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The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report – Tunnel T-8

(Sta. 11+600L to 11+720L)

Geocres No. 40J3-16

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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 geotechnical design of Tunnel T- 8 (Geraedts Tunnel), located in the LaSalle sector of the Windsor-Essex Parkway (WEP) project. The proposed 2 span Tunnel T-8 will carry parkland landscape and local traffic along Geraedts Drive over Highway 401 between Sta. 11+600L and Sta. 11+720L. A trail and associated pedestrian tunnel is located north and parallel to the tunnel. Tunnel T-8 comprises semi-integral abutments and a centre pier founded on deep end bearing piles. As for all other tunnels at this project, Tunnel T-8 will be a cut-and-cover construction.

The report includes the results of the additional geotechnical investigation carried out to support the design and addresses review comments from peer reviews and MTO. This report is issued for construction (IFC). 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]. The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines as well as the Parkway Infrastructure Constructors (PIC).

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-8 complies with the requirements of the execution version of the Project Agreement (PA) Schedule 15-2 Part 2, Article 5.

[1] References are listed in Section 9.

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 (ref. 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 successive 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, in turn, 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 Zv = 0 and Acceleration seismic zone Za = 0. Zonal Velocity ratio V and Zonal Acceleration ratio A are both 0.

In accordance with the CHBDC and the results of a series of cross-hole tests completed during the background investigation program (ref. R-21), the soil profile at the project site generally meets the description for Soil Profile Type III (soft clay and silts greater than 12 m in depth). These cross-hole tests were completed during the background investigation program at locations distributed along the project alignment between Howard Road (east end) and Matchette Road (west end). The measured velocities of the shear waves were consistently over 200 m/s, with the bulk of results ranging between 200 and 300 m/s.

2.3 Existing Site Conditions and Proposed Tunnel Layout

Tunnel T-8 site is situated near the center of the Windsor segment of the Parkway. The tunnel structure will be constructed under WEP Phase I development and will be used to carry parkland and local traffic (Geraedts Drive) over Highway 401. Highway 3 in the vicinity of Tunnel T-8 will be situated on the south side of the proposed depressed Highway 401. The Cahill Drain (and Culvert CV-4) and Pedestrian Tunnel TB-6 are located on the north side of Highway 401. Highway 401 at this location will be constructed within permanent cut.

The topography of the lands immediately adjacent Tunnel T-8 at Highway 401 is generally flat with elevation ranging from approximately 182[2] to 183 in the area of both north and south abutments. Adjacent land use is typically commercial to the north and undeveloped to the south. The proposed top of the tunnel deck elevations range from 183.610 to 184.674 and the finished grades along the tunnel walls will be raised to about elevation 184.497 to 185.674.

2.4 Frost Depth

In accordance with MTO–SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-33) and OPSD 3090.101, the frost depth below the ground surface in Windsor area is estimated to 1.0 m[3]. 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 and R-23) to develop the conceptual design and serve as background information for development of the WEP proposal designs. Additional geotechnical investigation was completed 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-8 comprised a total of 5 boreholes, 1 Nilcon vane test, 3 CPT 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-8 Site

Reference	Boreholes	Nilcon Vane Tests	CPT	DMT
This Investigation	BH T8-1	NIL T8-1	CPT T8-1	DMT T8-1
	BH TB6-1		CPT 43-RW	
	BH CV4-1		CPT 44-RW	
	BH HGMW-3			
	BH PS5-1			
Previous Studies	BH-7	NIL-7	CPT-7	
	BH-118		BH/CPT-315	
	BH-314		BH/CPT-316	

Drawing 285380-04-090-WIP1-2802 shows the locations of the test holes and an interpreted soil stratigraphic profile along the WEP centreline for the general area from Sta. 11+500L to Sta. 12+300L. The test hole locations and stratigraphic sections at the tunnel location are illustrated on Drawings 285380-04-090-WIP1-2803 and 285380-04-091-WIP1-2804.

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 a 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 (OD=88.9 mm) casing.

Soil sampling was carried out using 50 mm diameter split spoon samplers or thin-walled Shelby tubes (70 mm diameter by 600 mm long). 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 by a field technologist, placed in airtight containers and transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing[4]. Rock coring of the bedrock was completed using a 1.5 m long NQ (OD=75.7 mm)) or HQ (OD=96.0 mm) sized core barrels.

Standard Penetration Tests (SPT, ASTM D1586[5]) 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 typically adjacent the boreholes. Table 3-2 summarizes the depths of overburden penetration and rock coring as well as the list of instruments at each borehole location 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 testes were carried out on rock core samples selected from across the WEP length.

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

Nilcon vane blade was pushed into the ground from the bottom of shallow pre-augered holes through surficial soils using the hydraulic ram of the drill rig. The Nilcon vane tests were conducted in accordance with ASTM D2573-01.

The CPT cone was pushed at a constant rate into the ground using the hydraulic ram system of the drill rig. The tests were conducted following the provisions of ASTM D 5778. CPT T8-1, CPT 43-RW and CPT 44-RW were advanced to refusal. CPT-7, CPT-315 and CPT-316 (completed by Golder Associates) were all terminated at elevations between 157 and 158. Pore pressure dissipation tests were carried out at selected depth in CPT 43-RW and CPT 44-RW.

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

The locations of test holes and inferred soil profile at the tunnel location are shown on Drawing 285380-04-090-WIP2-2103. Borehole, Nilcon testing, CPT and DMT logs from the additional investigations are included in Appendix A. Relevant borehole and CPT logs from the previous investigations are included in Appendix B.

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[4] Advanced laboratory tests (consolidation, consolidated undrained triaxial tests) were carried out in AMEC's Scarborough laboratory.

[5] American Society for Testing and Materials.

Table 3-2: Overburden Thickness and Instrumentation in Boreholes

Borehole	Location	Overburden Thickness, m	Test or Instrument Name & Elevation					
			Rock Coring	Nilcon Vane	S-Piez.	VWP	MHSG	IN
BH T8-1	N 4,678,790 E 333,365	32.6	150.2 to 148.1	175.9 to 161.9		172.1 & 162.2	171.3 & 163.1	148.1*
BH TB6-1	N 4,678,910 E 333,353	> 10.1 (BTWO)						
BH CV4-1	N 4,678,868 E 333,368	> 10.4 (BTWO)						
BH HGMW-3	N 4,678,887 E 333,396	> 3.5 (BTWO)			181.4 to 179.8			
BH PS5-1	N 4,678,814 E 333,219	32.6	150.1 to 149.3			164.5, 156.9 & 150.8		
BH-7	N 4,678,848 E 333,325	33.2	150.0 to 145.3	177.1 to 164.1		167.4 & 147.1		
BH-118	N 4,678,904 E 333,303	33.3	150.3 to 146.6		147.5			
BH-314	N 4,678,751 E 333,462	33.1	150.0 to 144.8			145.0		

Legend:
- S-Piez. Standpipe Piezometer (Screen elevations)
- VWP Vibrating Wire Piezometer (Sensor elevations)
- MSG Spider Magnet Heave/Settlement Gauge
- IN Inclinometer Casing
- BTWO Borehole Terminated within Overburden
- * Bottom elevation of inclinometer casing/standpipe piezometer

3.3 Instrumentation

Geotechnical instruments (vibrating wire piezometers – VWP, spider magnets heave/settlement gauges – MHSG and inclinometers – 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: The VWP transducers (RST Model VW2100, 0.35 MPa for shallow to mid-depth and 0.7 MPa for deep installations) were installed in drilled holes at selected depths and electrical wires extended to the monitoring station at the ground surface to measure pore water pressures in soil strata.. The borehole was filled with a bentonite-cement mixture designed to match, as close as practical, the permeability and strength-deformation characteristics of the native soils. Sensor elevation and details of installations are provided in Table 3-2 and applicable borehole logs.

Magnetic Settlement/Heave Gauges: Spider magnets (RST, Model SSMM100 mechanical release spider target for 25 mm pipe) were installed in boreholes at 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. Elevations of the magnetic torus are provided in Table 3-2 and applicable borehole logs.

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 T8-1. 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 2.1 m into bedrock, 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.

3.4 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 tests).

Selected samples of the silty clay and silt samples obtained from Boreholes T8-1, TB6-1 and PS5-1 were sent to the ALS Environmental Analytical Laboratory in London, Ontario to determine the pH, redox potential, resistivity, sulphide and sulphate content of the soil to assess corrosion potential.

The results of geotechnical and geochemical 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.

3.5 Data Interpretation

Field Vane Test Data Correction: The chart (Figure 3.1[6]) developed initially by Bjerrum (1972) and updated subsequently by Ladd et al (1977) based on circular failure surfaces analyses of embankment failures suggest correction by multiplying the field vane data by 1.05 to 1.10 for soils with plasticity index (PI) of about 15 (ref. R-5 and R-31). However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), the Canadian Foundations Engineering Manual suggests that the vane test data for clays with PI<20 should not be corrected (ref. R-1 and R-43, and Figure 3.2). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI.

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

Strength Profiles from Cone Penetration Tests: The undrained shear strength of the silty clay/clayey silt 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 Su profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles. In this regard, the N_{kt} factor values used to calibrate the CPT strength profiles varied slightly for different segments of the WEP and the soil strata. Thus, 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.

Figure 3.3 presents the undrained shear strength profiles for WEP segment between Sta. 11+500L and Sta. 11+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. 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).

Pre-Consolidation Pressures from Cone Penetration Tests: The approach used for estimating the pre-consolidation pressures from the estimated Su profiles follows the Stress History and Normalized Soil Engineering Properties (SHANSEP) method developed at MIT (Ladd and Foott, 1974, ref.R-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, preliminary values of S = 0.18 and m = 0.95 were chosen to estimate the maximum past pressures from the inferred undrained shear strength profile. The maximum past pressure, σ'_p can then be estimated as:

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

Flat Blade Dilatometer (DMT) Test Data: DMT tests were conducted following the ASTM D6635-01 (2007) method. The soil properties from the results of these tests were developed in general using the guidelines layout in ISSMGE, 2001 (ref. R-27), except that the undrained shear strength values for the clay deposits were estimated using the relationship $S_u = S\ \sigma'_{vo}\ (0.5\ K_d)^{1.25}$, where S = 0.18 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 DMT results at this location suggest a relatively uniform undrained shear strength profile for the unweathered portion of the firm silty clay deposit with values close to the average strength measured by Nilcon vane apparatus.

The undrained shear strength (S_u), pre-consolidation pressure (σ_p'), natural water content (w_N) and compression index (Cc) profiles based on field and laboratory testing from boreholes, CPTs and DMT carried out between Sta. 11+500L and 11+800L are presented on Figure 3.3. Also included on this figure are $0.18x\sigma_{vo}'$ curve (representing undrained strength for OCR=1 condition) and simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units.

4 Subsurface Conditions

The general soil stratigraphy at the borehole locations consists of the following successive strata: topsoil, surficial layers of occasional fills,and upper granular deposit; an extensive cohesive clayey silt to silty clay deposit below about elevation 183, and a lower granular deposit below about elevation 153, overlying limestone and dolostone bedrock below about elevation 150. The thickness of the Clayey Silt to Silty Clay deposit varies between about 27.3 m and 32.9 m. The lower granular deposit (sandy silt / silty sand / sand and gravel) varied in thickness between 0 to 3.4 m. The bedrock was encountered at depths ranging from about 32.3 m to 33.2 m below the ground surface.

4.1 Topsoil and Surficial Fills and Upper Granular Deposit

All boreholes, except Boreholes CV4-1 and CPT T8-1 encountered up to 1.4 m thick layer of brown to black topsoil. The thickness of the topsoil is expected to vary through the project area.

Boreholes CV4-1 and CPT T8-1 encountered 2.1 and 0.8 m of clayey silty fill, respectively. Clayey silty fill over sand and gravel fill (1.4m thick) was also encountered underlying topsoil at Borehole BH-314.

A 0.9 m thick unit of sand was encountered 1.5 m below ground surface at Borehole HG-MW-3. Based on experience in the general area and confirmation at Borehole HG-MW-3, a discontinuous upper granular deposit may be expected.

4.2 Silty Clay to Clayey Silt 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 0.1 to 2.1 m. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into 4 layers as follows: brown desiccated stiff to very stiff clay crust, transition zone, upper grey silty clay to clayey silt deposit (referred to hereafter as upper silty clay), and then a lower grey clayey silt deposit (referred to as lower clayey silt). The natural water content, Atterberg limits and bulk unit weights determined on the samples recovered during the pre-bid and additional geotechnical investigation of the clay sub-strata are summarized in Table 4-1.

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

Property[1]	Clay Crust	Transition	Upper Silty Clay	Lower Silty Clay	Upper Clayey Silt	Lower Clayey Silt
Elevation Range, m	183[2] to 178	178 to 175	175 to 166	166 to 163	163 to 161	161 to 153
Natural Water Content, w_N, %	7 to 22	14 to 17	17 to 37	16 to 38	13 to 16	9 to 32
Liquid Limit, w_L, %	25 to 27	23	23 to 43	21 to 38	24 to 26	25 to 31
Plastic Limit, w_P, %	13	14	13 to 22	13 to 19	14 to 15	12 to 18
Plasticity Index, PI, %	12 to 14	9	10 to 21	8 to 19	10 to 11	12 to 14
Liquidity Index, LI	0.00	0.10	0.21 to 0.73	0.29 to 0.96	0.04 to 0.11	0.23 to 0.93
Unit Weight, γ, kN/m^3	-	21.9	20.4 to 21.4	21.1	22.0	N/A

Notes:
1. Index Properties are based on laboratory results on samples recovered from Boreholes BH T8-1, BH TB6-1, BH CV4-1, BH HG-MW-3, BH/CPT 43-RW, BH/CPT 44-RW, BH-7, BH-118 and BH-314.
2. Elevation varies

The undrained shear strength (S_u) profiles of the stratum between Sta. 11+500L and Sta. 12+300L are illustrated on Figure 3.3.

As illustrated on Figure 3.3, the measured and inferred undrained shear strength of the clay stratum varied with depth generally as follows:

- Crust layer: > 100 kPa
- Transition layer: 90±15 kPa to 60±15 kPa
- Upper silty clay: 60±15 kPa to 80±15 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 at 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-8 site are summarized in Table 4-2.

Table 4-2: Summary of Interpreted Compressibility Properties

Property[1]	Clay Crust	Transition	Upper Silty Clay	Lower Silty Clay	Upper Clayey Silt	Lower Clayey Silt
Elevation Range	183[2] to 178	178 to 175	175 to 166	166 to 163	163 to 161	161 to 153
Average Natural Water Content, w_N, %	16	15	21	22	15	21
Virgin Compression Index, C_c	0.13	0.12	0.18	0.18	0.12	0.17
Recompression Index, C_r	0.014	0.013	0.019	0.020	0.013	0.019
Swelling Index, C_s	0.031	0.030	0.044	0.045	0.029	0.043
Secondary Compression Index, C_α	0.0035	0.0033	0.0049	0.0051	0.0033	0.0049

Notes:
1. Index Properties are based on laboratory results on samples recovered from Boreholes BH T8-1, BH TB6-1, BH CV4-1, BH HG-MW-3, BH/CPT 43-RW, BH/CPT 44-RW, BH-7, BH-118 and BH-314.
2. Elevation varies

An oedometer test carried out on a grey clayey silt sample obtained from Borehole T8-1 at a depth of 17.1 m below ground surface with a W_N = 28.5% indicated the following compressibility indices: C_c = 0.221, C_r = 0.034 and C_s = 0.060. These compression index values are in general agreement with the interpreted compressibility characteristics summarized in Table 4-2.

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). .

$$E_u = 300\ S_u$$

$$E' = 0.9E_u$$

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.

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	32	0.35
Transition	21		19	
Grey Silty Clay	16		14	
Clayey Silt	19		17	

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

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

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

(*) Based on triaxial tests (ref. R-17 and R-19)

A Consolidated Undrained Triaxial Compression (CIUC) test carried out on a clayey silt sample obtained from Borehole T8-1 at a depth of about 17.1 m below ground surface indicated an effective friction angle of 28 degrees.

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"value ranging generally from 16 to greater than 100, this material is considered to be in a compact to very dense state of compactness. This layer was approximately 0 to 4.6 m thick but will 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 148.1 to 150.3 in the vicinity of Tunnel T-8. The Rock Quality Designation (RQD) of the recovered rock cores varied on average between 60 to 100 per cent, indicating a fair to excellent quality. Based on this core logging the rock mass classification was estimated to range from 2.8 to 5 for the Q-System (Barton *et. al.*, 1974) and 53 to 58 for the Rock Mass Rating (RMR) based on Bieniawski (1976) and indicates that the rock mass can be considered as a Fair quality rock mass based on the later system. With the exception of Borehole BH-314, rock quality generally increases with depth. A rock core sample from Borehole BH-118 located in the vicinity of Tunnel T-8 was tested and had unconfined compressive strength of 27.9 MPa. The RQD for the core that contained this sample was 25%. Photographs of rock cores recovered from the additional investigation are provided in Appendix E.

Based on this core logging the rock mass classification was estimated to range from 2.8 to 5 for the Q-System (Barton et. al., 1974, ref. R-3) and 53 to 58 for the Rock Mass Rating (RMR) based on Bieniawski (1976, ref. R-5) and indicates that the rock mass can be considered as a Fair quality rock mass based on the later system.

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
Number of Samples, N	12	12	16

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

4.5 Groundwater Conditions

Shallow and deep standpipe and vibrating wire piezometers were installed in selected boreholes during pre-bid and additional investigations to measure the water levels within overburden and bedrock, respectively (Table 3-2). The piezometric water levels within the the lower granular/bedrock and the overburden were observed between elevations 177.6 and 180.2, and between elevations 178.7 and 181.2, respectively (Table 4-5). The readings in piezometers in Borehole BH-7 suggest a downward gradient between the overburden and the bedrock. However, the readings in piezometers in Borehole BH PS5-1 suggest an upward gradient between the lower granular and the overburden. It is recognized that these piezometric water levels (particularly in the overburden) may not have fully stabilized.

In consideration of the findings at other locations along the project alignment, occurrence of local artesian condition in bedrock cannot be ruled out.

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.

Table 4-5: Summary of Measured Water Levels

Borehole	Surface Elevation	Piezometer Type	Screen / Sensor Elevation	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	Elevation
BH T8-1	182.8	VWP	172.1	Silty Clay	August 29, 2011	181.2
		VWP	162.2	Silty Clay	August 29, 2011	179.9
HGMW-3	182.9	S-Piez.	179.4 to 181.4	Silty Clay	July 29, 2011	180.9
PS5-1	182.8	VWP	164.5	Silty Clay	November 3, 2011	179.6
		VWP	158.9	Silty Clay	November 3, 2011	178.7
		VWP	150.8	Lower Granular	November 11, 2011	177.6
		S-Piez	150.9	Limestone	November 11, 2011	180.2
BH-7	183.2	S-Piez	167.2 to 167.6	Silty Clay	November 23, 2011	180.9
		S-Piez	146.3 to 147.8	Limestone	July 10, 2011	178.1
BH-118	182.7	S-Piez	146.7 to 148.2	Limestone	July 10,2011	178.7

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

4.6 Subsurface Gases

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

The H_2S gas can frequently be detected by odour at concentrations on the order of 0.5 mg/L (ppm) and can be corrosive at concentrations of about 2 to 3 mg/L in the groundwater.

A summary of sampling and testing of the groundwater by Golder (ref. R-17) and the recent investigation, in the boreholes near Tunnel T-8 is presented in Table 4-6.

Table 4-6: Summary of Natural Groundwater Chemistry

Borehole	Surface El, m	Sample El, m	Strata Type at Screen / Sensor Depth	H_2S	CH_4
				mg/L	µg/L
BH-118	186.66	146.66	Bedrock	2.55	65

Although the H_2S and CH_4 gases was not confirmed during the 2011 geotechnical investigation at Tunnel T-8 site, their presence cannot be ruled out.

Pumping tests were conducted at three locations across the proposed parkway to determine concentration levels of hydrogen sulphide gas in the groundwater of the area. A summary of the results of these tests is provided in Table 4-7.

Table 4-7: Pumping Tests Data

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

Dissolved methane was also sampled by Golder (ref. R-17) with most samples below detection (<5 μg/L) with the largest values (up to 485 μg/L) generally measured where artesian conditions occurred. These data are consistent with general water chemistry sampling taken at the end of the pumping tests

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 Designs

5.1 Tunnel Configuration

Tunnel T-8 will be constructed along the below-grade section of the WEP between Stations 11+600L and 11+720L, and will accommodate the below-grade traffic of Highway 401 (Drawing 285380-03-060-SEG1-2801). The proposed Tunnel T-8 is 120.2 m long and its width varies from 43.9 m at the east end to 46.8 m at the west end.

Tunnel T-8 is a 2-span deck-on-girder structure incorporating semi-integral abutments and centre pier founded on deep end bearing HP 310x110 steel piles (Drawing 285380-03-061-SEG1-2805and Reinforced Soil System (RSS) walls as false abutments. Geraedts Drive, which carries local traffic over the Highway 401, will be constructed at the centerline of the tunnel. The wing walls will comprise RSS return walls and tapered RSS portal walls extending beyond the tunnel portals. RSS return walls flared to the tunnel diaphragm is indicated at each corner of the structure.

The geotechnical design of the RSS walls with various sections of approved regular backfill, granular backfill and EPS is shown in Appendix I. The RSS abutments will be placed over a reinforced granular mat (RGM) placed, which in turn will be built over undisturbed native silty clay subgrade. The retained backfill will be completed with a combination of approved conventional soil fills and EPS.

Table 5-1 provides a summary of control elevations at the tunnel used for the geotechnical design development.

Table 5-1: Summary of Interpreted Elevations at Abutments

Location	Approximate Existing Ground Surface Elevation	Approximate Top of Finished Grade Elevation	Top of Deck Elevation	Approximate Top of Pile Cap Elevation	Approximate Pavement Subgrade Elevation
North Abutment Sta.11+660.000L (WP#1)	183.5	184.5	183.910	182.0	176.6
Centerline Tunnel & Hwy 401 Sta.11+660L (WP#2)	183.3	185.0	184.140	175.9	176.7
South Abutment Sta.11+660.000L (WP#3)	183.0	185.7	184.373	182.5	176.5

5.2 Geotechnical Design Criteria and Considerations

The geotechnical design has been completed in compliance with the requirements of the executed version of the Project Agreement Schedule 15-2 Part 2, Article 5 (PA) for the Windsor-Essex Parkway Project. The foundations' designs 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).

Working Stress Design (WS Method) was employed for global stability of the earthworks and soil mass containing earth retaining structures. The stability of the soil mass containing the false abutments and wing-walls (return wall and portal wall) is checked for all potential surfaces of sliding.

WS method was also used for the external stability (bearing, sliding and overturning) of the RSS structures.

Tunnel T-8 construction is expected to involve the following main sequence of earthwork, design elements and loading stages:

- Temporary excavations up to about 7.9 m depth below existing grade;
- Installation of a 1.5 m thick Reinforced Granular Mat (RGM) foundation at the north and south abutments (void forms may be used to accommodate pile installation at later stage through the RGM);
- Temporary sub-excavation to the underside of the pile cap for the centre pier;
- Installation of piles (HP310×110) for all tunnel supports;
- Completion of the pier footings;
- Installation of 500 mm diameter CSP around the abutment pile stickup with temporary supports;
- Construction of the RSS structures and associated permanent sub-drainage works at the south abutment;
- Filling of the CSP casing with concrete;
- Construction of the pile caps, abutment stubs, piers and tunnel deck;
- Completion of the base and sub-base for Highway 401 (this phase is required along the south abutment only to meet the necessary factor of safety for global stability during construction);
- Completion of the backfill above abutments and final topsoil placement; and
- Completion of the pavements for Highway 401.

5.3 Design Soil Properties

As described in Sections 3 and 4, the design soil properties for the silty clay to clayey silt deposit were interpreted from the CPT, DMT and Nilcon vane test profiles and the laboratory test results. The undrained shear strength (S_u) and preconsolidation pressure (σ'_p) profiles were estimated from Nilcon vane tests, DMT and the CPT based on the calibration described in Section 3.5. The S_u and σ'_p profiles inferred from the tests advanced around Tunnel T-8 and the design values obtained from these profiles are shown in Figure 3-3 and summarized hereafter in Table 5-2.

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

Clay Substratum	Elevation Range	Undrained Shear Strength (S_u), kPa	Effective Strength Parameters	Pre-consolidation Pressure, σ_p', kPa	Over Consolidation Ratio
Clay Crust	183* to 178	75 (**)	Cohesion, c' = 0 Effective Friction angle, ǿ = 30°	600	>10
Transition	178 to 175	75 to 60		600 to 400	4
Upper Silty Clay	175 to 166	60 to 50		400 to 280	1.3
Lower Silty Clay	166 to 163	50 to 57		280 to 310	1.3
Upper Clayey Silt	163 to 161	57 to 80		310 to 450	1.3
Lower Clayey Silt	161 to 153	80		450	1.2

(*) Elevations vary
(**) For global stability purposes
Note: The undrained shear strength and pre-consolidation pressure values vary with depth as illustrated in Figure 3-3.

The design values of the coefficient of horizontal permeability (k_h), the hydraulic conductivity anisotropy ratio ($A = k_h/k_v$) and in-situ void ratios required for the analysis of stress and deformation response of the soils 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	0.44
Transition	3.9×10^{-7}	2	0.42
Upper Silty Clay	1.1×10^{-7}		0.60
Lower Silty Clay	1.1×10^{-7}		0.62
Upper Clayey Silt	1.1×10^{-7}		0.41
Lower Clayey Silt	1.1×10^{-7}		0.59

For design purposes the initial groundwater level in the overburden was considered to be at elevation 180.0.

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 for north and south abutments (including sub-excavation for Highway 401 pavement) are expected to encounter topsoil and surficial fills and will be extended within native silty clay to the depth of about 6.7 to 8 m (approximate range of elevation 174.1 to 174.6) below existing grade. Pier excavation will be extended to the elevation ranging from 173.5 to 172.5 in the native grey silty clay soils.

Basal hydrostatic uplift was calculated at pier location based on the highest measured water level in the bedrock (elevation 180.2), anticipated deepest excavation depth, and a 15.6 m thick silty clay/clayey silt layer below the deepest excavation. The calculated factor of safety against hydrostatic uplift was about 1.5. The water level in new piezometers and piezometers installed in Boreholes T8-1, BH-7 and BH-118 advanced for this structure should be measured on regular basis and based on the results obtain, the basal uplift hydrostatic pressure should be reassessed. The calculated factor of safety against hydrostatic uplift instability at the abutment locations was about 1.6. These factors of safety are based on the weight only of the clay cap between the base of the excavation and the lower granular deposit.

As described in Section 4.6, gassy soils near bedrock surface could potentially be encountered during construction, which could impact the pore pressure and undrained shear strength condition of the lower part of the silty clay deposit. Given the significant soil stress relief due to depth of excavations, it is recommended that in the case of excavations deeper than 5 m, careful monitoring of basal heave and pore water pressures below of the bottom of the excavations be carried out during construction.

Adequate number of heave gauges and low-displacement type piezometers should be installed prior to initiation of the major excavations. If significant heave and pore water pressures are indicated by the monitoring during the excavation progress, the excavation rates will have to be adjusted to allow sufficient time to dissipate the pore water 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 by the Contractor with approval of the Engineer. A number of static load tests should be carried out at key locations along the alignment of WEP in conjunction with Pile Driving Analyzer (PDA) testing to facilitate proper calibration of the PDA, and determine the hammer performance and appropriate driving criteria (set).

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

For piles driven to bedrock the Serviceability Limit States (SLS) resistance of the HP310x110 piles, based on the conventional 25 mm settlement, is estimated to exceed the ULS resistance due to the unyielding nature of the bearing surface. Hence, the SLS resistance does not govern the design. In an unlikely event that some of the piles stop in very dense till (such as cobbles and boulders layer) the SLS resistance can decrease to not less than 1800 kN.

Based on the available borehole data at this structure, the bedrock surface elevation varies between 148.1 and 150.3, 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.
- 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 80 kN along the strong axis, and 55 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 235 kN, and 115 kN along the strong axis and weak axis, respectively.

The above SLS and ULS resistances were estimated using the “p-y” model (LPile 5.0 model Ensoft 2010). The pile model assumed to be embedded within firm to stiff silty clay below elevation 175.5. The “p-y” curves were generated using the Reese method described in the Technical manual for LPILE, using the Reese “Stiff-Clay without free water” and Matlock “Soft Clay” models in conjunction with the soil parameters described in Tables 5-4 and 5-5.

Table 5-4: Soil Parameters for p-y curve calculation

Soils Around the Piles	Elevation Range	Design Bulk Unit Weight (kN/m^3)	Undrained Shear Strength, S_u (kPa)	ε_{50}
Native Silty Clay Crust	Above 178	22	75	0.007
Native Transition Clay	178 to 175	22	Decreases linearly with depth from 75 to 60	0.007
Upper Silty Clay	175 to 166	20	Decreases linearly with depth from 60 to 50	0.007 to 0.010
Lower Silty Clay	166 to 163	21	Increase linearly with depth from 50 to 57	0.010 to 0.007
Upper Clayey Silt	163 to 161	22	Increases linearly with depth from 57 to 80	0.007
Lower Clayey Silt	161 to 153	21	80	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 Su and ε_{50}.

Table 5-5: Fill Properties for Pile Interaction Assessment

Material	Soil Model in L-Pile	Design Bulk Unit Weight, kN/m³	ø°	n_h, MPa/m
RSS Fill (Granular*) & Compacted Granular Slope	Sand (Reese)	21	35	10

(*)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.

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

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

It should be noted that during driving, significant soil disturbance and damage occur around the pile shaft forming sizeable gaps between the pile and the surrounding soils. These gaps cause 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 compacted reinforced RSS fill will develop lateral resistances to lateral loads larger than the above listed conventional ULS and SLS resistances.

Significant lateral loads in excess of the values previously cited may be resisted fully or partially by the use of battered piles. For ease of constructability and to limit the loss of hammer energy for pile driving, batters are usually limited to no steeper than 1H:5V. However, greater batter up to 1H:3V may be considered.

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	$= n_h$ −	for cohesionless soils, and
Where:	= 67 —	for cohesive soils.
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 ranges of 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, as 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 For Subgrade Reaction Method

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

d = pile diameter

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 can be considered using the nonlinear "p-y" interaction method and elastic continuum theory as discussed in the Canadian Foundation Engineering Manual (ref. R-8). The p-y curves describe the lateral soil resistance along the pile depth. For each soil layer along the pile shaft, the p-y curves describe lateral soil pressure 'p' (kPa) per unit length mobilized by the pile lateral deflection 'y' (m). Where only pile head loads are applied and there are no lateral movements of the surrounding soil mass, 'y' is the absolute lateral deflection. Where lateral ground movements occur, 'y' is the relative movement between the pile and the soil. The p-y curves reflect the non-linear soil behaviour under moderate to high stress levels where the more traditional elastic modeling of the soil response is considered to be insufficient.

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

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

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

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

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: Lateral Load Capacity Reduction Factor For Pile Groups for p-y Method

Relative Pile Position	Pile Spacing Ratio, s/d	β_{ki}
In Row (perpendicular to the load direction)	< 3.75	$0.64(s/d)^{0.34} \leq 1$
Leading 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 modifier factor applies to the "p" values.

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 9 m to accommodate the future depressed highways, followed by partial re-placement of fills 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: after RSS completion (where applicable), after completion of the top backfill against the tunnel diaphragm (End of Construction - EC) and in long-term (LT), and associated vertical effective stresses are presented in Figures G-11 and G-13. The analyses indicate the following:

- Ground settlements are expected to occur along the pile shaft during construction of the RSS wall, tunnel and the associated backfill, and continue for approximately 2 years; and

- Ground rebound is expected to occur after substantial completion of the ground surface loading (approximately 2 years after end of construction).

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

a) Maximum transient downdrag of 820 kN plus structural dead load only (pile cap and tunnel roof) occurring during backfilling against the tunnel diaphragm.

b) Residual (long-term) downdrag of less than 200 kN plus total design dead loads (structural and topsoil/landscape materials over tunnel roof) after the completion of construction.

If the staging is such that piles are driven after the installation of the RSS wall, the estimated maximum short-term (transient) negative skin friction would reduce to 440 kN.

The above estimates assume that the placement of the soil fill over the tunnel roof occurs after substantial completion of the final grading along the tunnel sides.

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

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

No downdrag is anticipated at the pier piles.

Shaft Bending due to Lateral Soil Displacement:

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 embedded within native soils. .
- The ground lateral movement (Figure G-13) along the pile shaft anticipated to occur after the installation of the piles was estimated using the stress-deformation analysis described below in Section 5.8.2.
- 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.

Based on the above approach and anticipated lateral ground displacement, the estimated maximum unfactored bending moment in the shaft was 65 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 55 kN. The calculated maximum pile deflection at the underside of the RSS wall base was 9 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 the tunnel loads applied to the piles.

The maximum computed moment in the pile under an assumed pile head load equal to the conventional SLS resistance (80 kN) was 85 kN-m for the strong axis pile loadings. Accordingly, a potential combination of the maximum bending stresses from pile head shear force and ground displacement field would lead to a maximum bending moment of 155 kN-m, which is less than the yield moment of the pile.

As indicated, the stress and deformation discussed above are in addition to the stress and deformation caused by the tunnel loads.

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 35 and 20 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 two options of abutment 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) may 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:

$$p_h = K_r K_a \sigma_v + \Delta\,\sigma_H \qquad \text{(FHWA-NHI-10-024, ref. R-39)}$$

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 can 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:

Table 5-8: Estimated Earth Pressures on Pile Cap Straps

Abutment	**Earth Pressure, kN/m**		
	ELL	**EDS**	**EB**
North Wall – West Segment	6	24	10
North Wall – Geraedts Drive	10	19	10
North Wall – East Segment	6	24	10
South Wall – West Segment	6	21	9
South Wall – Geraedts Drive	10	13	10
South Wall – East Segment	6	21	9

Legend:
ELL (kN/m) Earth pressure from live loads (assumed 9 kPa within landscape areas and 16 kPa within roadways)
EDS (kN/m) Earth pressure from Dead Surcharge load above the pile cap
EB (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 strip should be carried out by the supplier of the reinforced soil structures.

5.6 RSS False Abutment Walls

As mentioned earlier, false abutments using RSS wall system were included at both abutments. The general configurations developed for the typical abutments at Tunnel T-8 are shown in Figure I.1. The abutments generally comprise reinforced soil structure (RSS) founded on the reinforced granular mat (RGM), approved regular backfill and EPS.

These configurations and dimensions were developed at representative sections along the tunnel to verify the geotechnical design requirements with respect to (a) the ground deformations, (b) the global stability of the soil mass containing the structure and (c) the foundation soil bearing resistances.

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 where applicable 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.

The RSS and its RGM foundation are 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 described in Tables 5-9 and 5-10.

Table 5-9: Assumed Proprietary Product Properties

Material	Unit weight, kN/m^3	Limit Equilibrium Analyses (Slope/W Models)			Stress Deformation Analyses (Sigma/W Models)	
		Undrained	Drained			
		Undrained Shear Strength, kPa	Friction Angle, °	Apparent Cohesion, kPa	Modulus of Elasticity, E, MPa	Poisson's ratio, μ
RSS with Approved Granular Fill	21	50	35	50	40	0.35
RGM	21	40	35	40	60	0.35
EPS	0.5	10	0	10	10	0.20

Table 5-10: Assumed Backfill Material Properties for Global Stability Analyses

Backfill Material	Unit weight, kN/m^3	Undrained Shear Strength, kPa	Drained Angle of Internal Friction*, °	Modulus of Elasticity, E, MPa	Poisson's ratio, μ
Compacted Clay Fill	21	50	30	22.5	0.35
Granular Backfill	21	N/A	33	22.5	0.35

Preliminary dimensions of the abutments are listed in Table 5-14.
* $\varphi' = 30°$ 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-21 illustrate the stability models for the north and south abutments of the tunnel. The global stability analyses have been carried out for short-term during construction (undrained soil properties), end of construction (undrained soil properties) and long-term (drained soil properties with stabilized water levels) loading conditions. The short-term analysis completed for temporary conditions using undrained soil properties represents the condition in which the pavement structure over the Highway 401 subgrade tunnel is not present or is removed (e.g., to simulate future pavement repairs). The drained analyses assumed that all the components of the structure are present. The presence of the piles was not considered in the stability models (somewhat conservative approach). Live load of 12 kPa in the area of highway pavement where present and surcharge of 9 kPa in the area of landscape and trail were applied at the top of ground surface for short-term and long-term model, while tension crack was assumed for short-term only.

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

As mentioned before, 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 strength profiles. The calculated factors of safety (FS) against global instability of the abutments are shown in Figures F-1 to F-21 and summarized in Table 5-11.

Table 5-11: Summary of the Results of Abutment Slope Stability Analyses

Abutment	Factor of Safety for Loading Condition				Figure
	Short-term Undrained Loading Condition(1)	Short-term Undrained Loading Condition(2)	End of Construction Undrained Loading Condition(3)	Long-term Drained Loading Condition(4)	
North Wall – West Segment	1.6 (1.3)	-	1.8 (1.5)	1.8 (1.7)	F-1 to F-3
North Wall – Geraedts Drive	1.6 (1.3)	-	1.7 (1.3)	1.7 (1.6)	F-4 to F-6
North Wall – East Segment	1.7 (1.4)	-	1.9 (1.6)	1.8 (1.7)	F-7 to F-9
South Wall – West Segment	1.4 (1.1)5	1.5 (1.3)	1.6 (1.3)	1.6 (1.5)	F-10 to F-13
South Wall – Geraedts Drive	1.4 (1.2)5	1.4 (1.3)	1.6 (1.3)	1.7 (1.5)	F-14 to F-17
South Wall – East Segment	1.4 (1.0)5	1.5 (1.3)	1.6 (1.3)	1.7 (1.5)	F-18 to F-21

Note: Values outside parentheses refer to circular failure surfaces and the values in parentheses refer to non-circular failure surface.
(1) Short-term (temporary) undrained response without pavement box over Highway 401 subgrade
(2) Short-term (temporary) undrained response with base – sub-base over Highway 401 subgrade
(3) Undrained response with pavement box over Highway 401 subgrade
(4) Drained response with all design components present
(5) Granular base of Highway 401 must be placed before any backfill is placed above the deck seat level

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 SDA were carried out using an effective stress-based model for south abutment (Figure G-1). The SIGMA model was developed for the south abutment (east segment) where the height of the retained soils measured from the top of finished grade to the bottom of the RSS is 9.0 m high and the Highway 3 section is in the closest proximity to the RSS wall. The south abutment (east segment) model will provide the upper limits for the deformation estimates. The long-term phreatic surface was assumed to correspond to the initial groundwater level at elevation 180.0 and follow the excavation and subgrade surfaces. Elastic-plastic Mohr-Coulomb models were used for all soil layers except for the unweathered firm and 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 stratigraphy and selection of the soil properties (except for the RSS structure and pavement box) was based on the design soil properties discussed at Section 5.3. The RSS structure, RGM and pavement were assumed as a homogeneous elastic materials described in Table 5-9.

The configuration of the calculation model is presented in Figures G-1 to G-4. The calculation model typically assumed the following loading steps:

a) Definition of the initial (in-situ) stress condition for level ground assuming an average bulk unit weight of 21 kN/m^3 and an at-rest earth pressure coefficient K_0 of 0.75 for the soil deposit (based on published data [ref. R-42] and confirmed by DMT at the site) for the soil deposit (0 days);;

b) Bulk excavation to the subgrade level under the highway pavement (60 day duration – day 0 to 60);

c) Construction of the RGM and RSS structures, and the associated backfill (60 day duration – day 60 to 120);

d) Completion of the remaining fill above the RSS structure (60 day duration – day 120 to 180);

e) Completion of the pavement structure for Highway 401 (1 day duration – day 180 to 181); and

f) Dissipation of excess pore pressure.

The construction stages were represented by excavation, completion of the RSS and completion of the entire abutment followed by the placement of the pavement box. The excavation was assumed to occur in 60 days, construction of RGM and RSS structures in 60 days, completion of remaining fill above RSS in 60 days and remaining stage (completion of pavement structure) was assumed to occur rapidly (1 day stage).

Figures G-1, G-3 and G-4 show the cumulative settlement/heave for the end of excavation (60 days), end of construction ("181 days") undrained conditions for the tunnel and the long-term ("9,306 days") drained loading conditions. Figure G-2 shows the cumulative lateral movement for the end of excavation. Figure G-5 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 5.8.2. The cumulative deformations are summarized in Table 5-12. The ground movements generated by the construction loads are anticipated to stabilize within approximately 13 years following completion of construction. Due to the relatively smooth changes in the geometry of the tunnel, the above settlement changes along Highway 401 are anticipated to be gradual in longitudinal profile.

Figure G-6 shows cumulative ground surface settlement along the tunnel approachway and along Geraedts Drive/Highway 3 at end of excavation and end of RSS construction. Figure G-7 shows the cumulative ground settlement at the tunnel approachway and along Geraedts Drive/Highway 3 at end of construction and at long term condition. Figure G-8 shows the cumulative settlement at the top of the RSS wall facing and Figure G-9 shows the cumulative lateral displacement along the RSS wall facing. Figure G-10 shows the cumulative settlement and heave along Highway 401. Figures G-11, G-12 and G-13 show soil settlement, lateral soil displacements and vertical effective stress (respectively) along the pile line determined from SDA, which were used in pile calculations in Section 5.5.

