801.1002.101

1/16/2012

Name: Clay Crust Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Clay Transition Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Upper Silty Clay O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Phi': 25 °
 Name: Mid Silty Clay O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.086 Kappa: 0.0094 Initial Void Ratio: 0.65 Unit Weight: 20.5 kN/m³ Phi': 26 °
 Name: Lower Silty Clay Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 20.5 kN/m³
 Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³



Legend:
 (-) Sign on Contourline Label = Settlement
 No Sign on Contourline Label = Heave



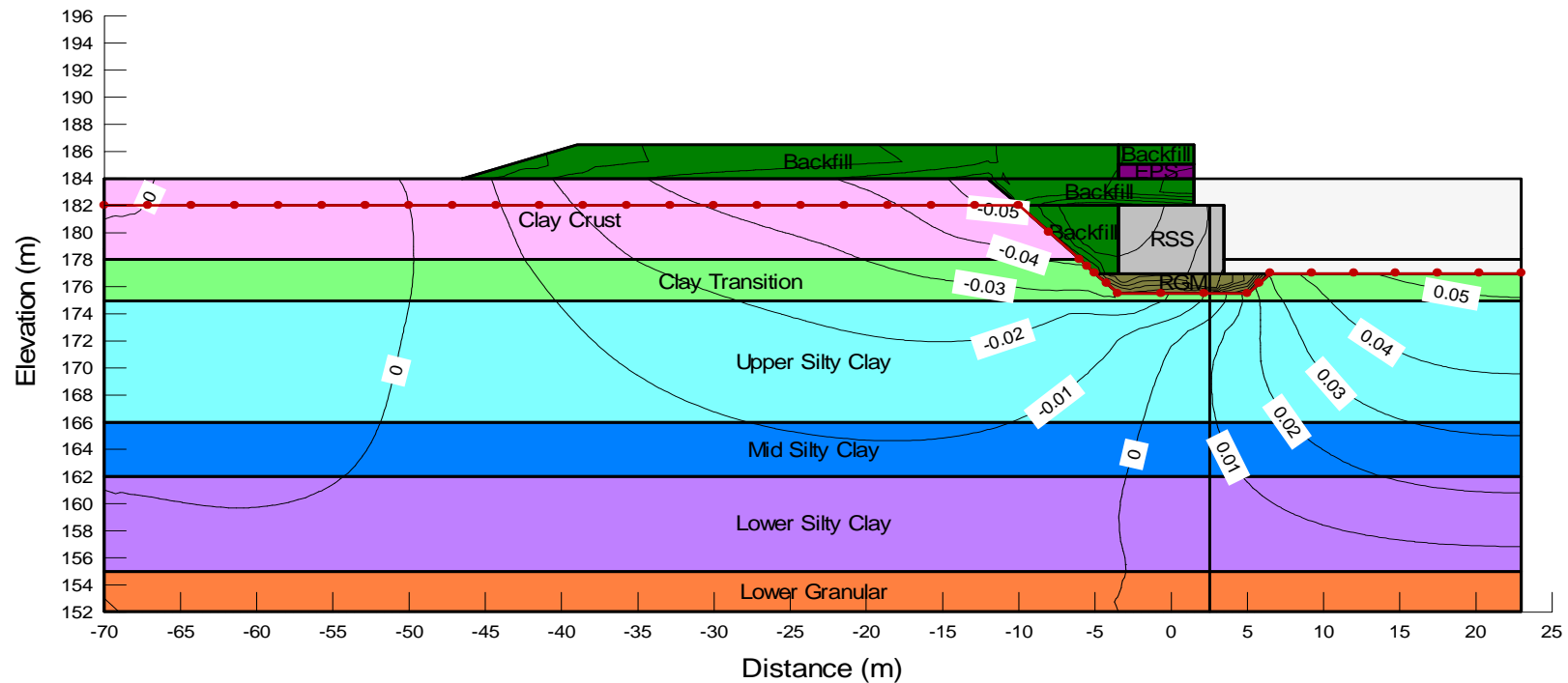
PROJECT: WINDSOR ESSEX PARKWAY			
TITLE: STRESS-DEFORMATION MODEL OF TUNNEL T-9 CUMULATIVE SETTLEMENT/HEAVE CONTOURS AT END OF EXCAVATION (m)			
DATE: Jan 2011	JOB NO.:	CAD FILE:	FIGURE NO.: G.1
			REV.

Tunnel T-9-RSS Wall-Central Segment-Deformation.gsz

WEP SW8801.1002.101

1/16/2012

Name: Clay Crust Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 22 kN/m³
 Name: Clay Transition Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 22 kN/m³
 Name: Upper Silty Clay O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Φ' : 25 °
 Name: Mid Silty Clay O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.086 Kappa: 0.0094 Initial Void Ratio: 0.65 Unit Weight: 20.5 kN/m³ Φ' : 26 °
 Name: Lower Silty Clay Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 20.5 kN/m³
 Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 22 kN/m³
 Name: RSS Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
 Name: RGM Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
 Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Φ' : 30 ° Unit Weight: 21 kN/m³
 Name: EPS Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m³ Poisson's Ratio: 0.2



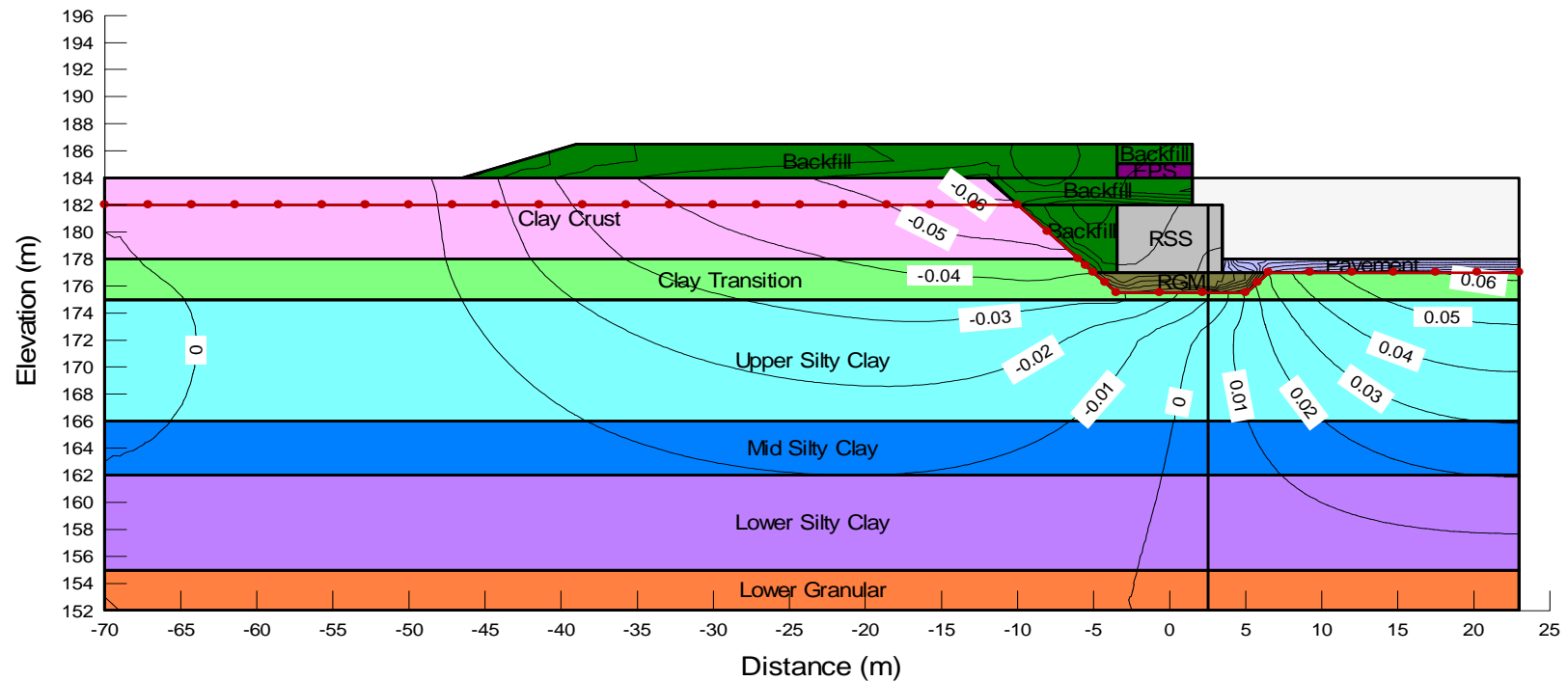
Legend:
 (-) Sign on Contourline Label = Settlement
 No Sign on Contourline Label = Heave

Tunnel T-9-RSS Wall-Central Segment-Deformation.gsz

WEP SW8801.1002.101

1/16/2012

Name: Clay Crust Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Clay Transition Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Upper Silty Clay O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Phi': 25 °
 Name: Mid Silty Clay O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.086 Kappa: 0.0094 Initial Void Ratio: 0.65 Unit Weight: 20.5 kN/m³ Phi': 26 °
 Name: Lower Silty Clay Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 20.5 kN/m³
 Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: RSS Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
 Name: RGM Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
 Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³
 Name: EPS Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m³ Poisson's Ratio: 0.2
 Name: Pavement Young's Modulus (E): 54000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35



Legend:
 (-) Sign on Contourline Label = Settlement
 No Sign on Contourline Label = Heave



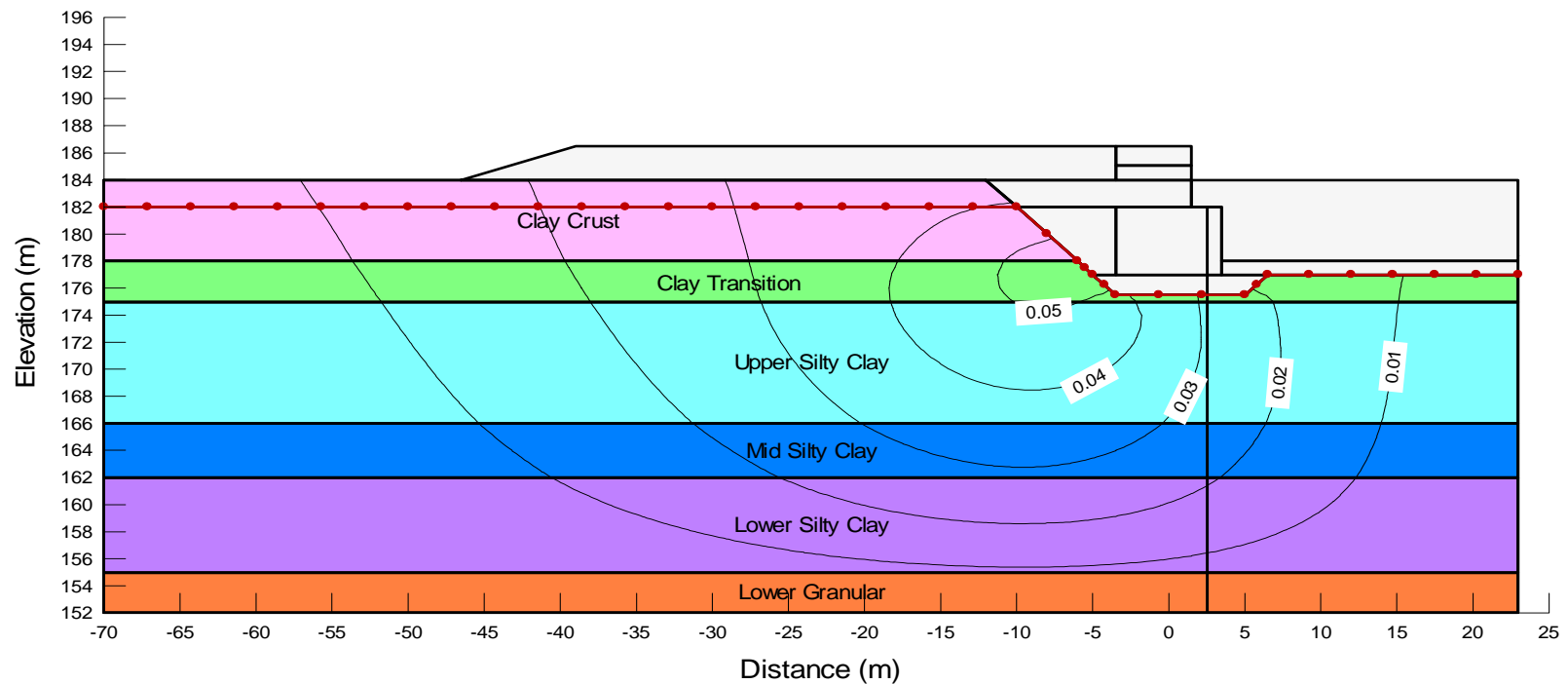
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TITLE: STRESS-DEFORMATION MODEL OF TUNNEL T-9 CUMULATIVE SETTLEMENT/HEAVE CONTOURS IN LONG-TERM (m)				
DATE: Jan 2011	JOB NO.:	CAD FILE:	FIGURE NO.: G.3	REV.