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

Table 5-12: Summary of Calculated Cumulative Deformations

Parameter	**End of Excavation (60 days)**	**End of RSS Construction (120 days)**	**End of Construction (181 days)**	**Long-term (Drained) (9306 days)**	**Remarks**
Settlements on Top of Ground at Distances (m) from the Edge of Deck of (†)					
• 0 m†	N/A	N/A	-10 mm (*)	-10 mm	Figures G-6 & G-7
• 5 m	N/A	N/A	-10 mm (*)	-10 mm	
• North Edge of Highway 3	N/A	N/A	-10 mm (*)	-10 mm	
• Center of Highway 3	-25 mm	-20 mm (*)	-10 mm (*)	-20 mm	
• South Edge of Highway 3	-25 mm	-15 mm (*)	-5 mm (*)	-20 mm	
• Centerline of Pedestrian Trail	-20 mm	-15 mm	-20 mm	-30 mm	
Settlement at the top of RSS facing (mm)	N/A	-55 mm (*)	-60 mm (*)	-60 mm	Figure G-8
Lateral displacement at the base of RSS facing (mm)	N/A	< 5 mm	< 10 mm	< 5 mm	Figure G-9
Rotation of the RSS facing	N/A	0.004	0.004	0.003	
Maximum Heave (rebound) at Highway 401	55 mm	65 mm	65 mm	90 mm	Figure G-10

N/A Not Applicable – Area located within the temporary excavation.
(-)ve denotes settlements
(†) Distances measured perpendicular to the tunnel abutment.
(*) Indicates calculated movement that is corrected during constructions
Note: The abutment design and soil properties assumed represent the south abutment configuration.

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

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.

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 Short-term (undrained) net ultimate geotechnical bearing capacity is 290 kPa (based on an average shear strength of 57 kPa).

The long-term (drained) net bearing capacity is 490 kPa based on a friction angle of 30° and an embedment of at least 1.5 m below finished grade.

Sliding Resistance:

The ultimate geotechnical resistance can be determined in accordance with the following expression:

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

Where: A' = effective contact area of the base (m^2).

c' = cohesion/adhesion at sliding interface

δ = friction angle at sliding interface

V = vertical force (kN)

H_f = design horizontal load (kN).

Based on Highway Flood Hazard Analysis[7], it is understood that Tunnel T-8 will not be flooded in the occurrence of 1:100 year storm and regional storm. However, due to the estimated elevation of 177.4 for the 100-year flooding event and 177.7 for the regional storm event from Pump Station 5 in the vicinity of Tunnel T-8, flooding of the roadway in Tunnel T-8 is expected to occur. As the EPS and LWF incorporated in Tunnel T-8 abutments and return walls are located above the base of the pile cap at elevations greater than 181.3, submergence of the material is not anticipated to occur in the area of Tunnel T-8.

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'
RSS to RGM	30	0	30	0
RGM to Silty Clay	0	55	30	0

Based on geotechnical analyses discussed in Sections 5.6.1 to 5.6.4, preliminary abutment configurations and dimensions were determined (Table 5-14). As noted previously in Section 5.6, the abutment configurations and dimensions indicated in these analyses are the minimum required and are to be finalized by proprietary suppliers. The final design of the abutments is to be developed in consultation

[7] HMM document no. 285380-70-126-0010, Rev.0.

with the suppliers of the proprietary components. The proposed abutment configurations are shown in Figure I.1 in Appendix I.

Table 5-14: Tentative Abutment Dimensions(5)

Abutment Location	Assumed Total Height(1), m	RGM(2) Size (Thickness x Min. Width at Base), m	EPS(2) Volume, m^3/m	RSS Structure Size (Width x Height)(3), m
North Wall – West Segment	8.8	1.5 x 8.0	4.5	6.5 x 4.4(4)
North Wall – Geraedts Drive	8.3	1.5 x 8.0	4.5	6.5 x 4.3(4)
North Wall – East Segment	8.8	1.5 x 8.0	4.5	6.5 x 4.4(4)
South Wall – West Segment	9.3	1.5 x 8.0	5.9	6.5 x 4.9(4)
South Wall – Geraedts Drive	8.9	1.5 x 8.0	6.8	6.5 x 5.0(4)
South Wall – East Segment	8.7	1.5 x 8.0	5.9	6.5 x 4.3(4)

(1) Measured from top of finished grade at tunnel edge to the base of the RSS structure.
(2) In general, the use of RGM and EPS is required to meet the design compliance for undrained short-term condition.
(3) The RSS supplier may require wider structures to meet the internal design requirement. The effects of a wider structure on bearing capacity will need to be assessed.
(4) Unit weight of RSS wall was assumed to be 21.0 kN/m^3 as an approved granular material.
(5) RSS minimum dimensions for external stability purposes.

5.7 RGM Foundation Loads

A 1.5 m thick 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 approach was used considering that the RGM foundation distributes the vertical pressures at the base of the RSS walls to the subgrade below the RGM at a 45 degree angle. The following loads (Table 5-15) were estimated to act on top of the RGM on the basis of conventional calculation of the bearing pressures under gravity retaining walls.

Table 5-15: Estimated load on RGM

Abutment Location	Maximum Unfactored Bearing Pressure below RSS wall, kPa	Average Unfactored Bearing Pressure below RSS wall, kPa
North Wall	160	150
South Wall – Geraedts Drive	190	150
South Tapered Walls	220	170

Based on the above load on RGM, an estimated factored horizontal tensile load of 67 kN per meter of RGM was estimated across the entire height of 1.5 m. For cost estimates, this tensile load can be accommodated by 3 layers of geogrid which has long-term load capacity of 23 kN/m.

The above loads are for the use by the RGM suppliers to assist in the RGM's internal design. The associated soil resistances at the underside of the RGM at ULS are provided in Section 5.6.4.

5.8 Wing Walls (Return Walls and Portal Walls)

As mentioned earlier, an RSS return walls flared at 90^0 to the tunnel diaphragm is indicated at each corner of the structure. The tapered RSS portal walls are extended beyond the tunnel portals. Similar to the RSS walls at the abutments, the RSS wing walls have been preliminarily checked for bearing capacity and sliding resistances. Light weight fill (LWF) was required for RSS return walls and south abutment tapered walls.

The global stability analyses have been carried out on RSS return walls and the highest RSS tapered wall. The calculated factors of safety are in excess of 1.3 against global instability for short term conditions and over 1.5 for long-term conditions. The calculated factors of safety are summarized in Table 5-16. Figures F.22 to F.27 in Appendix F illustrate the stability models for the return walls.

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

Wing Wall Locations	Wing Wall Components	Factor of Safety for Loading Condition			Figure
		Short-term (Undrained Loading)(1)	End of Construction (Undrained Loading)(2)	Long-term (Drained Loading)(3)	
South Wall (West Segment)	Tapered Wall	1.4 (1.3)	1.6 (1.4)	1.6 (1.5)	F-22 to F-24
	4.0 m wide Return Wall	2.0 (1.8)	1.8 (1.6)	1.8 (1.7)	F-25 to F-27

Note: Values outside parentheses refer to circular failure surfaces and the values in parentheses refer to non-circular failure surface.
(1) Short-term (temporary) undrained response without pavement box over Highway 401 subgrade
(2) Undrained response with pavement box over Highway 401 subgrade
(3) Drained response with all design components present

Based on geotechnical analyses, tentative wing wall configurations and dimensions summarized in Table 5-17 were determined. The wing wall 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.

Table 5-17: Wingwall Dimensions(2)

Wing Wall	RSS Structure (Width × Height)(1), m	Quantity of Lightweight Fill, m^3/m
Tapered South Walls (Highest section)	7.5 × 5.0	11.25
Tapered North Walls (Highest section)	5.5 × 5.0	-
Return Wall (Highest Section)	4.0 × 4.5	12.8

(1) Measured between the underside of the stem (pile cap) and the top of the RGM at the tapered walls and between the top grade and the underside of the stem (pile cap) at the return walls.
(2) RSS minimum dimension for external stability purposes.

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

Table 5-18 summarizes the net ultimate bearing capacity values (q_u) determined for the cohesionless materials supporting the return walls and native silty clay supporting the portal walls.

Table 5-18: Ultimate Bearing Capacity at Return Walls and Portal Walls

Type of Wing Walls	q_u (kPa)
Return Walls	545 (1)
Portal Walls	290(2)

(1) Based on an assumed friction angle, phi = 33^0 and unit weight = 21 kN/m^3 within the assumed zone of influence of RSS return wall.
(2) Based on an average cohesion of 57 kPa within the assumed zone of influence of RSS portal wall.

5.9 Backfilling

Construction notes for backfill are provided in Drawing 285380-04-094-WIP1-2839. Construction notes for lightweight fill material (LWF) and EPS are provided in Drawings 285380-04-094-WIP1-2140 and 285380-04-094-WIP1-2141, respectively.

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 Canadian Highway Bridge Design Code CAN/CSA-S6-06.

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 5-19. Compactable Group III soils may be used as general backfill within approved areas.

Table 5-19: 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	0.310 to 0.35	0.33 to 0.45
'At Rest' or Restrained, $K_o^{(*)}$	0.43 to 0.46	0.47 to 0.52	0.50 to 0.62
'Passive', $K_p^{(*)}$	3.3 to 3.7	2.9 to 3.2	2.2 to 3.0

Note: Values are given for level backfill and ground surface behind the wall compacted to > 95% Standard Proctor maximum dry density. The coefficients of lateral earth pressure should be adjusted if there is sloping ground at the back of the wall.
Group I Soils: Coarse grained soils (e.g. Granular A and B Type 2)
Group II Soils: Finer grained than Group I noncohesive soils (e.g. Granular B Type1, pit run, etc)
Group III Soils: Finer grained soils (e.g. approved site generated silty clay).

5.10 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 for the retained soil mass within the RSS structures and the RGM, as recommended, will ensure that these structures will act as a "natural" drain convening the seepage from the 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 manholes or sumps.

Depending on the grain size of the backfill, RSS and RGM materials, a filter layer may be required at the interface between the native soil excavation slope and the backfill. Given the grain size uniformity, the LWF should be wrapped in filter fabric to prevent the migration of the fines from adjacent fills and soils.

Simplified steady-state models (Appendix H) were used to estimate seepage rates associated with the long-term drawdown of the groundwater along a typical cross-sections of the north and south abutments of Tunnel T-8. SEEP/W 2007 software was used for these analyses. The initial groundwater table was assumed at elevation 180 for both abutment models and a high water level of elevation 182.1 m was used for the Cahill Drain in the north abutment model. 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 of $1x10^{-4}$ m/day 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 3.4 litre/day per meter length of the tunnel. This is an approximate estimate and the actual quantities could differ significantly from this magnitude. The above flow rates do not include additional seepage that may occur from other external sources, perched groundwater within the upper fills / granular layers, utility trenches, and runoff from ground surface.

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. Free draining granular material (Granular B Type 1 or approved equivalent) should be installed immediately adjacent to walls to prevent water pressures acting on the walls and to permit downward flow of surface water down into the wall sub-drains. The subdrains should be surrounded by approved granular material and discharged via gravity flow to the storm drain or road ditch system along Highway 401.

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

6.2 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.
- The design and construction should address the potential presence of soil gases. Air monitoring should be considered during construction. In general, it is recommended that equipment operating in confined spaces 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 that may occur if gassy soils are encountered.

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 samples of the silt and clay stratum obtained in Boreholes BH T8-1 (Sample 6), BH TB6-1 (Sample 10) and BH PS5-1 (Sample 23). Table 6-3 summarizes the results of various analyses carried out on the soil samples to assess the potential for corrosion on concrete.

Table 6-1: Results of Analytical Testing on Soils

Location of Soil Samples	Elevation of Soil Sample	pH	Redox Potential, mV	Resistivity, ohm.cm	Sulphide, mg/kg	Sulphate, mg/kg
Borehole BH T8-1 (Sample 6)	179.0	7.84	100	4670	<0.2	112
Borehole BH TB6-1 (Sample 10)	173.6	7.86	125	3700	<0.2	100
Borehole BH PS5-1 (Sample 23)	153.3	7.90	230	2580	<0.2	486

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-8).

As discussed in the sections above, dissolved hydrogen sulphide at concentrations of 7 mg/L were encountered in the groundwater pumping tests north of Tunnel T-8, therefore construction materials should be selected accordingly.

Based on the measured electrical resistivity, pH, redox potential, sulphide contents etc., the soil would be considered to have a potential for corrosion to buried metallic elements AWWA (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-8 was prepared by Mr. Ganan Nadarajah, P.Eng under the design direction of 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.

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

Yours truly,
AMEC Environment & Infrastructure
a Division of AMEC Americas Limited

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Drawings

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers — Hatch Mott MacDonald

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

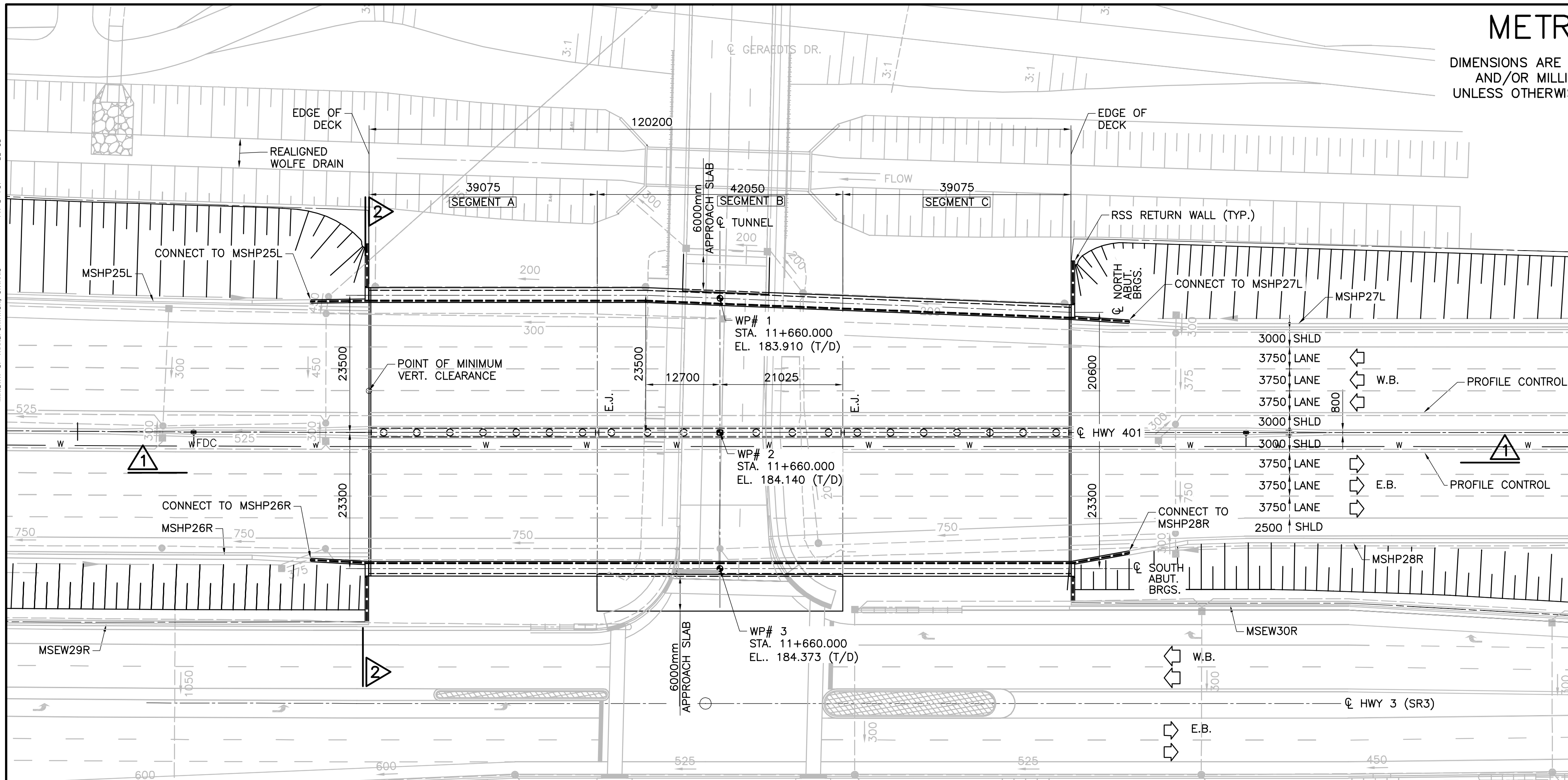
NEW CONSTRUCTION
HWY 401
GERAEDTS TUNNEL T-8
GENERAL ARRANGEMENT

SHEET S2801
Phase 1
IFC

NORTH FOR CONSTRUCTION

NOTES:

1. FOR GENERAL NOTES SEE SHEET S2802.



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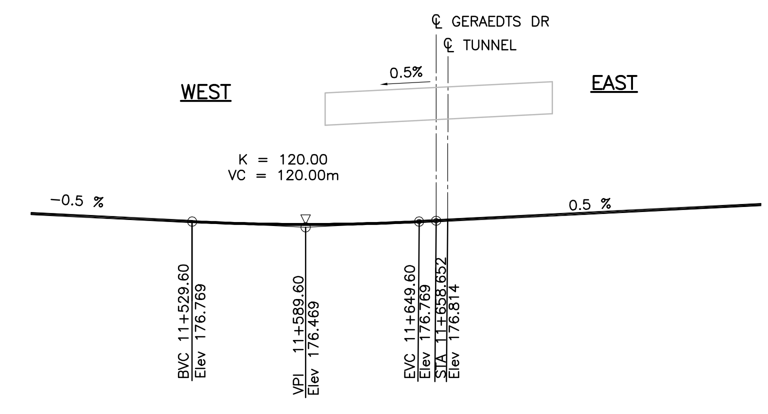

PROFILE OF HWY 401
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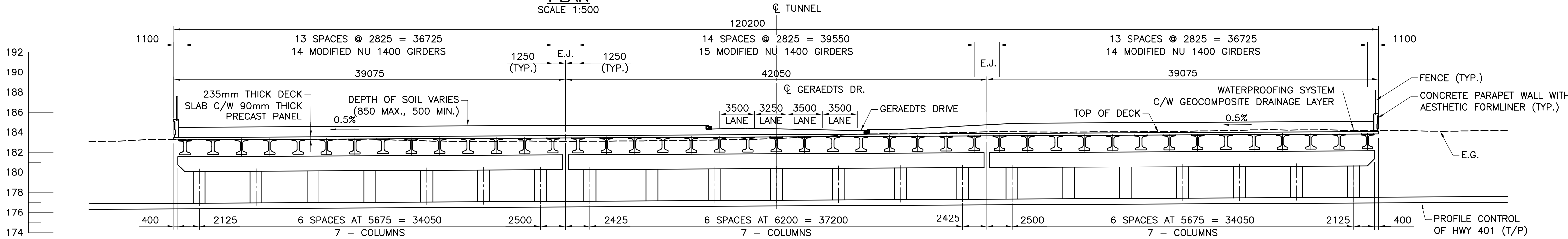

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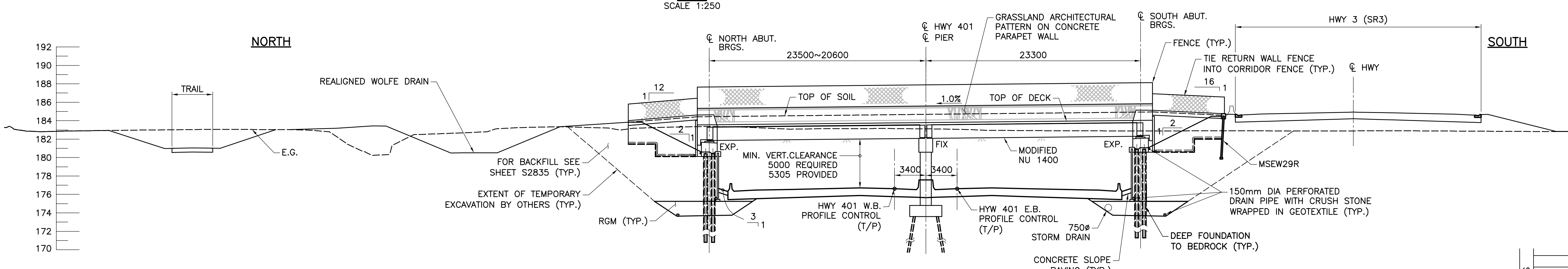

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DRAWING NOT TO BE SCALED
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DRAWN DM	CHK MAS/JL	SITE 6–708	DATE 12–JUL–11

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DATE PLOTTED: 9/7/2012 9:56:00 AM
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TOP OF ABUTMENT ELEVATIONS				
LOCATION	A	B	C	D
NORTH ABUTMENT	181.621	181.816	182.029	182.251
SOUTH ABUTMENT	182.086	182.281	182.491	182.687

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

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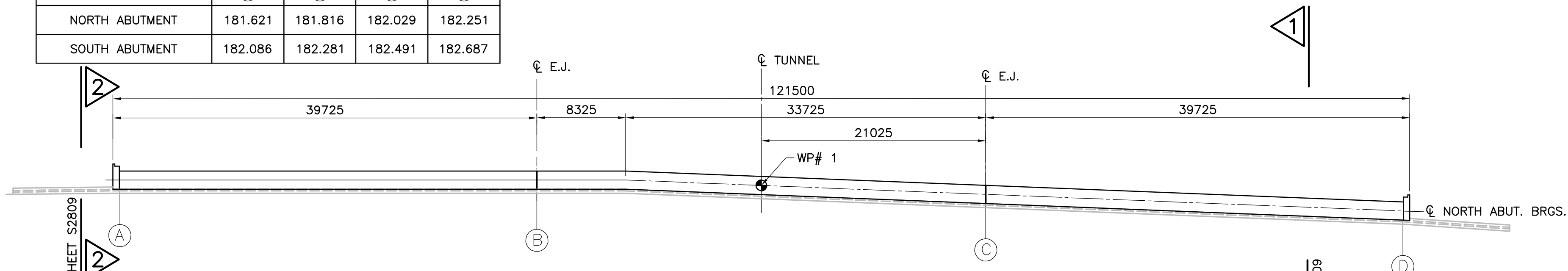
Windsor–Essex Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
GERAEDTS TUNNEL T–8
ABUTMENT LAYOUT I

SHEET S2808

Phase 1

IFC



NORTH ABUTMENT – PLAN

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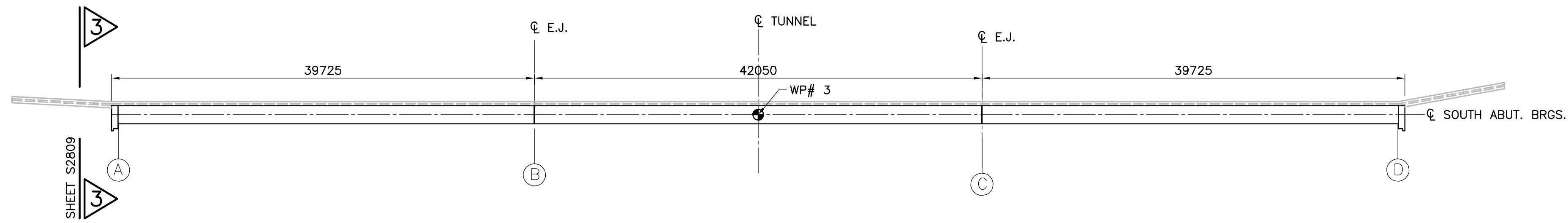

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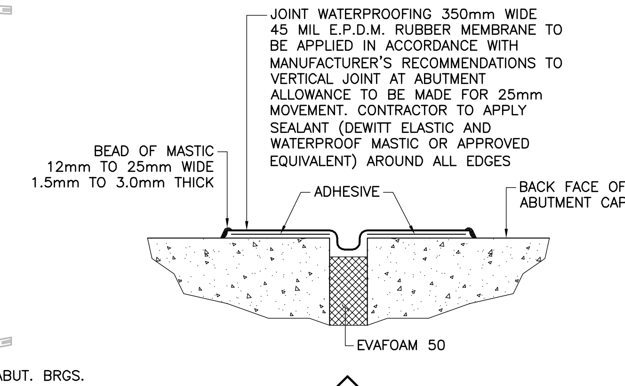

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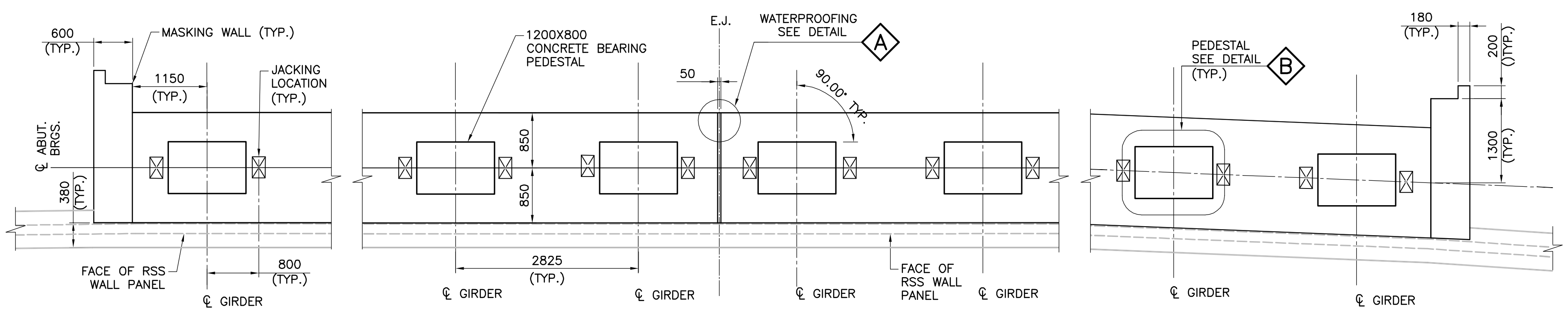
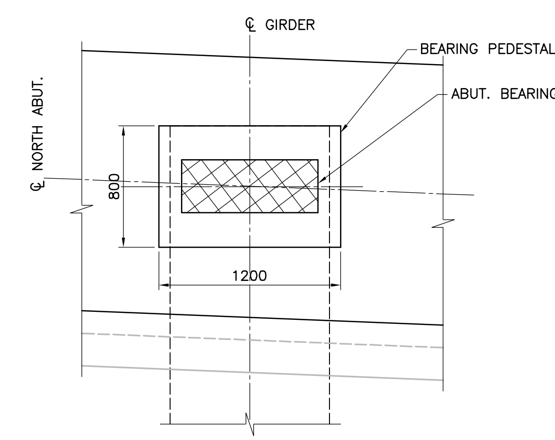
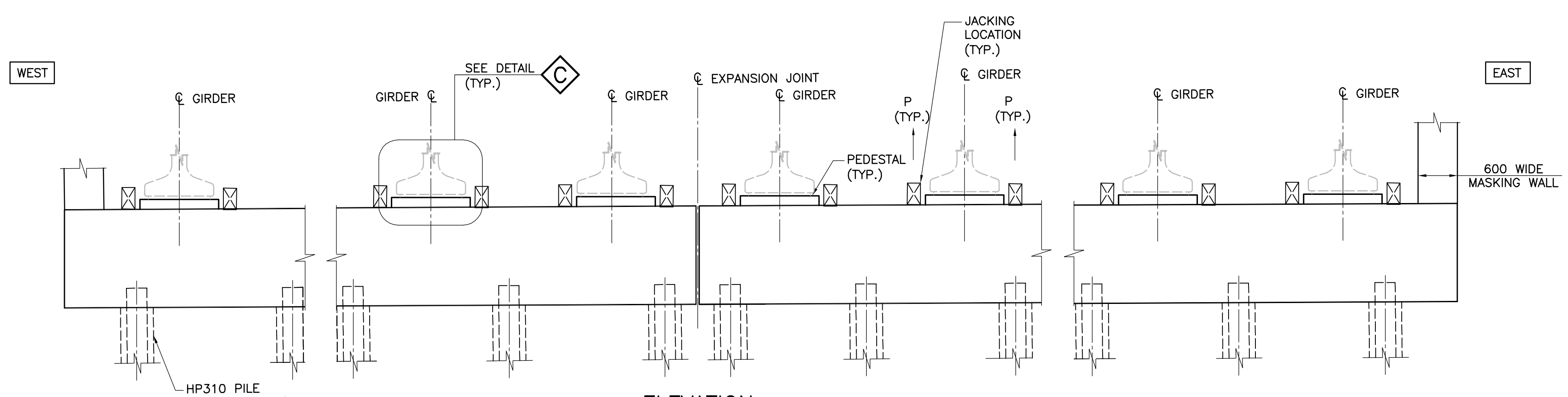


PLAN DETAILS

SCALE 1:50
NORTH ABUTMENT
SOUTH ABUTMENT SIMILAR

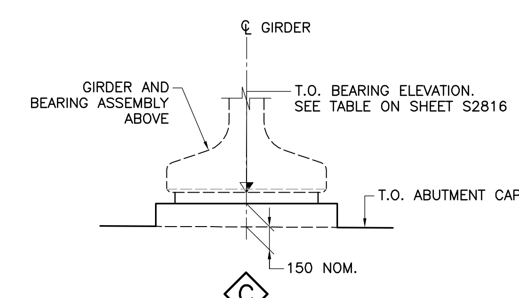


B
SCALE 1:25



ELEVATION

SCALE 1:50



C
SCALE 1:25
(AT ℄ OF BEARING)

NOTES:

1. FOR SEGMENTAL LAYOUT SEE SHEET S2801.
2. MAXIMUM JACKING LOAD 2P (kN) PER GIRDER LINE FOR JACKING OF THE GIRDERS:

NORTH ABUTMENT (2P)	1979
SOUTH ABUTMENT (2P)	1961

3. SEE SHEETS S2835 TO S2837 FOR SOIL REINFORCING STRIPS NOTES AND LOAD INFORMATION.
4. REINFORCING STRIPS AND CONNECTION DETAIL TO BE DESIGNED BY CONTRACTOR AND RSS WALL SUPPLIER.

CONSTRUCTION NOTES:

1. CONCRETE PLACEMENT FOR FOUNDATION IS TO BE PLACED IN A MINIMUM OF 4 SEGMENTS BETWEEN EXPANSION JOINTS AND POURED IN ALTERNATE SEGMENTS.
2. ALLOW 7 DAYS BETWEEN POURS.
3. CONSTRUCTION JOINT POSITIONS ARE SUGGESTED LOCATIONS TO CONTROL UNWANTED CRACKING. CONTRACTOR TO CONSULT DESIGNER IF ALTERNATE LOCATIONS ARE TO BE CONSIDERED.
4. GRANULAR BASE AND SUBBASE BELOW HWY401 PAVEMENT MUST BE IN PLACE BEFORE ANY BACKFILL IS PLACED ABOVE THE DECK SEAT LEVEL.
5. NO BACKFILL SHALL BE PLACED UNTIL CONCRETE IN ABUTMENT DIAPHRAGMS HAS REACHED 30MPa.
6. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATIONS BE GREATER THAN 500mm.
7. SEE SHEET S2802 FOR ADDITIONAL CONSTRUCTION NOTES.

CONSTRUCTION SEQUENCE:

1. CONSTRUCT THE BEARING PEDESTALS.
2. PLACE WATERPROOFING.
3. PLACE BEARING PADS.
4. ERECT PRESTRESSED CONCRETE GIRDERS. CONTRACTORS TO DESIGN AND INSTALL TEMPORARY LATERAL BRACING TO ENSURE STABILITY DURING GIRDER ERECTION AND ABUTMENT DIAPHRAGM AND DECK CONCRETE PLACEMENT.
5. CAST ABUTMENT DIAPHRAGM INTEGRAL WITH DECK SLAB.
6. PLACE BACKFILL.

APPLICABLE STANDARD DRAWINGS

OPSD–4670.000 TYPICAL JOINT DETAILS

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS			
07–SEP–12	0	JL	ISSUED FOR CONSTRUCTION
DATE	REV.	BY	DESCRIPTION

DESIGN JL	CHK BR	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN DM	CHK MAS/JL	SITE 6–708	DATE 12–JUL–11

DOC: 285380–03–061–WIP1–2808

DATE PLOTTED: 9/7/2012 9:58:04 AM
FILE LOCATION: C:\pwworking\hmmg_285380\karey.thatcher@amec.com\dms03095\285380-03-061-WIP1-2808.dwg
MINISTRY OF TRANSPORTATION, ONTARIO PH–D–707 88–05

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | Hatch Mott MacDonald

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

NEW CONSTRUCTION
HWY 401
GERAEDTS TUNNEL T–8
ABUTMENT LAYOUT II

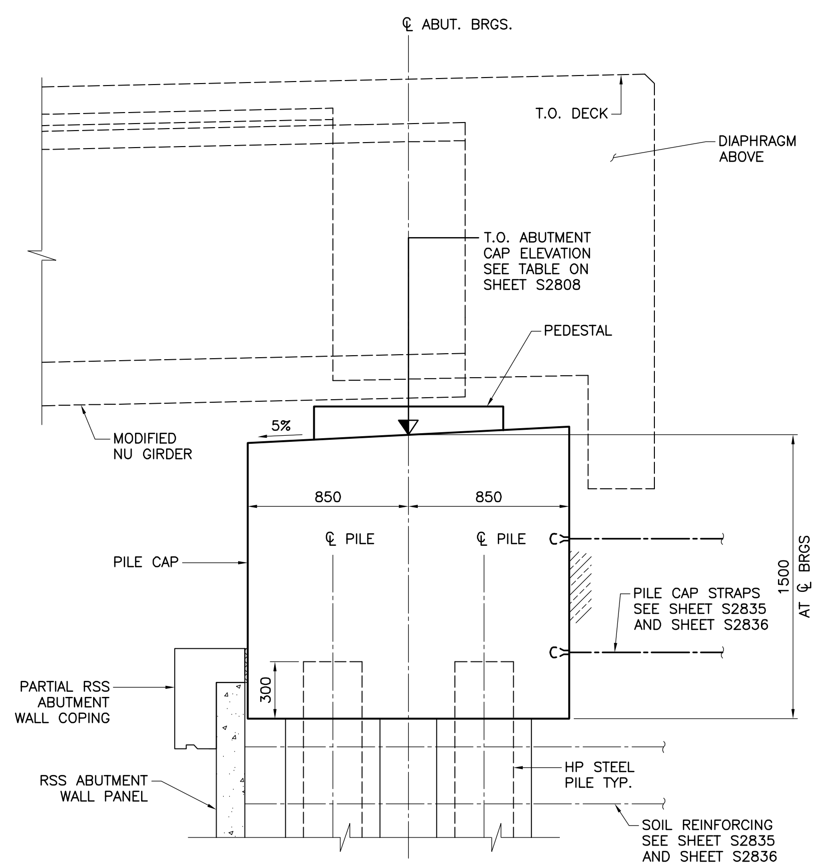
SHEET S2809

Phase 1

IFC

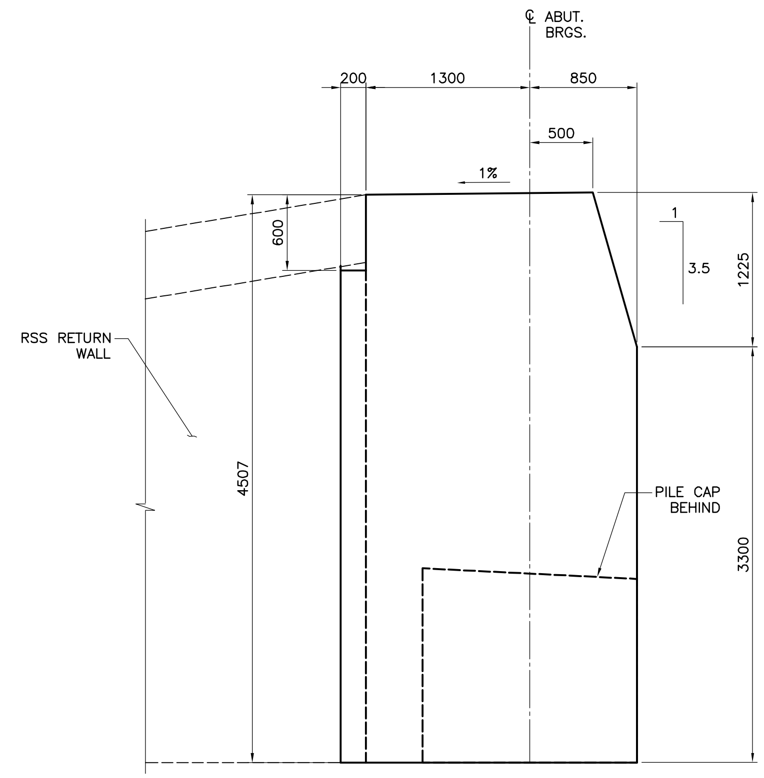
NOTES:

1. FOR BEARING DETAILS SEE SHEET S2817.
2. REINFORCING STRIP CONNECTION DETAIL TO BE DESIGNED BY CONTRACTOR AND RSS WALL SUPPLIER.



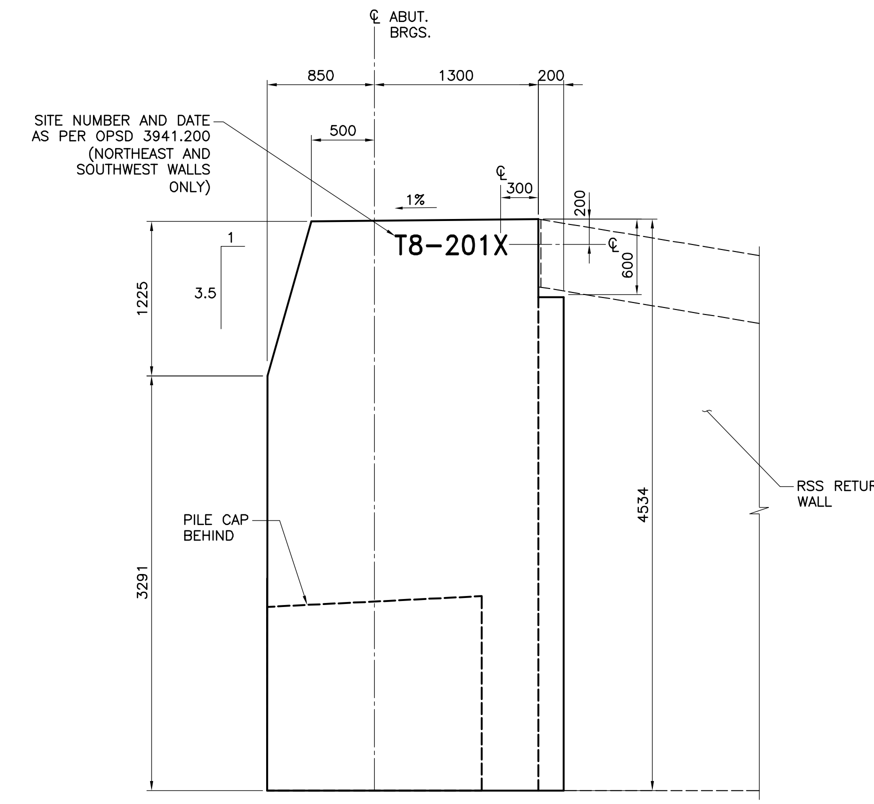
TYPICAL ABUTMENT PILE CAP OUTLINE

1
SCALE 1:20
SHEET S2808



MASKING WALL ELEVATION – NORTH ABUTMENT

2
SCALE 1:30
SHEET S2808



MASKING WALL ELEVATION – SOUTH ABUTMENT

3
SCALE 1:30
SHEET S2808

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS			
07–SEP–12	0	JL	ISSUED FOR CONSTRUCTIN
DATE	REV.	BY	DESCRIPTION

DESIGN JL	CHK BR	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN DM	CHK MAS/JL	SITE 6–708	DATE 12–JUL–11

DOC: 285380–03–061–WIP1–2809

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METRIC

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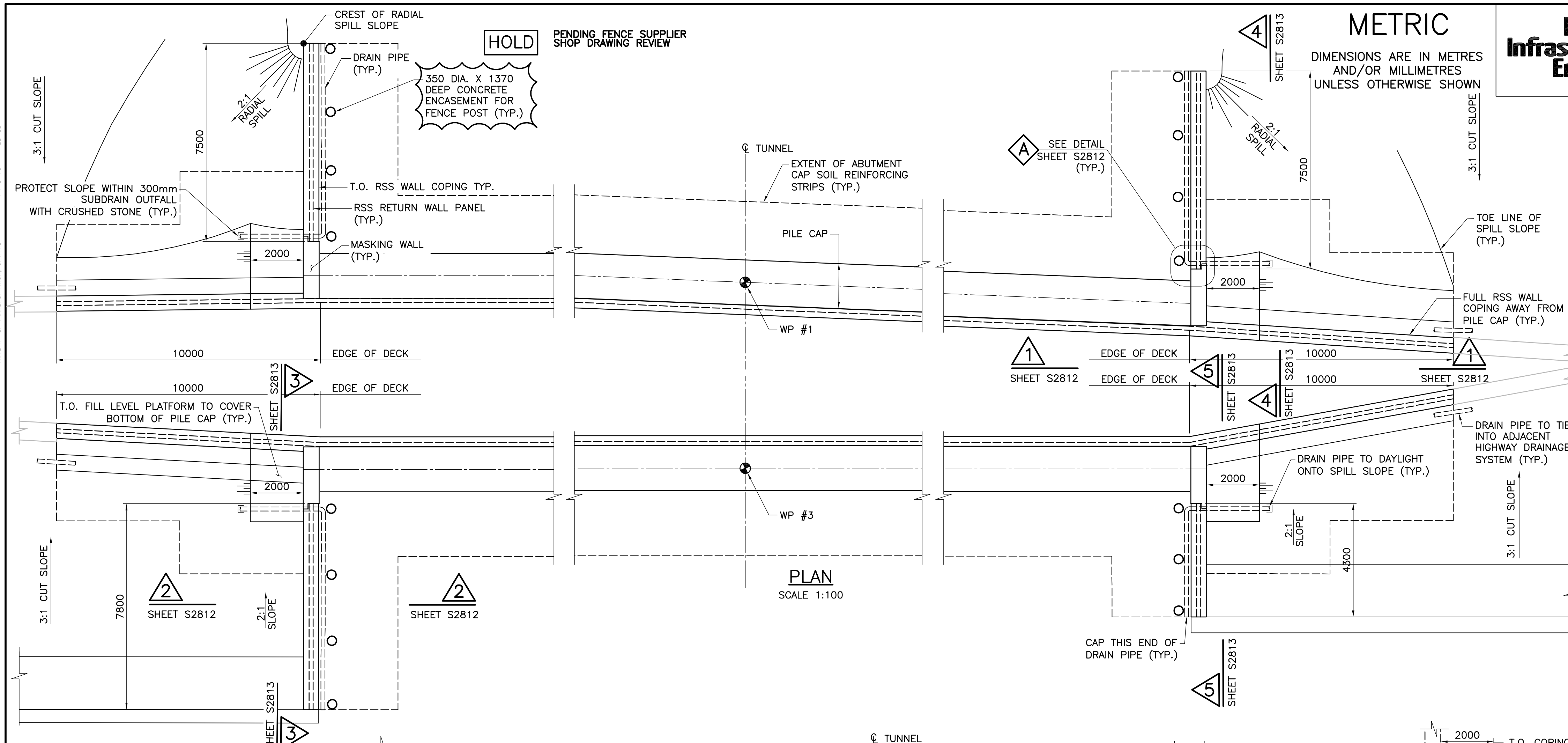
Windsor–Essex Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
GERAEDTS TUNNEL T–8
RSS WALL LAYOUT

SHEET S2811

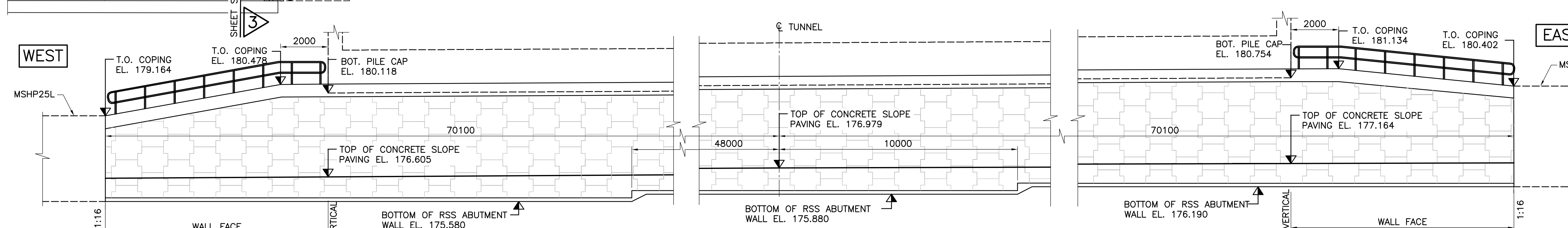
Phase 1

IFC



NOTES:

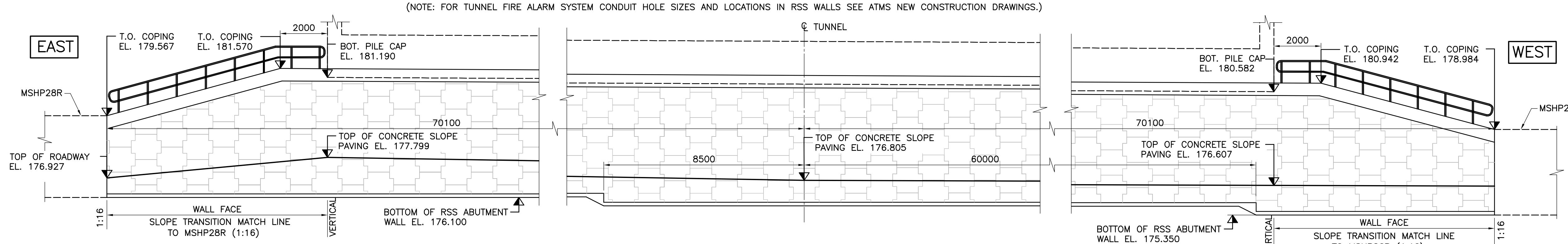
1. READ THIS DRAWING IN CONJUNCTION WITH SHEET S2805 AND S2807 TO S2809.
2. SEE SHEET S2807 FOR SOIL REINFORCING STRIPS NOTES.
3. SEE FOUNDATION INVESTIGATION REPORT FOR AVAILABLE GEOTECHNICAL INFORMATION.
4. CONTRACTOR SHALL REVIEW ANY TEMPORARY WORK RESTRICTIONS PRIOR TO RSS WALL SHOP DRAWING PREPARATION.
5. VERIFY ELEVATIONS AND DIMENSIONS BEFORE PREPARING SHOP DRAWINGS AND NOTIFY DESIGNER IF DISCREPANCIES EXIST.
6. RSS WALL ATTRIBUTES:
   APPLICATION: FALSE ABUTMENT AND RETAINING WALL
   PERFORMENCE: HIGH
   APPEREANCE: HIGH
7. FOR LOCATION OF ELECTRICAL PANELS AND CONDUITS SEE ELECTRICAL WORK DRAWINGS.
8. EPOXY COATED REINFORCEMENT SHALL BE USED IN THE FRONT SURFACE OF RSS PANELS AND ALL RSS COPING FOR ANY WALL WITHIN THE SPLASH ZONE. THIS INCLUDES PANEL SURFACES AND COPING WITHIN 10m OF AN EXISTING OR FUTURE ROADWAY, MEASURED HORIZONTALLY FROM THE EDGE OF PAVEMENT UNLESS THE SURFACE IS MORE THAN 5m ABOVE THE ROADWAY.
9. CONTRACTOR TO INSTALL CSP DURING RSS WALL INSTALLATION AND IS TO BE USED FOR FENCE POST FOOTING FORM.
10. OVER–EXCAVATED AREA TO BE FILLED WITH NON SHRINKABLE MATERIAL.



NORTH RSS WALL ELEVATION

SCALE 1:100

(NOTE: FOR TUNNEL FIRE ALARM SYSTEM CONDUIT HOLE SIZES AND LOCATIONS IN RSS WALLS SEE ATMS NEW CONSTRUCTION DRAWINGS.)



SOUTH RSS WALL ELEVATION

SCALE 1:100

(NOTE: FOR TUNNEL FIRE ALARM SYSTEM CONDUIT HOLE SIZES AND LOCATIONS IN RSS WALLS SEE ATMS NEW CONSTRUCTION DRAWINGS.)

DRAWING NOT TO BE SCALED 100mm ON ORIGINAL DRAWING

REVISIONS

DATE	REV.	BY	DESCRIPTION
07–SEP–12	0	JL	ISSUED FOR CONSTRUCTION

DESIGN JL	CHK BR	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN DM	CHK MAS/JL	SITE 6–708	DATE 12–JUL–11

DOC: 285380–03–061–WIP1–2811

DATE PLOTTED: 9/7/2012 10:00:48 AM
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METRIC

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

NEW CONSTRUCTION
HWY 401
GERAEDTS TUNNEL T–8
RSS WALL DETAILS

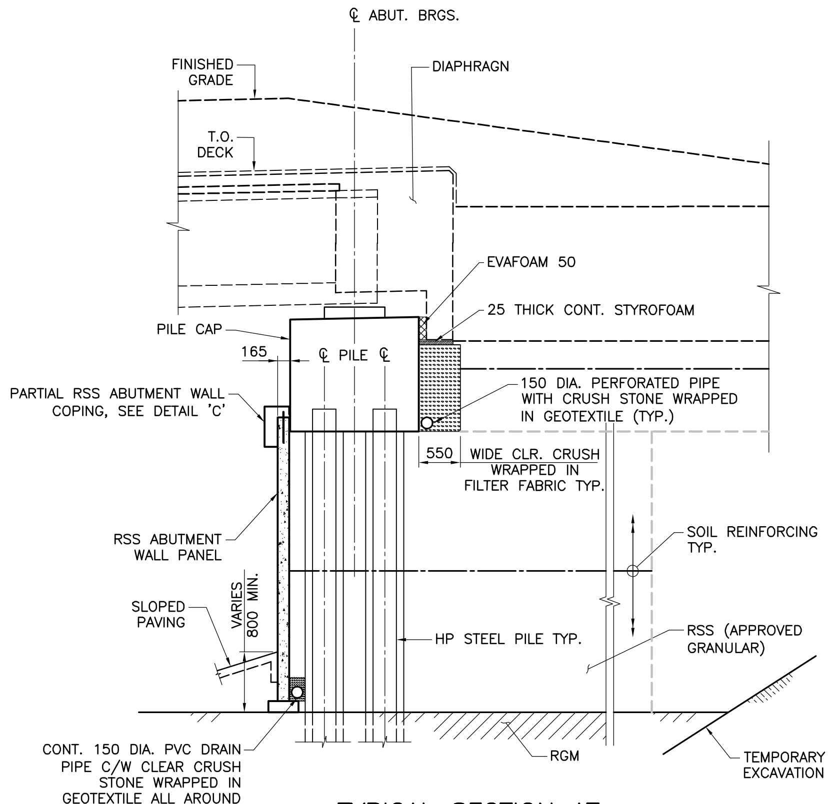
SHEET S2812

Phase 1

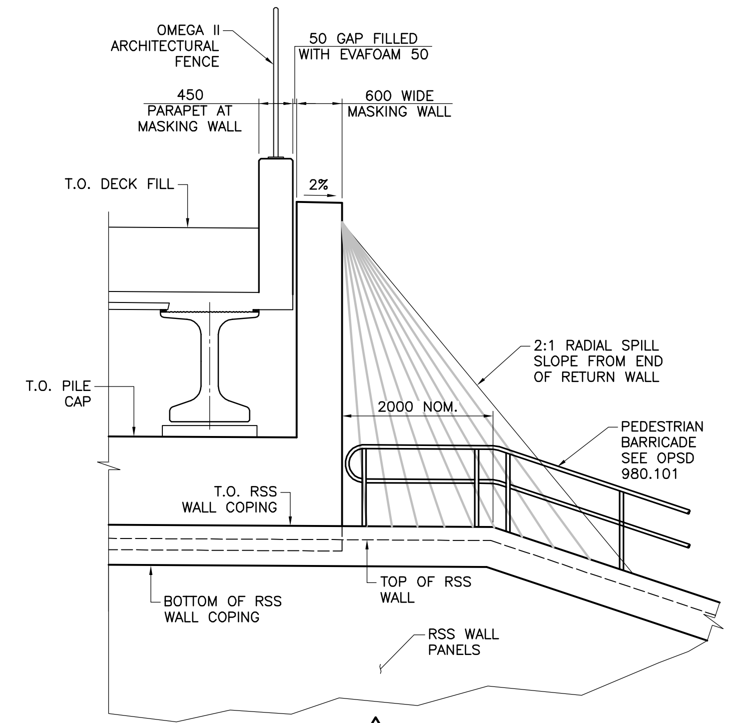
IFC

NOTES:

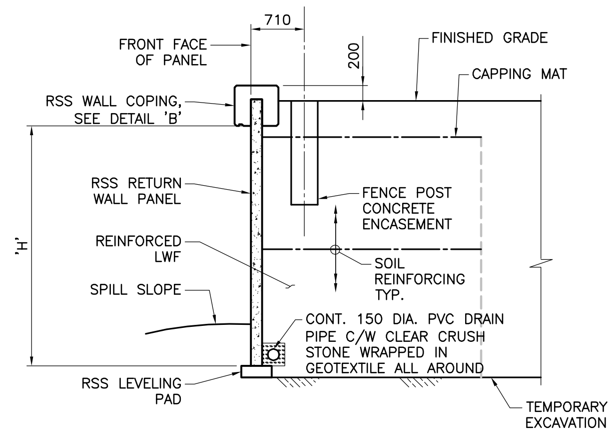
1. READ THIS DRAWING IN CONJUNCTION WITH SHEET S2811.
2. THE MIN. 600mm DEPTH FOR RSS WALL LEVELING PAD TO BE CONFIRMED BY RSS MANUFACTURER.
3. FOR BACKFILL DETAILS IN FRONT OF THE HARDWARE CLOTH WIRE AND TIE IN DETAILS OF THE WIRE ONTO CORRIDOR FENCE REFER TO ECOLOGICAL LANDSCAPE DRAWING PACKAGE.
4. REFER TO LANDSCAPE CONSTRUCTION DRAWINGS AND HIGHWAY NEW CONSTRUCTION DRAWINGS FOR FENCING DETAILS AND LAYOUT, INCLUDING RIGHT OF WAY FENCE, SECURITY FENCE, NOISE WALLS, LANDSCAPE AND TRAIL BARRIERS.



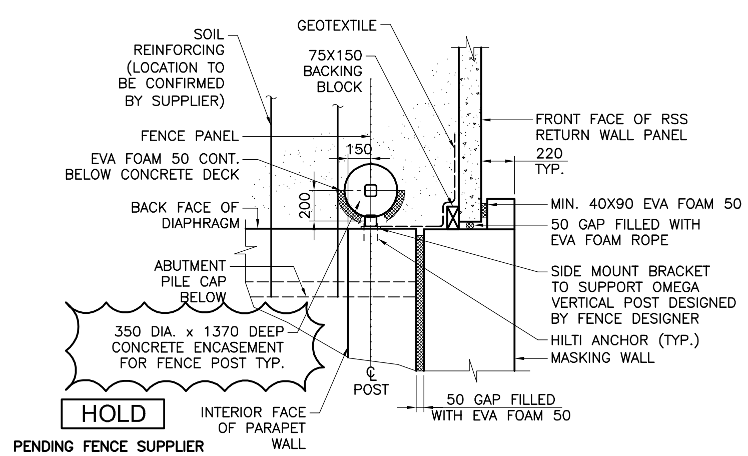
TYPICAL SECTION AT ABUTMENT DIAPHRAGM
SCALE 1:50



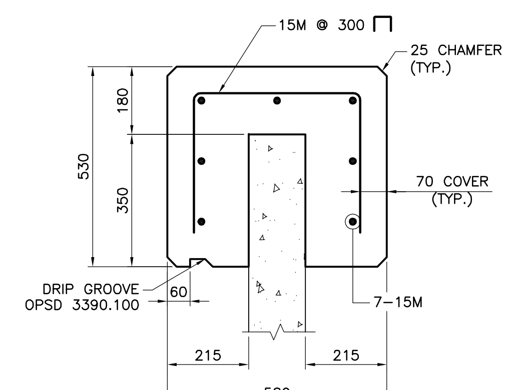
1
SCALE 1:50
SHEET S2811



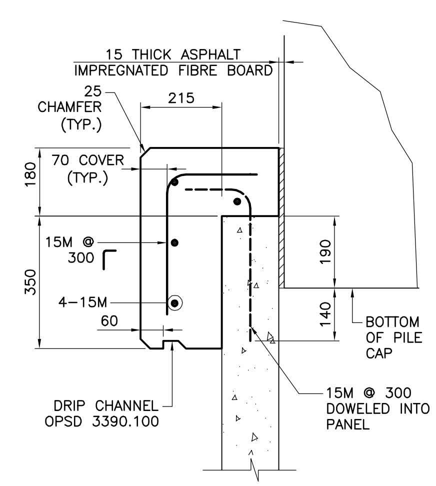
2
SCALE 1:50
SHEET S2811



A
SCALE 1:25
SHEET S2811



B
SCALE 1:10



C
SCALE 1:10

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS			
07–SEP–12	0	JL	ISSUED FOR CONSTRUCTION
DATE	REV.	BY	DESCRIPTION

DESIGN JL	CHK BR	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN DM	CHK MAS/JL	SITE 6–708	DATE 26–AUG–11

DOC: 285380–03–061–WIP1–2812

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METRIC

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Parkway Infrastructure Engineers | Hatch Mott MacDonald

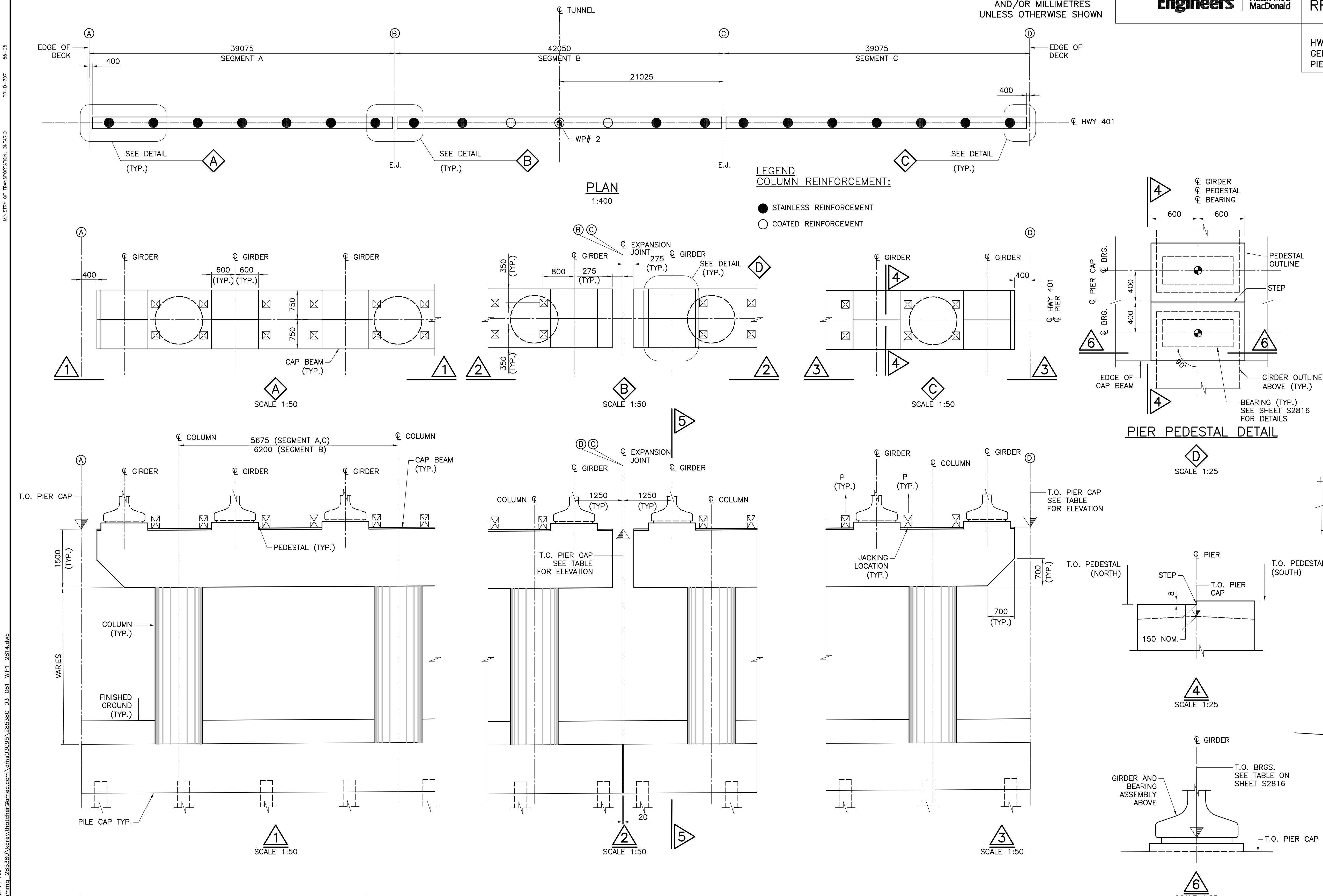
Windsor–Essex Parkway Project RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
GERAEDTS TUNNEL T-8
PIER LAYOUT

SHEET S2814

Phase 1

IFC



NOTES:

1. FOR SEGMENTAL LAYOUT SEE SHEET S2801.
2. MAXIMUM JACKING LOAD 2P=2567KN PER GIRDER LINE FOR JACKING OF GIRDERS AT PIER.

CONSTRUCTION SEQUENCE:

1. CONSTRUCT COLUMNS.
2. CONSTRUCT PIER CAPS TO UNDERSIDE OF BEARING PEDESTALS.
3. CONSTRUCT BEARING PEDESTALS.

TOP OF PIER CAP ELEVATION

	GRID A	GRID B	GRID C	GRID D
TOP OF PIER CAP	181.882	182.077	182.288	182.483

DRAWING NOT TO BE SCALED 100mm ON ORIGINAL DRAWING

DATE	REV.	BY	DESCRIPTION
07-SEP-12	0	JL	ISSUED FOR CONSTRUCTION

REVISIONS

DESIGN JL | CHK BR | CODE CAN/CSA S6-06 | LOAD CL-625-ONT
DRAWN DM | CHK MAS/JL | SITE 6-708 | DATE 12-JUL-11

DOC: 285380-03-061-WIP1-2814

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METRIC

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

REVISIONS				
22–AUG–12	0	GN	ISSUED FOR CONSTRUCTION	
DATE	REV.	BY	DESCRIPTION	
DESIGN NR	APR DD	DATE	15–JUL–11	

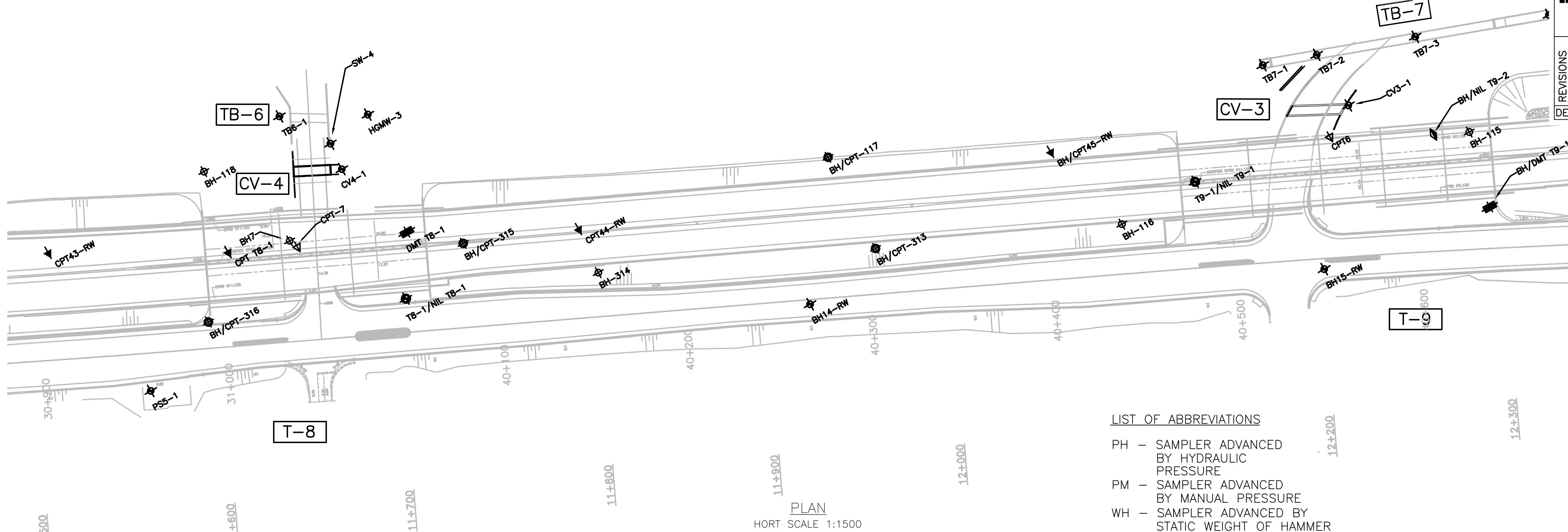
LOCATION PLAN & INTERPRETED STRATIGRAPHIC PROFILE

STA 11+500L TO STA 12+300L

SHEET G2802

Phase 1

IFC



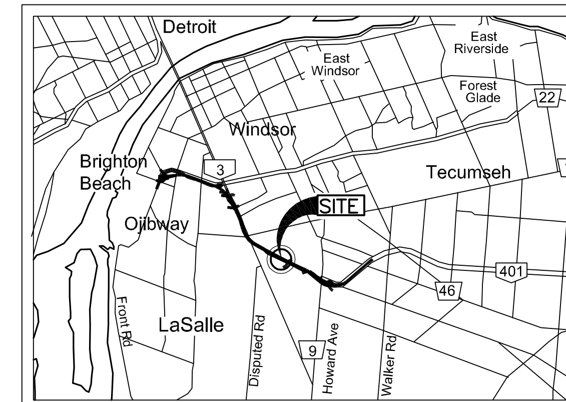
LIST OF ABBREVIATIONS

PH – SAMPLER ADVANCED BY HYDRAULIC PRESSURE

PM – SAMPLER ADVANCED BY MANUAL PRESSURE

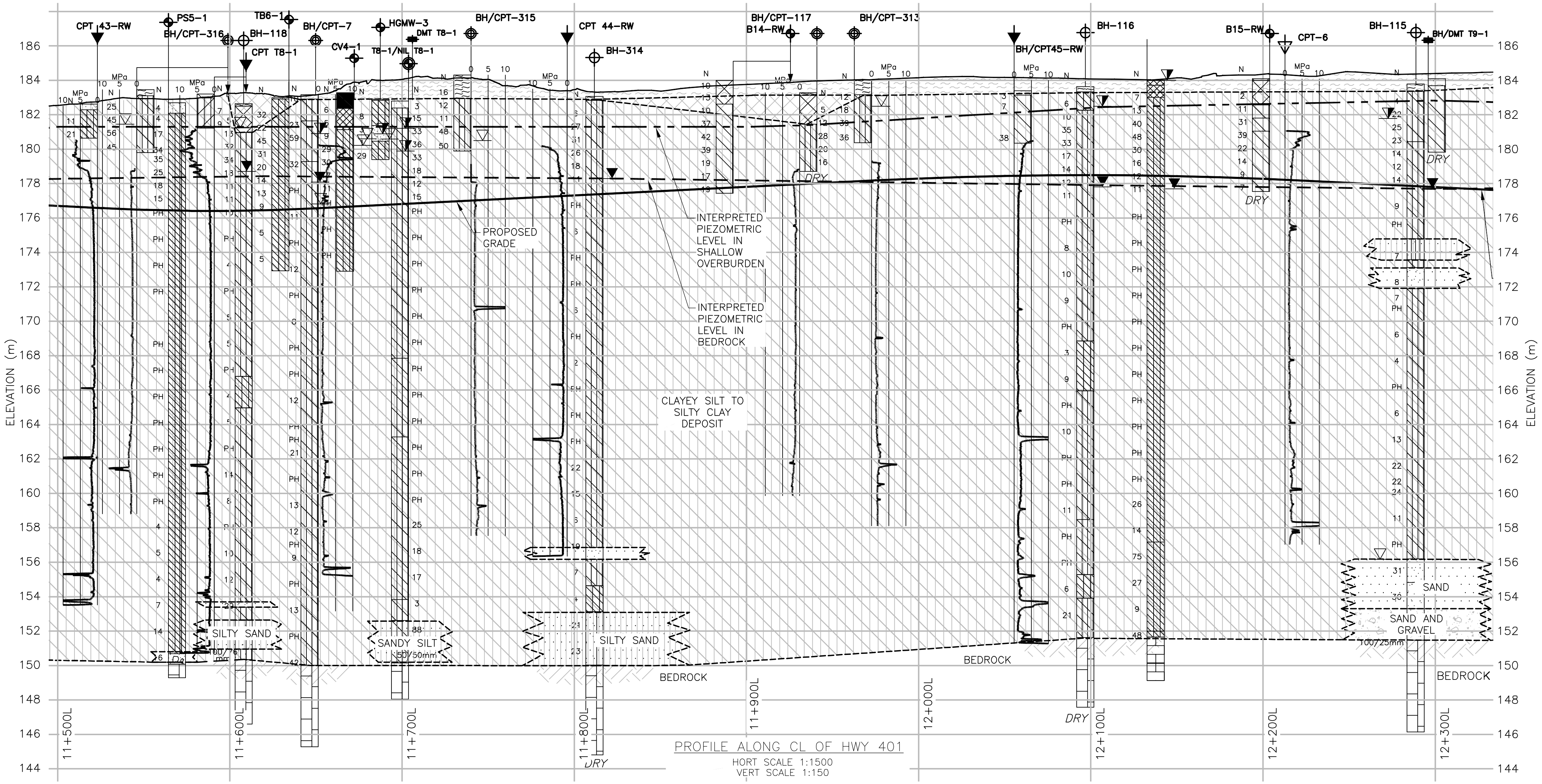
WH – SAMPLER ADVANCED BY STATIC WEIGHT OF HAMMER

WR – SAMPLER ADVANCED BY WEIGHT OF SAMPLER RODS



KEY PLAN

SCALE 1 0 2 4Km



PROFILE ALONG CL OF HWY 401

HORT SCALE 1:1500
VERT SCALE 1:150

LEGEND

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

NOTES

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

SCALES

15 0 30m Horizontal

1.5 0 3 Vertical

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

DATE PLOTTED: 9/7/2012 10:06:28 AM
FILE LOCATION: C:\pwworking\hmmg_285380\karey.thatcher@amec.com\dms09855\285380-04-090-WIP1-2802.dwg

DOC: 285380-04-090-WIP1-2802

METRIC

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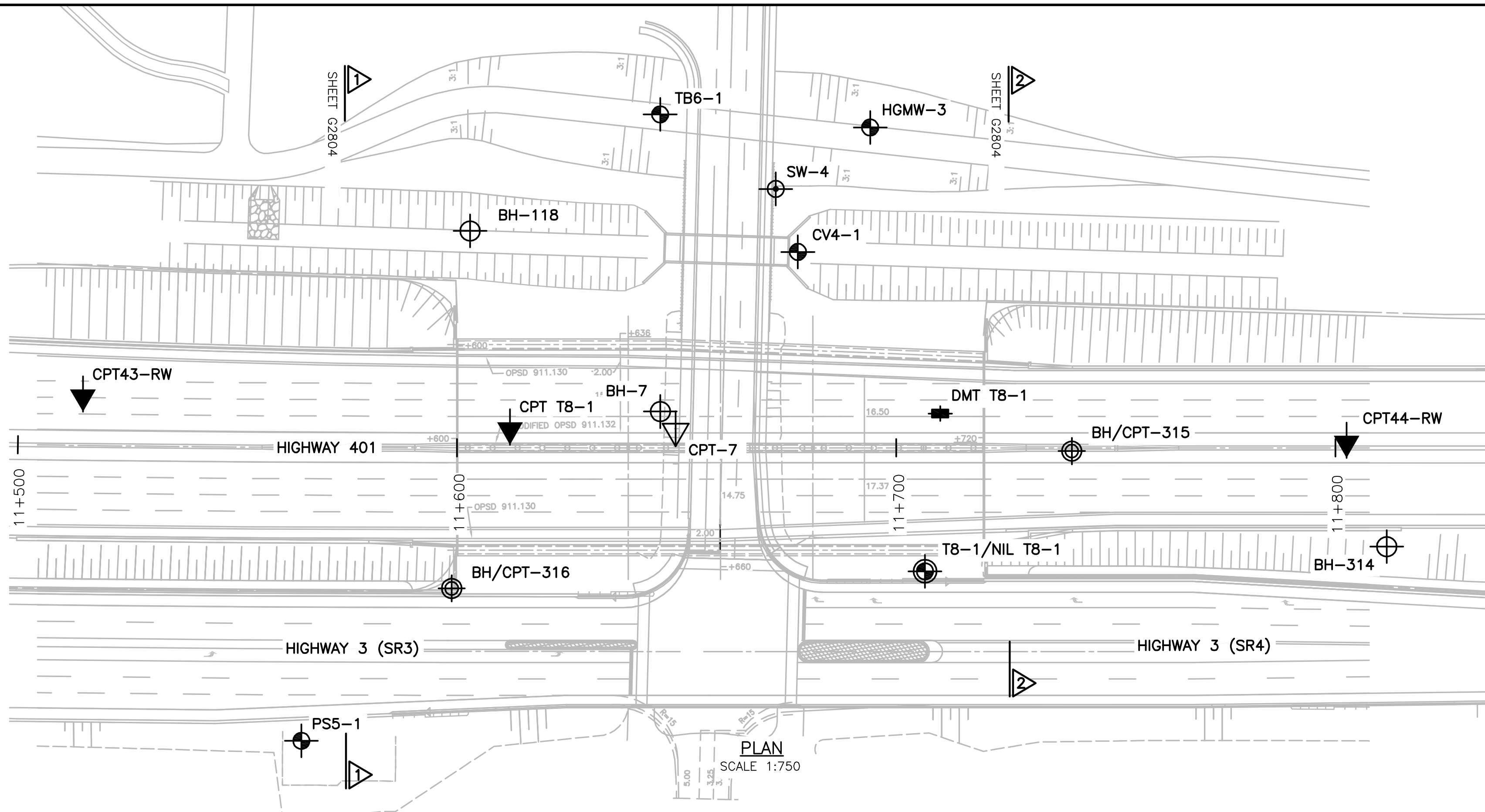
Windsor–Essex Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
GERAEDTS TUNNEL T–8
BOREHOLE LOCATIONS & SOIL STRATA

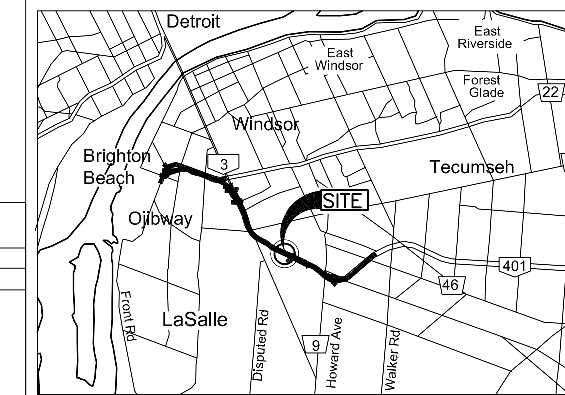
SHEET G2803

Phase 1

IFC



No.	ELEVATION	CO–ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC BOREHOLES			
T8–1/NIL T8–1	182.8	4678789.7	333364.5
CPT T8–1	183.2	4678860.0	333292.9
DMT T8–1	183.0	4678820.9	333382.7
CV4–1	183.3	4678867.9	333368.7
CPT 43–RW	182.6	4678907.6	333207.7
CPT 44–RW	183.1	4678777.0	333464.0
HGMW–3	182.9	4678886.8	333395.5
TB6–1	183.0	4678909.5	333353.3
PS5–1	182.8	4678814.1	333219.4
PREVIOUS BOREHOLES			
BH–7	183.17	4678848.0	333325.0
BH–118	182.66	4678903.5	333302.9
BH–314	183.07	4678750.8	333462.3
BH/CPT–315	184.31	4678800.6	333406.3
BH/CPT–316	182.99	4678831.3	333265.0
CPT–7	183.18	4678844.0	333327.0



KEY PLAN
SCALE
1 0 2 4Km

LEGEND

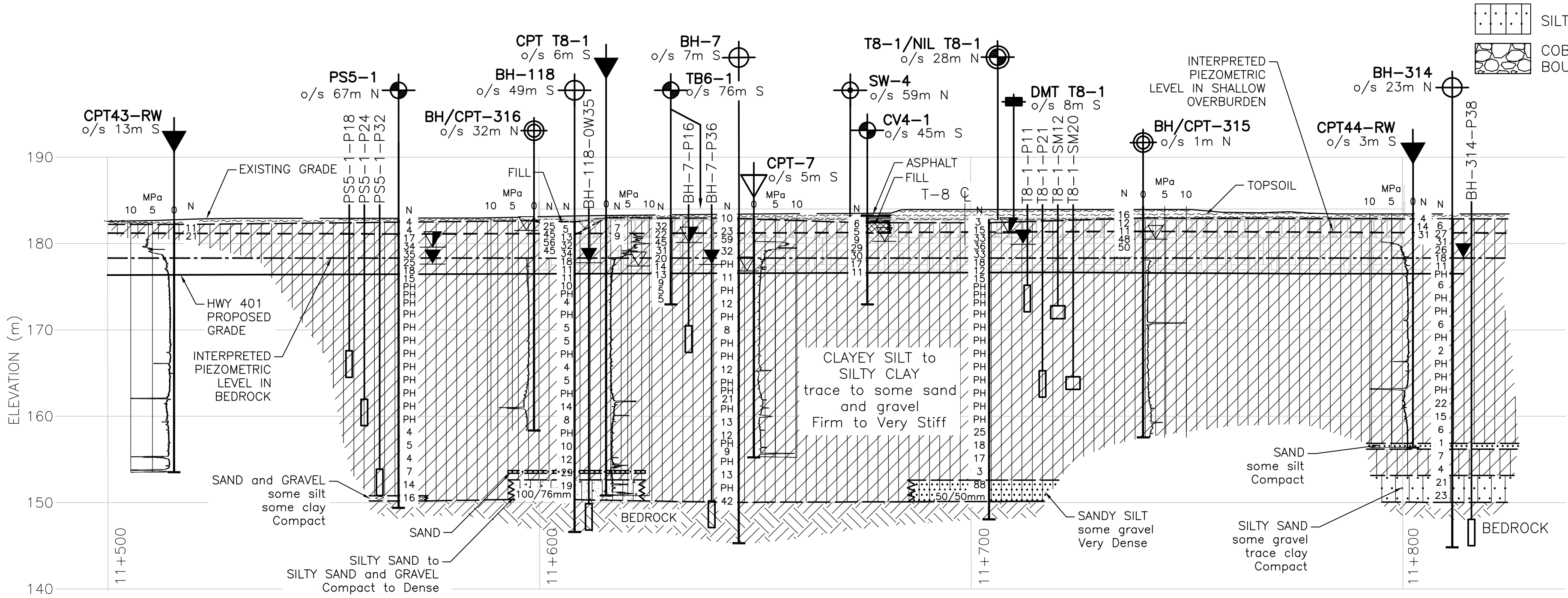
- BOREHOLE CURRENT INVESTIGATION
- BOREHOLE AND NILCON VANE CURRENT INVESTIGATION
- SW/SP HOLE (HYDROGEOLOGY) CURRENT INVESTIGATION
- NILCON VANE CURRENT INVESTIGATION
- CPT – CURRENT INVESTIGATION
- DMT – CURRENT INVESTIGATION
- BOREHOLE PREVIOUS INVESTIGATION
- BOREHOLE, CPT AND NILCON VANE PREVIOUS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

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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



PROFILE ALONG CL OF HIGHWAY 401
HORT SCALE 1:750
VERT SCALE 1:375

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOTES

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

REVISIONS

DATE	REV.	BY	DESCRIPTION
07–SEP–12	0	GN	ISSUED FOR CONSTRUCTION

DESIGN JF	CHK JF	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN MM	CHK NSV	SITE 6–708	DATE 30–SEP–11

DOC: 285380–04–090–WIP1–2803

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

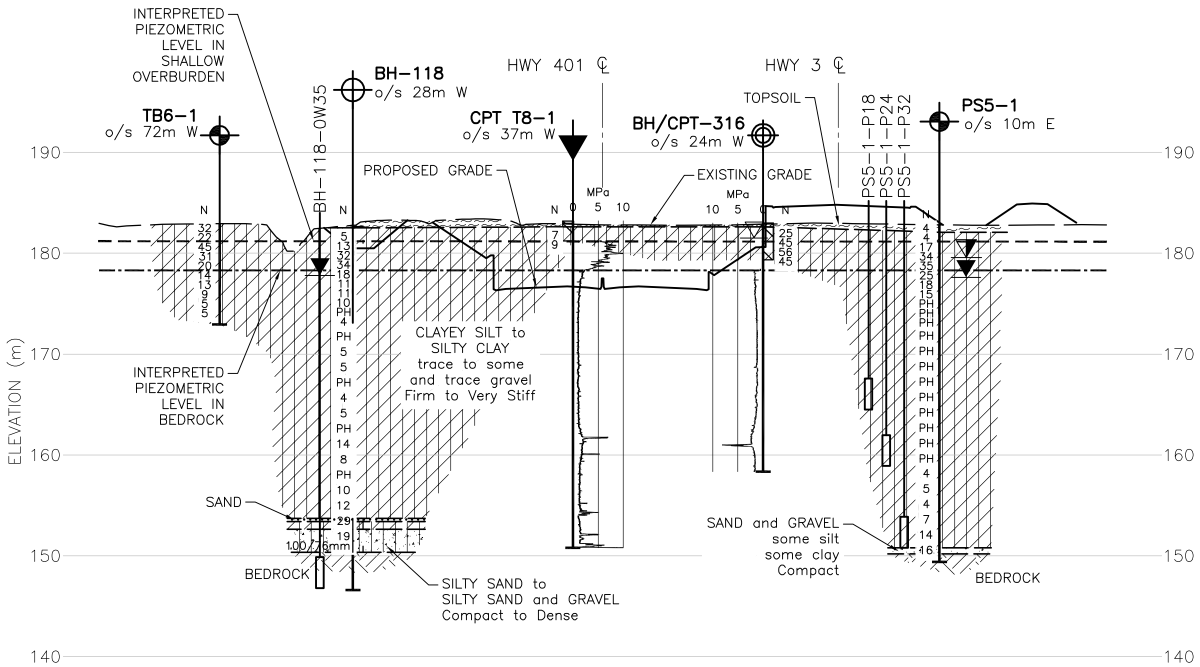
Windsor–Essex Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
GERAEDTS TUNNEL T–8
SOIL STRATIGRAPHY