Tunnel T-9-RSS Wall-Central Segment-Deformation.gsz

WEP SW8801.1002.101

1/16/2012

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 Name: Upper Silty Clay O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Φ' : 25 °
 Name: Mid Silty Clay O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.086 Kappa: 0.0094 Initial Void Ratio: 0.65 Unit Weight: 20.5 kN/m³ Φ' : 26 °
 Name: Lower Silty Clay Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 20.5 kN/m³
 Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 22 kN/m³



Legend:
 (-) Sign on Contourline Label = Lateral Deformation opposite to Highway 401 Excavation
 No Sign on Contourline Label = Lateral Deformation towards Highway 401 Excavation



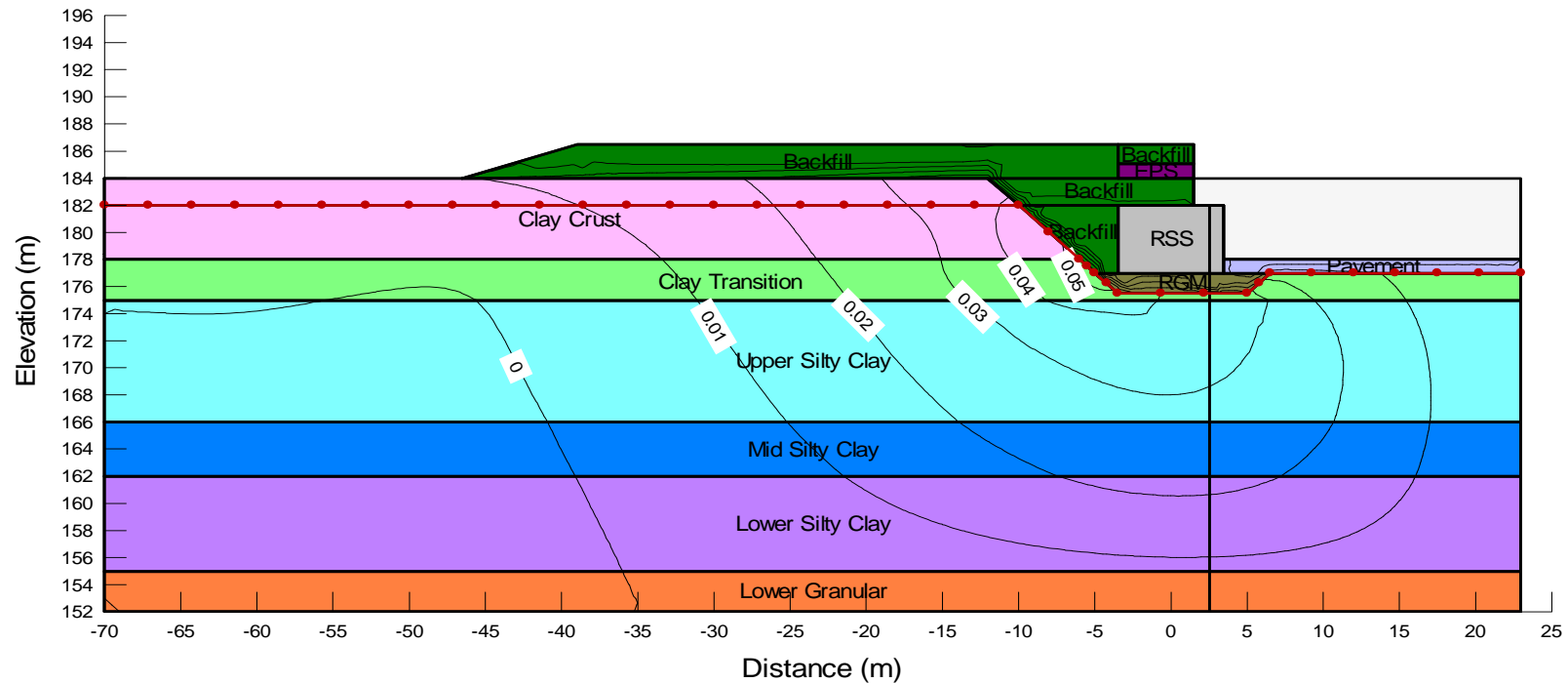
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DATE: Jan 2012	JOB NO.:	CAD FILE:	FIGURE NO.: G.4
			REV.

Tunnel T-9-RSS Wall-Central Segment-Deformation.gsz

WEP SW8801.1002.101

1/16/2012

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 Name: Mid Silty Clay O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.086 Kappa: 0.0094 Initial Void Ratio: 0.65 Unit Weight: 20.5 kN/m³ Φ' : 26 °
 Name: Lower Silty Clay Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 20.5 kN/m³
 Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 22 kN/m³
 Name: RSS Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
 Name: RGM Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
 Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Φ' : 30 ° Unit Weight: 21 kN/m³
 Name: EPS Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m³ Poisson's Ratio: 0.2
 Name: Pavement Young's Modulus (E): 54000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35



Legend:
 (-) Sign on Contourline Label = Lateral Deformation opposite to Highway 401 Excavation
 No Sign on Contourline Label = Lateral Deformation towards Highway 401 Excavation



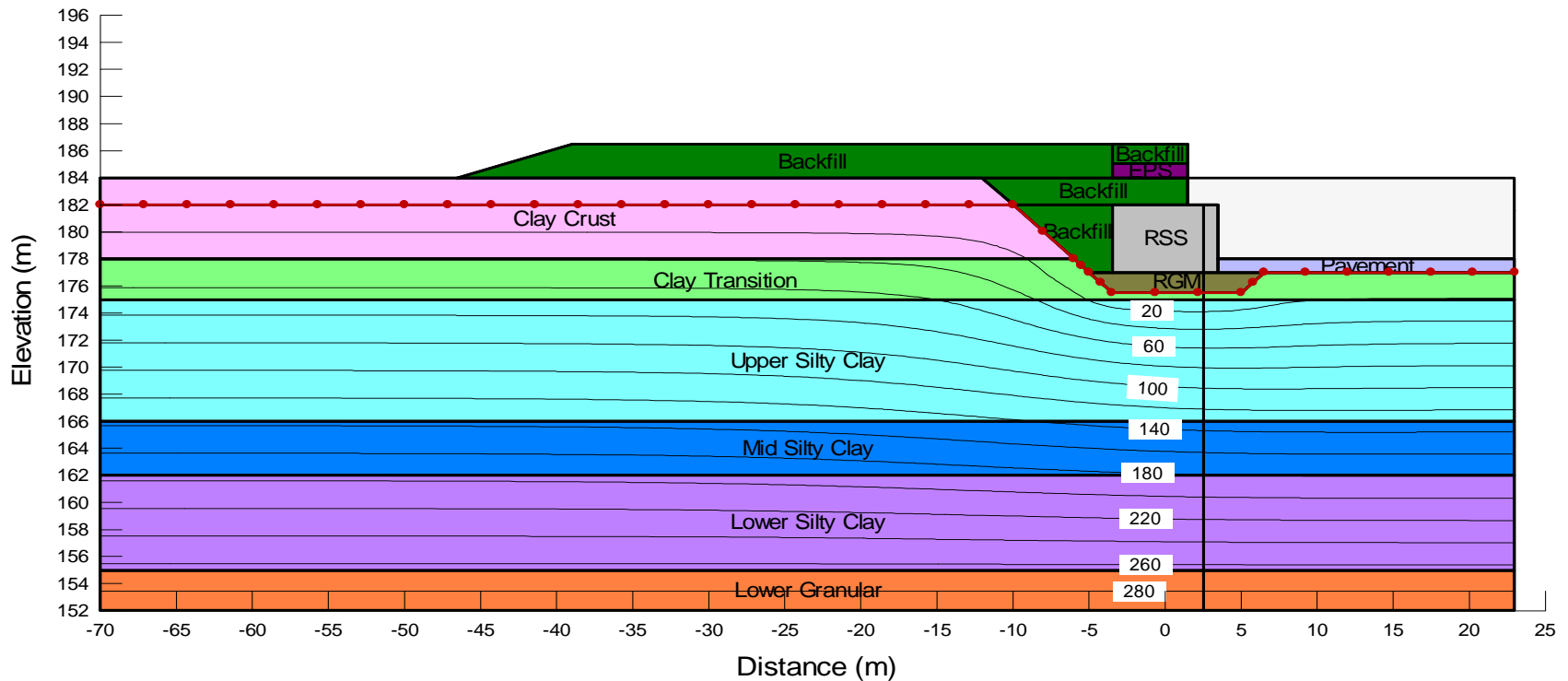
PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: STRESS-DEFORMATION MODEL OF TUNNEL T-9 CUMULATIVE LATERAL DEFORMATION CONTOURS IN LONG-TERM (m)				
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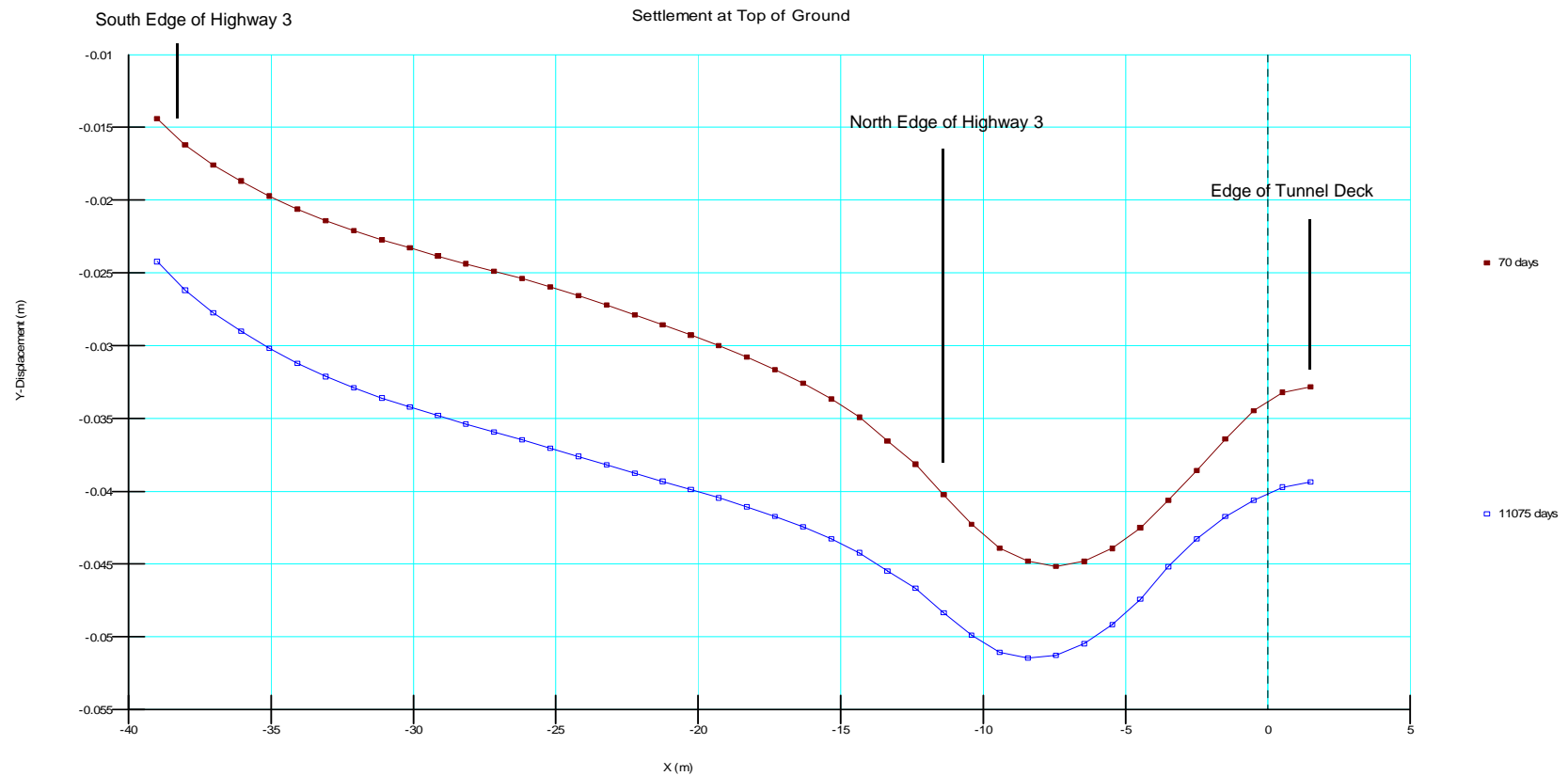
Tunnel T-9-RSS Wall-Central Segment-Deformation.gsz