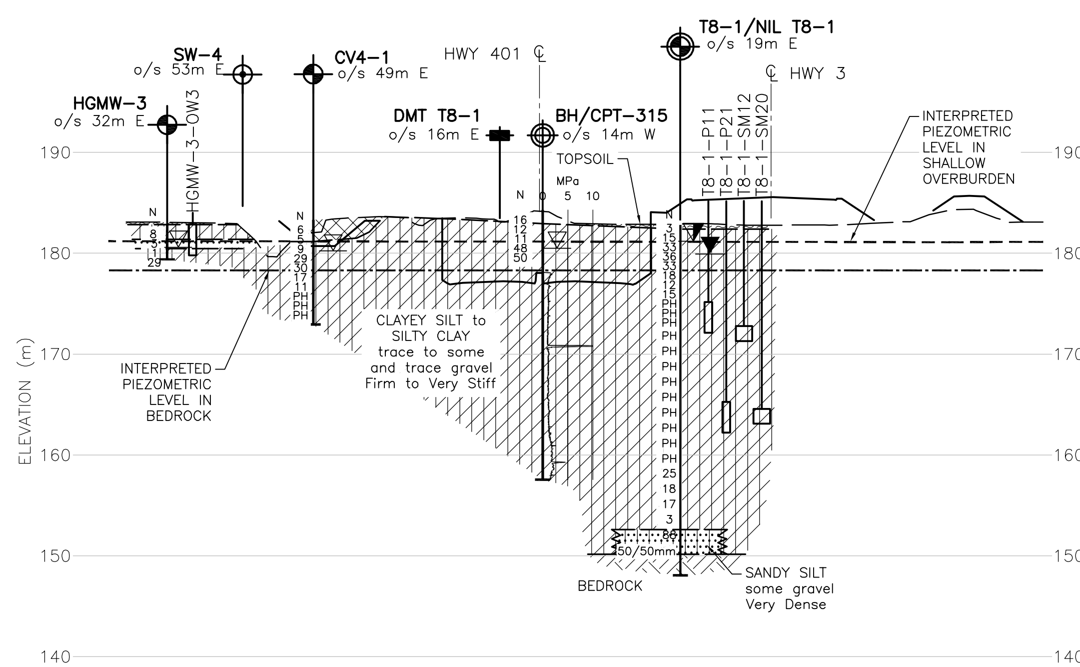
SHEET G2804

Phase 1

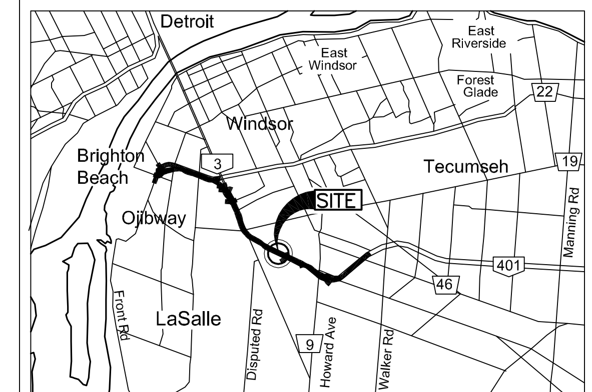
IFC



1
HORT SCALE 1:750
VERT SCALE 1:375



2
HORT SCALE 1:750
VERT SCALE 1:375



KEY PLAN
SCALE
1 0 2 4Km

LEGEND

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

NOTES

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

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

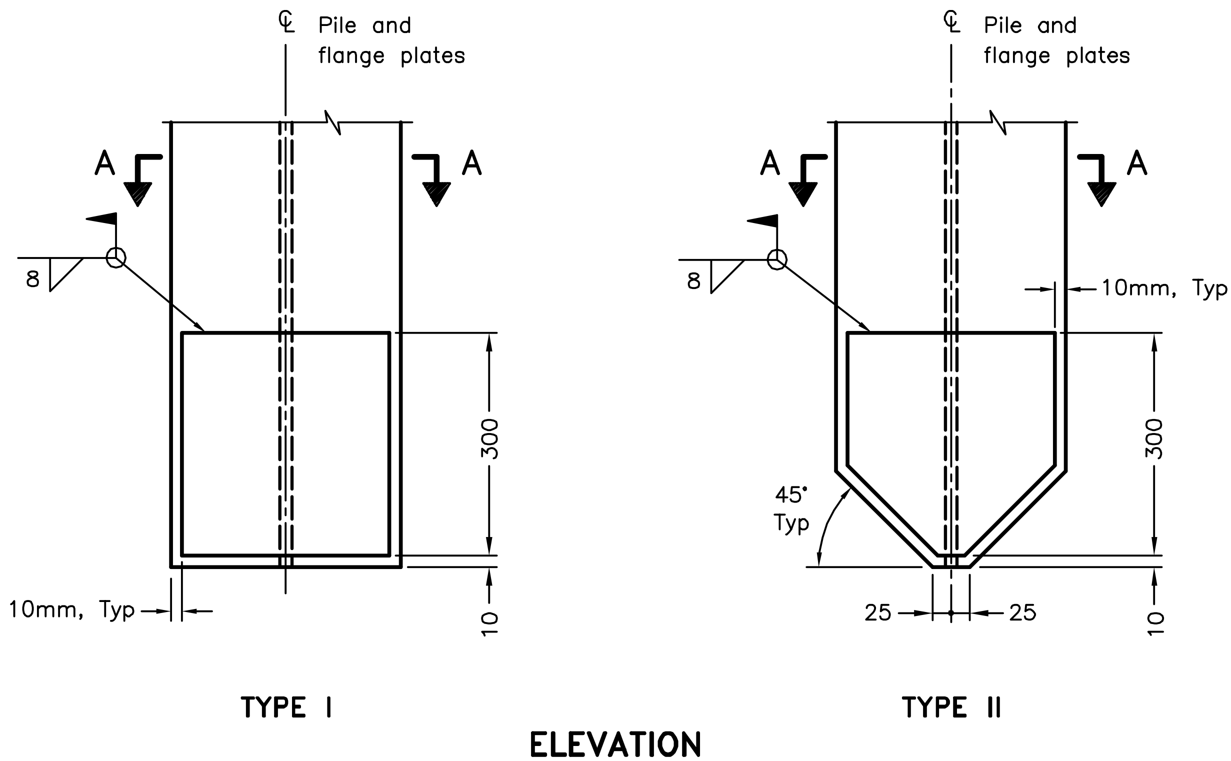
DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

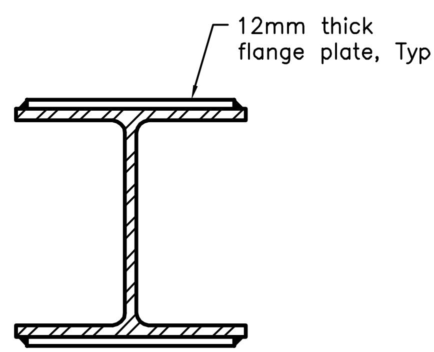
REVISIONS			
07–SEP–12	0	GN	ISSUED FOR CONSTRUCTION
DATE	REV.	BY	DESCRIPTION

DESIGN JF	CHK JF	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN MM	CHK NSV	SITE 6–708	DATE 30–SEP–11

DOC: 285380–04–091–WIP1–2804

Applicable OPSDs





PILE DRIVING SHOE

SECTION A–A

NOTES:

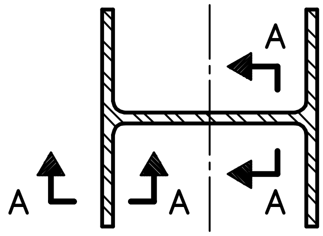
A Flange plates shall be according to CSA G40.20/G40.21, Grade 300W.

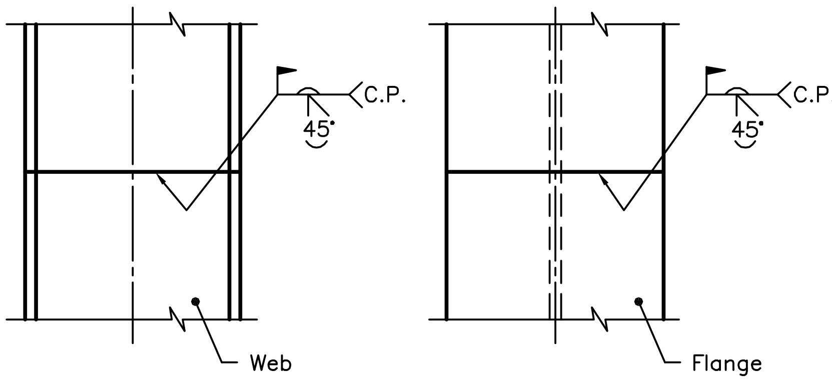
B Welding shall be according to CSA W59.

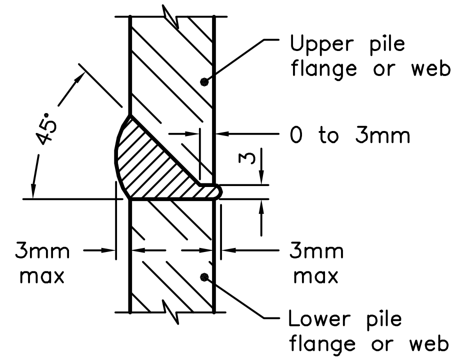
C Driving shoe Type I shall be used unless Type II is specified.

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	

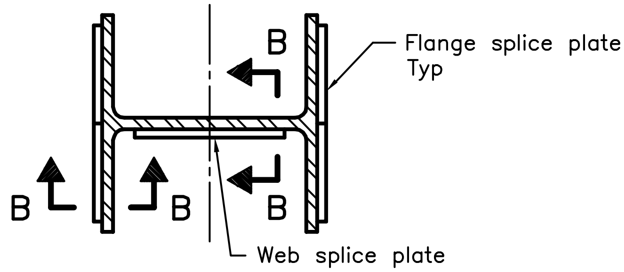


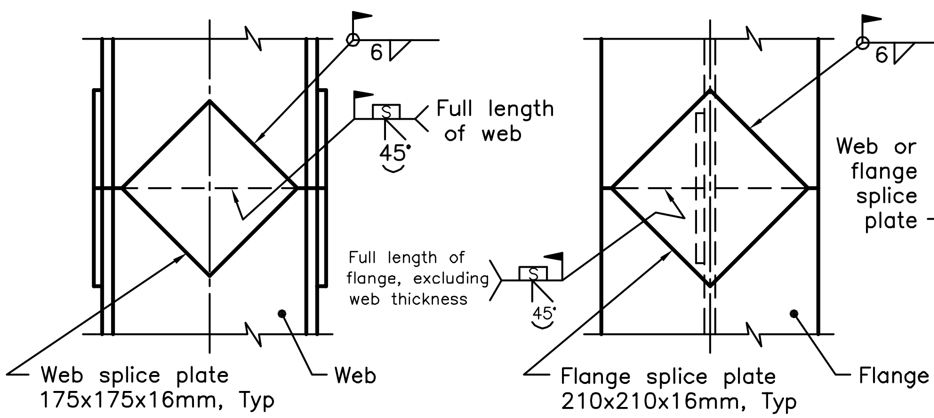


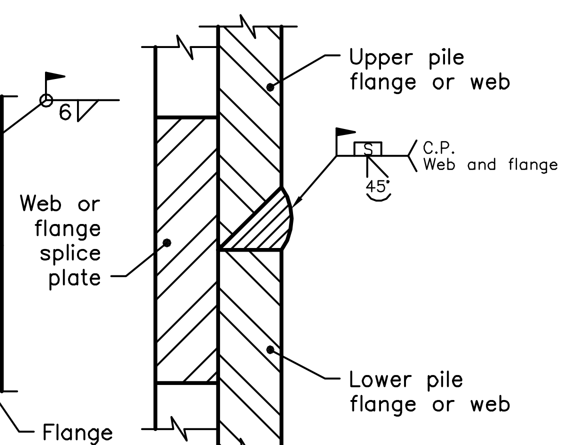


BUTT WELD

SECTION A–A







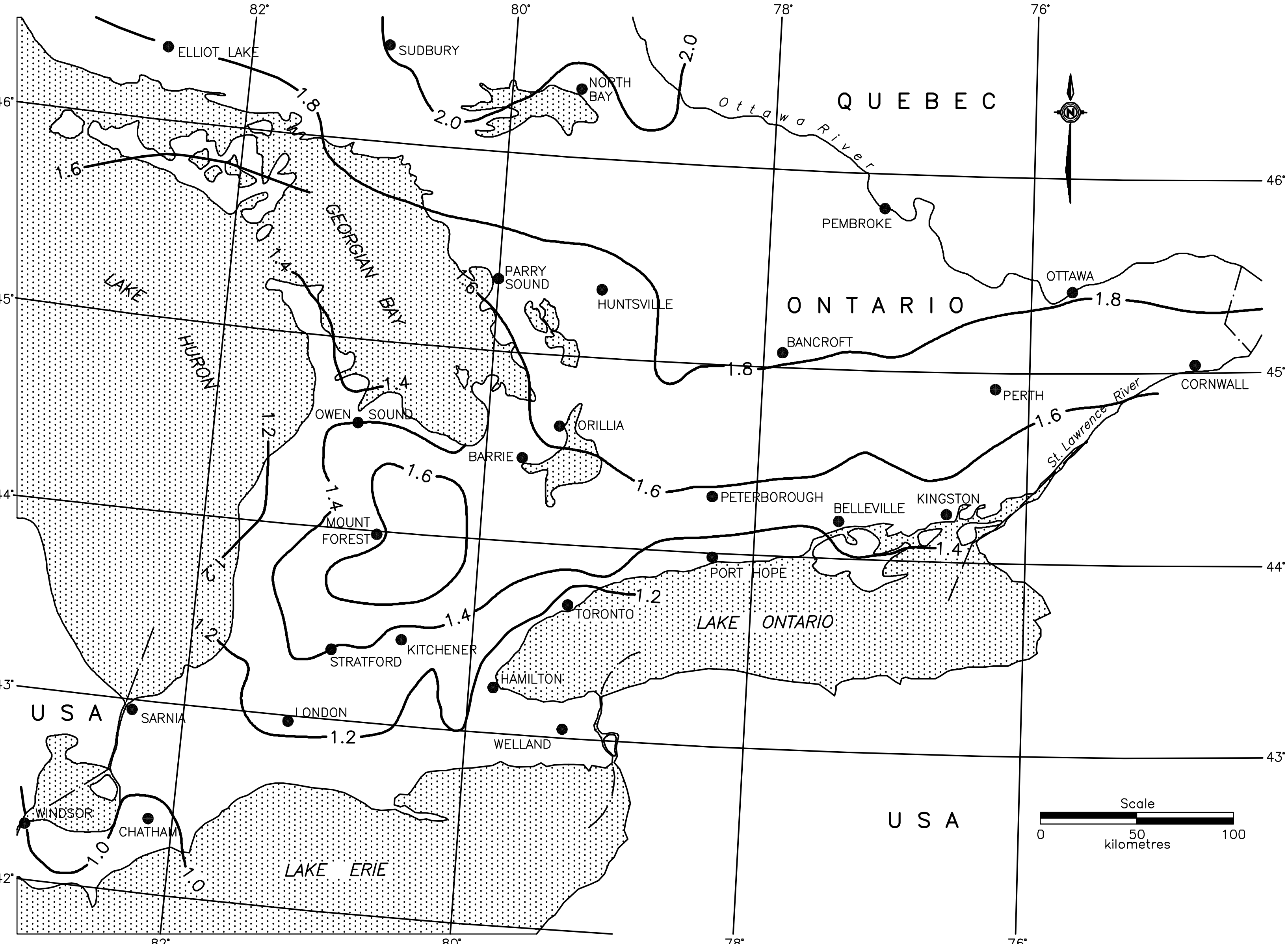
BUTT WELD WITH SPLICE PLATES

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.

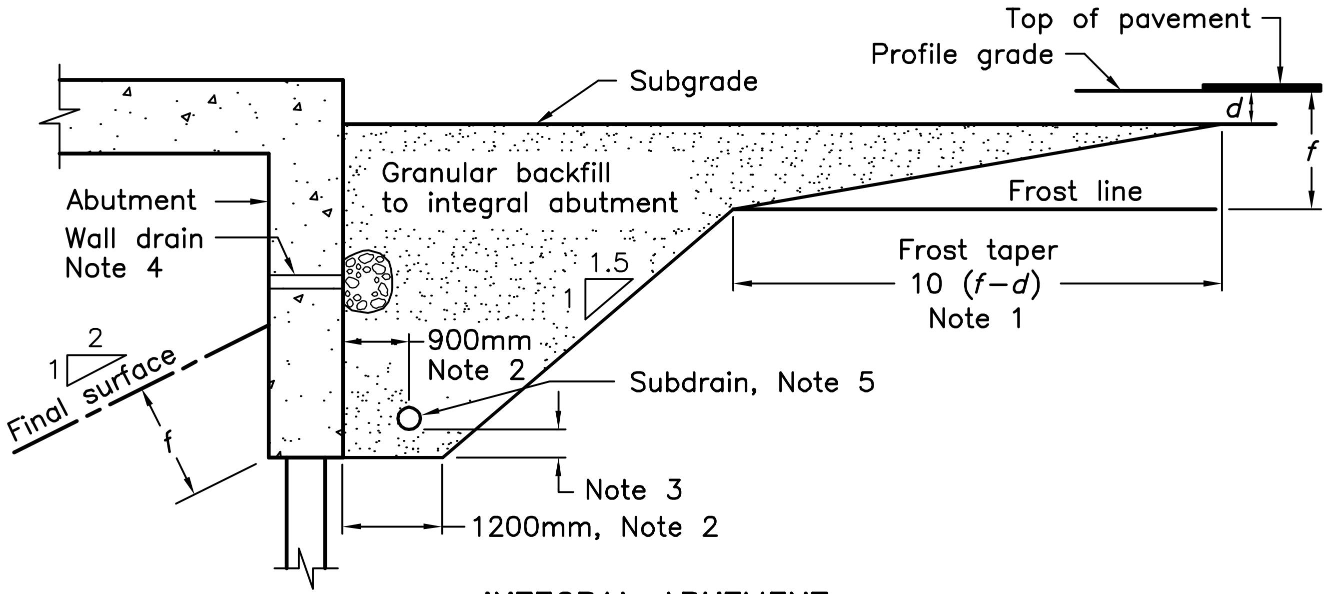
D Frost penetration depths are in metres.

ONTARIO PROVINCIAL STANDARD DRAWING

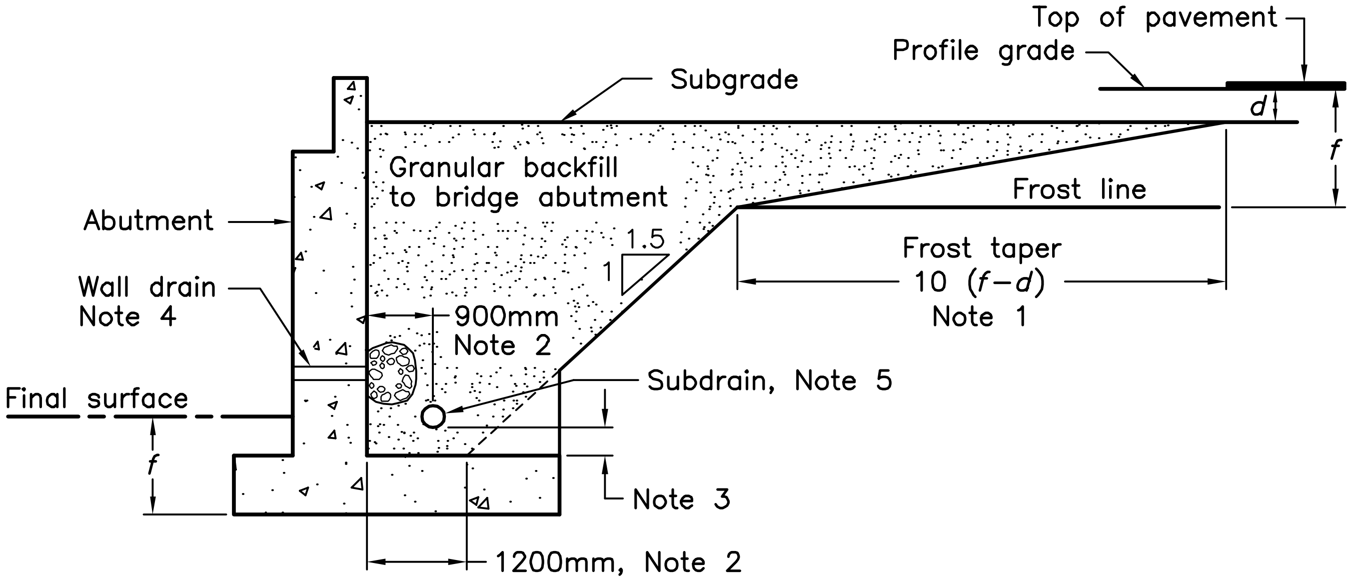
Nov 2010 | Rev 1

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
(Figure 5.1, Ladd & DeGroot, 2004, ref. R-29)

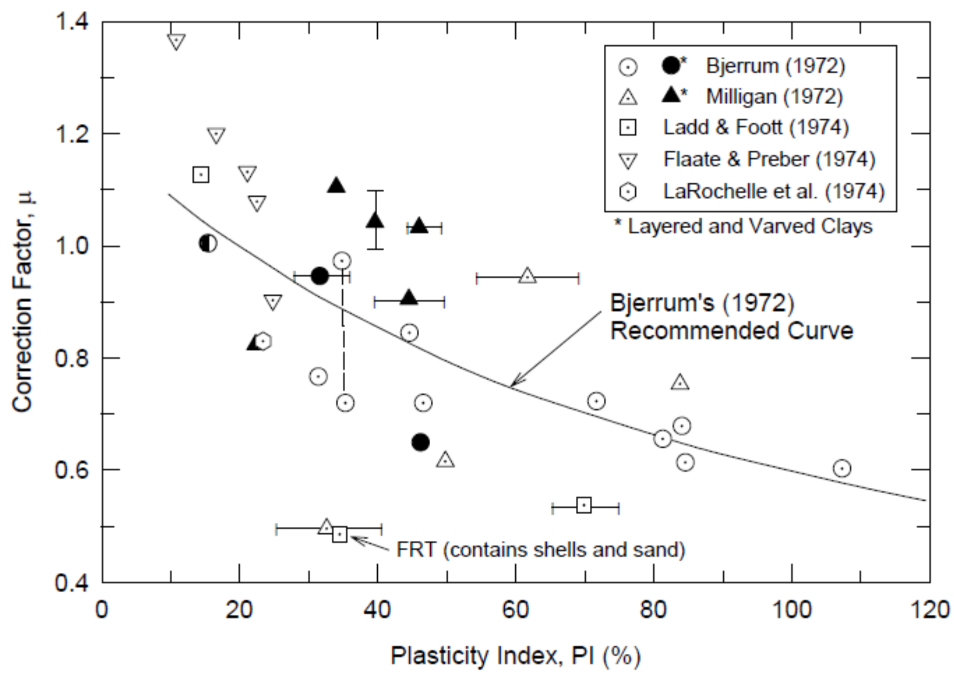
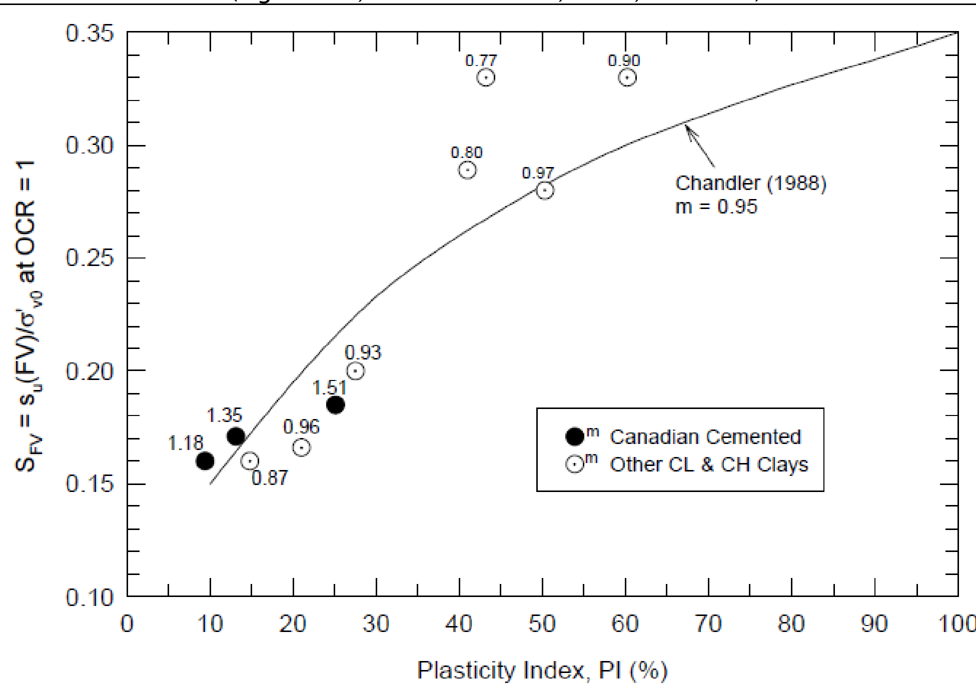


Figure 3-2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays
(Figure 5.2, Ladd & DeGroot, 2004, ref. R-29)



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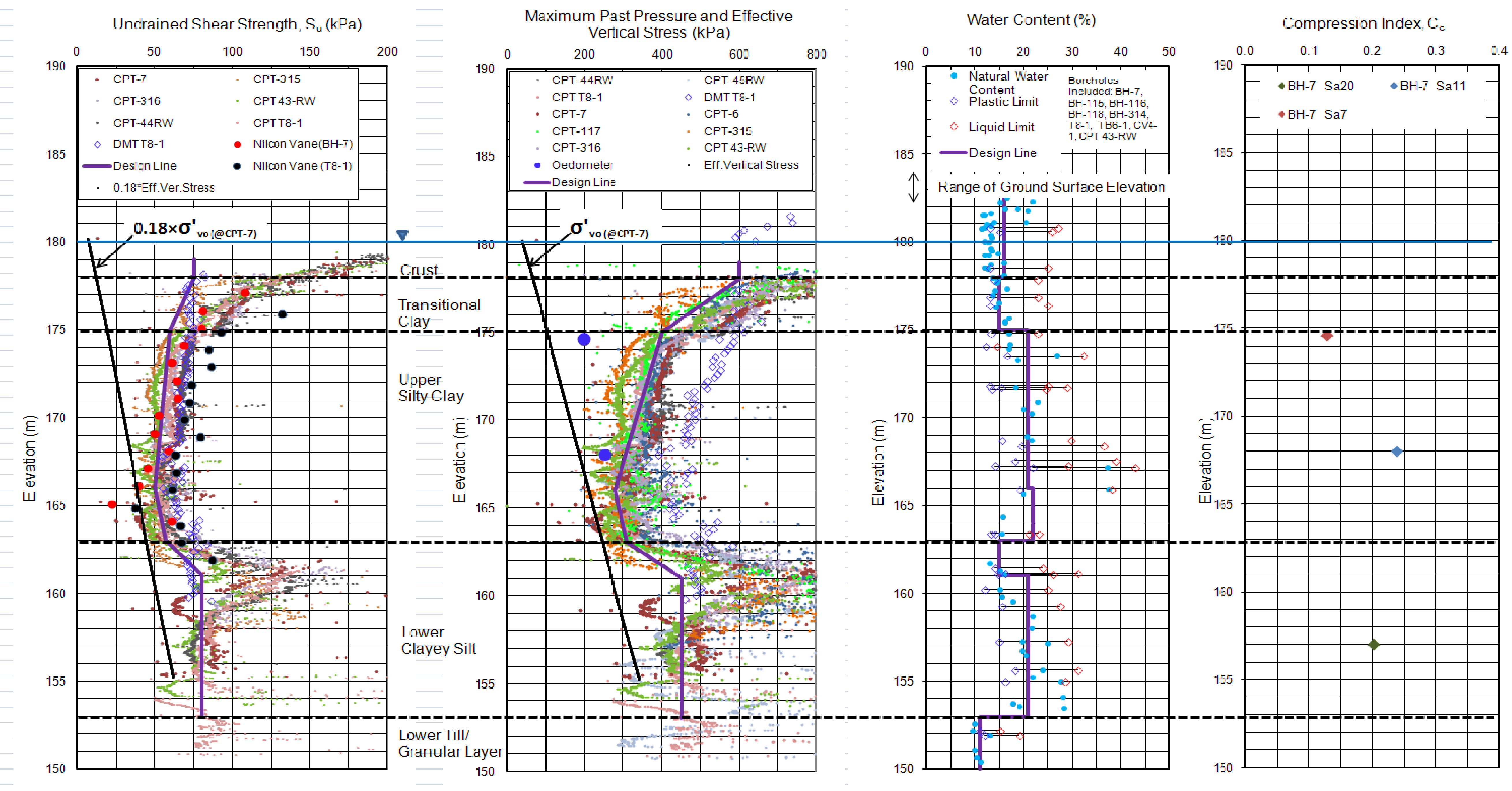
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

Figure 3-3: Soil Property Profiles for Tunnel T-8



Notes:

1. Shear strength profiles were estimated from CPT data using the equation $S_u = (q_t - \sigma_{VO}) / N_{KT}$. The cone factor N_{KT} was estimated by comparing the CPT profiles with a nearby Nilcon Vane profile.
2. Maximum past pressure profiles estimated using SHANSEP method. $OCR = [(S_u/\sigma'_V)/S]^{1/m}$
3. Historical boreholes and tests by Golder (ref R-17 and R-19)

amec Earth & Environmental	PROJECT: WINDSOR ESSEX PARKWAY
CLIENT:	TITLE: SOIL PROPERTIES PROFILES TUNNEL T-8
	DATE: Jul 2012 · JOB NO.: SW8801.1002 · CAD FILE: · FIGURE NO.: · REV.:

Figure 4-1: Compressibility Parameters at WEP

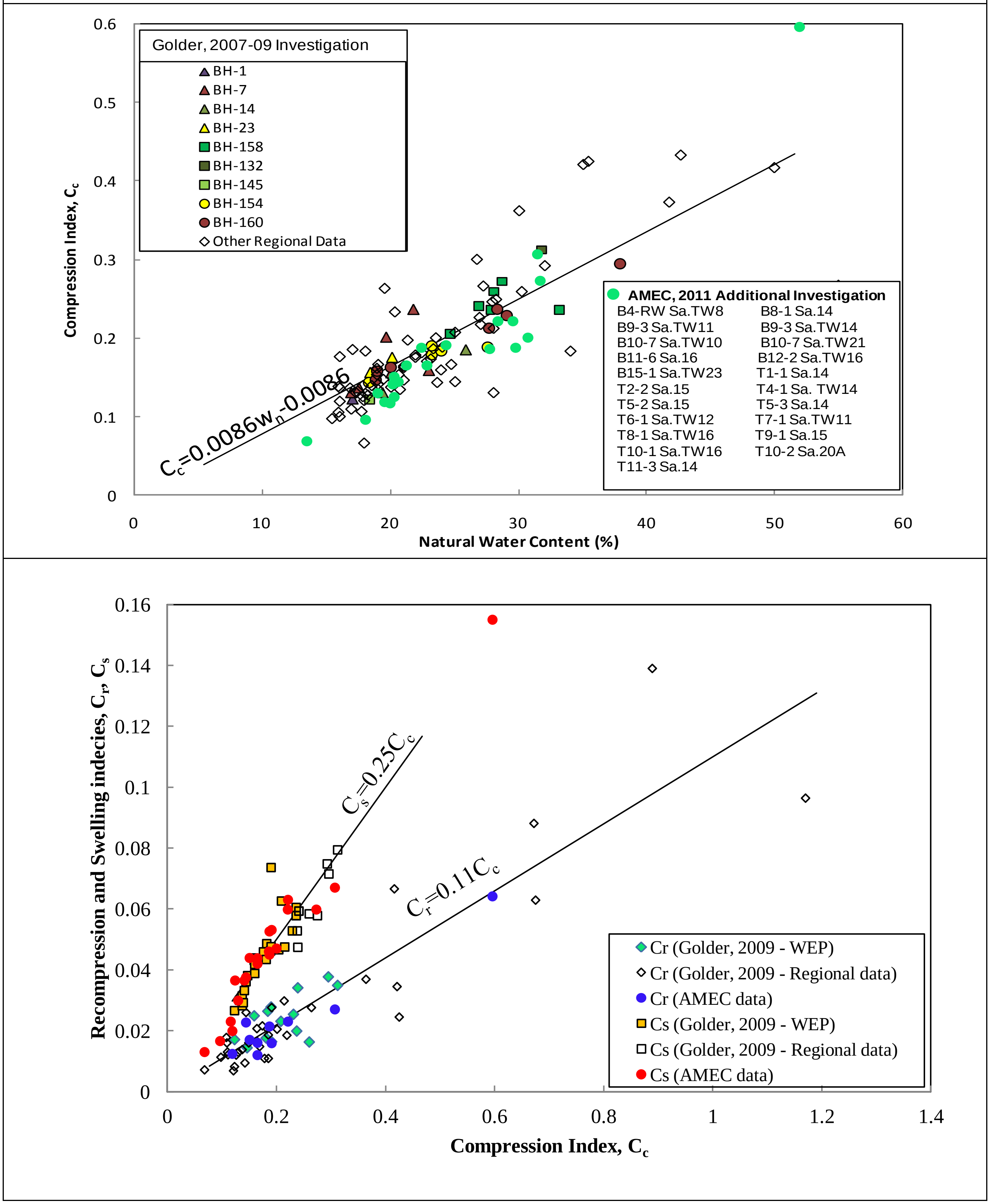


Figure 4-2: Cc versus Cα Relationship at WEP

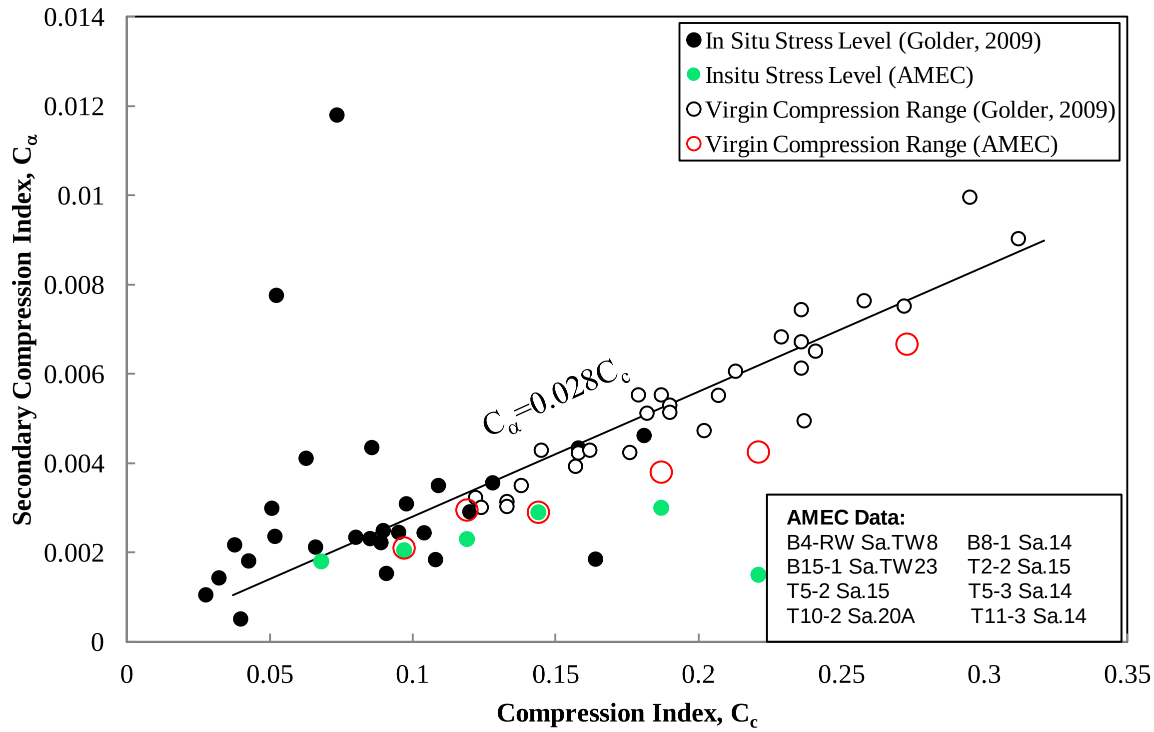
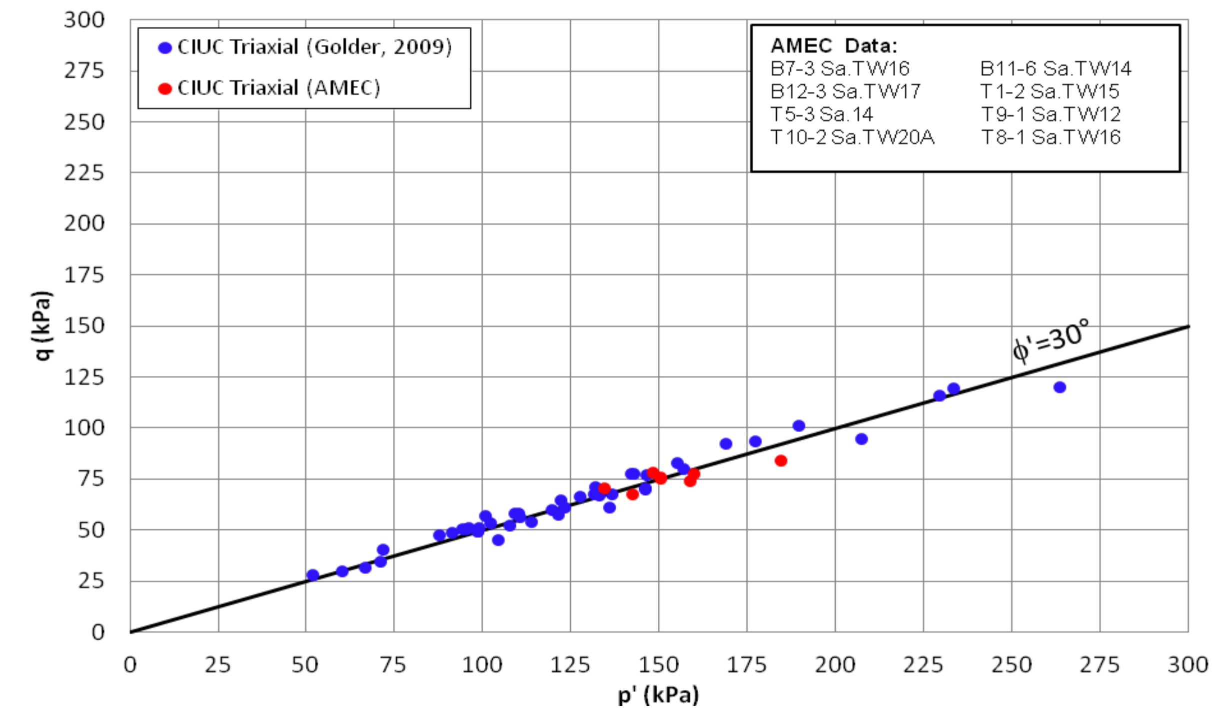


Figure 4-3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP



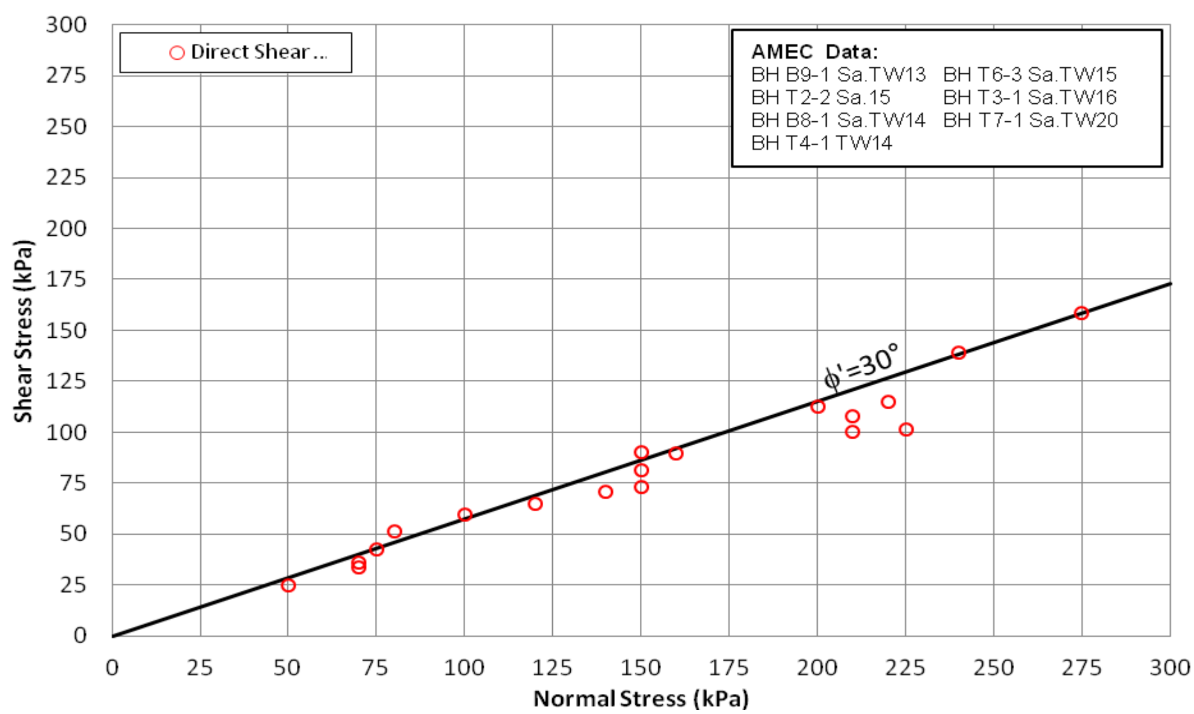


Figure 4-4: Relationship between sin ϕ' and Plasticity Index for Normally Consolidated Soils

(Kenney, 1959)

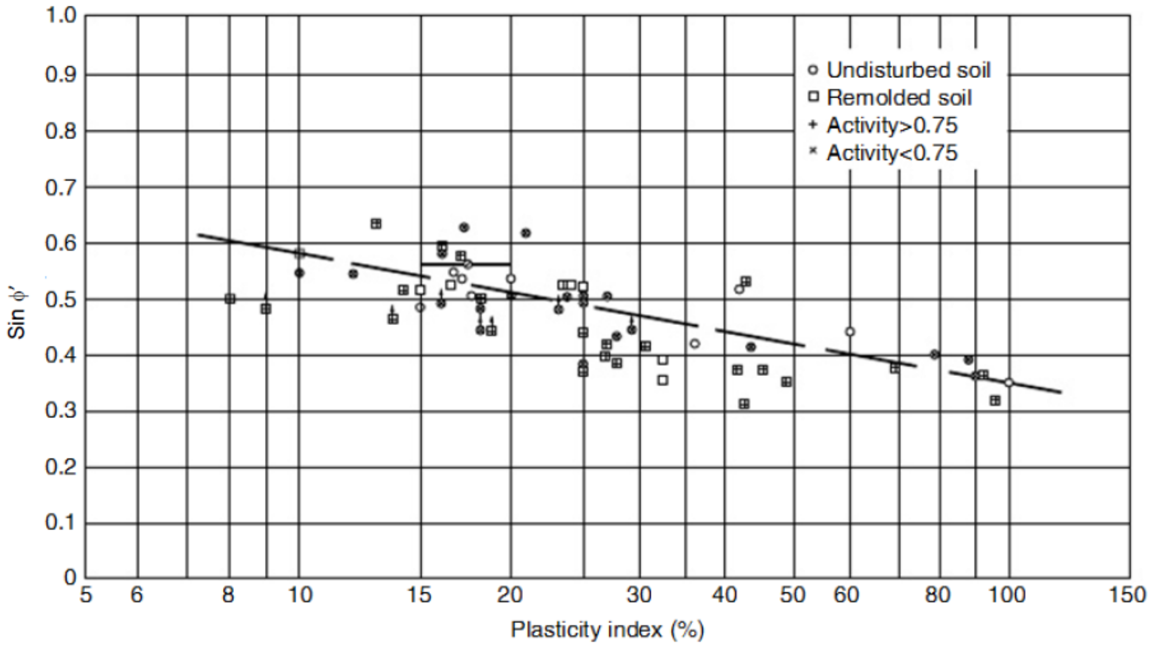
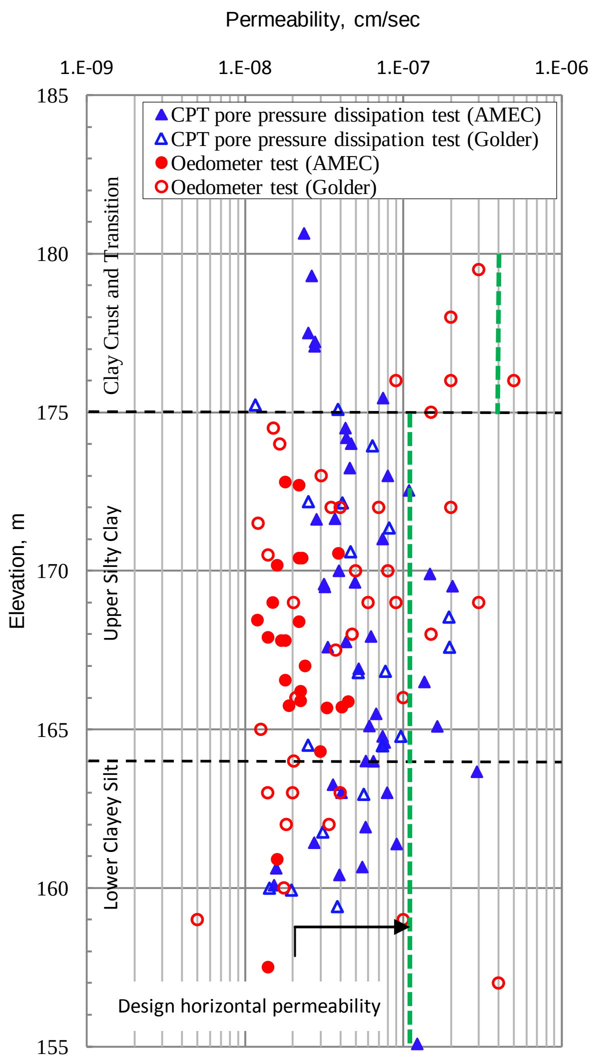
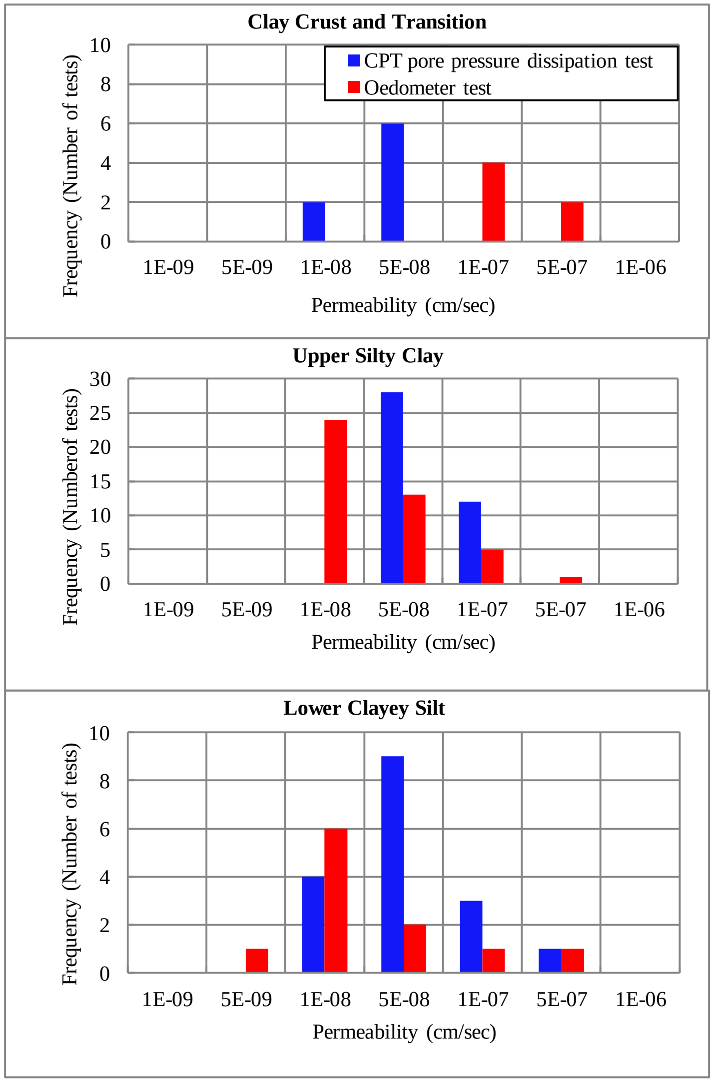


Figure 4-5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests





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

RECORD OF BOREHOLE No T8-1

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4678789.7, E333364.5		ORIGINATED BY NB
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic		DATE 19 Jul 11 - 20 Jul 11	CHECKED BY MSO

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
182.8	Ground Surface
0.0 / 182.4	400mm **TOPSOIL**
0.4	**CLAYEY SILT** Some sand, trace gravel Stiff to hard Brown-grey Trace rootlets in upper 2 m
	Grey
	-Trace cobbles retained in sampler
167.9	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	3		
2	SS	15		
3	SS	33		
4	SS	36		
5	SS	33		
6	SS	18		
7	SS	12		
8	SS	15		
9	TW	PH	21.9	
	VT			
10	TW	PH		
11	TW	PH	21.4	4 30 40 26
	VT			
12	TW	PH		
13	TW	PH		
	VT			
14	TW	PH		

GROUND WATER CONDITIONS

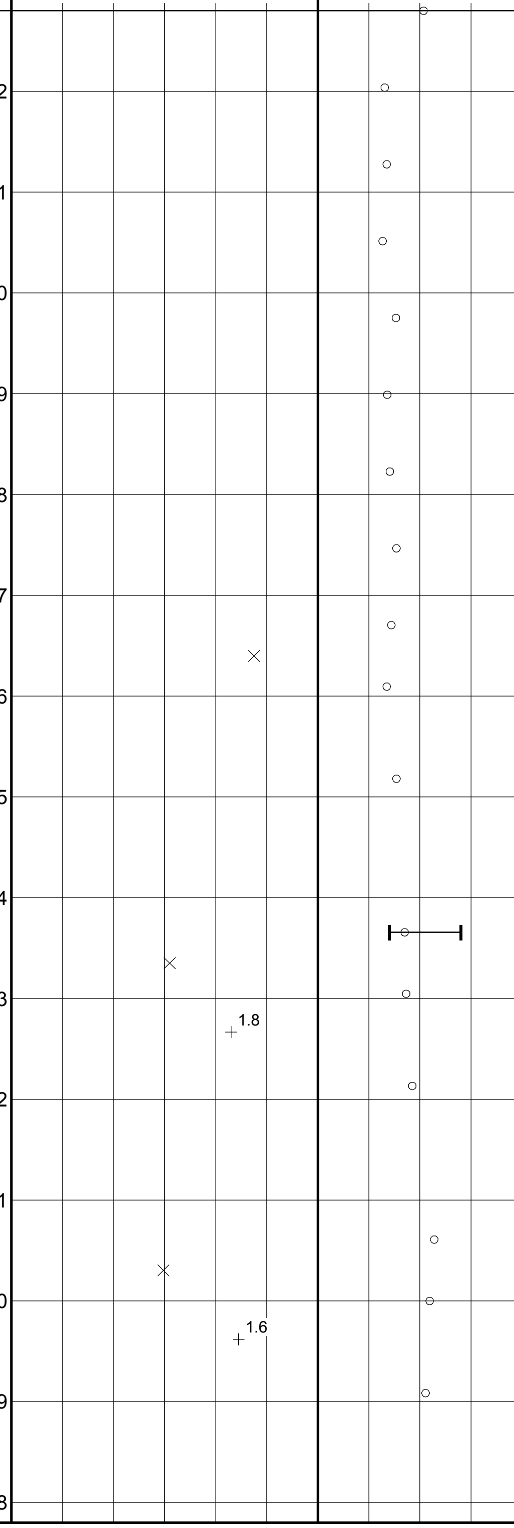
ELEVATION SCALE: 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30



REMARKS

-Vibrating Wire Piezometers (VWP) installed in adjacent boring at N4678788.9, E333366.7, Spider magents (MG) installed in adjacent boring at N4678787.9, E333368.8, Slope inclinometer casing installed in sampled borehole Nilcon vane advanced adjacent to sampled borehole from 7 m to 21 m (El. 175.8 m to El. 161.8 m) -corrosivity sample

-vanes shear test exceeded max torque of apparatus (greater than 140 kPa)

-VWP T8-1-P11 installed at 10.7m below ground surface (El. 172.1 m) -MG T8-1-SM12 installed at 11.5 m below ground surface (El. 171.3 m)

-switched to wash boring method

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

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RECORD OF BOREHOLE No T8-1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678789.7, E333364.5 ORIGINATED BY NB

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

DATUM Geodetic DATE 19 Jul 11 - 20 Jul 11 CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14.9	**SILTY CLAY** Some silt nodules Firm to stiff Grey, some pink nodules *(continued)*	15	TW	PH		20.4	1 19 35 45
			VT		167		
		16	TW	PH	166		
		17	TW	PH	165, 164	21.1	
			VT				-end of drilling July 19; continued July 20
163.3 19.5	**CLAYEY SILT** Some sand, trace gravel Very stiff Grey	18	TW	PH	163		-VWP T8-1-P21 installed at 20.6m below ground surface (El. 162.2 m) -MG T8-1-SM20 installed at 19.7 m below ground surface (El. 163.1 m)
		19	TW	PH	162, 161	22.0	2 28 46 24
			VT				-attempt at vane shear test exceeded max torque of apparatus
		20	TW	PH	160		
		21	SS	25	159, 158		
		22	SS	18	157		
		23	SS	17	156, 155		
153.8 29.0	**SILTY CLAY** Some silt seams Soft Grey Wet	24	SS	3	154, 153		

Field vane annotation: 2.3

Continued Next Page

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

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Foundation Design

RECORD OF BOREHOLE No T8-1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678789.7, E333364.5 ORIGINATED BY NB

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

DATUM Geodetic DATE 19 Jul 11 - 20 Jul 11 CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
152.6 30.2	**SANDY SILT** With clayey silt layers,some gravel Very dense Grey	25	SS	88	152	~9	-SPT refusal at 30.9m; augered to 32.0m
	-Some limestone fragments	26	SS	50/ 50mm	151	~12	-broken rock pieces at tip of spoon -SPT refused at 32.4m; auger refusal at 32.6m
150.2 32.6	**LIMESTONE** Fine grained, laminated, pitted Rubble between 33.0m and 33.2m White-Grey	27	RC		150		RQD = 45% TCR = 100% SCR = 50%
148.6 34.2	**LIMESTONE** Fine grained, pitted, stylolitic contact with upper unit, porous Grey	28	RC		149		RQD = 94% TCR = 100% SCR = 76%
148.1 34.7	**END OF BOREHOLE**				148		

No groundwater observed during drilling from July 19 to July 20, 2011 due to wash boring

Water Level measured in Piezometer VWP T8-1-P11 at elevation 181.2m on August 29, 2011

Water Level measured in Piezometer VWP T8-1-P21 at elevation 179.9m on August 29, 2011

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

Elevation scale: 152 151 150 149 148 147 146 145 144 143 142 141 140 139 138

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

RECORD OF BOREHOLE No PS5-1

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4678814.1, E333219.4	ORIGINATED BY	LC
DIST		HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	24 Aug 11 - 24 Aug 11	CHECKED BY	MSO

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH	UNIT WEIGHT γ (kN/m³)	REMARKS
182.8	Ground Surface							
0.0	Clayey **TOPSOIL**	1	SS	4				-Observation Well (OW) installed in sampled borehole; Vibrating Wire Piezometers (VWP) installed in adjacent boring
182.2 / 0.6	**CLAYEY SILT** Some sand, trace gravel, soft to hard Brown changing to grey below approx 3m (EL. 179.7m) Trace silt/sand seams and lenses occur randomly throughout	2	SS	4	182			
		3	SS	17	181			
		4	SS	34	180			
		5	SS	35				
		6	SS	25	179			
		7	SS	18	178			
		8	SS	15	177			
		9	TW	PH	176	×	21.8	
		10	TW	PH	175			
					174			
		11	TW	PH	173			
			VT			>> 3.9		
		12	TW	PH	172	×	20.6	
					171			
		13	TW	PH	170			
			VT			>> 2.4		
		14	TW	PH	169			
					168			

Continued Next Page

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

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RECORD OF BOREHOLE No PS5-1

METRIC

W.P. RFP No. 09-54-1007 | LOCATION N4678814.1, E333219.4 | ORIGINATED BY LC

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

DATUM Geodetic | DATE 24 Aug 11 - 24 Aug 11 | CHECKED BY MSO

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT	REMARKS
	CLAYEY SILT Some sand, trace gravel, soft to hard. Brown changing to grey below approx 3m (EL. 179.7m). Trace silt/sand seams and lenses occur randomly throughout *(continued)*	15	SS	PH		-no recovery with shelby tube; sample retrieved by pushing split spoon
			VT			
		16	TW	PH		
		17	TW	PH		-VWP PS5-1-P18 installed at 18.3m below ground surface (EL. 164.5m)
		18	TW	PH	21.7	
		19	TW	PH		-end of drilling Aug 24; continue Aug 25
		20	TW	PH		
		21	SS	4		-VWP PS5-1-P24 installed at 23.9m below ground surface (EL. 158.9m)
		22	SS	5		
		23	SS	4		
154.5 / 28.3	**CLAYEY SILT** Firm, grey, and layers of **FINE SAND** and **SILT**	24	SS	7		

Elevation scale: 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153

Field vane: 1.6

Continued Next Page

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

RECORD OF BOREHOLE No PS5-1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678814.1, E333219.4 ORIGINATED BY LC

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

DATUM Geodetic DATE 24 Aug 11 - 24 Aug 11 CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	CLAYEY SILT Firm, grey, and layers of **FINE SAND** and **SILT** *(continued)*	25A,B,C	SS	14	
150.8 / 32.0	**SAND AND GRAVEL** Some silt, some clay, compact, grey	26	SS	16	-VWP PS5-1-P32 installed at 32.00m below ground surface (EL. 150.8m)
150.2 / 32.6	Grey to white **LIMESTONE** Fine Grained Pitted, porous	27	RC		
149.8 / 33.0					
149.4 / 33.4	Grey to white **LIMESTONE** Stylolites present, well crystallized, dense				
	END OF BOREHOLE				

No groundwater observed during drilling due to wash boring

Water levels in deep observation well:
Nov 3, 2011: EL. 180.1m
Nov 11, 2011: EL. 180.2m

Piezometric levels in VWP PS5-1-P18 (shallow):
Nov 3, 2011: EL. 179.6m
Nov 11, 2011: EL. 179.6m

Piezometric levels in VWP PS5-1-P24 (mid-depth):
Nov 3, 2011: EL. 178.8m
Nov 11, 2011: EL. 178.8m

Piezometric levels in VWP PS5-1-P32 (deep):
Nov 11, 2011: EL. 177.6m

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (152, 151, 150, 149, 148, 147, 146, 145, 144, 143, 142, 141, 140, 139, 138); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; GR SA SI CL

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No TB6-1

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4678909.5, E333353.3		ORIGINATED BY TA
DIST	HWY WEP	BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic		DATE 9 Jul 11 - 9 Jul 11	CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
183.0	Ground Surface					
182.9 / 0.1	**TOPSOIL**					
	CLAYEY SILT Some sand, trace gravel Stiff to hard Mottled brown and grey Sandy, dry	1	SS	32	182	-hit a stone which may have skewed blow counts
	-Trace fissures	2	SS	22	181	
	-Trace inferred cobbles, trace fissures	3	SS	45		
		4	SS	31	180	-sample very disturbed due to inferred cobbles
		5	SS	20	179	
		6	SS	14	178	
		7	SS	13	177	
		8	SS	9	176	
		9	SS	5	175	
			VT			
		10	SS	5	174	-corrosivity sample
			VT		173	
172.9 / 10.1	**END OF BOREHOLE** Borehole dry on completion				172	
					171	
					170	
					169	

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

Field vane values: +2 (at approx. 174.6), +2 (at approx. 173.0).

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

RECORD OF BOREHOLE No CV4-1

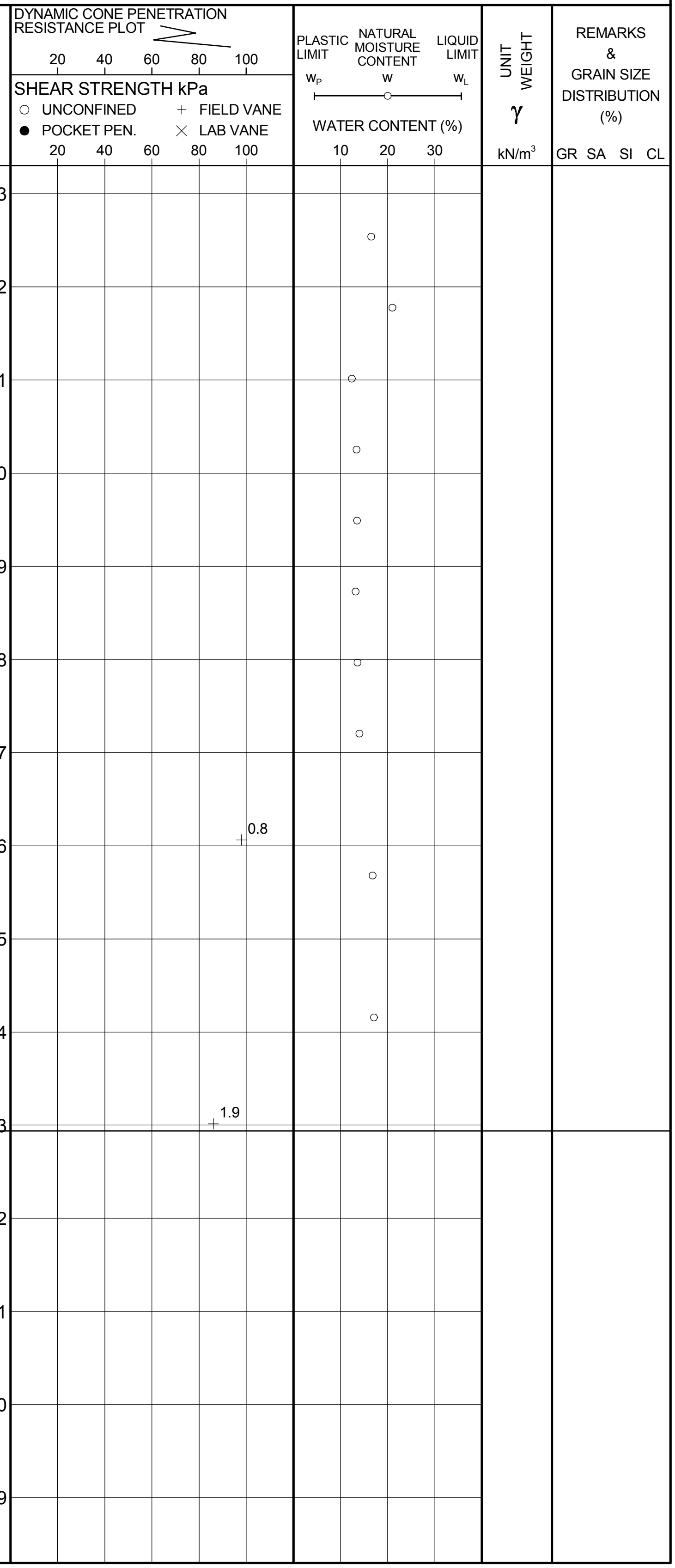
METRIC

W.P. RFP No. 09-54-1007 | LOCATION N4678867.9, E333368.7 | ORIGINATED BY DG

DIST ___ HWY WEP | BOREHOLE TYPE CME 850 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY SS

DATUM Geodetic | DATE 27 Aug 11 - 27 Aug 11 | CHECKED BY MSO

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
183.3	Ground Surface				
0.0	75mm **ASPHALT** Over **FILL,** sand and gravel				
182.4 / 0.9	**FILL** Silty Clay/Clayey Silt Some topsoil, trace fine gravel, trace sand, brown	1	SS	6	
		2	SS	5	
181.2 / 2.1	**CLAYEY SILT** Some sand, trace fine-coarse gravel Stiff to hard Mottled brown-grey	3	SS	9	
					∇ (El. 180.3)
		4	SS	29	
		5	SS	30	
	Grey	6	SS	17	
		7	SS	11	
		8	TW	PH	
			VT		
		9	TW	PH	
		10	TW	PH	
172.9			VT		
10.4	**END OF BOREHOLE** (no refusal) Groundwater observed at 3.0 m (El. 180.3 m) during drilling on Aug. 27, 2011				



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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No HG-MW-3

METRIC

W.P. RFP No. 09-54-1007 | LOCATION N4678886.8, E333395.5 | ORIGINATED BY TA

DIST | HWY WEP | BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY SS

DATUM Geodetic | DATE 9 Jul 11 - 9 Jul 11 | CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.9	Ground Surface				
182.8 0.1	**TOPSOIL**				-Observation Well installed in sampled borehole
	CLAYEY SILT Some sand, trace gravel, trace organics Brown	1	SS	8	
181.4 1.5	**SAND** Poorly-Graded, trace gravel, trace silt Brown	2	SS	3	9 68 13 10
180.5 2.4	**CLAYEY SILT** Some sand, trace gravel Trace fissures Brown	3A, B	SS	1	
		4	SS	29	
179.4 3.5	**END OF BOREHOLE**				

Water levels in observation well measured at elevation 180.9m on July 29, 2011
Water levels in observation well measured at elevation 180.6m on October 13, 2011

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (182 to 168); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No CPT T8-1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678860.0, E333292.9 ORIGINATED BY TA

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

DATUM Geodetic DATE 4 Aug 11 - 4 Aug 11 CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
183.2	Ground Surface					
183.0 0.0 / 0.2	**FILL** Crushed Limestone Grey					183
182.4 0.8	**FILL** Clayey silt, some gravel Brown	1	SS	7		
	SANDY SILT Some clay, trace gravel Mottled brown and grey Brown					182
181.2		2	SS	9		
2.0	**END OF SAMPLED BOREHOLE** Continued with CPT from 2 m to refusal at 32.4 m (El. 181.2 m to El. 150.8 m) Borehole dry on completion					181

Elevation scale: 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

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

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

RECORD OF BOREHOLE No CPT43-RW

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678907.6, E333207.7 ORIGINATED BY TA

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

DATUM Geodetic DATE 3 Aug 11 - 3 Aug 11 CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
182.6	Ground Surface							
0.0	**TOPSOIL**							
182.3 0.3	**CLAYEY SILT** Some sand, trace gravel Trace fissures Mottled brown and grey						182	
			1	SS	11			
	Brown						181	
180.6			2	SS	21			
2.0	**END OF SAMPLED BOREHOLE** Continued with CPT from 2.0 m to refusal at 29.1 m (El. 180.6 m to El. 153.5 m) Borehole dry on completion						180	
							179	
							178	
							177	
							176	
							175	
							174	
							173	
							172	
							171	
							170	
							169	
							168	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

GR SA SI CL

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No CPT44-RW

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678777, E333464 ORIGINATED BY TA

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

DATUM Geodetic DATE 4 Aug 11 - 4 Aug 11 CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
183.1	Ground Surface						
183.0 / 0.1	**TOPSOIL**						183
	CLAYEY SILT Some sand, occasional sand pockets Trace fissures Mottled brown and grey		1	SS	4		
			2	SS	14		182
	Brown						181
180.1			3	SS	31		
3.0	**END OF SAMPLED BOREHOLE** Continued with CPT from 3.3 m to refusal at 27.2 m (El. 179.8 m to El. 155.9 m) Borehole dry on completion						180
							179
							178
							177
							176
							175
							174
							173
							172
							171
							170
							169

SOIL PROFILE; SAMPLES

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P; NATURAL MOISTURE CONTENT w; LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

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

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

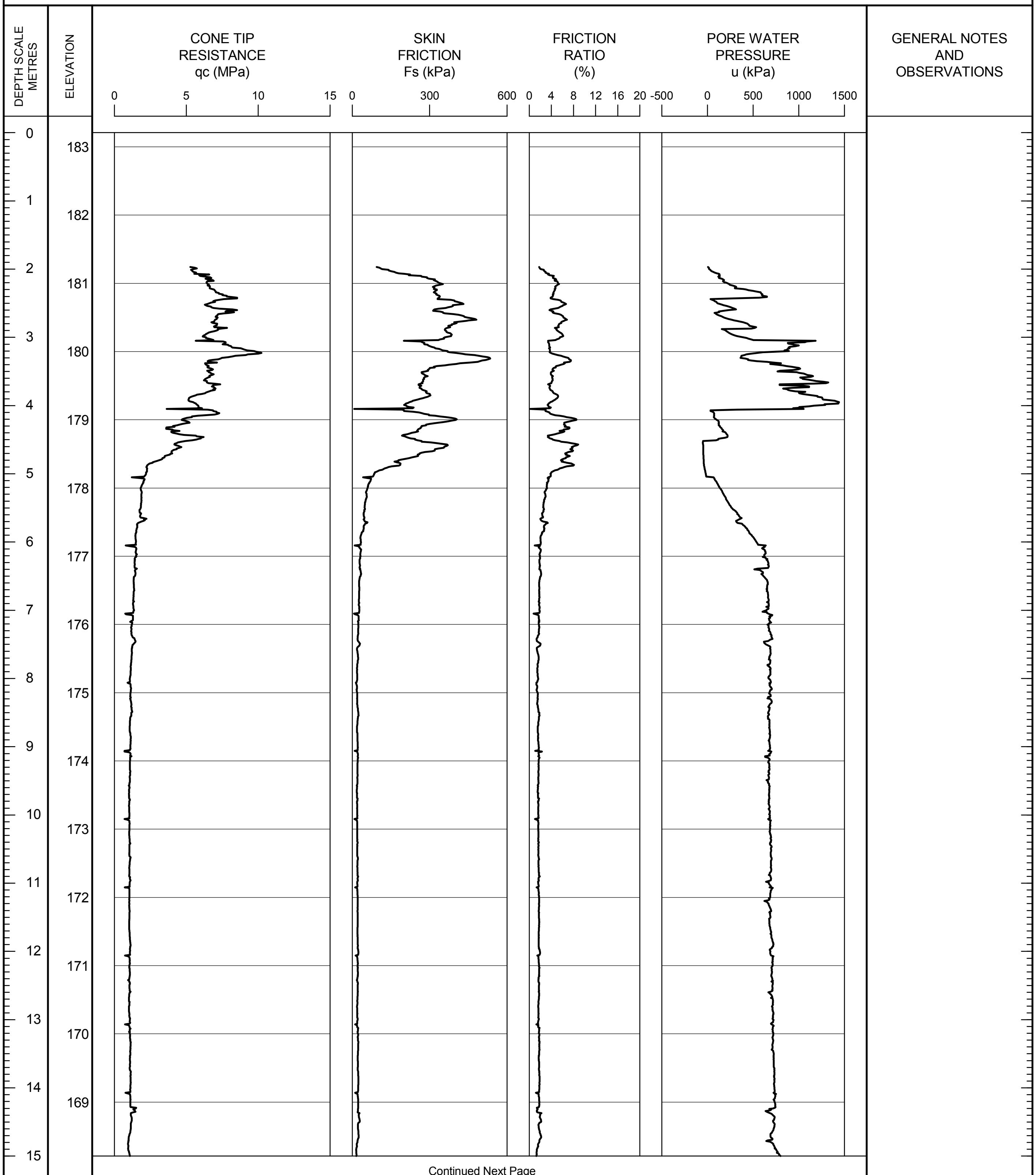
Foundation Design

RECORD OF CONE PENETRATION TEST CPT T8-1

METRIC

PROJECT	Windsor-Essex Parkway	TEST DATE	8/4/2011 - 8/4/2011	SHEET	1 OF 3
LOCATION	N4678860.0; E333292.9			DATUM	Geodetic

GROUND SURFACE ELEVATION: 183.2 PREDRILL DEPTH: 1.82 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

CHECKED: DD

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

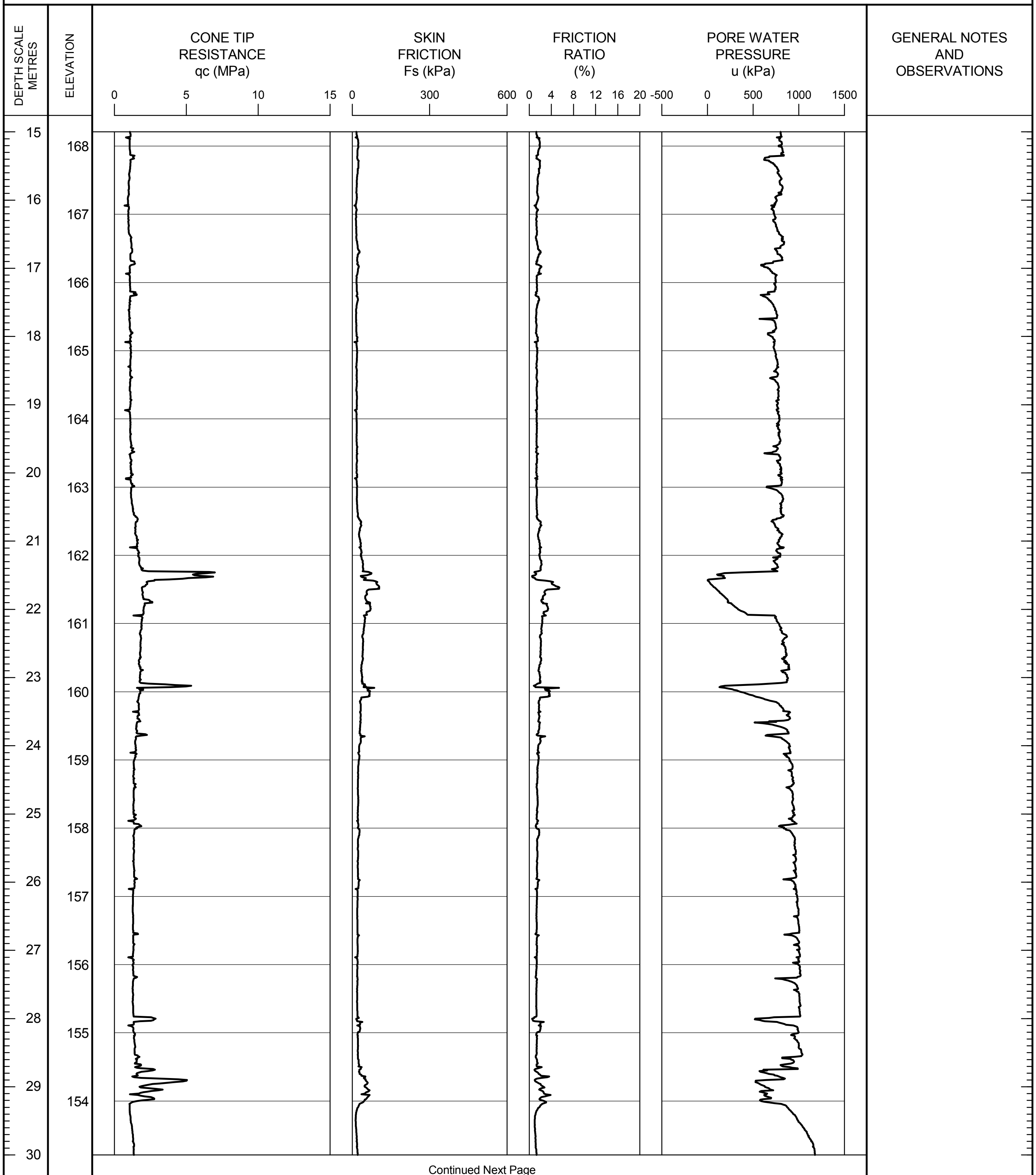
RECORD OF CONE PENETRATION TEST CPT T8-1

METRIC

PROJECT Windsor-Essex Parkway
TEST DATE 8/4/2011 - 8/4/2011

LOCATION N4678860.0; E333292.9
DATUM Geodetic

GROUND SURFACE ELEVATION: 183.2 PREDRILL DEPTH: 1.82 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



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OPERATOR: TA
CHECKED: DD

WEP CPT LOG CPT T8-1.GPJ ONTARIO MOT.GDT 22/12/11

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

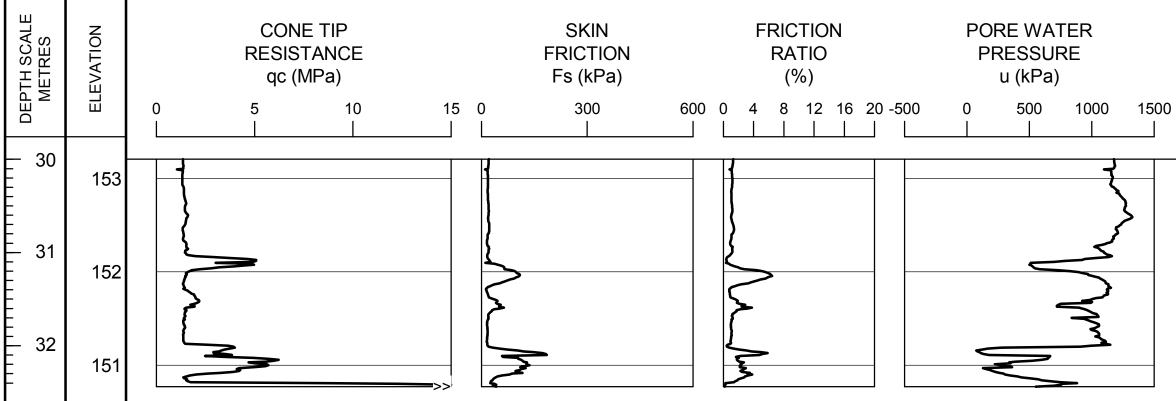
RECORD OF CONE PENETRATION TEST CPT T8-1

METRIC

PROJECT Windsor-Essex Parkway   TEST DATE 8/4/2011 - 8/4/2011   SHEET 3 OF 3

LOCATION N4678860.0; E333292.9   DATUM Geodetic

GROUND SURFACE ELEVATION: 183.2   PREDRILL DEPTH: 1.82   CORRECTION FACTOR A: 0.8   CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

OPERATOR: TA

CHECKED: DD

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

Foundation Design

RECORD OF CONE PENETRATION TEST CPT 43-RW

METRIC

PROJECT Windsor-Essex Parkway

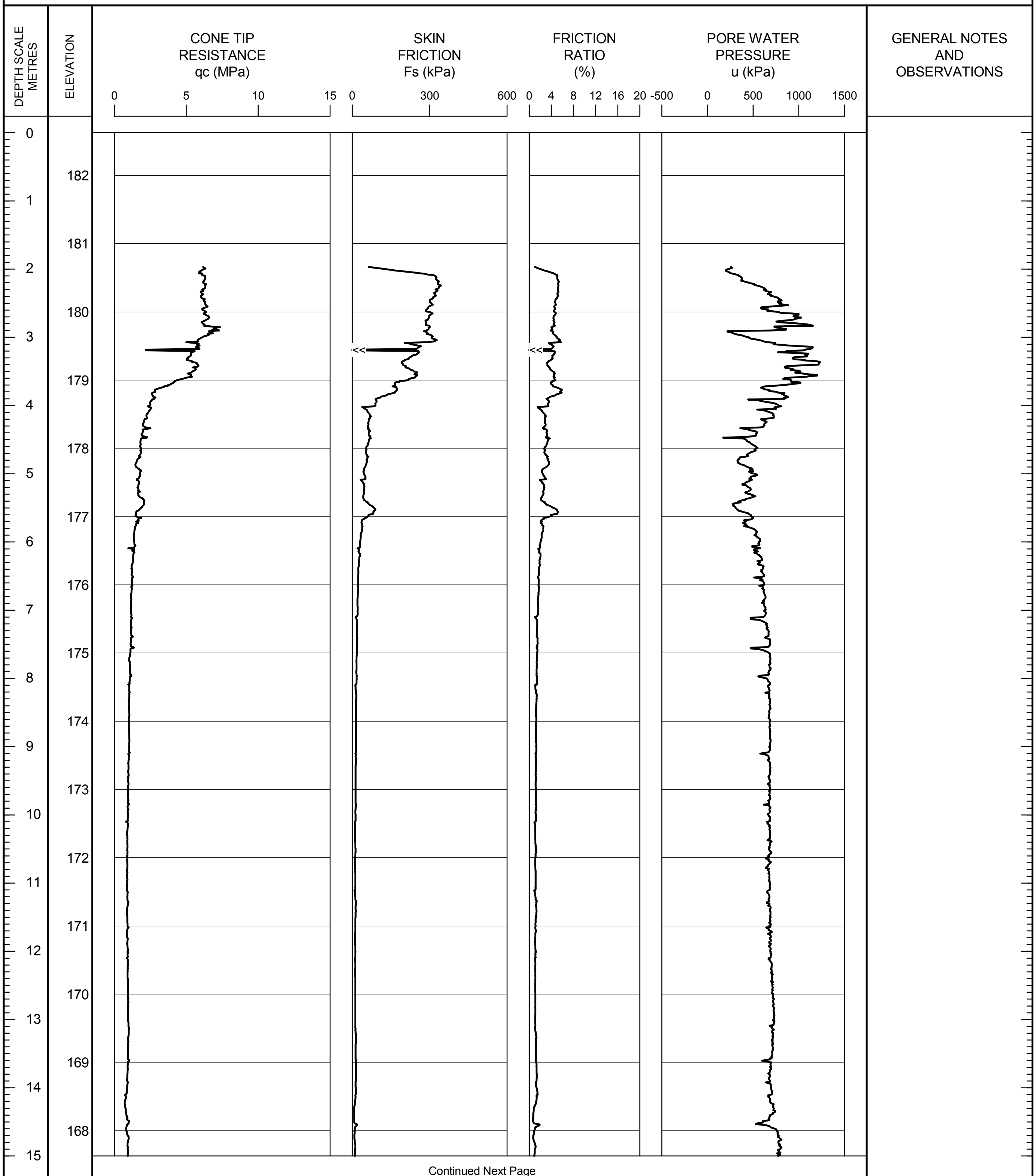
TEST DATE 8/3/2011 - 8/3/2011

SHEET 1 OF 2

LOCATION N4678907.6; E333207.7

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.6 PREDRILL DEPTH: 1.97 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