WEP SW8801.1002.101

1/16/2012

Name: Clay Crust Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Clay Transition Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Upper Silty Clay O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Phi': 25 °
 Name: Mid Silty Clay O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.086 Kappa: 0.0094 Initial Void Ratio: 0.65 Unit Weight: 20.5 kN/m³ Phi': 26 °
 Name: Lower Silty Clay Effective Young's Modulus (E'): 31000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 20.5 kN/m³
 Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
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 Name: RGM Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
 Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³
 Name: EPS Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m³ Poisson's Ratio: 0.2
 Name: Pavement Young's Modulus (E): 54000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35





Legend:

70 days = End of Construction

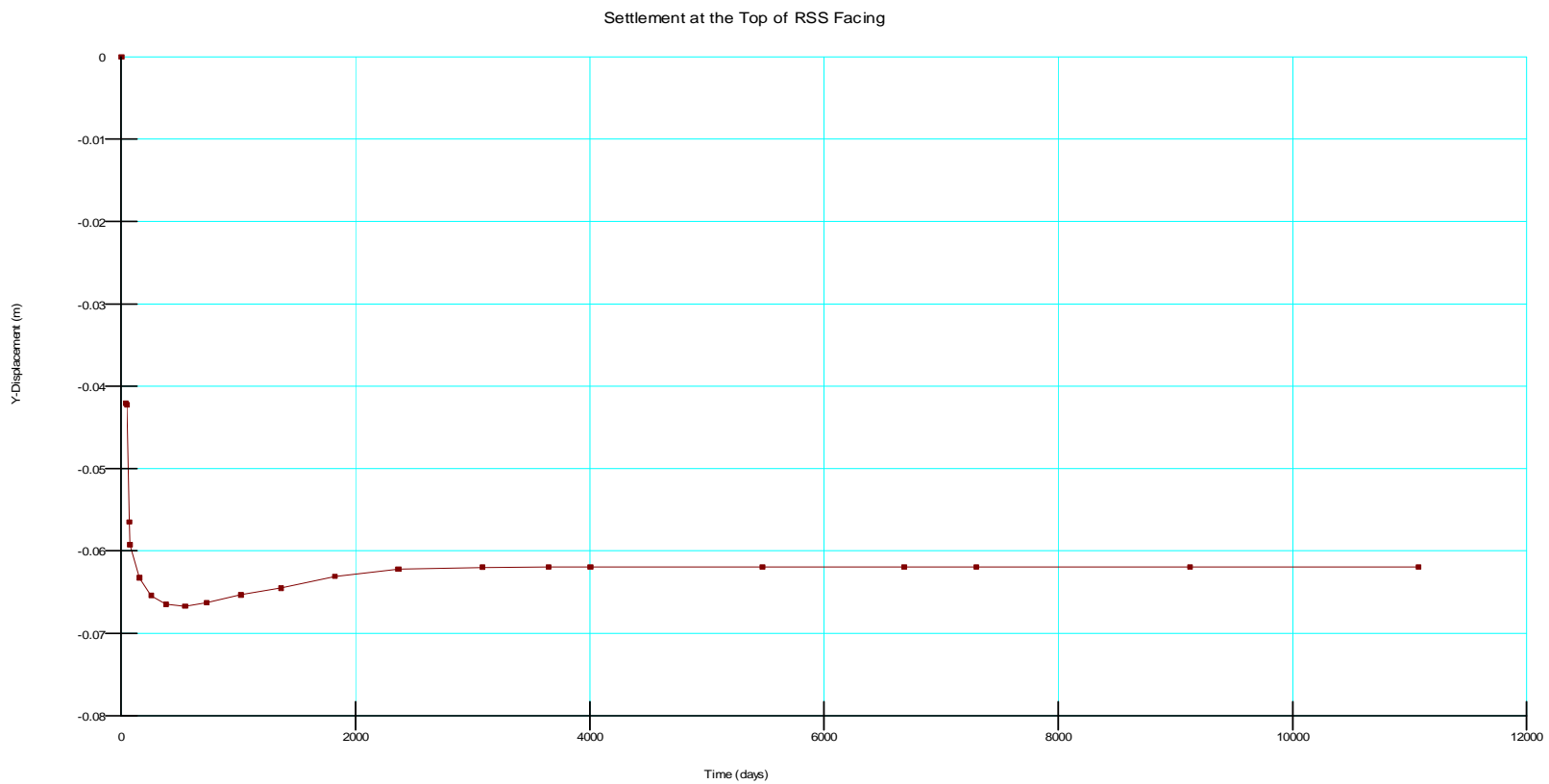
11075 days = Long-term Condition

(-) Displacement = Settlement

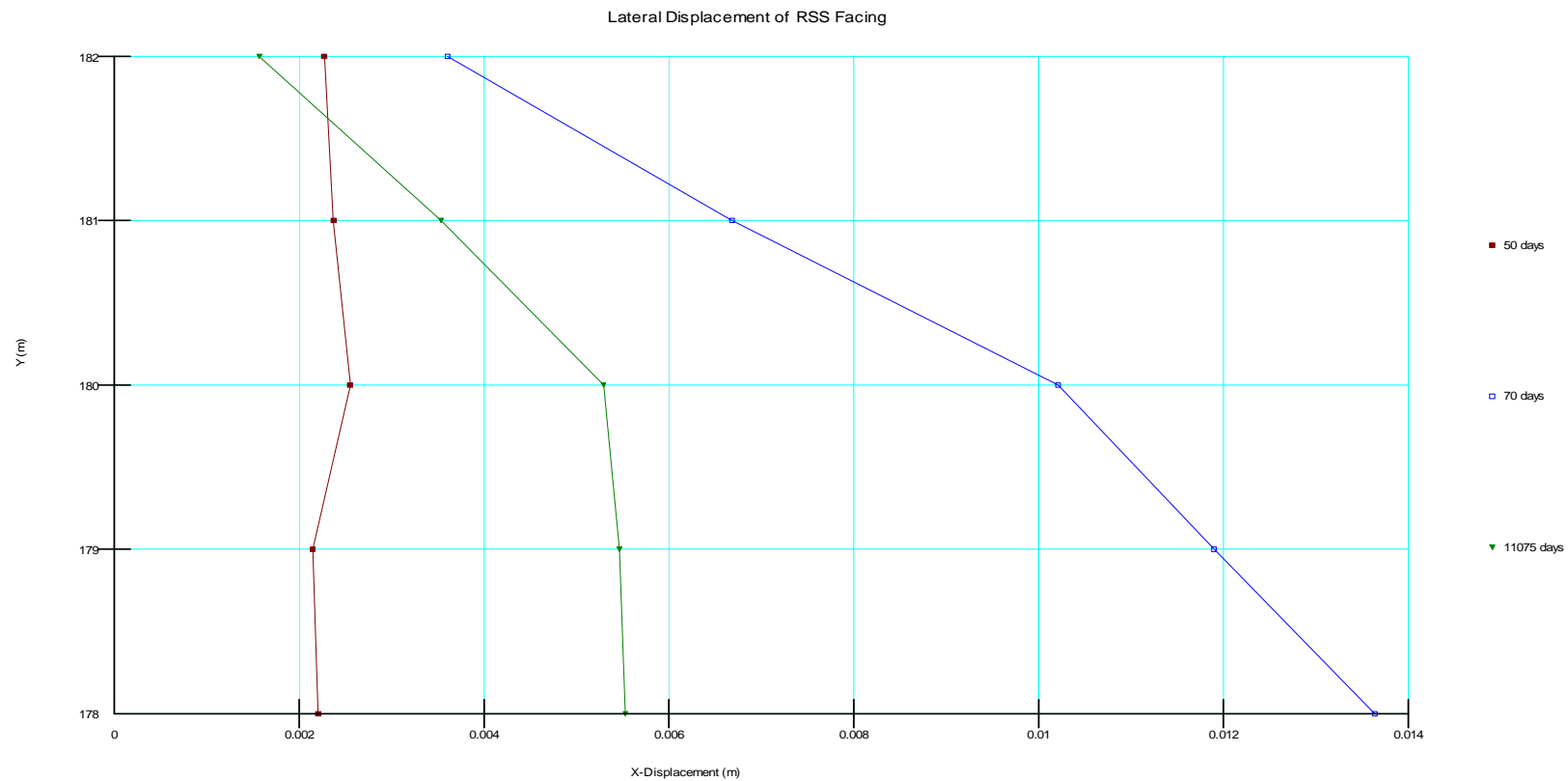
(+) Displacement = Heave

Y-Displacement = Vertical Displacement