CHECKED: DD

Foundation Design

RECORD OF CONE PENETRATION TEST CPT 43-RW

METRIC

PROJECT Windsor-Essex Parkway

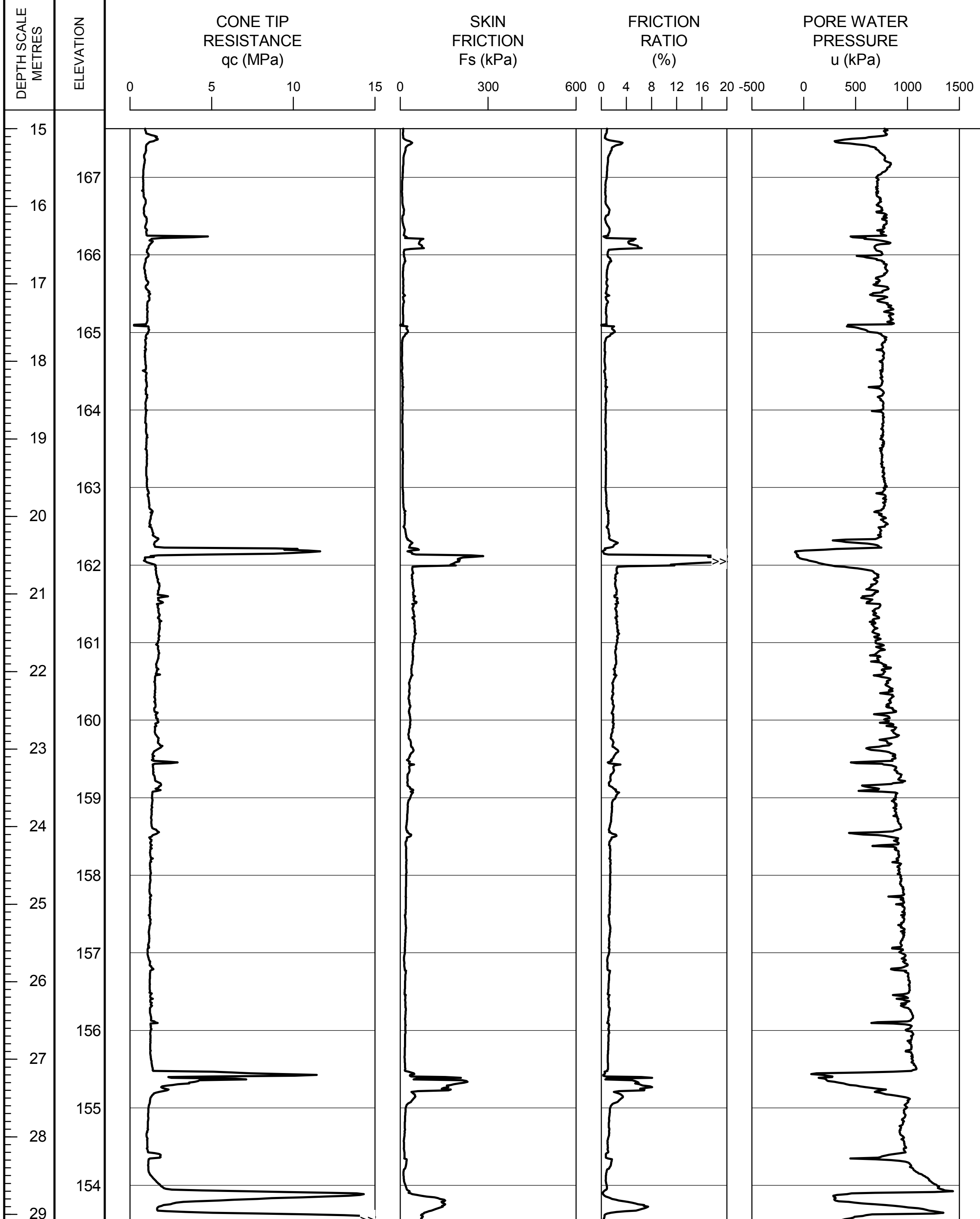
TEST DATE 8/3/2011 - 8/3/2011

SHEET 2 OF 2

LOCATION N4678907.6; E333207.7

DATUM Geodetic

GROUND SURFACE ELEVATION: 182.6 PREDRILL DEPTH: 1.97 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

OPERATOR: TA

CHECKED: DD

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

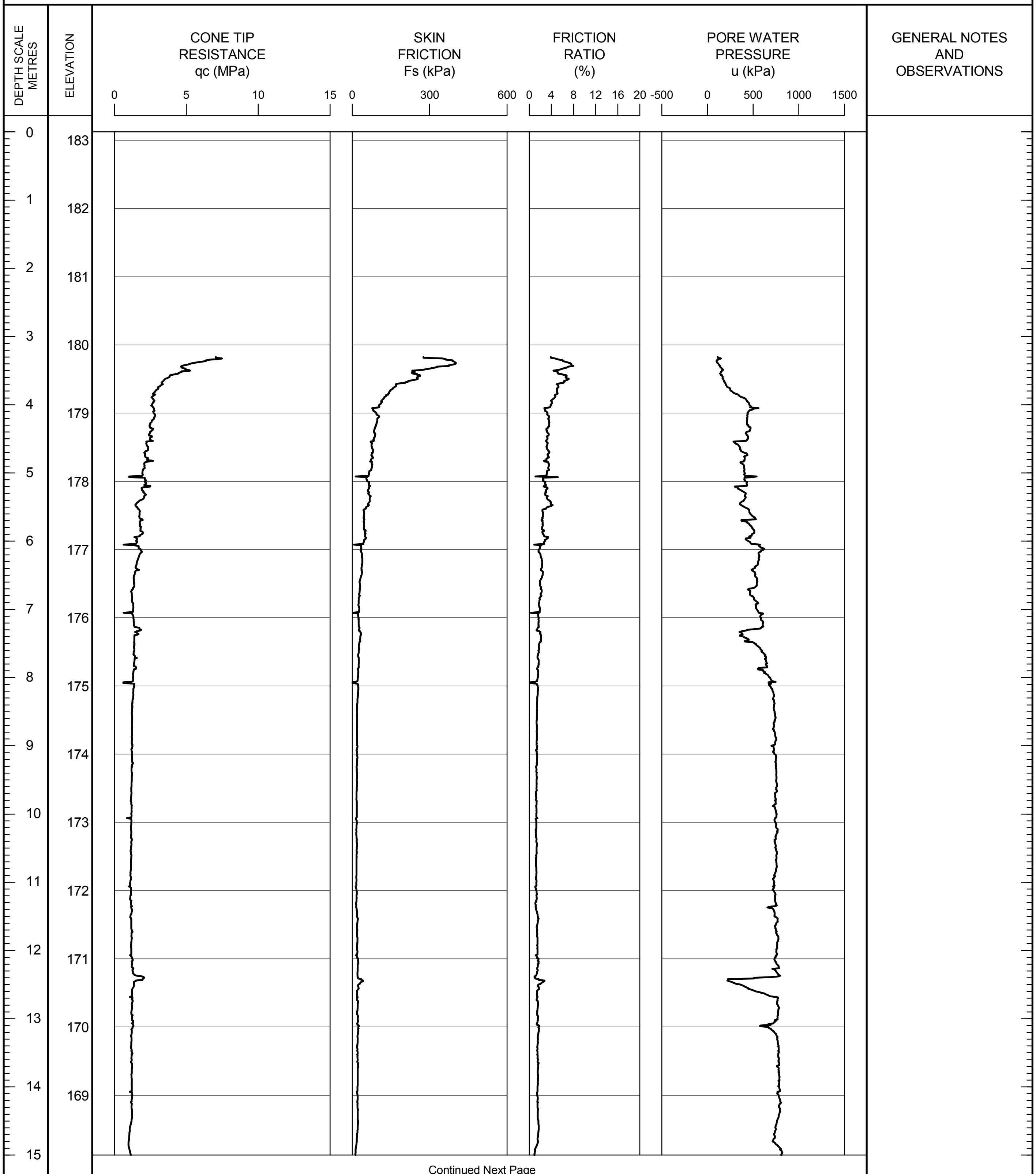
RECORD OF CONE PENETRATION TEST CPT 44-RW

METRIC

PROJECT Windsor-Essex Parkway
TEST DATE 8/4/2011 - 8/4/2011
SHEET 1 OF 2

LOCATION N4678777.5; E333464.6
DATUM Geodetic

GROUND SURFACE ELEVATION: 183.1 PREDRILL DEPTH: 3 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



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OPERATOR: TA

CHECKED: DD

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

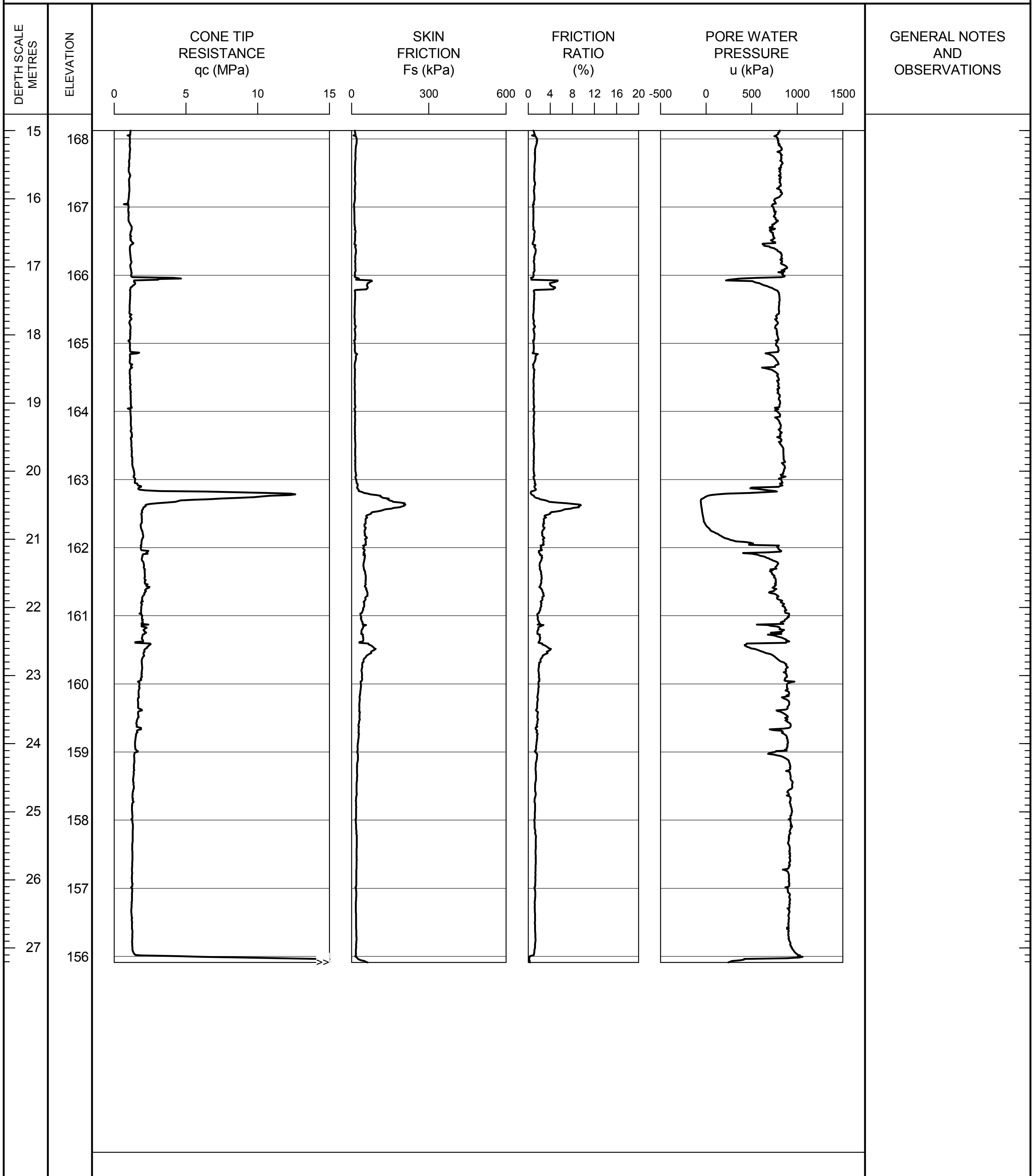
RECORD OF CONE PENETRATION TEST CPT 44-RW

METRIC

PROJECT Windsor-Essex Parkway
TEST DATE 8/4/2011 - 8/4/2011
SHEET 2 OF 2

LOCATION N4678777.5; E333464.6
DATUM Geodetic

GROUND SURFACE ELEVATION: 183.1 PREDRILL DEPTH: 3 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

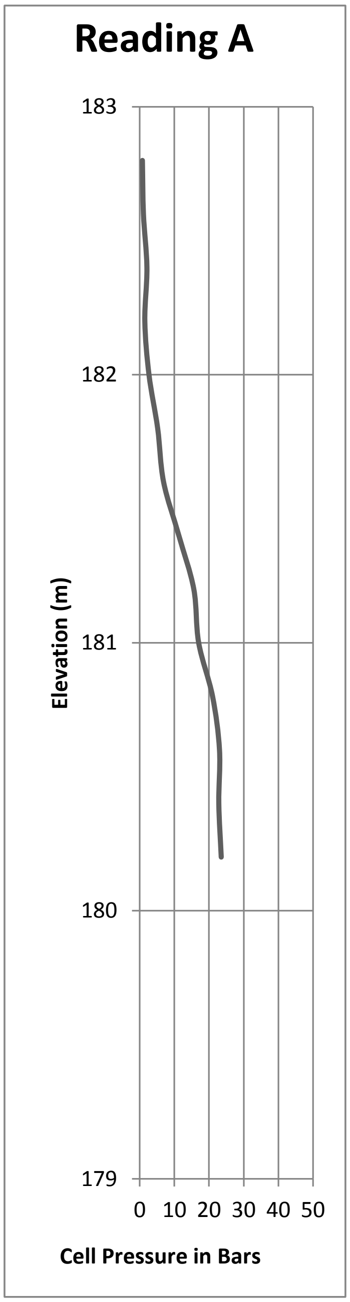
Foundation Design

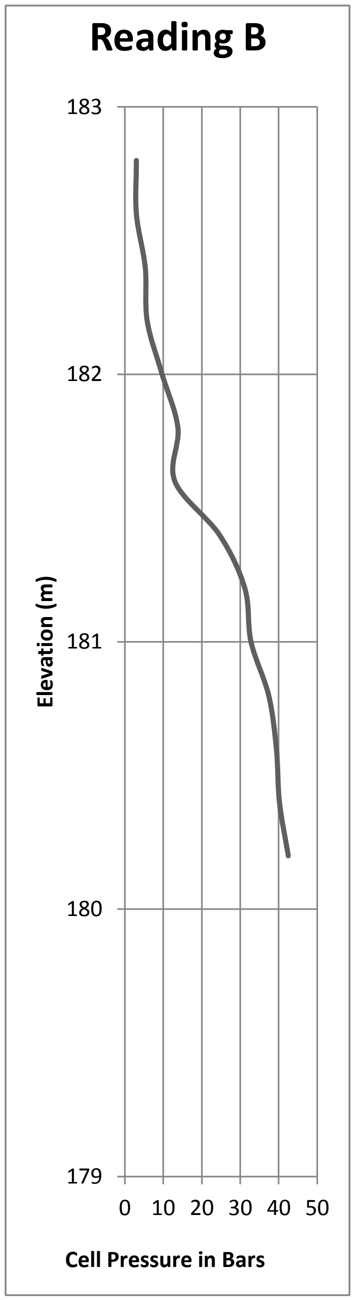
RECORD OF DILATOMETER TEST DMT T8-1-SHALLOW

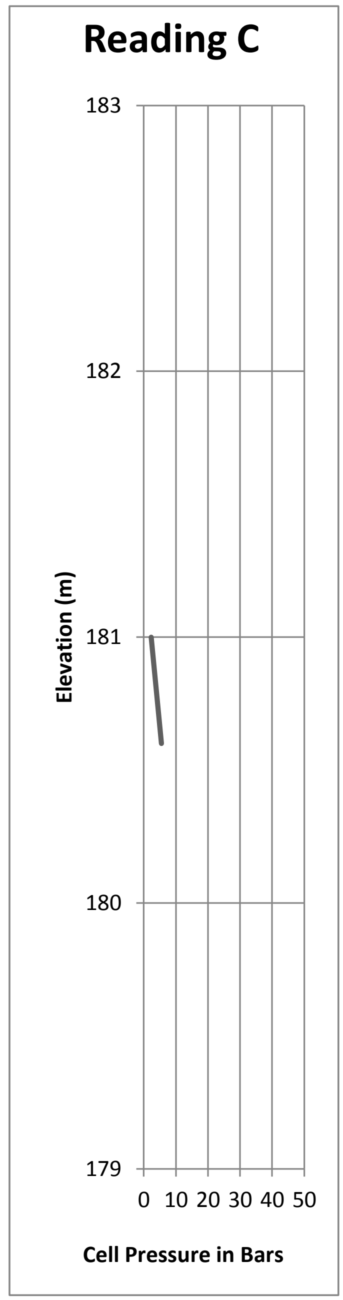
Project : Windsor-Essex Parkway
Location: N 4678820.9; E 333382.7
Ground Surface Elevation : 183.0

Test Date: 7/18/2011
Predrill Depth : 0.2 m
Delta A: 0.14 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.18 Bar







Note: DMT refusal at elevation 180.2m. Redrilled to elevation 178.4m.
Continued with DMT to elevation 159.6m

Operator: LC

Checked: DD

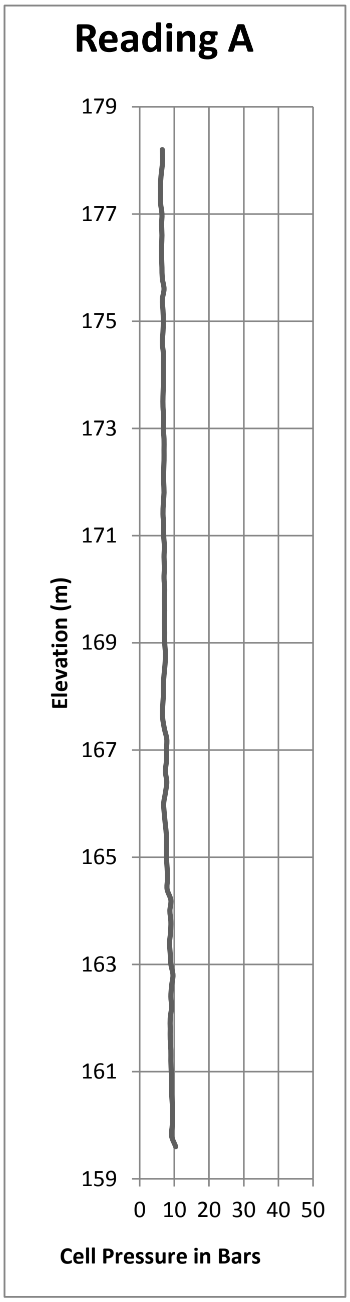
Foundation Design

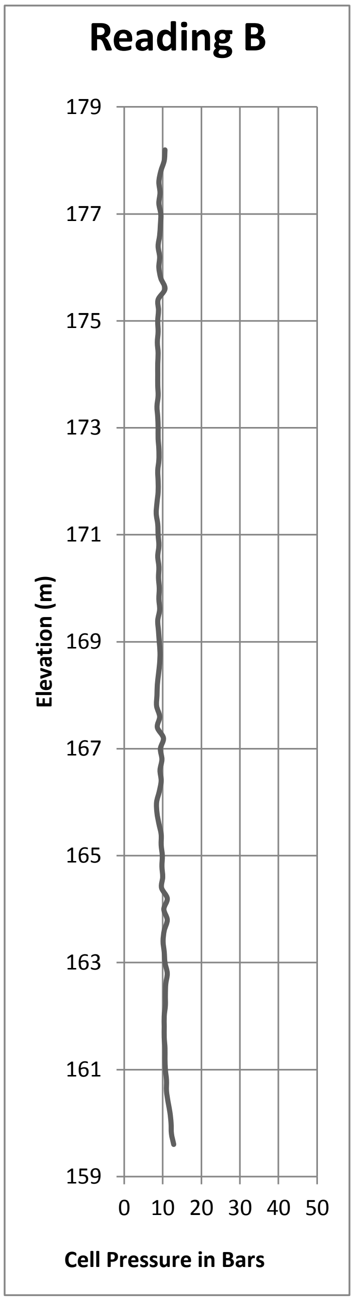
RECORD OF DILATOMETER TEST DMT T8-1-DEEP

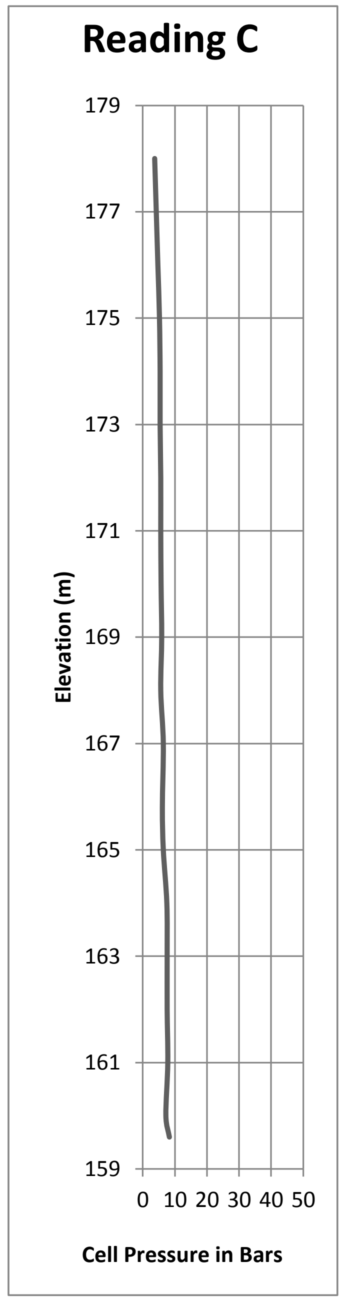
Project : Windsor-Essex Parkway
Location: N 4678820.9; E 333382.7
Ground Surface Elevation : 183.0

Test Date: 7/18/2011
Predrill Depth : 4.6 m
Delta A: 0.14 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.18 Bar







Operator: LC

Checked: DD

Foundation Design

RECORD OF NILCON VANE TEST NIL T8-1

Project : Windsor-Essex Parkway

Location: N4678784.8; E333381.3

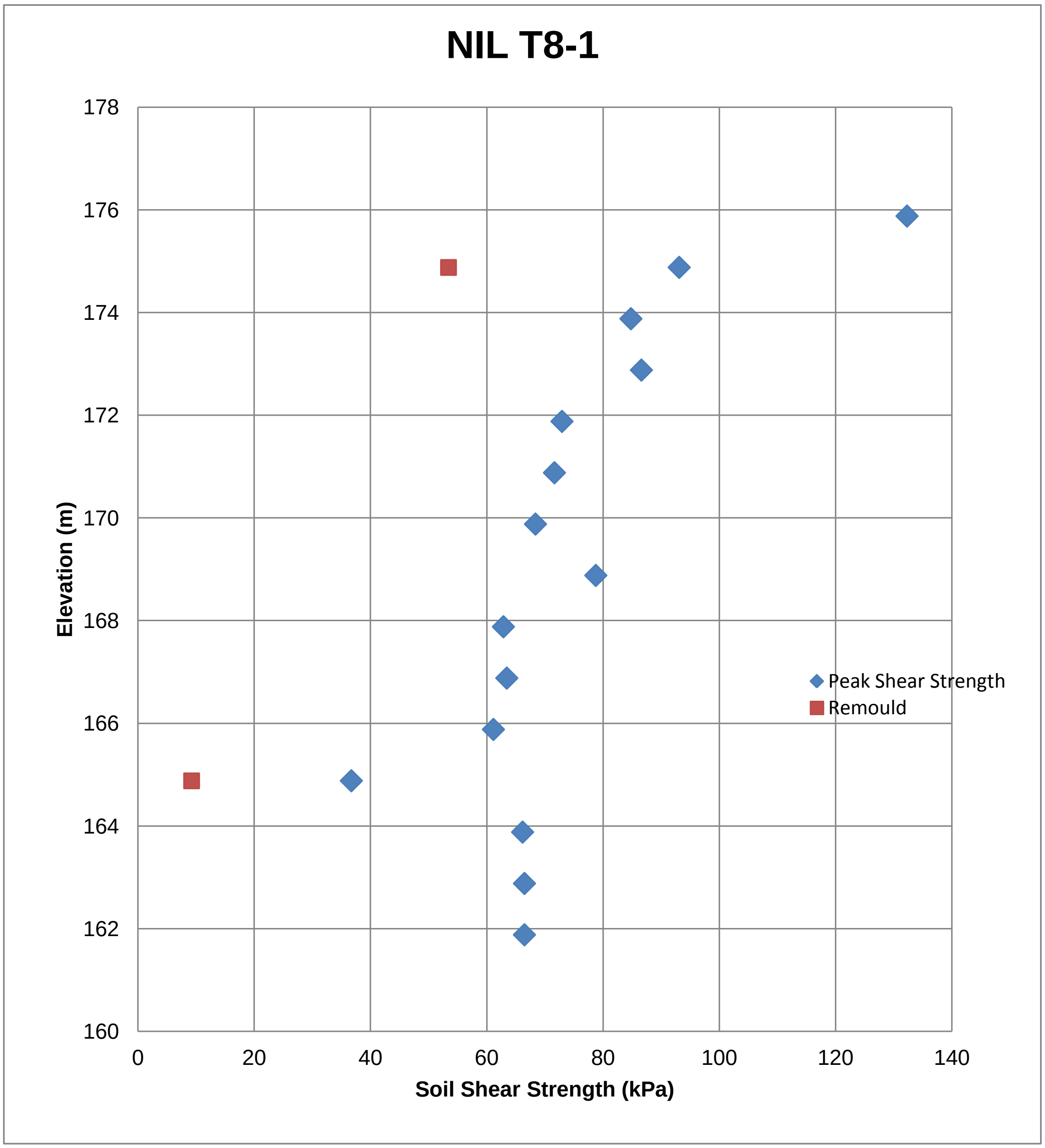
Ground Surface Elevation: 182.9 m

Test Date: 8/17/2011

Predrill Depth : 6.1 m

Sheet 1 of 1

Datum Geodetic



Operator: SD

Checked: DD

Appendix B Borehole and CPT Logs from Previous Investigations

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 7

METRIC

PROJECT 04-1111-060
W.P.
LOCATION N 4678848.0 ;E 333325.0
ORIGINATED BY C.C.
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
COMPILED BY T.M.
DATUM Geodetic
DATE November 10, 2006 - November 16, 2006
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	UNIT WEIGHT γ kN/m³
183.17	GROUND SURFACE						
0.00	TOPSOIL, clayey, Firm, Black				183		
0.25	CLAYEY SILT, some sand, trace gravel, Stiff, Mottled brown and grey	1	SS	10			
181.65 / 1.52	CLAYEY SILT, some sand, trace gravel, oxidized fissures, Very stiff to hard, Brown	2	SS	23	182, 181		
		3	SS	59	180		
179.28 / 3.89	CLAYEY SILT, some sand, trace gravel, silt and sandy silt pockets and laminated zones, Firm to very stiff, Grey	4	SS	32	179, 178		
		5	TO	PH	177		
		6	SS	11	176, 175		
		7	TO	PH	174	1 30 39 30, CIUC, Oedometer	21.4
		8	SS	12	173, 172		
		9	TO	PH	171		
		10	SS	8	170, 169		
		11	TO	PH		3 19 42 36	20.7

Ground water conditions: Grout; Bentonite Seal; Sand

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
Field vane sensitivities: 2.0, 1.7, 1.9, 1.8, 2.8
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
1.3

Continued Next Page

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

LDN_MTO_02 04-1111-060.GPJ GLDR_LON.GDT 6/29/09

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 7

METRIC

PROJECT 04-1111-060
W.P.
LOCATION N 4678848.0 ;E 333325.0
ORIGINATED BY C.C.
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
COMPILED BY T.M.
DATUM Geodetic
DATE November 10, 2006 - November 16, 2006
CHECKED BY SJB

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

CLAYEY SILT , some sand, trace gravel, silt and sandy silt pockets and laminated zones
Firm to very stiff
Grey

Ground water conditions: Sand (168 to 165); Grout (below 165)

Elevation scale	Sample number	Type	"N" values	Field vane (sensitivity)	Water content	Unit weight γ (kN/m³)	Remarks & grain size distribution
168							CIUC, Oedometer
				+ 3.7			
167	12	TO	PH		43		
166				+ 3.3			
165	13	SS	12				
				+ 1.8			
164	14	TO	PH				
163	15	TO	PH				
162	16	SS	21				
				>95.7 +			
161	17	SS	PH				
160				>143.6			
159	18	SS	13				
				+ 2.0			
158	19	SS	12				
157	20	TO	PH			21.0	2 19 42 37 CIUC, Oedometer
156	21	SS	9				
				>95.7 +			
155	22	SS	PH				
154							

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 7

METRIC

PROJECT 04-1111-060			
W.P.	LOCATION N 4678848.0 ;E 333325.0		ORIGINATED BY C.C.
DIST WEST	HWY 401 / 3	BOREHOLE TYPE POWER AUGER,HOLLOW STEM	COMPILED BY T.M.
DATUM Geodetic	DATE November 10, 2006 - November 16, 2006		CHECKED BY SJB

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT , some sand, trace gravel, silt and sandy silt pockets and laminated zones Firm to very stiff Grey		23	SS	13		153				
						Grout	152				
			24	SS	PH						
							151				
150.02			25	SS	42						
33.15	LIMESTONE, fresh, medium strong, laminated, very fine grained, moderately porous, light grey *(FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)*		26	NQ RC		Bentonite Seal	150				
			27	NQ RC		Sand	149				
			28	NQ RC			148 / 147				
			29	NQ RC			146				UC
145.28 / 37.89	END OF BOREHOLE										

Water level in borehole at about elevation 176.82m on October 16, 2006

Lower piezometer 32mm PVC screen and riser pipe. Second (Upper) piezometer 13mm porous tip and CPVC riser pipe.

Water level in Upper Piezometer at about elevation 180.06m on November 14, 2006.

Water level in Lower Piezometer at about elevation 177.59m on November 14, 2006.

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

PROJECT: 04-1111-060

RECORD OF DRILLHOLE: 7

LOCATION: N 4678848.0 ;E 333325.0

DRILLING DATE: November 10, 2006 - November 16, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG:

DRILLING CONTRACTOR:

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	ELEVATION	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
		ROCK SURFACE	150.04			
	MUD ROTARY / NW CASING	LIMESTONE, fresh, medium strong, laminated, very fine grained, faintly to moderately porous, light grey	149.64 / 33.13 – 33.53	1	150	
34 – 35		LIMESTONE, fresh, medium strong, laminated to thinly laminated, very fine grained, faintly to moderately porous, light grey	148.02 / 35.15	2	149	BD,UN,SM Cl ,IR,SM Cl JN,PL,SM Cl BD,CU,SM Cl BD,IR,VR Cl BD,PL,SM Cl JN,PL,Ro Cl BD,CU,Ro Cl
35 – 38	DIAMOND DRILLING / NQ ROCK CORE	LIMESTONE, fresh, medium strong, laminated to thinly laminated, very fine grained, faintly to moderately porous, light grey Moderately yo highly porous from 35.4m to 35.8m and from 36.5m to 36.9m depths.	145.28 / 37.89	3, 4	148, 147, 146	BD,UN,SM Cl BD,PL,Ro Cl ,PL,Ro Cl BD,CU,SM Cl JN,UN,Ro Cl BD,UN,Ro Cl ,IR,VR Cl BD,IR,SM Cl BD,CU,SM Cl
38		END OF BOREHOLE				

Column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.3; DIP w.r.t. CORE AXIS; ROCK STRENGTH INDEX (R4 R3 R2 R1); WEATHERING INDEX (W1 W2 W3 W4); NOTES WATER LEVELS INSTRUMENTATION

Depth scale: 39, 40, 41, 42, 43, 44, 45, 46, 47, 48

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: C.C.

CHECKED: SJB

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 118

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION N 4678903.5 ;E 333302.9

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 28, 2008 - March 4, 2008

CHECKED BY SSB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa / Sensitivity	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.66	GROUND SURFACE							
0.00	TOPSOIL, silty Brown				Concrete			
0.15	FILL, clayey silt, trace sand, trace gravel, with topsoil pockets Firm Brown				182			
		1	SS	5				
181.29 1.37	FILL, sand and gravel, some silt Compact Brown							
180.98 1.68	CLAYEY SILT, some sand, trace gravel Stiff to hard Brown, becoming grey at about elev. 179.0m	2	SS	13	181			
		3	SS	32	180			
		4	SS	34	179			
		5	SS	18				
		6	SS	11	178			
		7	SS	11	177			
		8	SS	10				
					176	136.5 / 1.6		
					Grout			
		9	TO	PH	175			
					174	1.2		
		10	SS	4	173			2 15 47 36
						1.6		
		11	TO	PH	172			
					171	1.5		
		12	SS	5	170			
		13	SS	5	169			
					168	2.4		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

Continued Next Page

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

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 118

METRIC

PROJECT 07-1130-207-0

W.P.

DIST WEST HWY 401/3

DATUM GEODETIC

LOCATION N 4678903.5 ;E 333302.9

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

DATE February 28, 2008 - March 4, 2008

ORIGINATED BY MA

COMPILED BY BRS

CHECKED BY SB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (kPa) / sensitivity	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		14	TO	PH	167			
166.81 15.85	SILTY CLAY, some sand, trace gravel Stiff to very stiff Grey					+ 1.6		
		15	SS	4	166		w_P – w_L	4 11 32 53
164.98 17.68	CLAYEY SILT, trace to some sand, trace gravel Stiff to very stiff Grey				165	+ 1.8		
		16	SS	5	164		o	
						+ 1.2		
		17	TO	PH	163			
					162			
		18	SS	14	161		o	
					160			
		19	SS	8	159		o	
		20	TO	PH	158			
					157			
		21	SS	10	156		o	
		22	SS	12	155		o	
					154			
153.70 28.96	SAND, with clayey silt layers Compact Grey	23	SS	29			o	
153.40 29.26	CLAYEY SILT, trace sand, trace gravel				153		o	
152.66								

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

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 118

METRIC

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4678903.5 ;E 333302.9
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE February 28, 2008 - March 4, 2008
ORIGINATED BY MA
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
30.02	Very stiff Grey										
	SILTY SAND, trace clay, trace gravel Compact Grey	24	SS	19	Grout	152					4 48 39 9
150.96 31.70	SILTY SAND AND GRAVEL, trace clay Dense Grey	25	SS	100/ 76mm	▽	151					
150.32 32.34	LIMESTONE, fresh, medium strong, thinly laminated, fine grained, moderately porous Whitish grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	26	NQ RC		Bentonite	150	100	60	25		UC
		27	NQ RC		Sand	149	99	98	82		
		28	NQ RC		Screen	148 / 147	99	98	90		
146.61 36.05	END OF BOREHOLE										

Water levels in borehole at about elev. 181.29m, 153.70m and 150.96m during drilling between February 28 and March 4, 2008.

Water level measured in deep piezometer at elev. 176.77m on March 4, 2008.

Water level measured in deep piezometer at elev. 177.30m on March 20, 2008.

Water level measured in deep piezometer at elev. 177.78m on July 24, 2008.

Water level measured in deep piezometer at elev. 177.32m on September 19, 2008.

Water level measured in deep piezometer at elev. 177.28m on November 14, 2008.

Water level measured in deep piezometer at elev. 177.40m on January 28, 2009.

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

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

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 118

SHEET 4 OF 4

LOCATION: N 4678903.5 ;E 333302.9

DRILLING DATE: February 28, 2008 - March 4, 2008

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	ELEVATION	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
	MUD ROTARY, NQ ROCK CORE	ROCK SURFACE	150.32			
		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous, whitish grey	32.34	1	150	JN,UN,SM Cl
33		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous, grey	149.56 / 33.10			JN,UN,Ro Si
		LIMESTONE, fresh, medium strong, thinly laminated, fine to very fine grained, pitted, whitish grey	149.22 / 33.44	2	149	JN,IR,Ro Si; JN,PL,SM Cl; JN,UN,
34						
35		LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, moderately porous, light grey	147.97 / 34.69		148	
		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, pitted to vuggy, light brown to grey	147.61 / 35.05	3	147	
36		END OF DRILLHOLE	146.60 / 36.06			

Depth scale: 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: SG
CHECKED: SJB

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 314

METRIC

PROJECT 09-1132-0080
W.P.
LOCATION N 4678750.8 ;E 333462.3
ORIGINATED BY SM
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY LMK/DMB
DATUM GEODETIC
DATE December 7, 2009 - December 9, 2009
CHECKED BY

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
183.07	GROUND SURFACE
0.00	TOPSOIL, clayey Black
0.23	CLAYEY SILT, some sand, trace gravel Firm to very stiff Mottled brown and grey, becoming grey below about elev. 179.4m

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	6
2	SS	27
3	SS	31
4	SS	26
5	SS	18
6	SS	11
7	TO	PH
8	SS	6
9	TO	PH
10	TO	PH
11	SS	6
12	TO	PH

GROUND WATER CONDITIONS: Bentonite; Grout

ELEVATION SCALE: 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivities (top to bottom): 1.6, 1.7, 2.8, 2.6, 1.6, 3.3, 2.2

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

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR	SA	SI	CL
0	32	42	26
1	33	41	25

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 314

METRIC

PROJECT 09-1132-0080
W.P.
DIST WEST HWY 401 / 3
DATUM GEODETIC

LOCATION N 4678750.8 ;E 333462.3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE December 7, 2009 - December 9, 2009

ORIGINATED BY SM
COMPILED BY LMK/DMB
CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa / DYNAMIC CONE PENETRATION	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, some sand, trace gravel Firm to very stiff Mottled brown and grey, becoming grey below about elev. 179.4m	13	SS	2					
					167	+ 5.8			
		14	TO	PH	166				
						+ 2.5			
		15	TO	PH	165				
					164	(>95.8)+			
		16	TO	PH	163				
						(>143.6)+			
		17	SS	22	162		o		
		18	SS	15	161 / 160		⊢o⊣		3 23 47 27
		19	SS	6	159		o		
					158	+ 2.0			
156.85 26.22	SAND, fine to coarse, some silt Compact Grey	20	SS	19	157		o / o		
156.17 26.90	CLAYEY SILT, some sand, trace gravel Very stiff Grey	21	SS	7	156		⊢o⊣		
154.64 28.43	SILTY CLAY, some sand, trace gravel Very stiff Grey				155	(>143.6)+			
		22	SS	4	154		o		
153.12						+ 1.5			

Continued Next Page

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

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

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London, Ontario

RECORD OF BOREHOLE No 314

METRIC

PROJECT 09-1132-0080
W.P.
LOCATION N 4678750.8 ;E 333462.3
ORIGINATED BY SM
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY LMK/DMB
DATUM GEODETIC
DATE December 7, 2009 - December 9, 2009
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	WATER CONTENT (%)	GR	SA	SI	CL
29.95	SILTY SAND, some gravel, trace clay Compact Grey	23	SS	21	Bentonite					o	11	44	35	10
						152								
						151								
		24	SS	23						o				
150.02 33.05	LIMESTONE, fresh, medium strong, weakly laminated, fine grained, faintly porous Light brown to grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)					150								
		25	NQ RC	-			90	68	46					
					Sand	149								
		26	NQ RC	-		148	100	87	87					
						147								
		27	NQ RC	-			100	53	58					
						146								
		28	NQ RC	-	Piezometer	145	100	100	100					
144.82 38.25	END OF BOREHOLE Borehole dry during drilling between December 7 and 9, 2009. Water level measured at elev. 178.35 on February 24, 2010. Water level measured at elev. 178.17 on January 6, 2010.													

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

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

PROJECT: 09-1132-0080

RECORD OF DRILLHOLE: 314

LOCATION: N 4678750.8 ;E 333462.3

DRILLING DATE: December 7, 2009 - December 9, 2009

DATUM: GEODETIC

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: AARDVARK

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	ELEVATION	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE	150.02				
	MUD ROTARY / NQ ROCK CORE	LIMESTONE, fresh, medium strong, weakly laminated, very fine to fine grained, faintly porous, light brown to grey	33.05	1	150	Broken core from 35.05m to 33.25m	
34		LIMESTONE, fresh, medium strong, massive to weakly laminated, very fine grained, faintly porous, light brown, occasional fossil fragments	148.93 / 34.14		149		
		LIMESTONE, fresh, medium strong, weakly laminated, fine to coarse grained, porous, light brown	148.60 / 34.47	2			
35					148		
36				3	147		
37		LIMESTONE, fresh, medium strong, weakly laminated, very fine grained, faintly porous, light brown to grey	146.04 / 37.03		146		
		LIMESTONE, fresh, medium strong, weakly laminated, fine to coarse grained, strongly porous with vugs, light brown to grey	145.58 / 37.49				
38			37.73	4	145		
		LIMESTONE, fresh, medium strong, massive to weakly laminated, very fine to fine grained, faintly porous, light brown to grey, occasional fossils and pits	144.82 / 38.25				
39		END OF DRILLHOLE					
40							
41							
42							
43							
44							
45							
46							
47							
48							

Column headings: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH; COLOUR % RETURN; RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 (5 10 15 20); DIP w.r.t. CORE AXIS (0 30 60 90); HYDRAULIC CONDUCTIVITY k, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: SG

CHECKED:

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-315

METRIC

PROJECT	09-1132-0080		
W.P.		LOCATION	N 4678800.6 ;E 333406.3
ORIGINATED BY	TA		
DIST	WEST	HWY	401 / 3
BOREHOLE TYPE	POWER AUGER, SOLID STEM	COMPILED BY	DMB
DATUM	GEODETIC	DATE	January 21, 2010
CHECKED BY			

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.31	GROUND SURFACE									
0.00	TOPSOIL, clayey Very stiff Black						184			
			1	SS	16					
182.94 1.37	CLAYEY SILT, some sand, trace gravel, with occasional fissures and silt partings Stiff to hard Brown						183			
			2	SS	12					
			3	SS	11		182			
			4	SS	48		181			
						∇				
			5	SS	50		180			
179.89 4.42	END OF BOREHOLE Groundwater encountered at about elev. 180.5m during drilling on January 21, 2010.									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

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

Golder Associates
London, Ontario

PROJECT 09-1132-0080 **RECORD OF BOREHOLE No CPT-316** 1 OF 1 **METRIC**

W.P. LOCATION N 4678831.3 ;E 333265.0 ORIGINATED BY TA

DIST WEST HWY 401 / 3 BOREHOLE TYPE POWER AUGER, SOLID STEM COMPILED BY DMB

DATUM GEODETIC DATE January 21, 2010 CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.99	GROUND SURFACE										
0.00	TOPSOIL, clayey Black										
182.63 0.36	CLAYEY SILT, some sand, trace gravel, with occasional fissures, silt partings and seams Very stiff to hard Brown										
			1	SS	25		182				
						▽					
			2	SS	45				o		
							181				
			3	SS	56				o		
							180				
			4	SS	45						
179.33 3.66	END OF BOREHOLE Groundwater encountered at about elev. 181.5m during drilling on January 21, 2010.										

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

NILCON FIELD VANE SHEAR TEST RESULTS

Windsor-Essex Parkway

Depth (m)	Elevation (m)	Undrained Shear Strength (kPa) Natural	Post-Peak	Remoulded	Sensitivity
Field Vane Location 1 (Borehole BH-1)					
5.1	181.6	145	104	93	1.6
6.1	180.6	109	94	73	1.5
7.1	179.6	81	65	66	1.2
8.1	178.6	107	90	64	1.7
9.1	177.6	90	77	62	1.4
10.1	176.6	75	60	59	1.3
11.1	175.6	84	65	62	1.4
12.1	174.6	83	60	47	1.7
13.1	173.6	60	48	55	1.1
14.1	172.6	62	45	47	1.3
15.1	171.6	50	43	49	1.0
16.1	170.6	64	45	47	1.3
17.1	169.6	60	39	51	1.2
18.1	168.6	52	35	45	1.2
19.1	167.6	60	35	53	1.1
20.1	166.6	71	33	45	1.6
21.1	165.6	58	20	42	1.4
22.1	164.6	60	19	49	1.2
23.1	163.6	43	34		
Field Vane Location 7 (Borehole BH-7)					
6.1	177.1	108	93	76	1.4
7.1	176.1	81	58	47	1.7
8.1	175.1	80	51	36	2.2
9.1	174.1	69	41	28	2.4
10.1	173.1	61	48	17	3.6
11.1	172.1	64	47	30	2.1
12.1	171.1	65	44	32	2.0
13.1	170.1	53	28	23	2.3
14.1	169.1	50	31	19	2.6
15.1	168.1	59	44	30	1.9
16.1	167.1	46	16	15	3.1
17.1	166.1	40	17	21	1.9
18.1	165.1	22	13	15	1.5
19.1	164.1	61	36	40	1.5
Field Vane Location 14 (Borehole BH-14)					
6.0	176.0	93	62	35	2.6
7.0	175.0	57	29	15	3.8
8.0	174.0	62	37	29	2.1
9.0	173.0	51	28	24	2.2
10.0	172.0	48	26	24	2.0
11.0	171.0	49	26	24	2.1
12.0	170.0	44	26	23	2.0
13.0	169.0	42	24	21	2.0
14.0	168.0	64	50	22	3.0
15.0	167.0	38	13	16	2.4
16.0	166.0	38	8	14	2.7

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-7

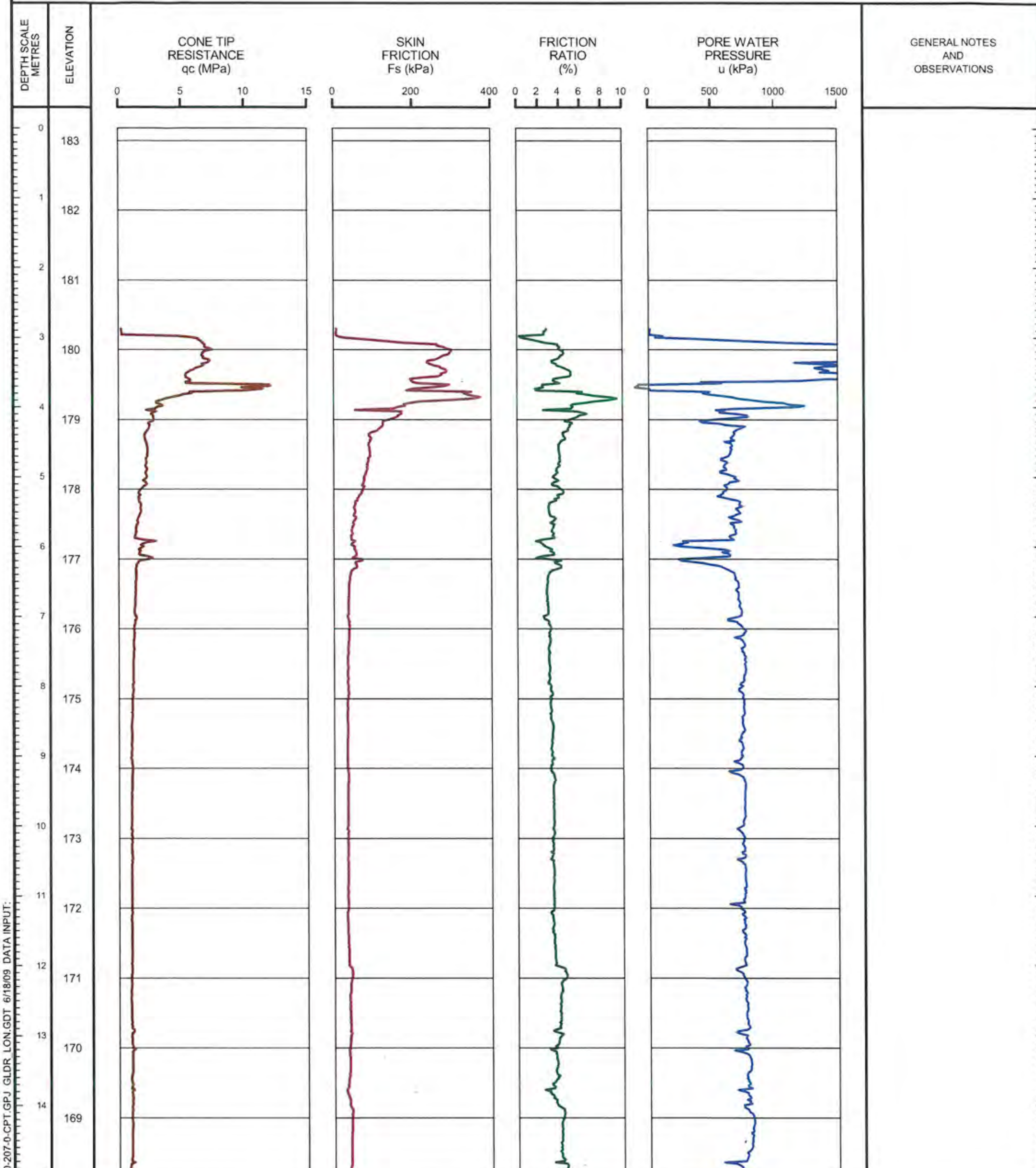
SHEET 1 OF 2

LOCATION: N 4678844.0 ;E 333327.0

TEST DATE: November 12, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 2.90m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



— CONTINUED NEXT PAGE —

DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

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

PROJECT: 07-1130-207-0

LOCATION: N 4678844.0 ;E 333327.0

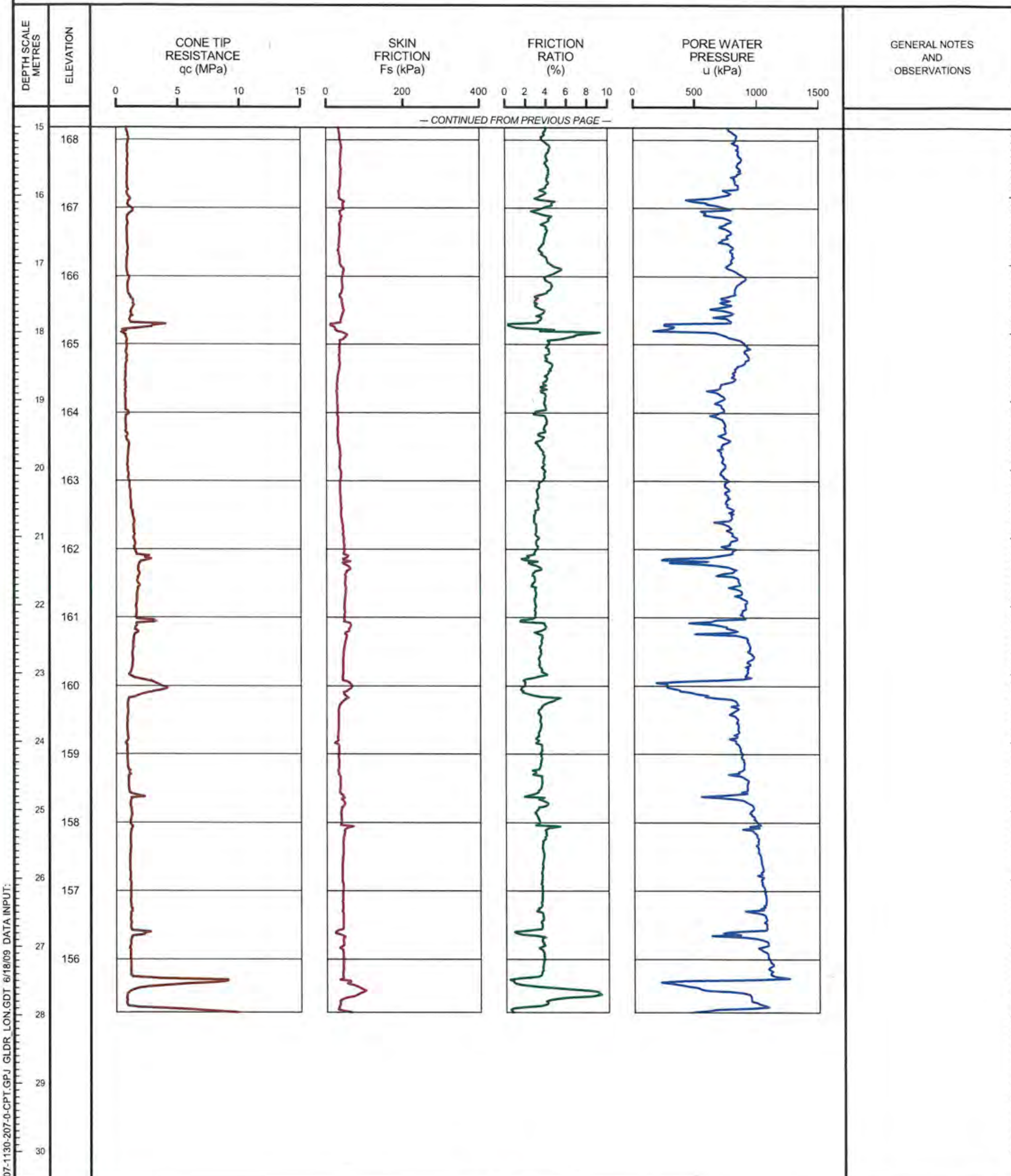
RECORD OF CONE PENETRATION TEST CPT-7

SHEET 2 OF 2

TEST DATE: November 12, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 2.90m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



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DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: GJB

PROJECT: 09-1132-0080

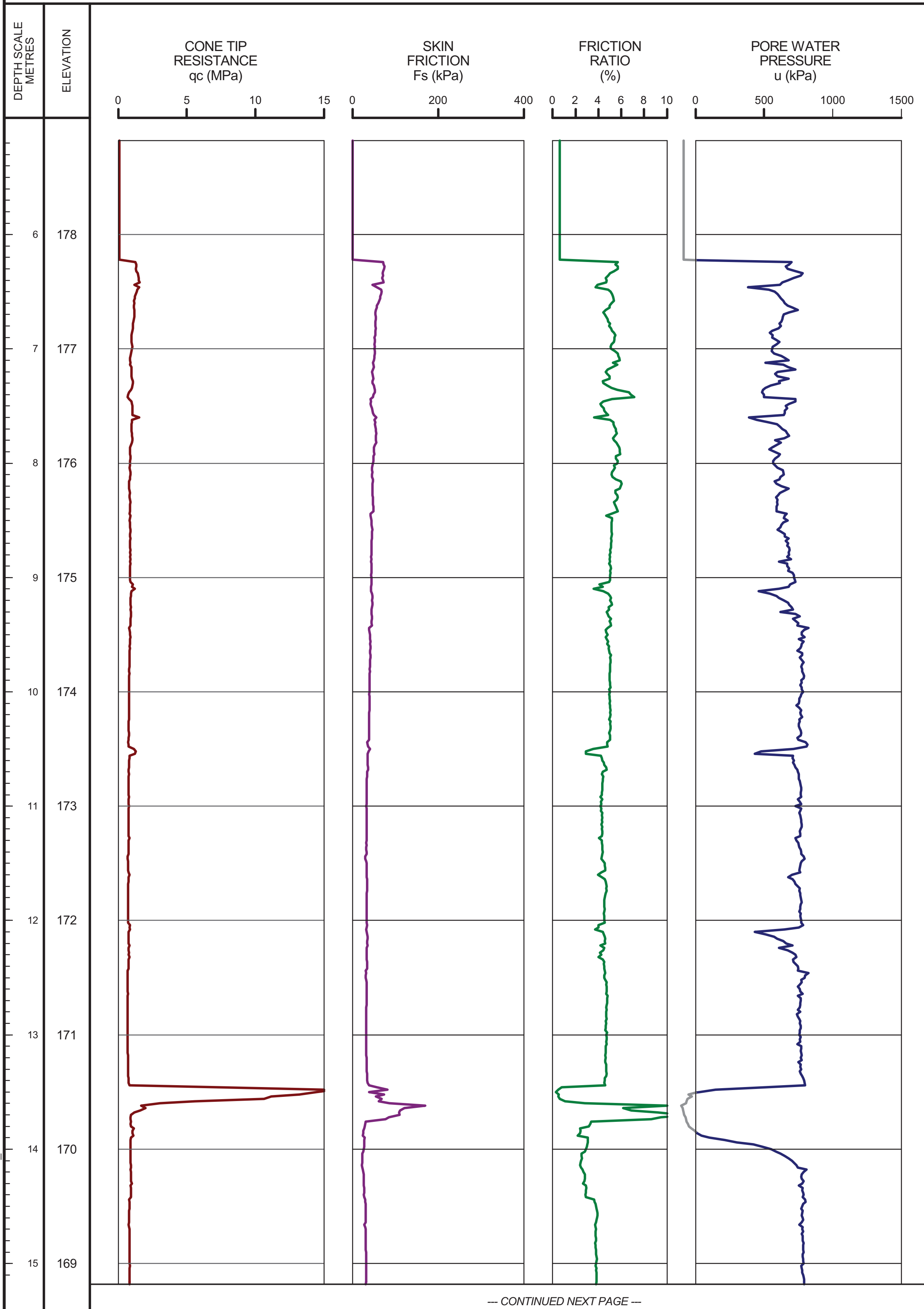
RECORD OF CONE PENETRATION TEST CPT-315

LOCATION: N 4678800.6 ;E 333406.3

TEST DATE: January 22, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 184.31m PREDRILL DEPTH: 5.18m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



GENERAL NOTES AND OBSERVATIONS

CPT PUSHED AFTER PREDRILLING SAMPLED HOLE.
FOR SOIL PROFILE, SEE RECORD OF BOREHOLE No. CPT-315.

--- CONTINUED NEXT PAGE ---

DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

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

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-315

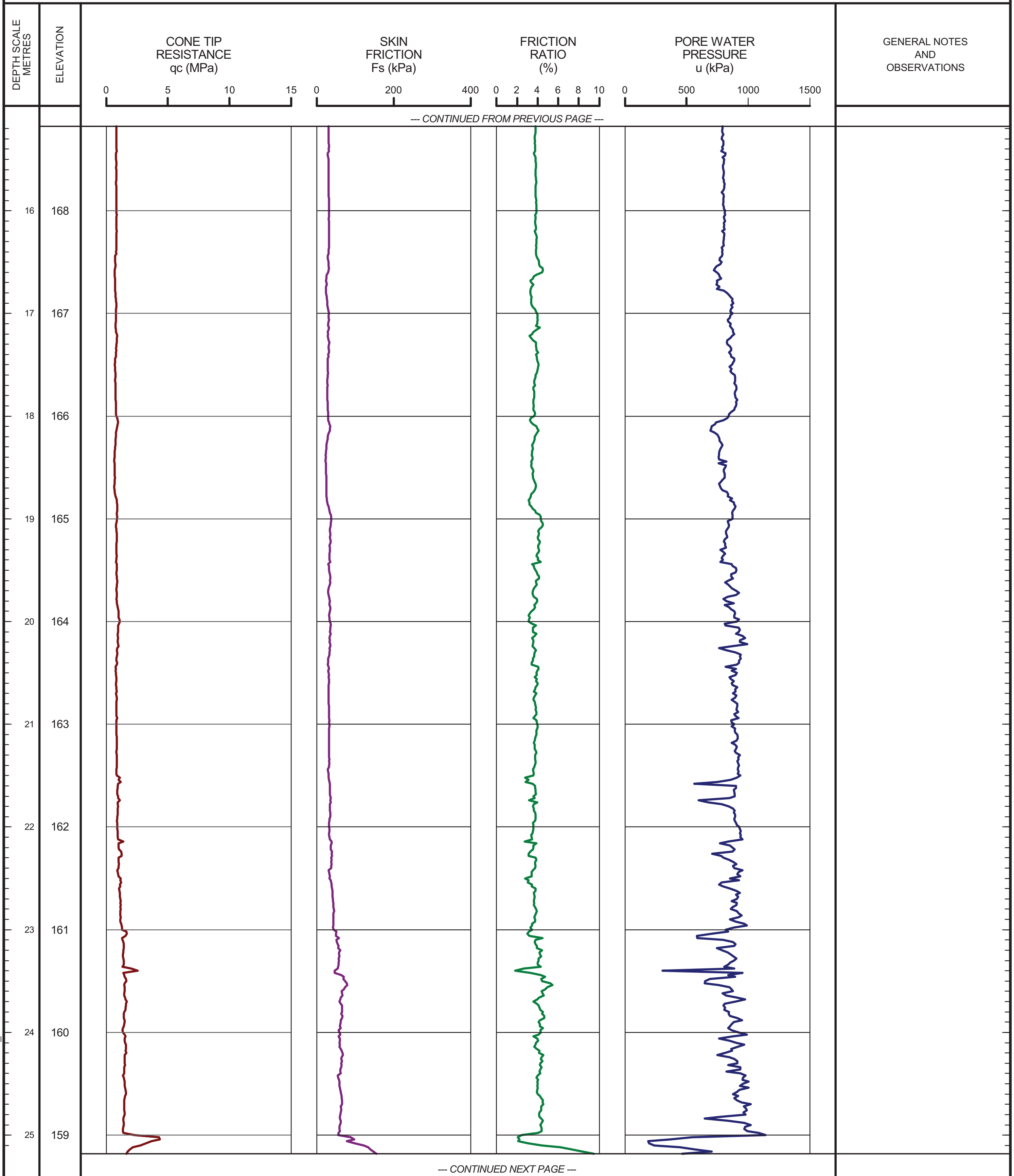
SHEET 2 OF 3

LOCATION: N 4678800.6 ;E 333406.3

TEST DATE: January 22, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 184.31m PREDRILL DEPTH: 5.18m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

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

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-315

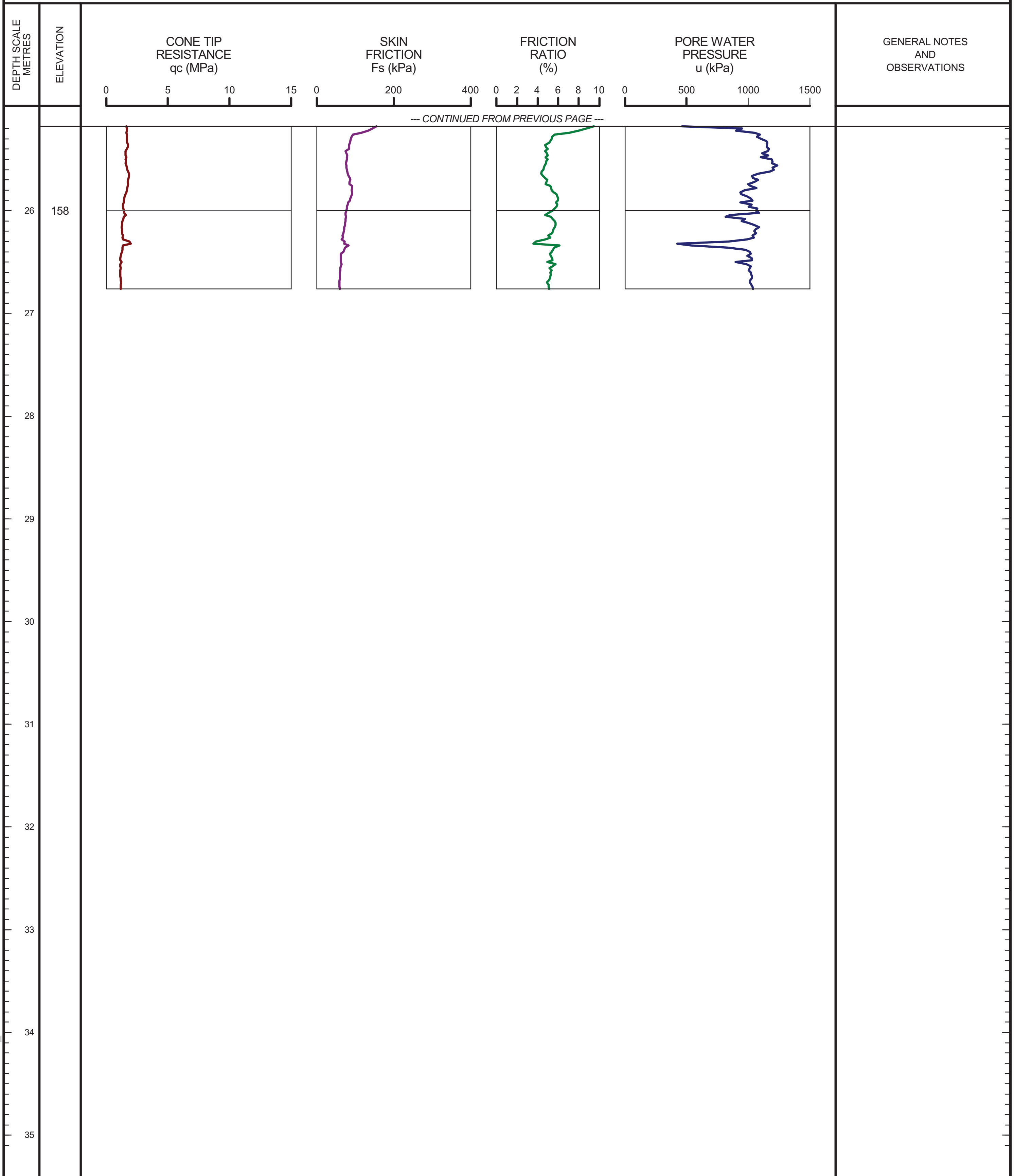
SHEET 3 OF 3

LOCATION: N 4678800.6 ;E 333406.3

TEST DATE: January 22, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 184.31m PREDRILL DEPTH: 5.18m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-316

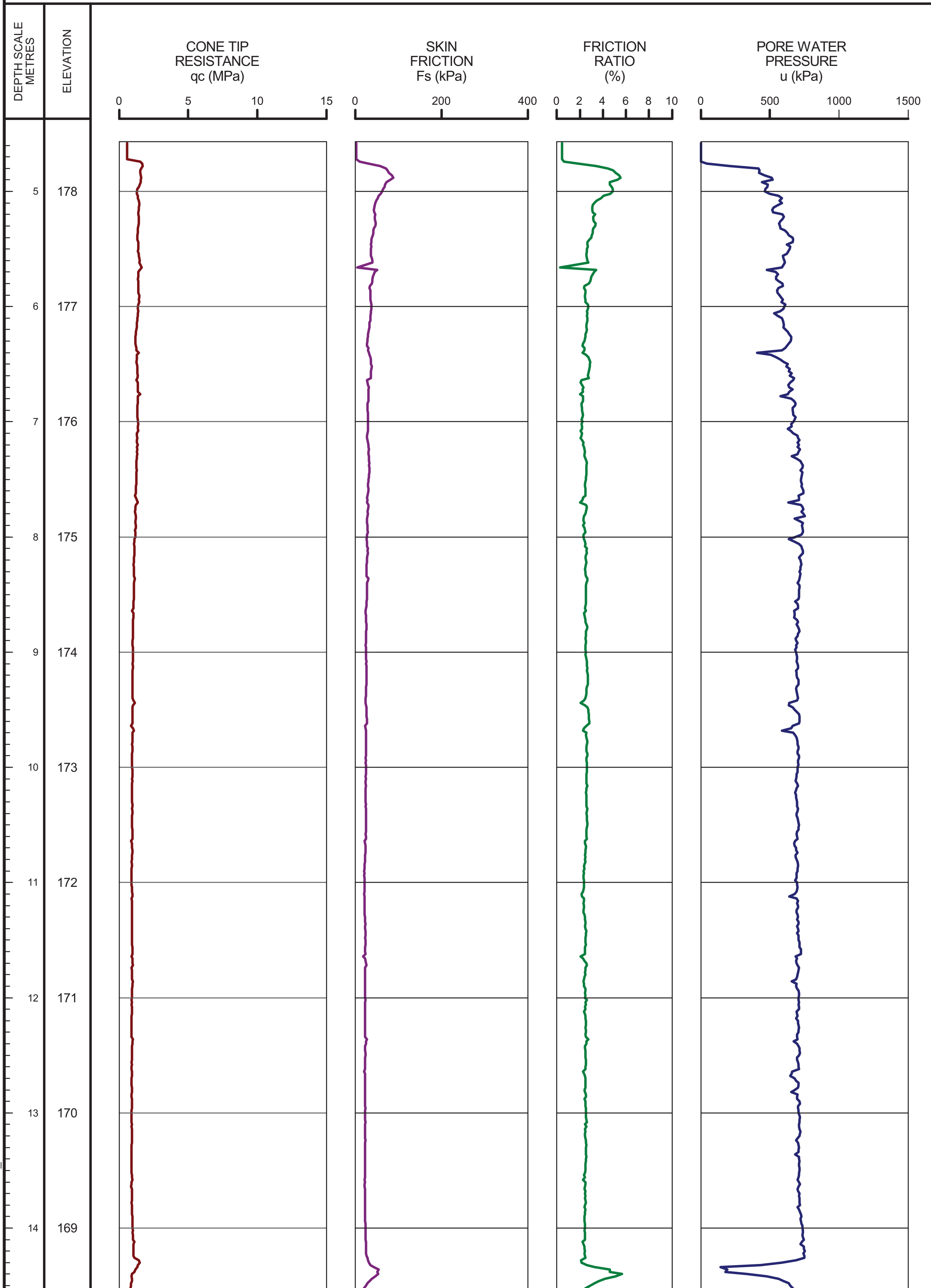
SHEET 1 OF 3

LOCATION: N 4678831.3 ;E 333265.0

TEST DATE: January 21, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 182.99m PREDRILL DEPTH: 4.57m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



GENERAL NOTES AND OBSERVATIONS

CPT PUSHED AFTER PREDRILLING SAMPLED HOLE.
FOR SOIL PROFILE, SEE RECORD OF BOREHOLE No. CPT-316.

--- CONTINUED NEXT PAGE ---

DEPTH SCALE
1 : 50

Golder Associates

OPERATOR: TA
CHECKED:

RECORD OF CONE PENETRATION TEST CPT-316

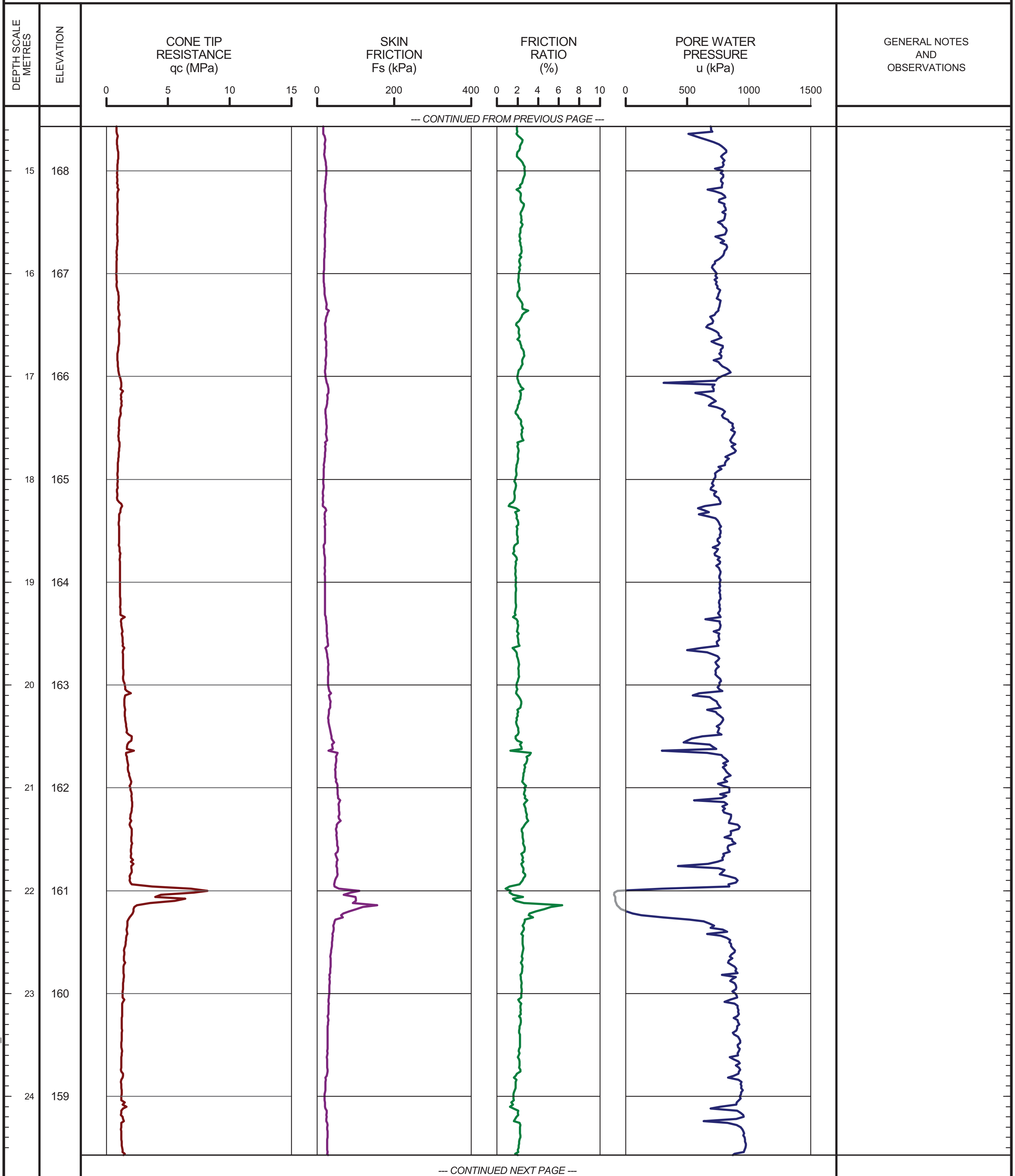
PROJECT: 09-1132-0080

LOCATION: N 4678831.3 ;E 333265.0

TEST DATE: January 21, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 182.99m PREDRILL DEPTH: 4.57m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



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

DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-316

LOCATION: N 4678831.3 ;E 333265.0

TEST DATE: January 21, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 182.99m PREDRILL DEPTH: 4.57m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013

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

--- CONTINUED FROM PREVIOUS PAGE ---

25
26
27
28
29
30
31
32
33
34

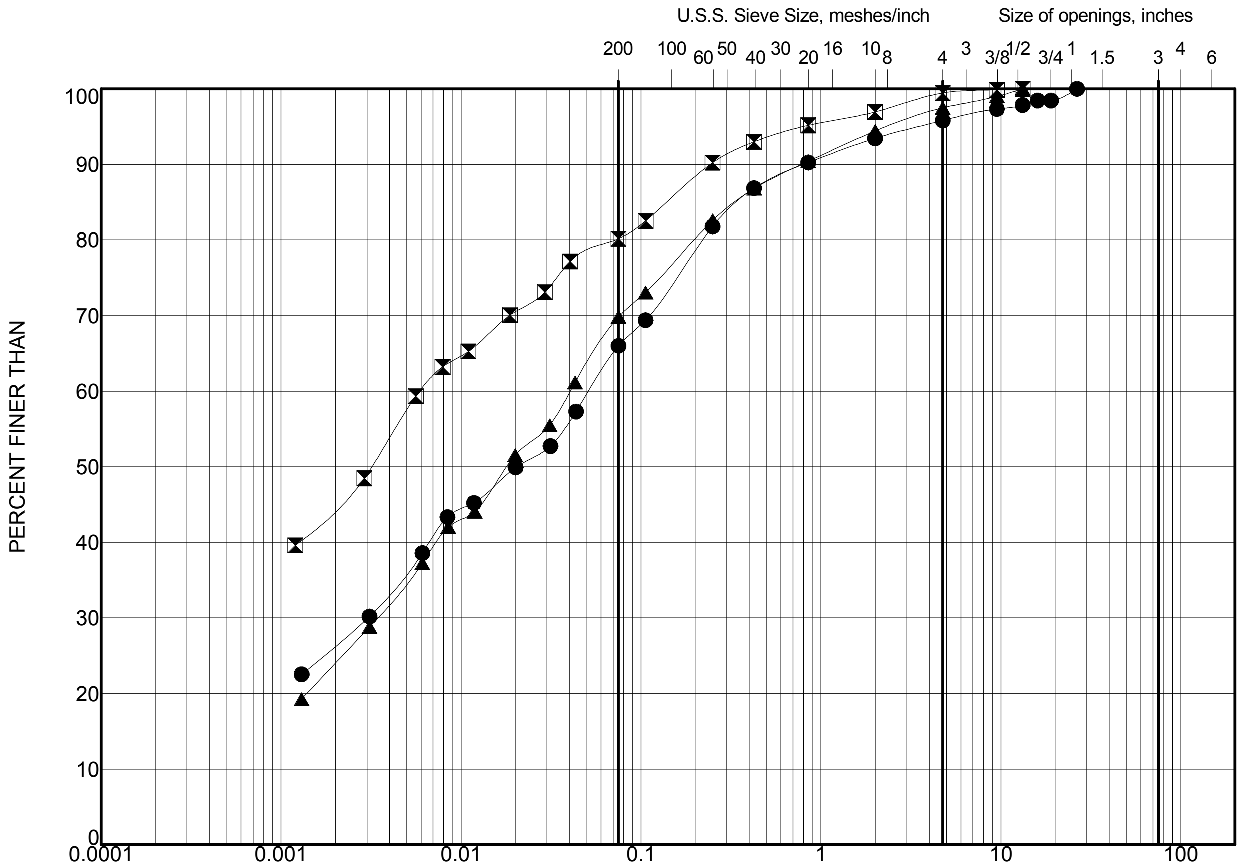
DEPTH SCALE
1 : 50

Golder Associates

OPERATOR: TA
CHECKED:

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

Appendix C Geotechnical Laboratory Test Results



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

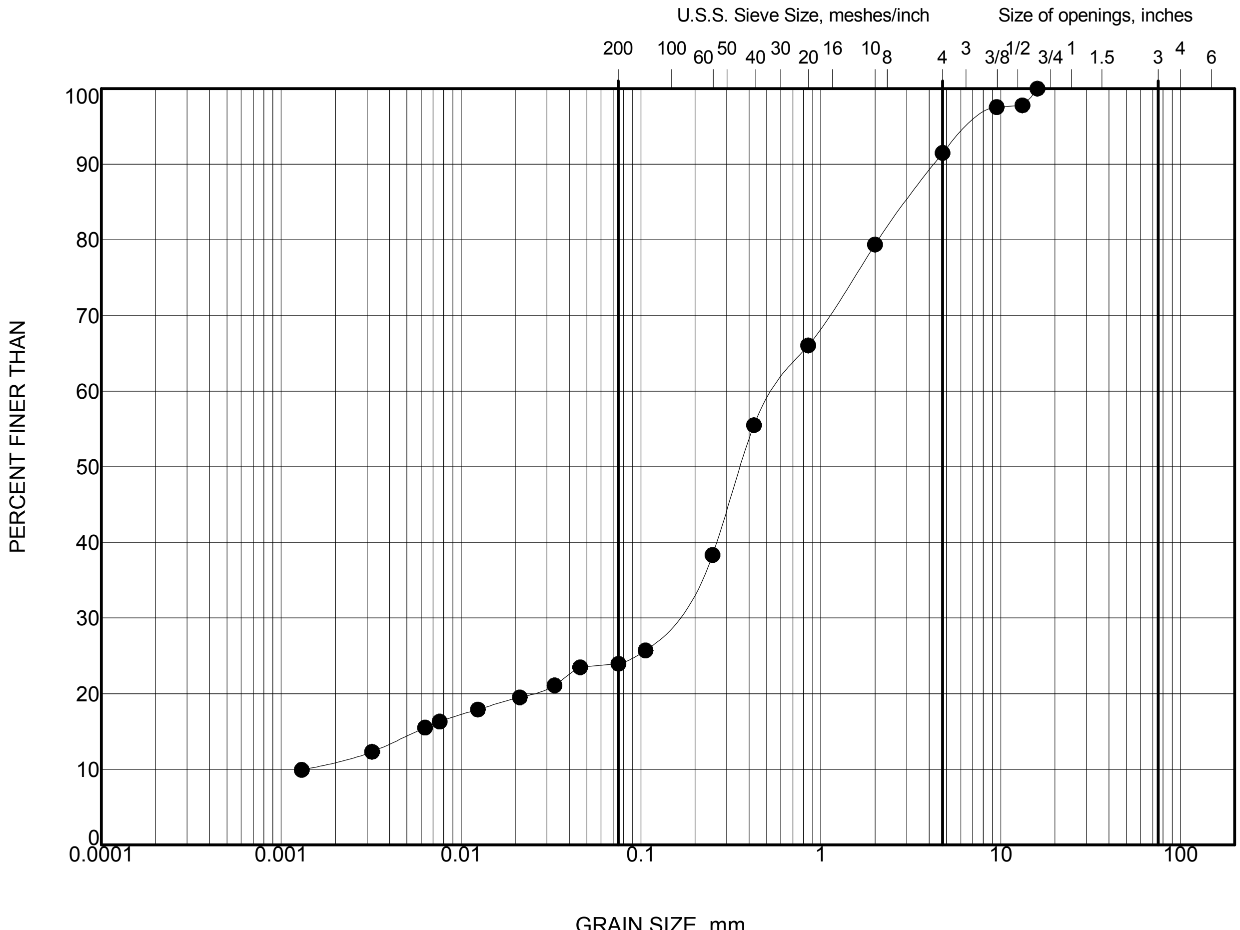
LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T8-1	11	9.1
⧗	T8-1	15	15.2
▲	T8-1	19	21.3

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

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.
		SCALE / REV.
DRAWN	EA	
CHECK	GN	FIGURE **C.1**