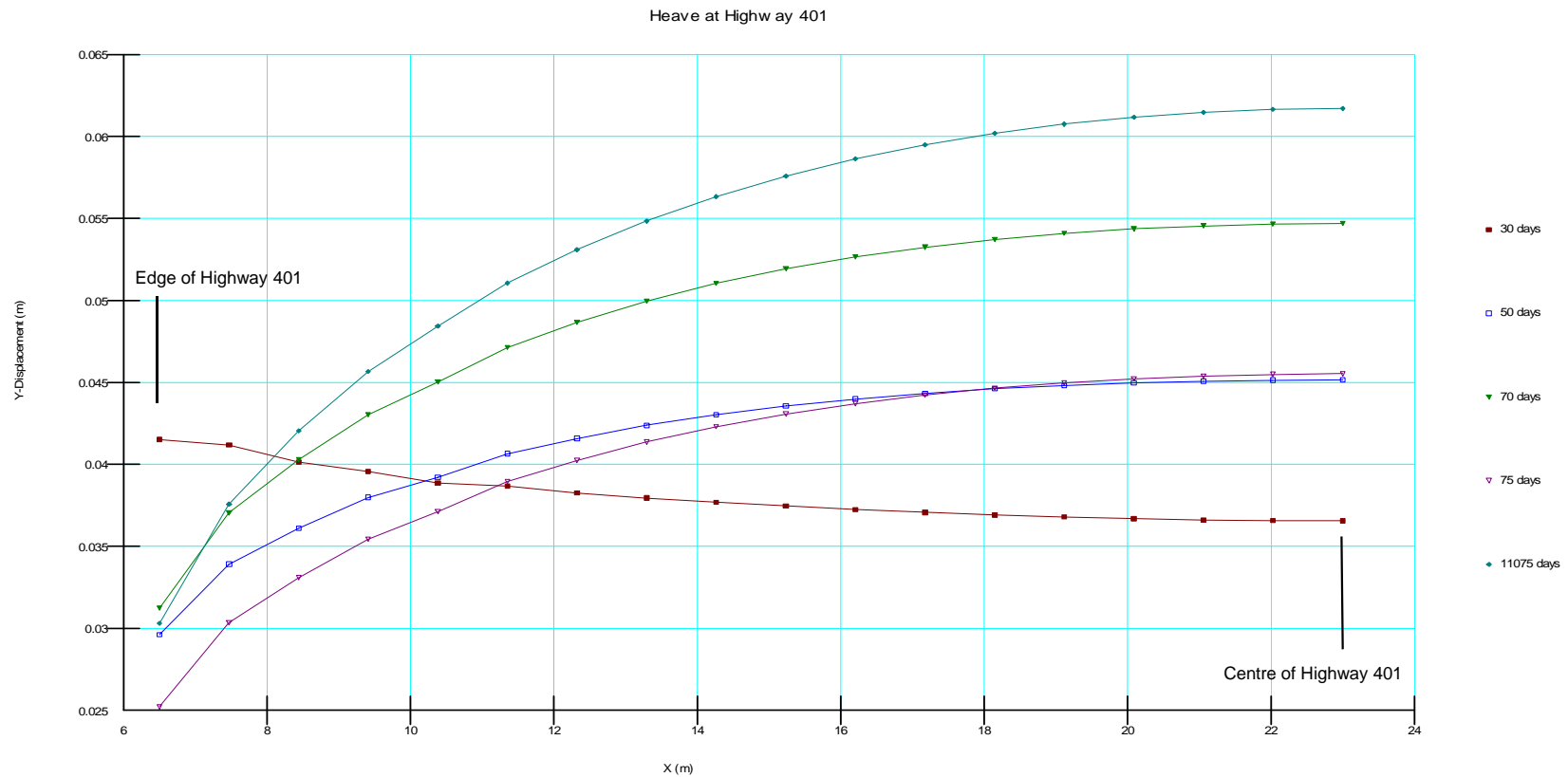
X = Horizontal Distance



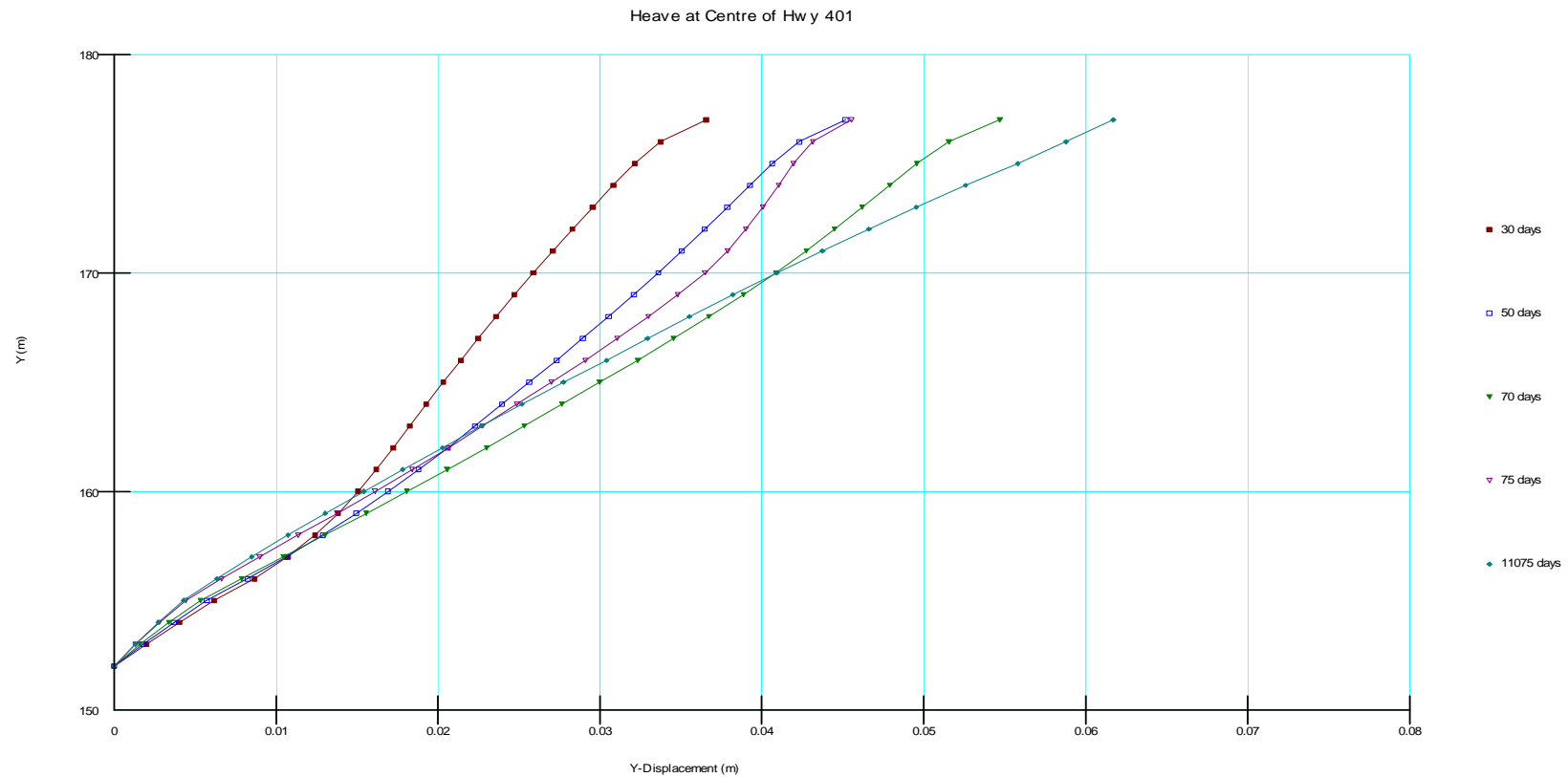
Legend:
 50 days = End of RSS Construction
 70 days = End of Construction
 11075 days = Long-term Condition
 Y-Displacement = Vertical Displacement



Legend:
 50 days = End of RSS Construction
 70 days = End of Construction
 11075 days = Long-term Condition
 (-) Sign = Lateral Deformation opposite to Highway 401 Excavation
 (+) Sign = Lateral Deformation towards Highway 401 Excavation
 Y = Elevation
 X-Displacement = Horizontal Displacement



Legend:
 30 days = End of Excavation for Tunnel
 50 days = End of RSS Construction
 70 days = End of Tunnel Abutment Construction
 75 days = End of Highway 401 Pavement Construction
 11075 days = Long-term Condition
 Y-Displacement = Vertical Displacement
 X = Horizontal Distance



Legend:

30 days = End of Excavation for Tunnel

50 days = End of RSS Construction

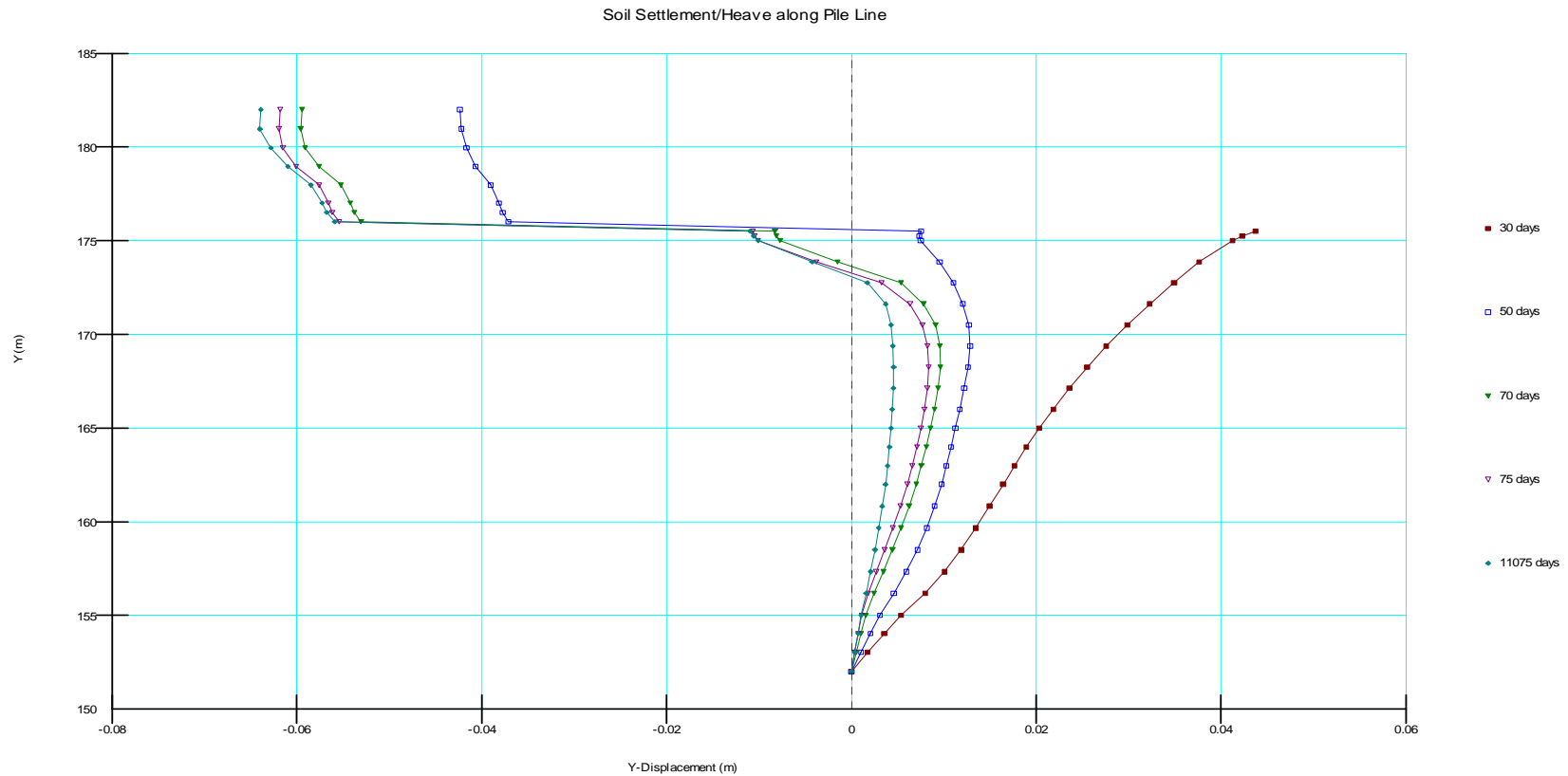
70 days = End of Tunnel Abutment Construction

75 days = End of Highway 401 Pavement Construction

11075 days = Long-term Condition

Y = Elevation

Y-Displacement = Vertical Displacement



Legend:

30 days = End of Excavation for Tunnel

50 days = End of RSS Construction

70 days = End of Tunnel Abutment Construction

75 days = End of Highway 401 Pavement Construction

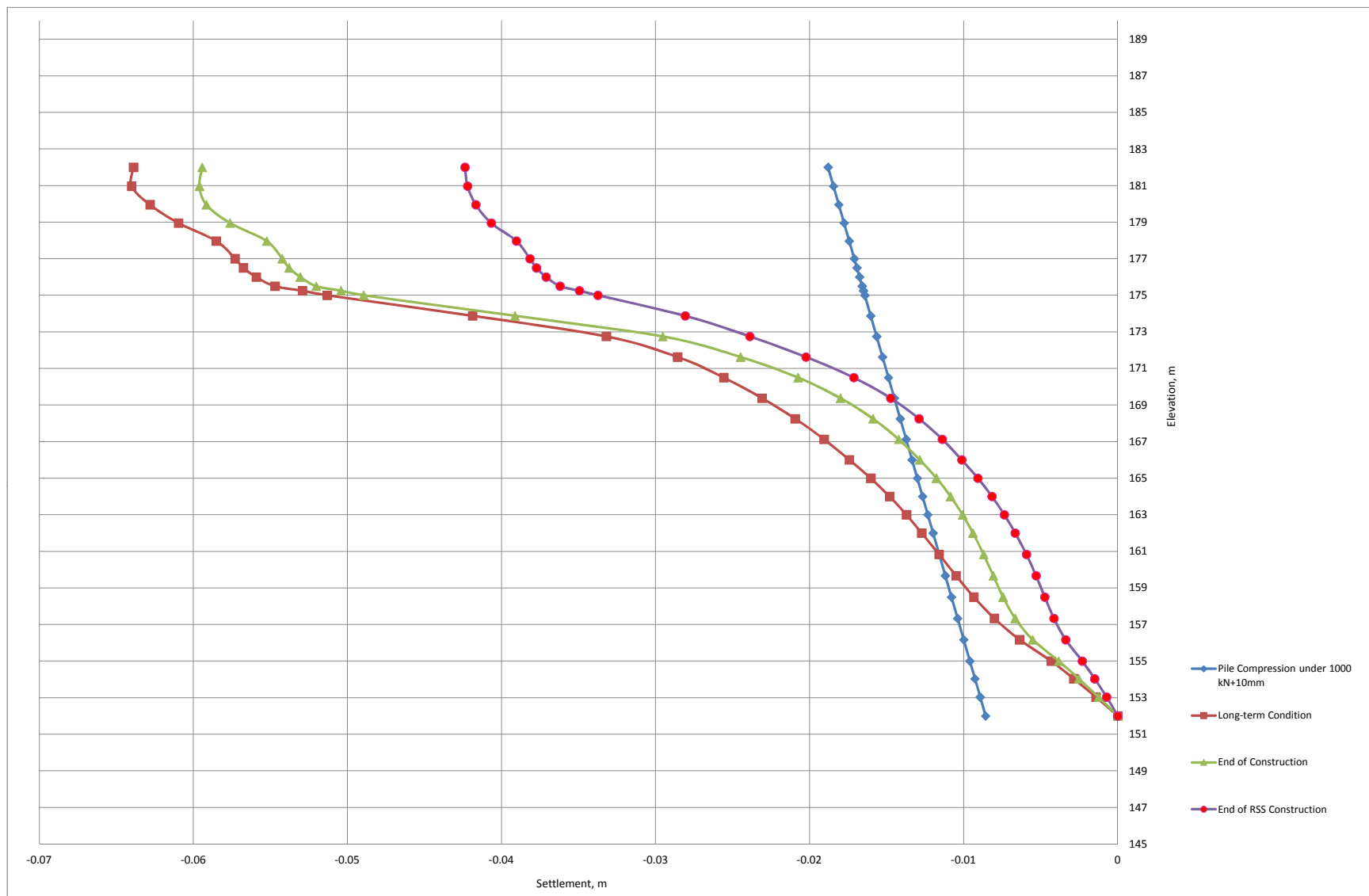
11075 days = Long-term Condition

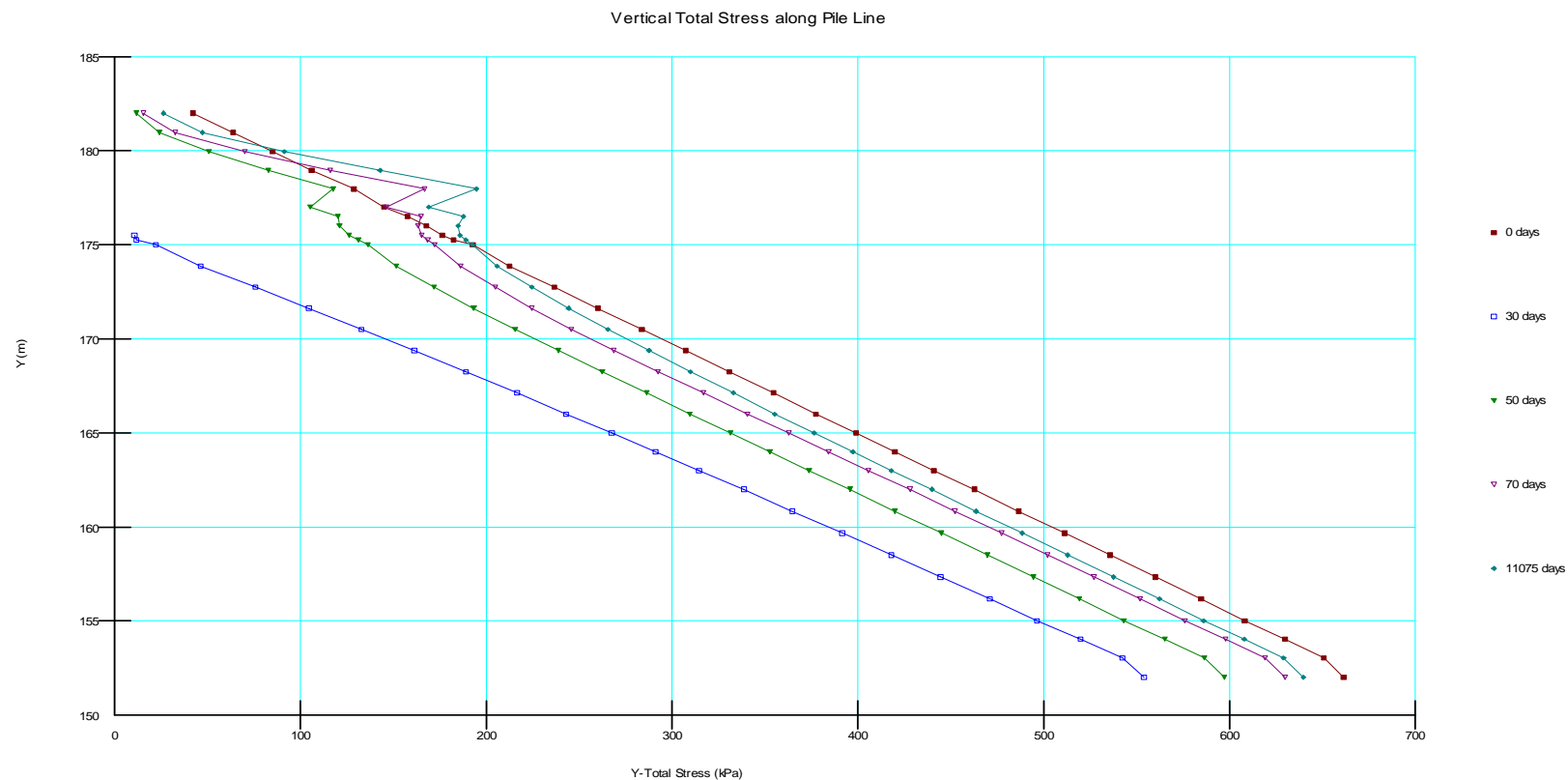
(-) Displacement = Settlement

(+) Displacement = Heave

Y = Elevation

Y-Displacement = Vertical Displacement





Legend:

0 days = In-situ Condition

30 days = End of Excavation for Tunnel

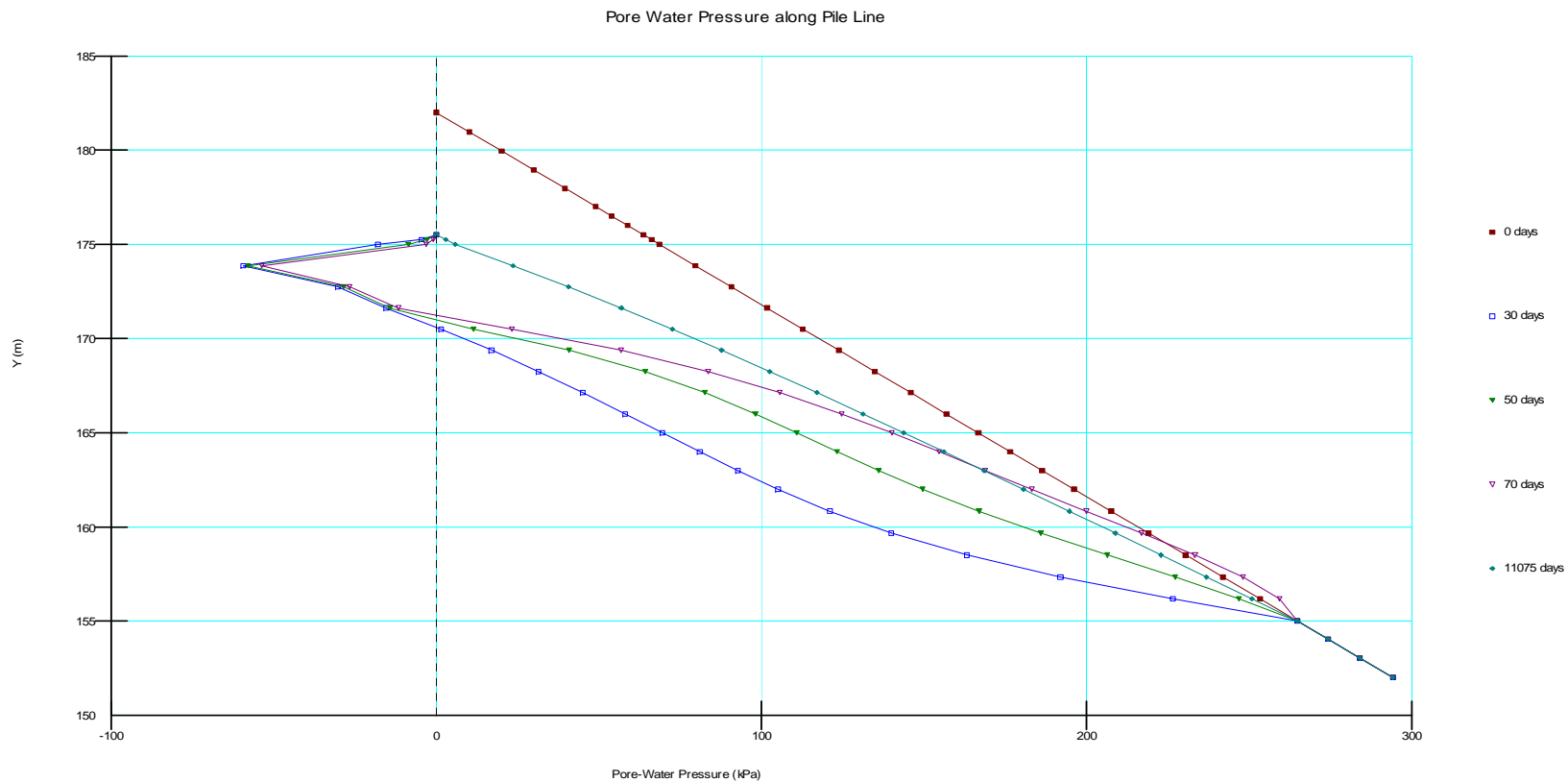
50 days = End of RSS Construction

70 days = End of Construction

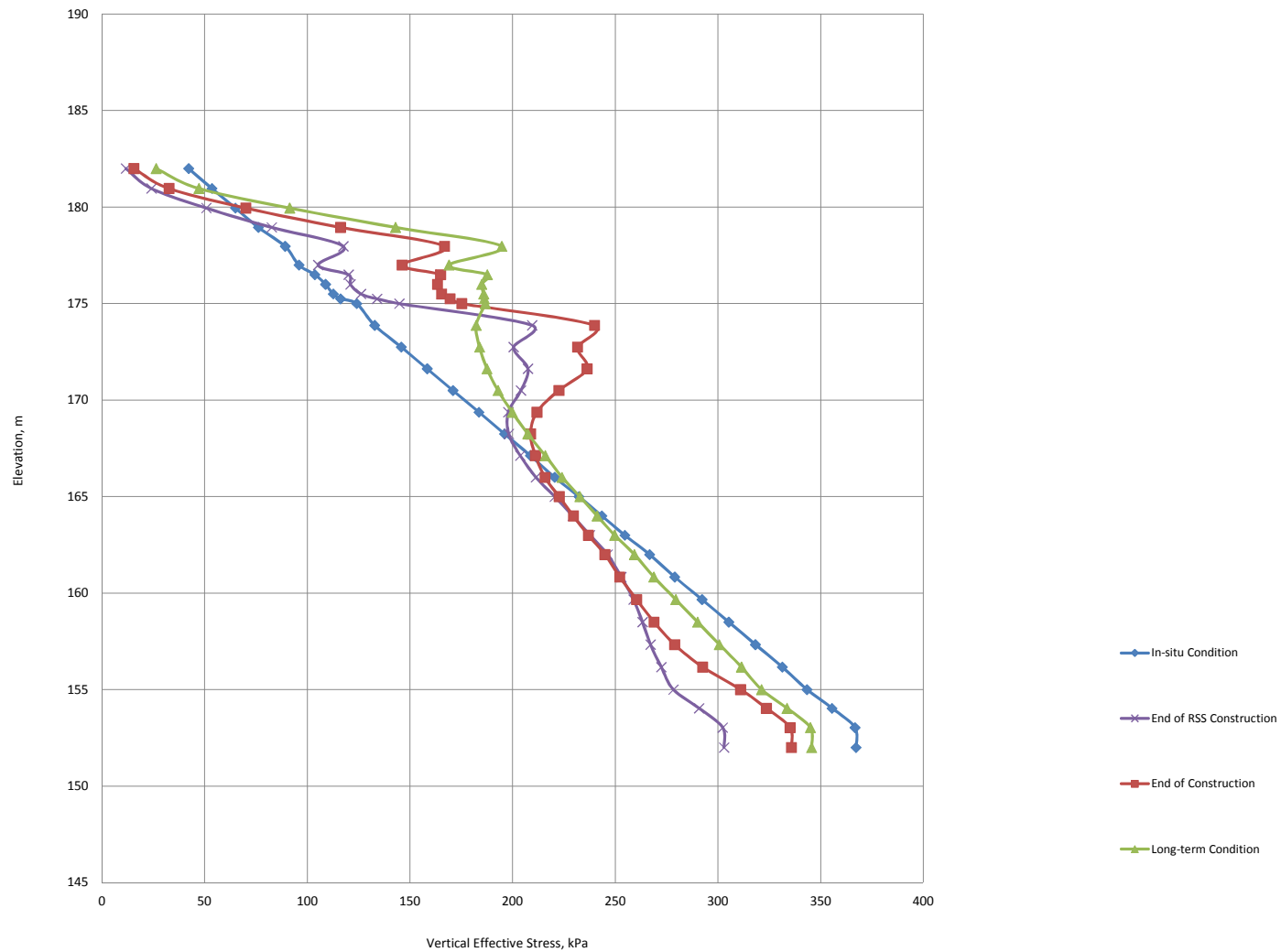
11075 days = Long-term Condition

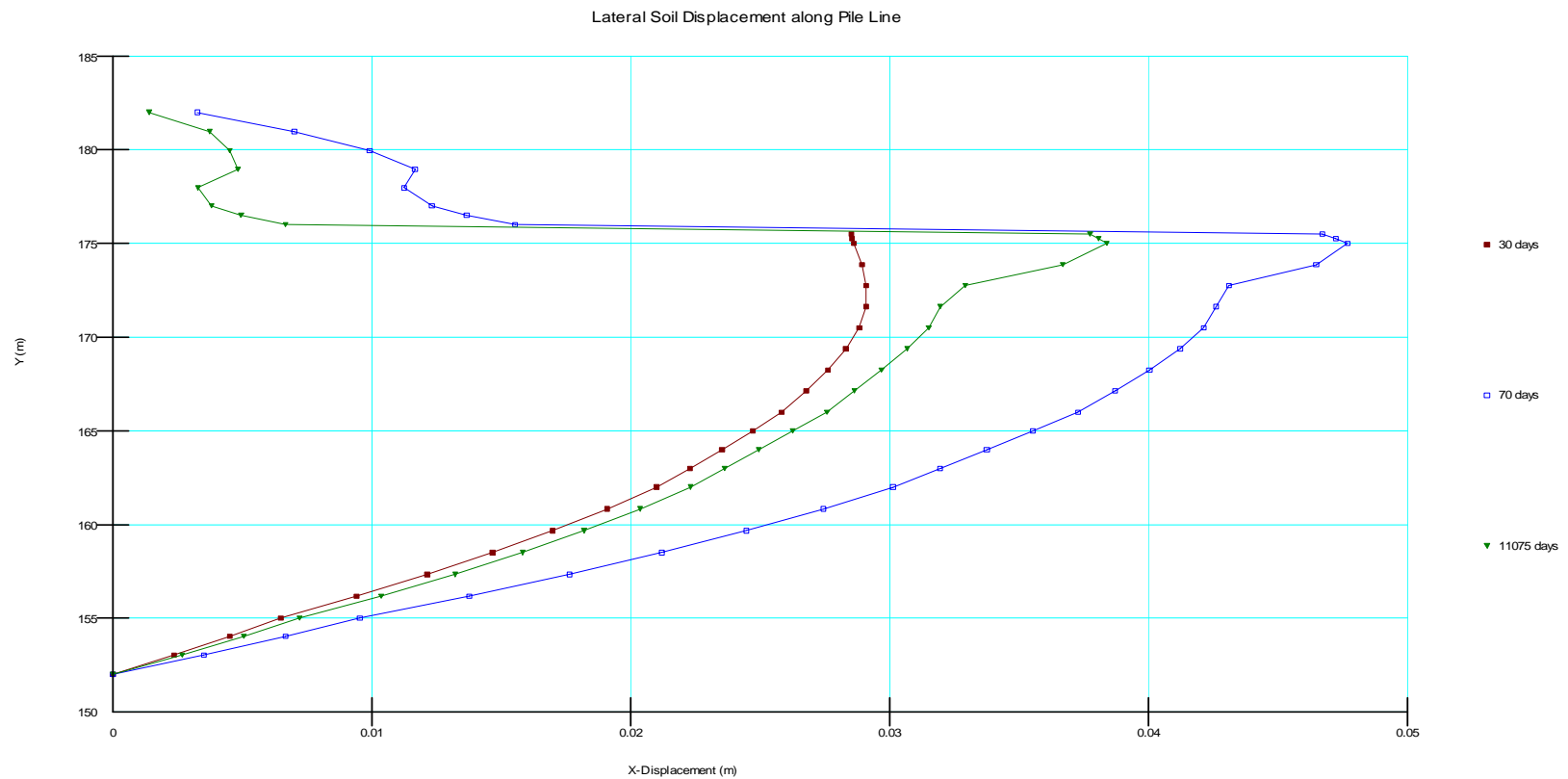
Y = Elevation

Y-Total Stress = Vertical Total Stress

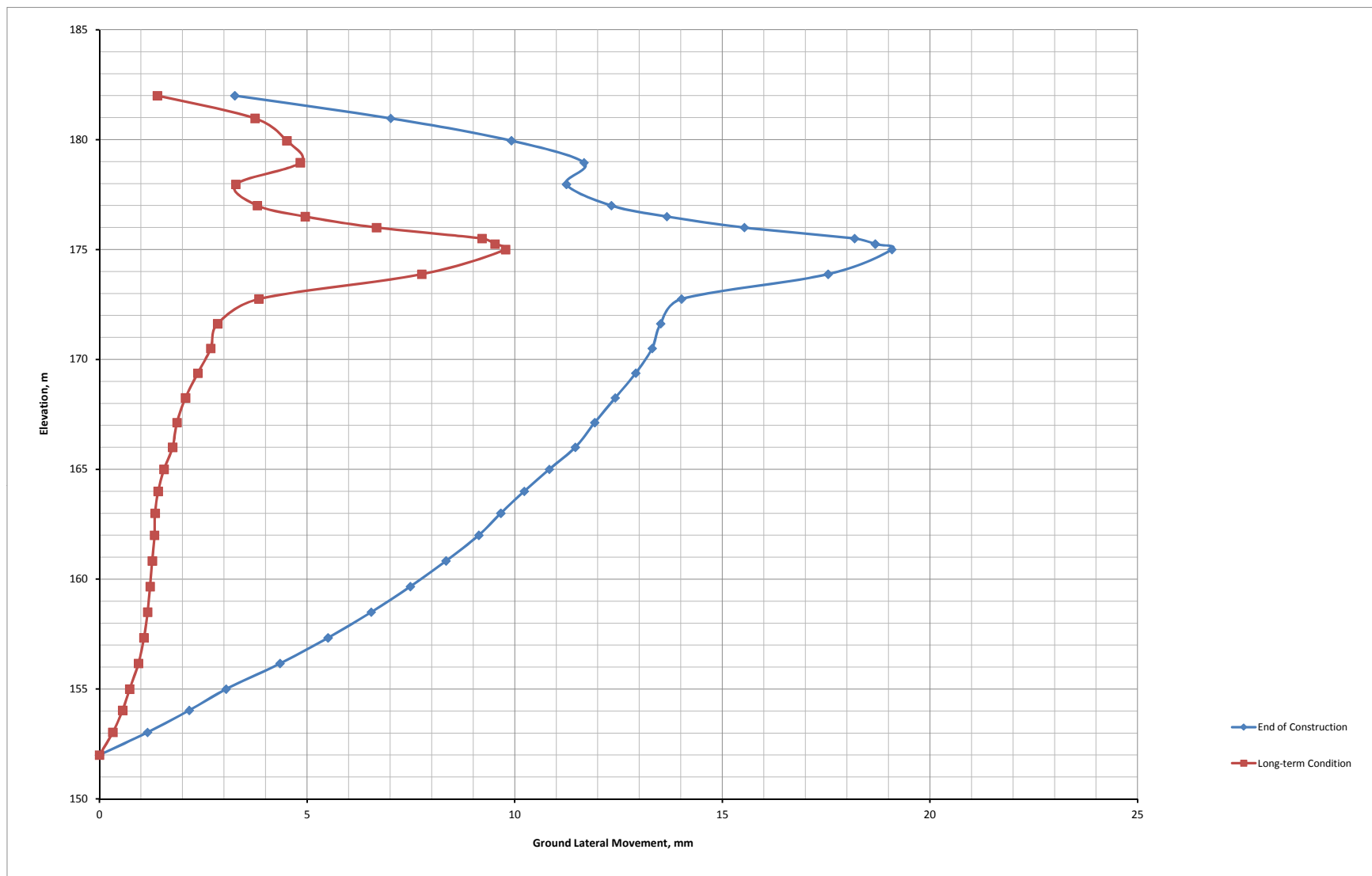


Legend:
 0 days = In-situ Condition
 30 days = End of Excavation for Tunnel
 50 days = End of RSS Construction
 70 days = End of Construction
 11075 days = Long-term Condition
 Y = Elevation





Legend:
 30 days = End of Excavation for Tunnel
 70 days = End of Construction
 11075 days = Long-term Condition
 Y = Elevation
 X-Displacement = Horizontal Displacement

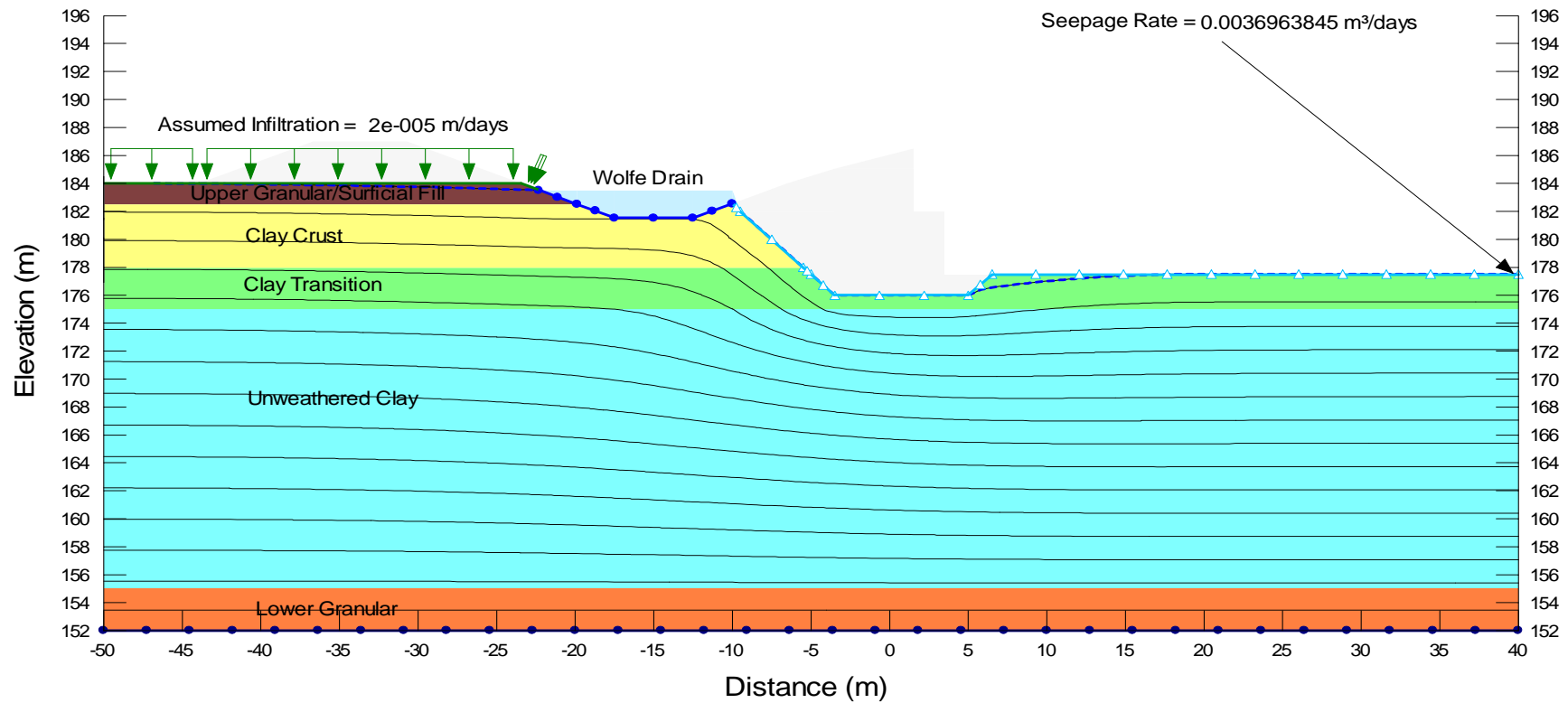


Appendix H Seepage Analyses Results

Tunnel T-9-North RSS Wall-Sta 12+130L-Seep.gsz

WEP SW8801.1002.101

Name: Clay Crust K-Sat: 0.00058752 m/days K-Ratio: 2
 Name: Clay Transition K-Sat: 0.00033696 m/days K-Ratio: 1
 Name: Unweathered Clay K-Sat: 9.504e-005 m/days K-Ratio: 0.5
 Name: Upper Granular/Surficial Fill K-Sat: 0.005 m/days K-Ratio: 2
 Name: Lower Granular K-Sat: 0.01 m/days K-Ratio: 1

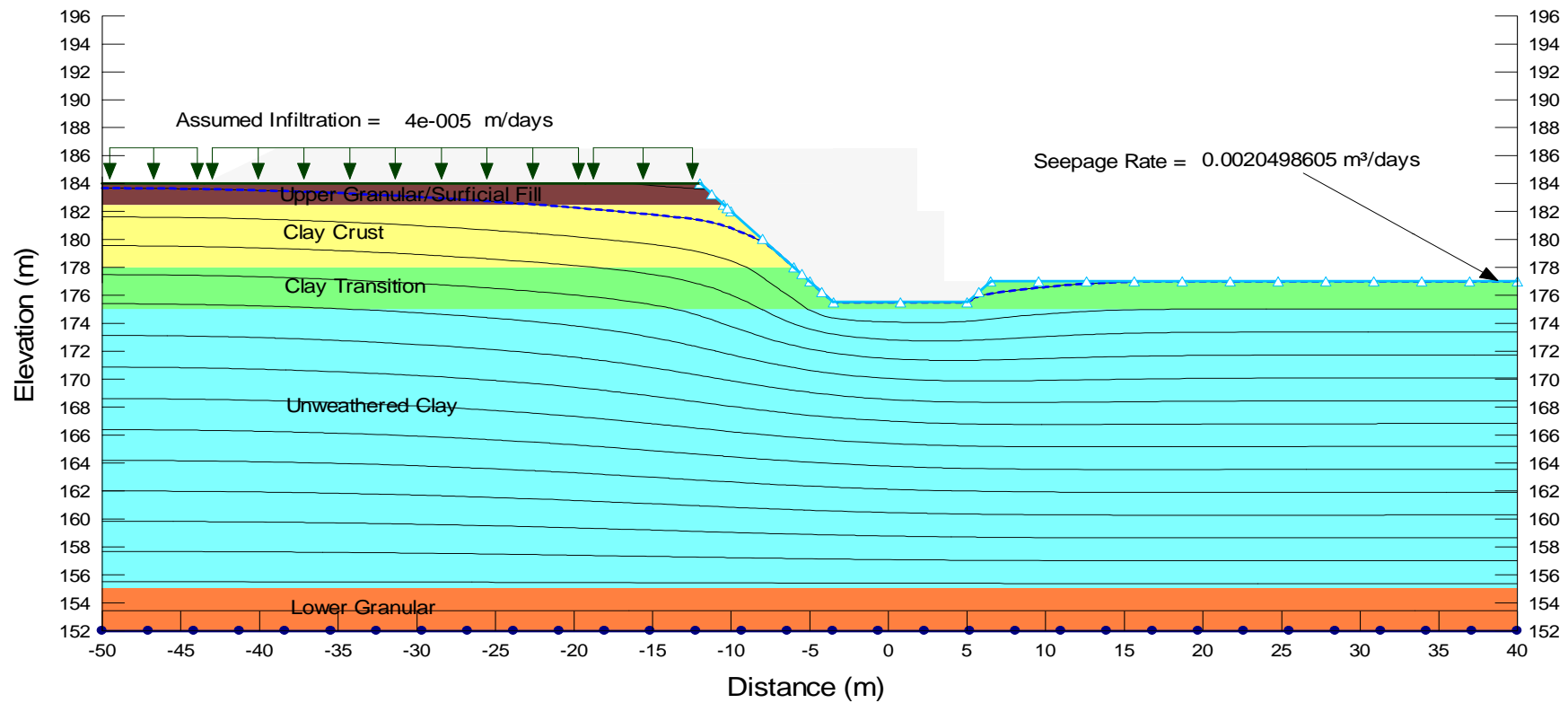


Tunnel T-9-South RSS Wall-Sta 12+215L-Seep.gsz

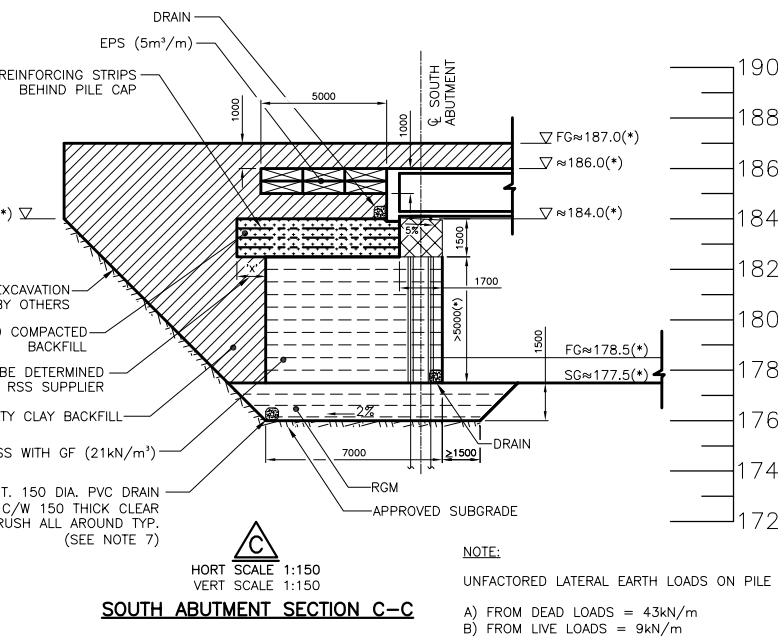
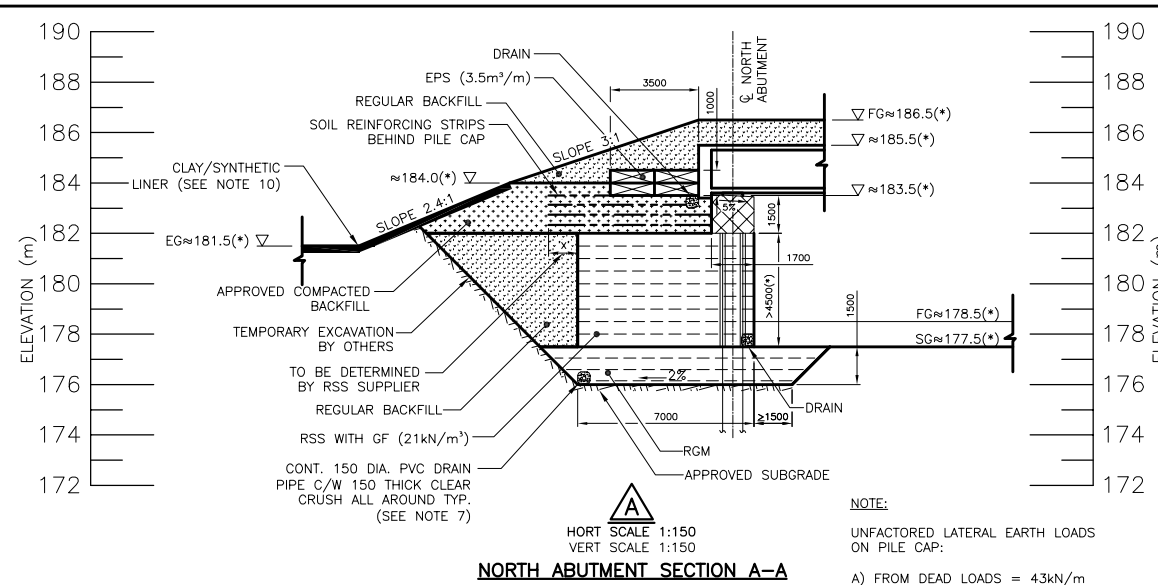
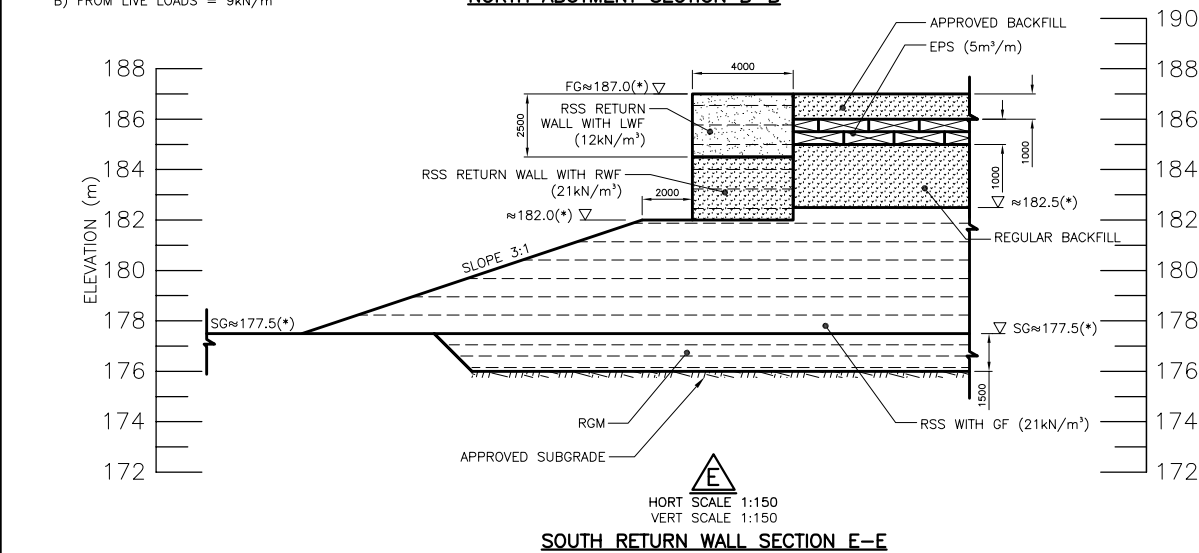
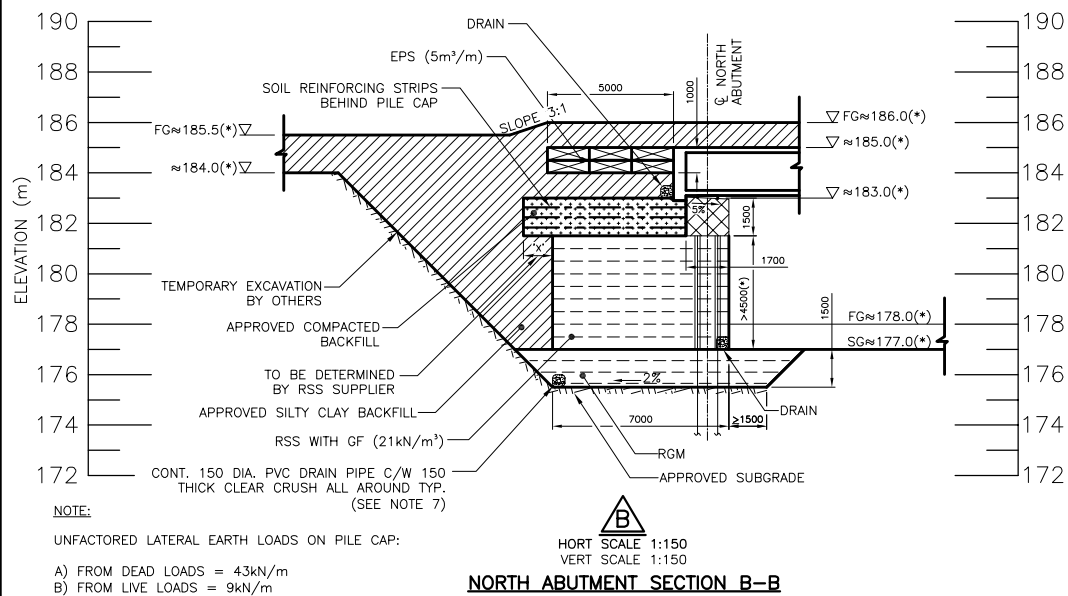
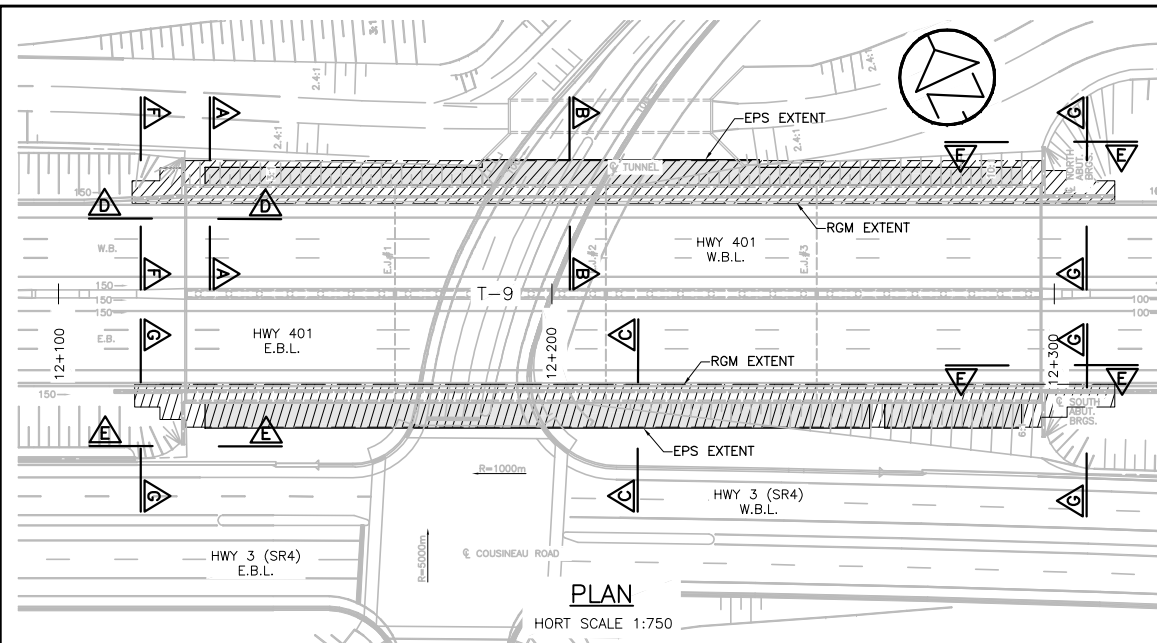
1/10/2012

WEP SW8801.1002.101

Name: Clay Crust K-Sat: 0.00058752 m/days K-Ratio: 2
 Name: Clay Transition K-Sat: 0.00033696 m/days K-Ratio: 1
 Name: Unweathered Clay K-Sat: 9.504e-005 m/days K-Ratio: 0.5
 Name: Upper Granular/Surficial Fill K-Sat: 0.005 m/days K-Ratio: 2
 Name: Lower Granular K-Sat: 0.01 m/days K-Ratio: 1

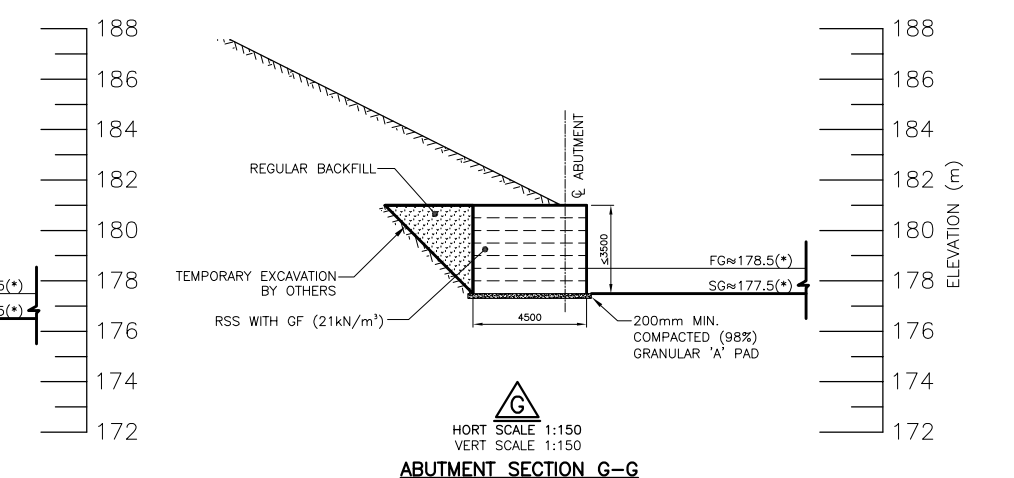


Appendix I Conceptual Drawings



NOTE:
UNFACTORED LATERAL EARTH LOADS ON PILE CAP:
A) FROM DEAD LOADS = 43kN/m
B) FROM LIVE LOADS = 9kN/m

NOTE:
UNFACTORED LATERAL EARTH LOADS ON PILE CAP:
A) FROM DEAD LOADS = 43kN/m
B) FROM LIVE LOADS = 9kN/m



NOTES:

- THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
- THIS DRAWING ILLUSTRATES THE GENERAL ARRANGEMENTS AT SELECTED REPRESENTATIVE LOCATIONS OF THE NORTH AND SOUTH ABUTMENTS OF TUNNEL T-9 BASED ON GEOTECHNICAL DESIGN ANALYSES.
- THE ILLUSTRATED RSS WALL AND RGM WIDTH REPRESENTS THE MINIMUM WIDTH BASED ON GEOTECHNICAL REQUIREMENTS. THE DESIGN OF THE RSS WALL AND RGM IS TO BE DEVELOPED BY OTHERS IN CONJUNCTION WITH RELEVANT INFORMATION FROM THE GEOTECHNICAL REPORT.
- TUNNEL ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGNS WERE OBTAINED FROM STRUCTURAL DRAWINGS AVAILABLE IN MAY 2012. ABUTMENT ELEVATIONS VARY ALONG THE TUNNEL.
- CLAY SUBGRADE IS SUSCEPTIBLE TO DISTURBANCE AND LOSS OF STRENGTH DUE TO WATER INFLOW/PONDING, CONSTRUCTION TRAFFIC AND THE LIKE. SUITABLE EXCAVATION METHODS, DEWATERING AND SUBGRADE PROTECTION MUST BE EXERCISED.
- CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS. EXCAVATED CLAY SURFACES ARE SUSCEPTIBLE TO DETRIORATION AND EXPERIENCE DEFORMATIONS AND INSTABILITY; THEY ARE TO BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED AND TREATED AS REQUIRED.
- HIGHEST SUBDRAIN ELEVATION NOT LESS THAN 500mm BELOW TOP OF RGM.
- RSS TOE SLOPES/BERMS AND BASE/SUBBASE GRANULAR MATERIALS MUST BE INSTALLED OVER HWY 401 SUBGRADE TO WITHIN AT LEAST 10m AND 20m DISTANCES FROM THE FACE OF THE NORTH ABUTMENT AND SOUTH ABUTMENT, RESPECTIVELY, BEFORE PLACING BACKFILL OVER RSS ABUTMENT ABOVE THE ELEVATION OF THE DECK SEAT LEVEL.
- SEE ACCOMPANYING DRAWINGS FOR APPLICABLE BACKFILL, LWF, AND EPS SPECIFICATIONS.
- RECOMMENDATIONS FOR LINERS ARE SUMMARIZED IN TABLE 6-1 IN THE ACCOMPANYING REPORT.

LEGEND:

RSS - REINFORCED SOIL STRUCTURE	EPS - EXPANDED POLYSTYRENE
GF - GRANULAR FILL	LWF - LIGHT WEIGHT FILL
RGM - REINFORCED GRANULAR MAT (LONG-TERM ALLOWABLE LOAD CAPACITY OF GEOGRID SHALL BE MINIMUM 22kN/m)	RWF - REGULAR WEIGHT GRANULAR FILL
	(*) - ELEVATION/DIMENSION VARIES