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	HG-MW-3	2	1.5

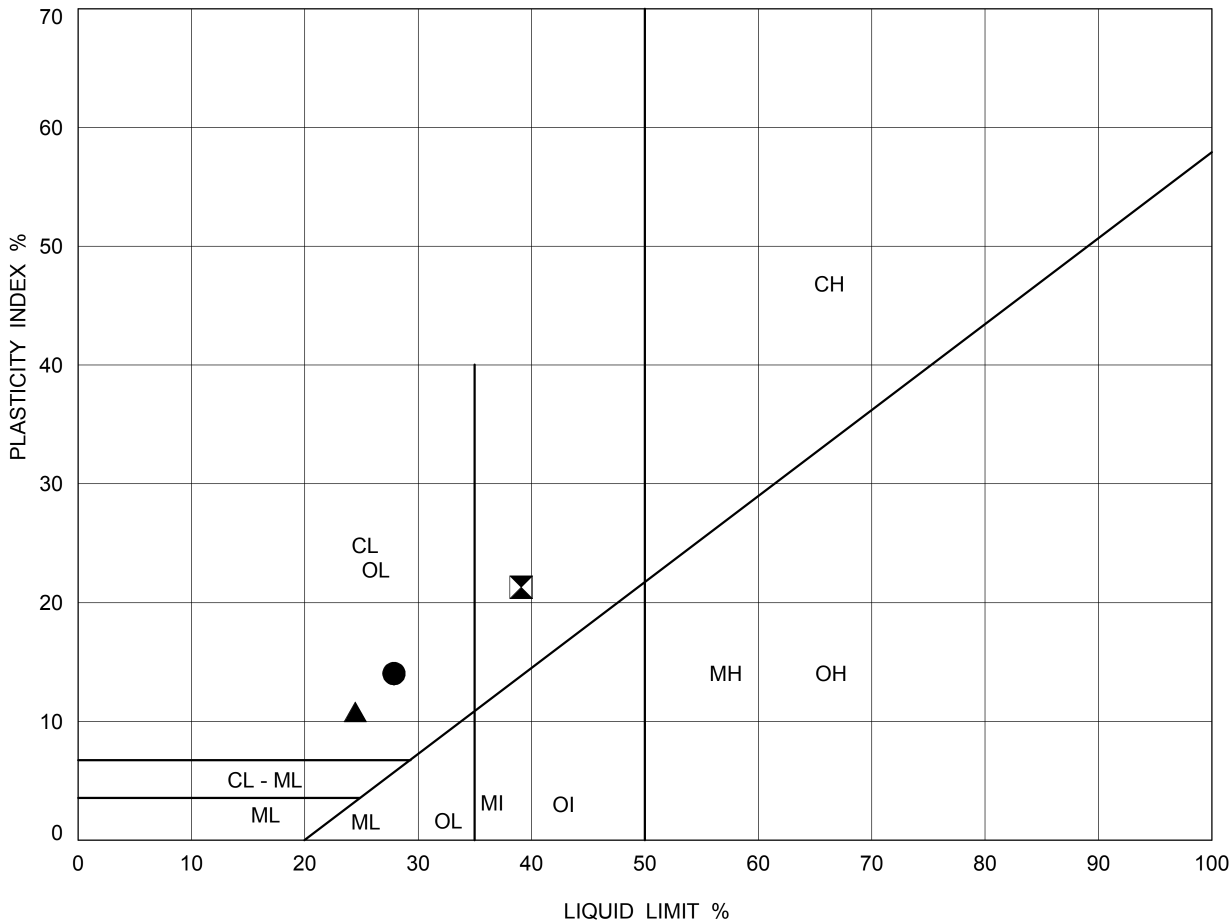
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION
Sand

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.	
		SCALE	REV.
DRAWN	EA	FIGURE **C.2**	
CHECK	GN		

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 06/09/12



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T8-1	11	9.1	**28**	**14**	**14**
⧗	T8-1	15	15.2	**39**	**18**	**21**
▲	T8-1	19	21.3	**24**	**14**	**10**

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE	PLASTICITY CHART **Silt and Clay**		
Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PROJECT No. SW8801.1004.101		FILE No.
			SCALE / REV.
	DRAWN	EA	FIGURE **C.3**
	CHECK	GN	

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 06/09/12

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 5-Jan-12 **Sample ID:** T8-1_TW16 **Depth(m):** 16.8 to 17.4

Test Data

Ring # :	A	Ring Height (in) =	0.755	Wt of dry filter paper (g)	0.69
Wet soil + Ring Wt (g)			192.35	Wt of ring (g)	76.58
Wet soil + Wet Paper + Ring (g)			191.27	Wet Paper (g)	1.91
Dry Soil + Dry Paper + Ring (g)			167.40	Ring Dia (in)	2.498
Initial moisture Content (%)			28.45	Final moisture Content (%)	25.13
Area of Ring (in^2)			4.90	Initial Volume (in^3)	3.7002
Initial Bulk Density (kg/m^3)			1909	Initial Dry Density (kg/m^3)	1486
Specific Gravity of Soil			2.74	Eqiv. Thick. of solids (mm)	10.396
Final Bulk Density (kg/m^3)			2031	Final Dry Density (kg/m^3)	1623
Initiall gauge reading for Load 1			0.2553	Gauge reading for last Loading	0.1918
Initial Voids Ratio			0.845	Final Void Ratio	0.690
Initial Degree of Saturation (%)			92	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	3.5	5.0	8.0	12.0	17.5	26.5	40.0
Load (tsf)	0.0364	0.052	0.083	0.125	0.182	0.276	0.416
Gauge Reading (in)	0.2553	0.2553	0.2550	0.2538	0.2522	0.24975	0.2469
(H-Hs) mm	8.781	8.781	8.772	8.743	8.701	8.640	8.567
Voids ratio	0.845	0.845	0.844	0.841	0.837	0.831	0.824
t90 (min)			0.25	2.56	4.41	3.61	4.00
Cv (m^2/day)			0.449	0.044	0.025	0.031	0.028
k' (MPa)			6.471	2.625	2.512	2.819	3.489
Mv (mm^2 / N)			0.1545	0.3810	0.3981	0.3547	0.2866

Trial #	8	9	10	11	12	13	14
Load (kPa)	60	90.0	135.0	200.0	300.0	200.0	135.0
Load (tsf)	0.624	0.936	1.404	2.080	3.120	2.080	1.404
Gauge Reading (in)	0.24285	0.2378	0.2309	0.2213	0.2085	0.2095	0.2105
(H-Hs) mm	8.465	8.337	8.160	7.918	7.592	7.618	7.643
Voids ratio	0.814	0.802	0.785	0.762	0.730	0.733	0.735
t90 (min)	2.56	2.25	2.56	2.40	2.56		
Cv (m^2/day)	0.043	0.048	0.041	0.043	0.039		
k' (MPa)	3.733	4.411	4.775	4.972	5.633		
Mv (mm^2 / N)	0.2679	0.2267	0.2094	0.2011	0.1775		

Trial #	15	16	17	18	19	20	21
Load (kPa)	90.0	60.0	40.0	60.0	90.0	135.0	200.0
Load (tsf)	0.936	0.624	0.416	0.624	0.936	1.404	2.080
Gauge Reading (in)	0.21247	0.2148	0.2177	0.2164	0.2144	0.2121	0.2094
(H-Hs) mm	7.693	7.752	7.826	7.794	7.741	7.684	7.616
Voids ratio	0.740	0.746	0.753	0.750	0.745	0.739	0.733
t90 (min)				2.25	1.00	1.44	1.21
Cv (m^2/day)				0.045	0.101	0.070	0.082
k' (MPa)				11.209	10.379	14.281	17.264
Mv (mm^2 / N)				0.0892	0.0964	0.0700	0.0579

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 5-Jan-12 **Sample ID:** T8-1_TW16 **Depth(m):** 16.8 to 17.4

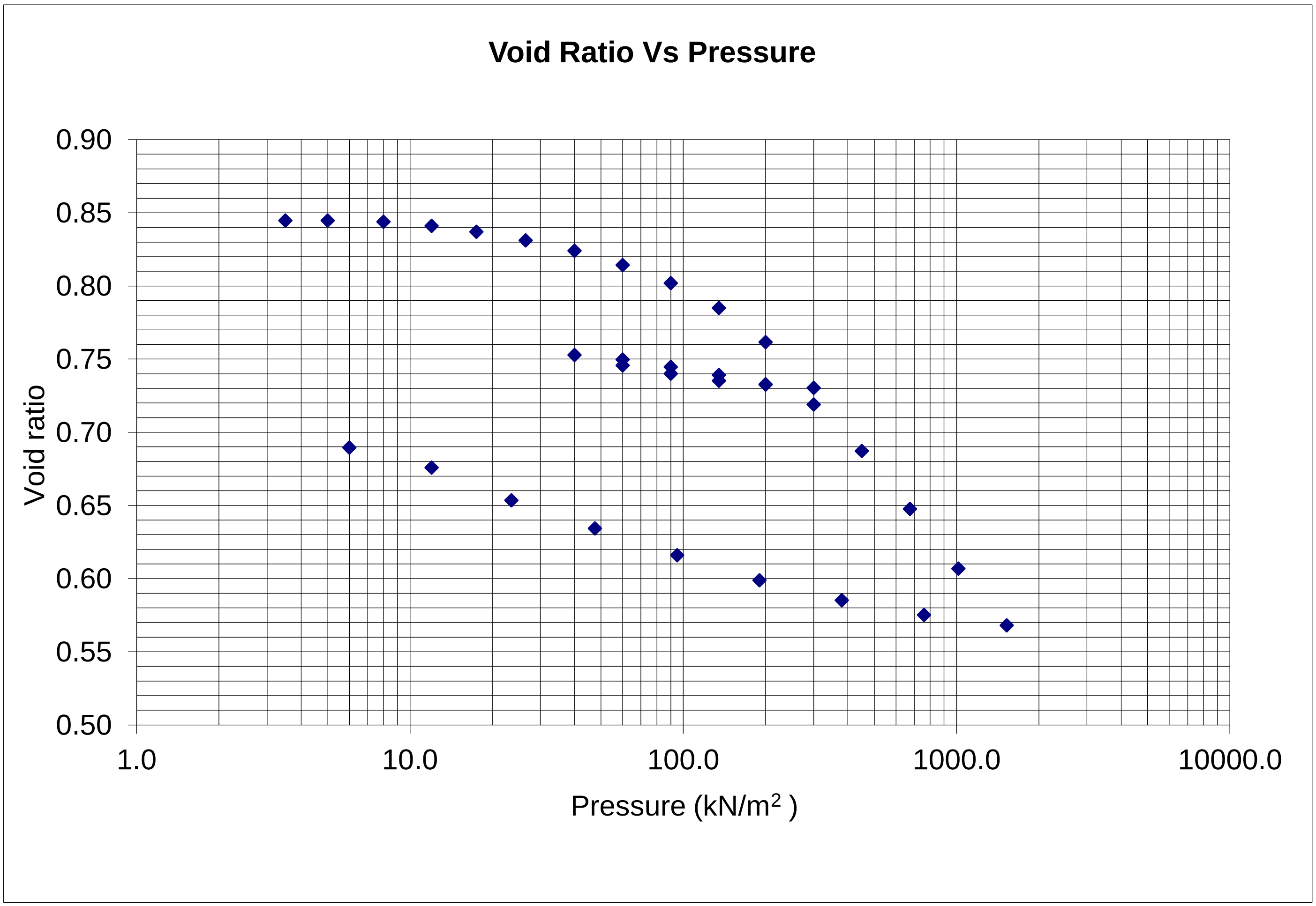
Trial #	22	23	24	25	26	27	28
Load (kPa)	300.0	450.0	675	1015.0	1525.0	760.0	380.0
Load (tsf)	3.12	4.680	7.02	10.556	15.860	7.904	3.952
Gauge Reading (in)	0.20385	0.1909	0.17465	0.1580	0.1421	0.1450	0.1491
(H-Hs) mm	7.474	7.144	6.733	6.308	5.905	5.980	6.084
Voids ratio	0.719	0.687	0.648	0.607	0.568	0.575	0.585
t90 (min)	1.44	2.56	3.24	2.56	1.96		
Cv (m^2/day)	0.045	0.037	0.028	0.034	0.042		
k' (MPa)	12.731	7.968	9.366	13.389	20.624		
Mv (mm^2 / N)	0.0785	0.1232	0.1043	0.0728	0.0473		

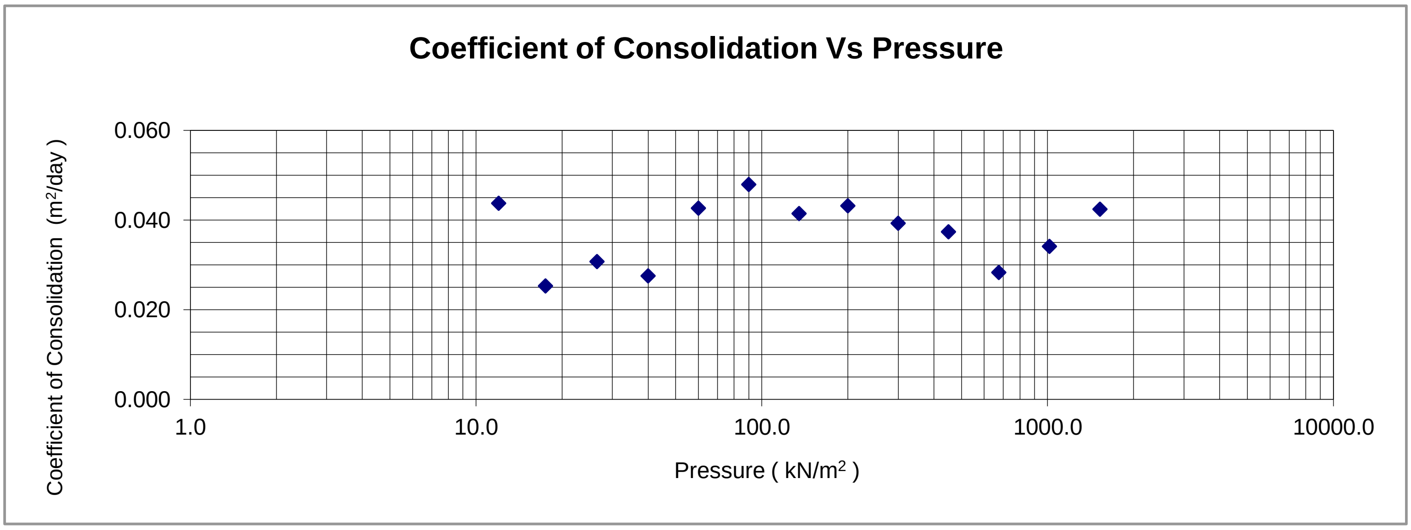
Trial #	29	30	31	32	33	34	
Load (kPa)	190.0	95.0	47.5	23.5	12.0	6.0	
Load (tsf)	1.976	0.988	0.494	0.244	0.125	0.062	
Gauge Reading (in)	0.1547	0.1617	0.1692	0.1771	0.1862	0.1918	
(H-Hs) mm	6.226	6.404	6.594	6.794	7.026	7.169	
Voids ratio	0.599	0.616	0.634	0.653	0.676	0.690	
t90 (min)							
Cv (m^2/day)							
k' (MPa)							
Mv (mm^2 / N)							

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 5-Jan-12 **Sample ID:** T8-1_TW16 **Depth(m):** 16.8 to 17.4

σ'_v versus e and c_v





ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 5-Jan-12 **Sample ID:** T8-1_TW16 **Depth(m):** 16.8 to 17.4

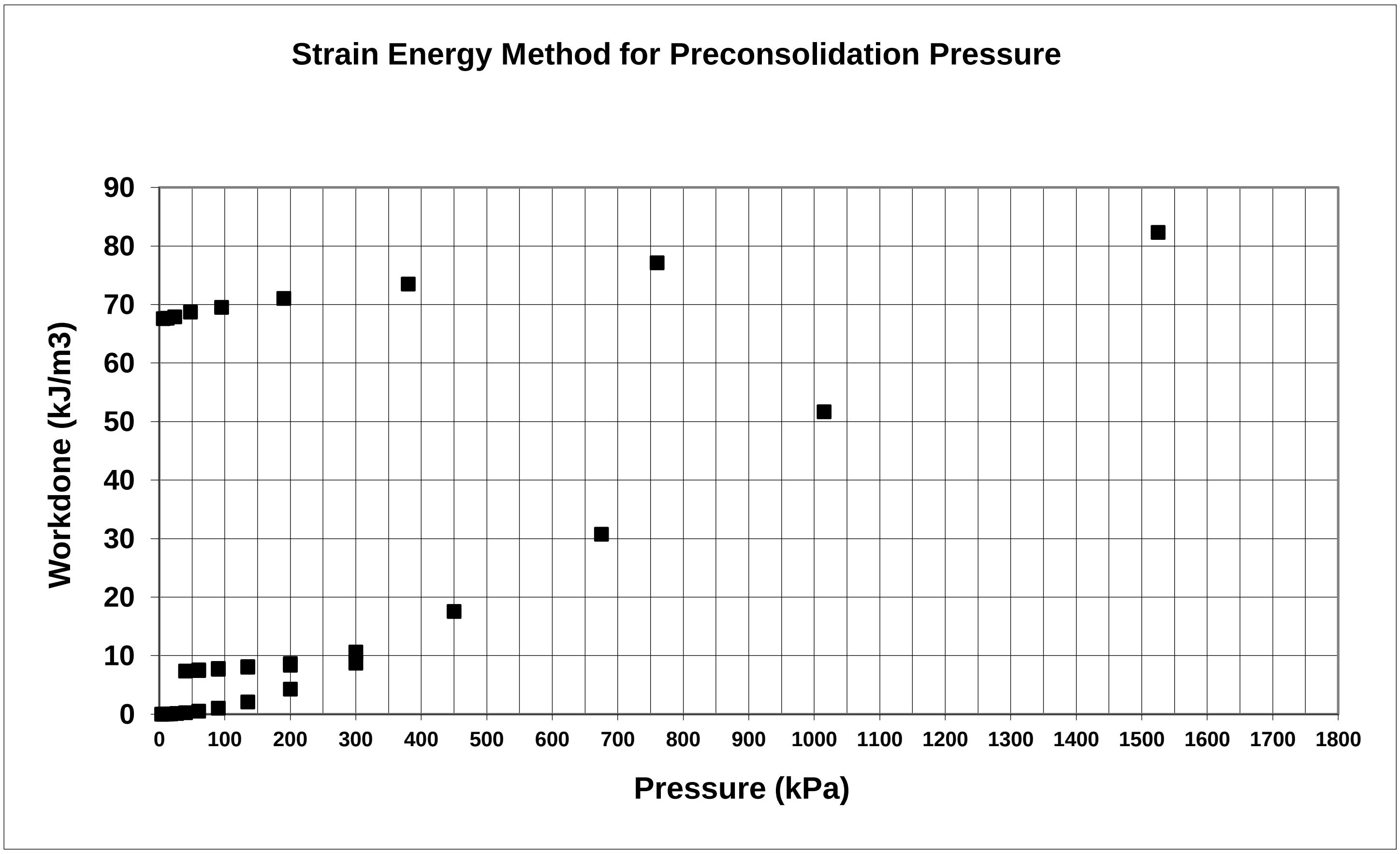
Strain Energy Data

Presssure (kN/m^2)	c_v (m^2/day)	Void ratio
3.5		0.845
5.0		0.845
8.0		0.844
12.0	0.044	0.841
17.5	0.025	0.837
26.5	0.031	0.831
40.0	0.028	0.824
60.0	0.043	0.814
90.0	0.048	0.802
135.0	0.041	0.785
200.0	0.043	0.762
300.0	0.039	0.730
200.0		0.733
135.0		0.735
90.0		0.740
60.0		0.746
40.0		0.753
60.0		0.750
90.0		0.745
135.0		0.739
200.0		0.733
300.0		0.719
450.0	0.037	0.687
675.0	0.028	0.648
1015.0	0.034	0.607
1525.0	0.042	0.568
760.0		0.575
380.0		0.585
190.0		0.599
95.0		0.616
47.5		0.634
23.5		0.653
12.0		0.676
6.0		0.690

Presssure (KN/m^2)	Height mm	Total Work (KJ/m^3)
3.5	19.177	0.000
5.0	19.177	0.000
8.0	19.168	0.003
12.0	19.139	0.018
17.5	19.097	0.051
26.5	19.036	0.121
40.0	18.962	0.249
60.0	18.861	0.517
90.0	18.733	1.027
135.0	18.556	2.088
200.0	18.313	4.277
300.0	17.988	8.715
200.0	18.014	8.362
135.0	18.039	8.126
90.0	18.089	7.814
60.0	18.148	7.572
40.0	18.222	7.367
60.0	18.189	7.456
90.0	18.137	7.673
135.0	18.080	8.027
200.0	18.012	8.658
300.0	17.870	10.622
450.0	17.540	17.551
675.0	17.128	30.747
1015.0	16.704	51.673
1525.0	16.301	82.320
760.0	16.375	77.122
380.0	16.480	73.497
190.0	16.622	71.037
95.0	16.800	69.513
47.5	16.990	68.705
23.5	17.380	67.890
12.0	17.612	67.653
6.0	17.755	67.580

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 5-Jan-12 **Sample ID:** T8-1_TW16 **Depth(m):** 16.8 to 17.4



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

Project: WEP **Project No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited **Date:** 9-Jan-12
Location: Windsor, ON. **Sample ID:** T8-1_TW16 **Depth(m):** 16.8 to 17.4

Sample Description: Inorganic Clay Medium Plasticity

Sample Parameters

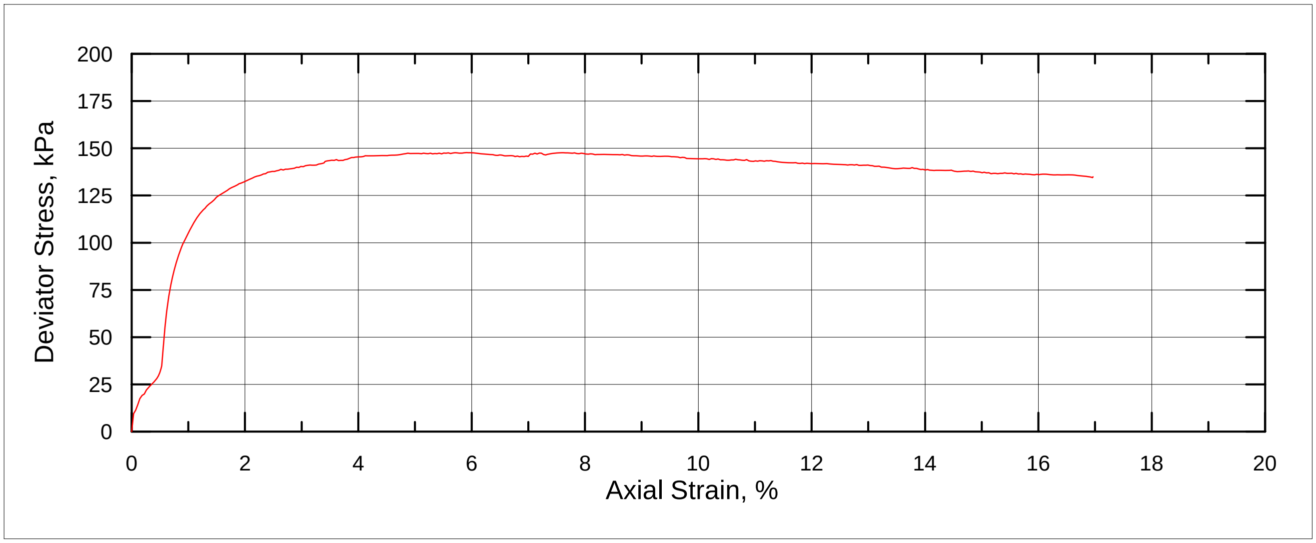
Initial		Specimen 1	Specimen 2	Specimen 3
Diameter	cm	6.925		
Height	cm	14.015		
Volume	cm^3	527.865		
Wet Mass	g	1057.37		
Dry Density	kg/m^3	1613		
Water Content	%	24.2		
Specific Gravity	Actual	2.742		
Void Ratio		0.70		
Degree of Saturation		94.8		
Before Shear (after consolidation)				
Volume	cm^3	501.965		
B - Value		1.00		
After Shear				
Wet Mass	g	1039.49		
Dry Density	kg/m^3	1673		
Water Content	%	23.8		
Void Ratio		0.64		
Degree of Saturation		100.0		

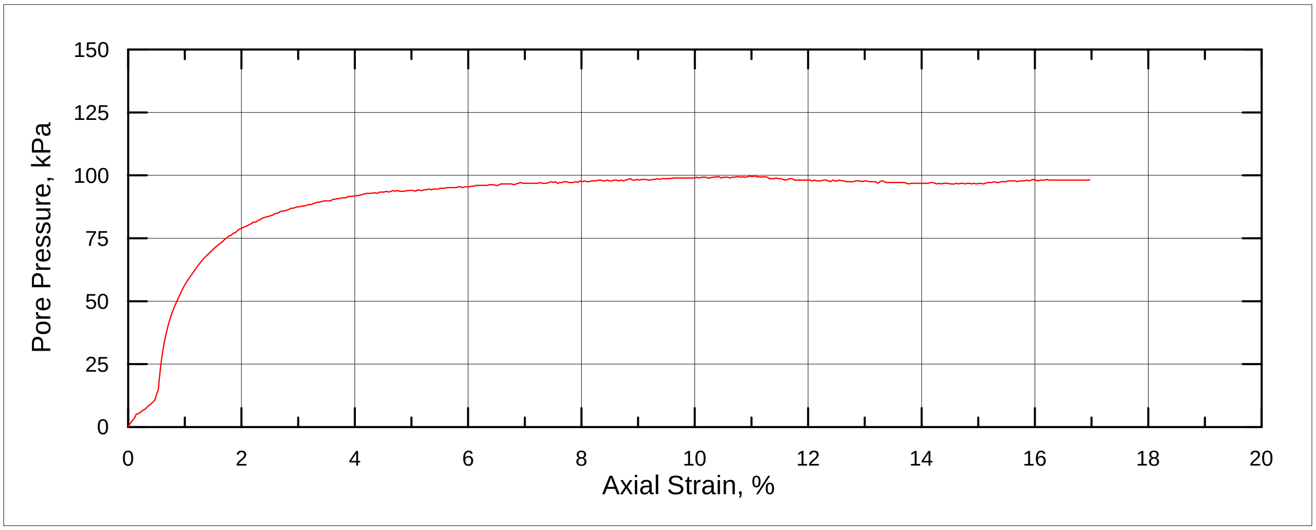
Stress - Strain				
Cell Pressure	kPa	370.00		
Back Pressure	kPa	190.00		
Consolidation Stress	kPa	180.00		
Rate of Strain	mm/min	0.0180		
Vertical Strain at Failure	%	5.90		
Deviator Stress at Failure	kPa	147.77		
Pore Pressure at Failure	kPa	95.10		

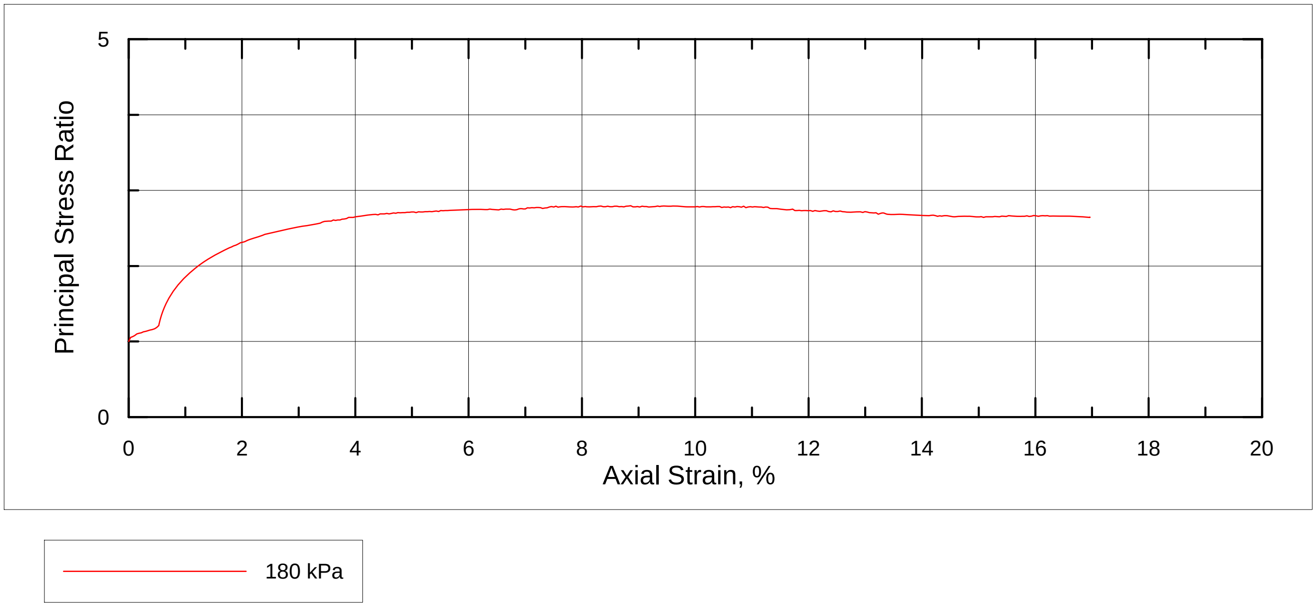
Total Stress				
Minor Principal Stress, σ_3	kPa	180.00		
Major Principal Stress, σ_1	kPa	327.77		
Radius, $(\sigma_1-\sigma_3)/2$	kPa	73.89		
Intersection Point, $(\sigma_1+\sigma_3)/2$	kPa	253.89		

Effective Stress				
Minor Principal Stress, σ_3'	kPa	84.90		
Major Principal Stress, σ_1'	kPa	232.67		
Radius, $(\sigma_1'-\sigma_3')/2$	kPa	73.89		
Intersection Point, $(\sigma_1'+\sigma_3')/2$	kPa	158.79		

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

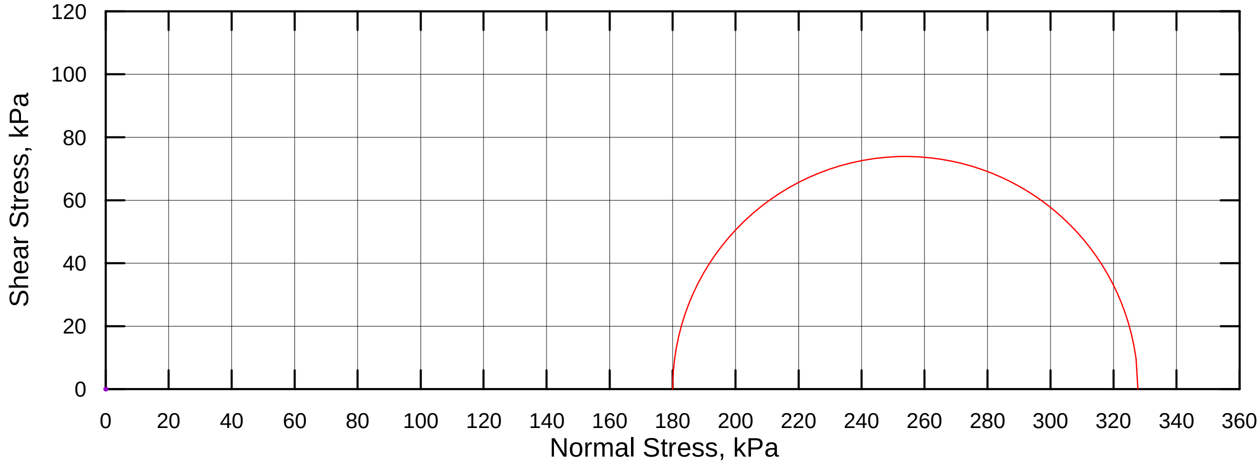




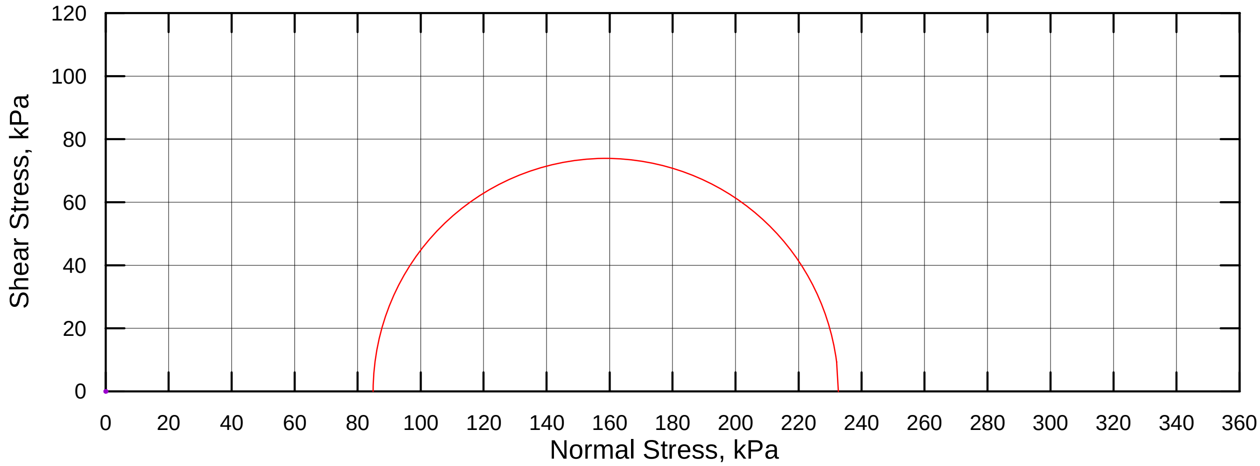


CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST
FOR COHESIVE SOILS (ASTM D- 4767)
(Multi specimen - single stage)
(Failure based on maximum deviator stress)

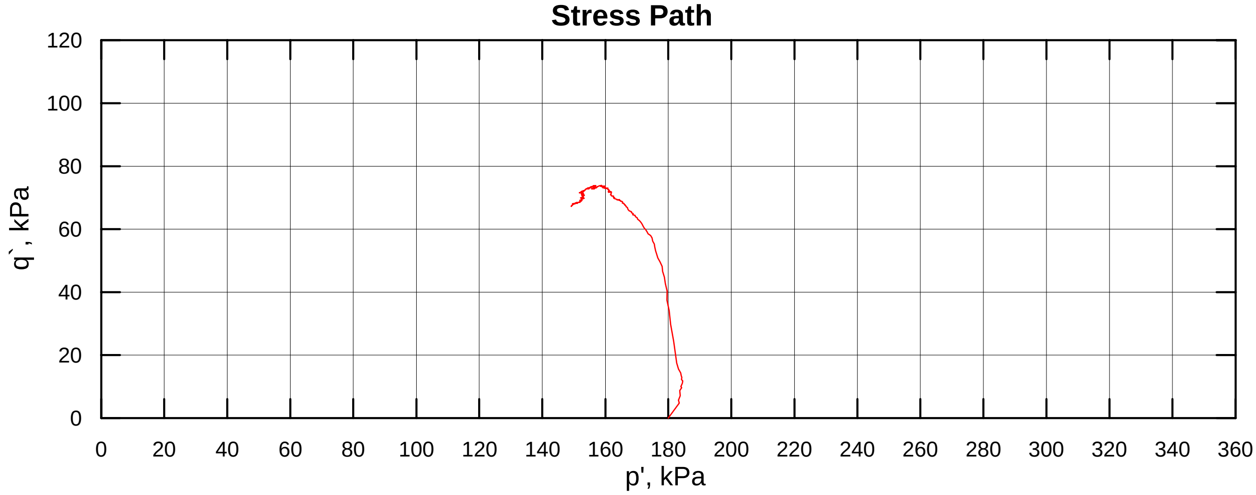
Mohr Stress Circles based on Total Stress



Mohr Stress Circles based on Effective Stress



Stress Path



180 kPa

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	7
Borehole Number	7	Sample Depth, m	8.5-8.9

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	11/13/2006		
Date Completed	11/25/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m^3	21.44
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	18.35
Area, cm^2	31.65	Specific Gravity, measured	2.73
Volume, cm^3	60.45	Solids Height, cm	1.309
Water Content, %	16.88	Volume of Solids, cm^3	41.43
Wet Mass, g	132.19	Volume of Voids, cm^3	19.02
Dry Mass, g	113.10	Degree of Saturation, %	100.4

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.910	0.459	1.910				
4.70	1.875	0.432	1.893	8	9.49E-02	3.90E-03	3.63E-05
9.54	1.865	0.425	1.870	7	1.06E-01	1.08E-03	1.12E-05
19.26	1.853	0.416	1.859	43	1.70E-02	6.46E-04	1.08E-06
38.70	1.837	0.403	1.845	46	1.57E-02	4.31E-04	6.63E-07
77.44	1.819	0.390	1.828	53	1.34E-02	2.43E-04	3.19E-07
154.87	1.794	0.371	1.807	76	9.10E-03	1.69E-04	1.51E-07
309.20	1.757	0.342	1.776	94	7.11E-03	1.26E-04	8.75E-08
618.55	1.711	0.307	1.734	124	5.14E-03	7.79E-05	3.92E-08
1241.52	1.660	0.268	1.686	68	8.86E-03	4.29E-05	3.72E-08
2478.24	1.608	0.228	1.634	146	3.88E-03	2.20E-05	8.36E-09
1241.52	1.614	0.233	1.611				
309.20	1.633	0.248	1.624				
77.44	1.659	0.267	1.646				
19.29	1.691	0.292	1.675				
4.85	1.717	0.312	1.704				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

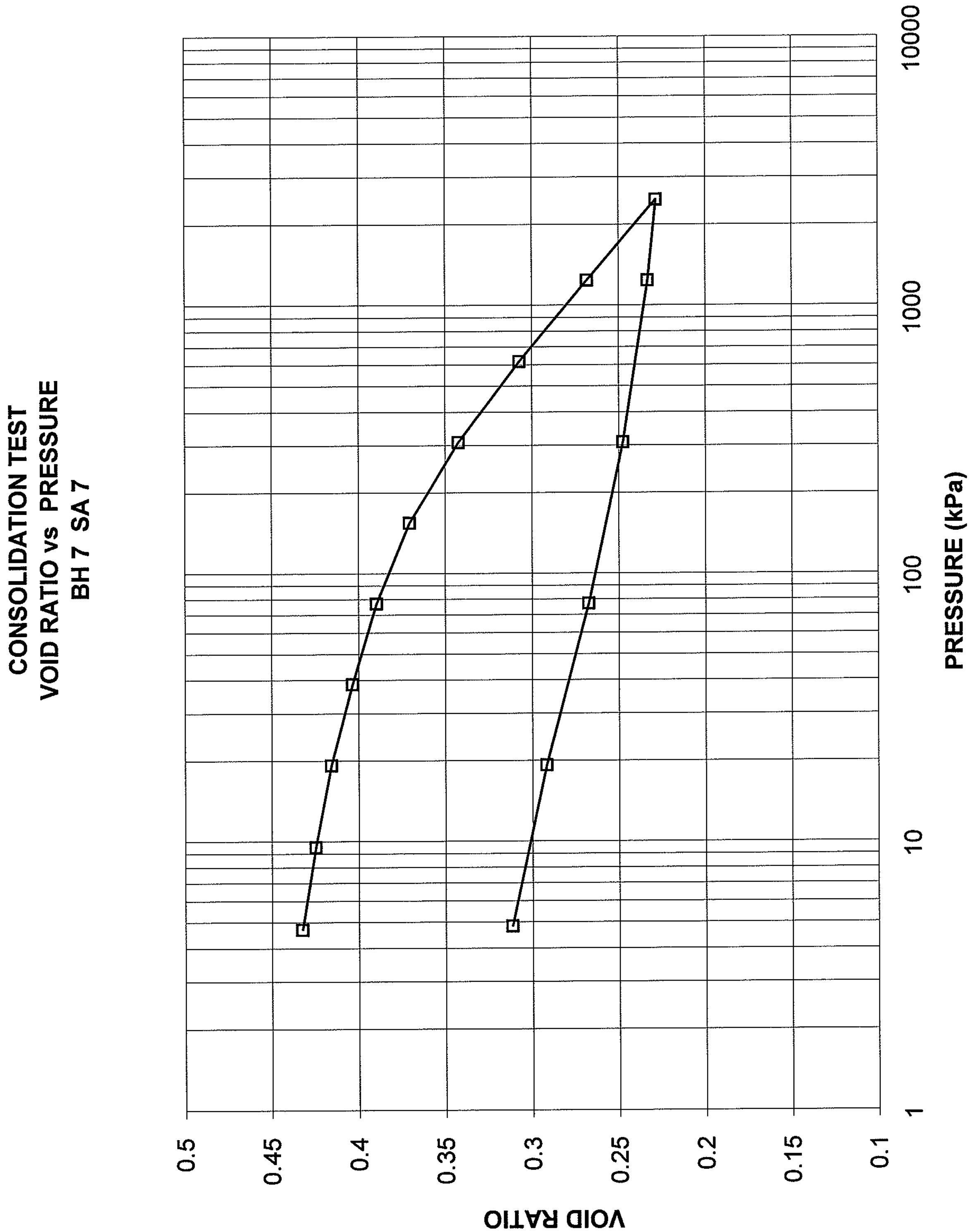
Sample Height, cm	1.72	Unit Weight, kN/m^3	23.31
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	20.41
Area, cm^2	31.65	Specific Gravity, measured	2.73
Volume, cm^3	54.34	Solids Height, cm	1.309
Water Content, %	14.20	Volume of Solids, cm^3	41.43
Wet Mass, g	129.16	Volume of Voids, cm^3	12.91
Dry Mass, g	113.1		

Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

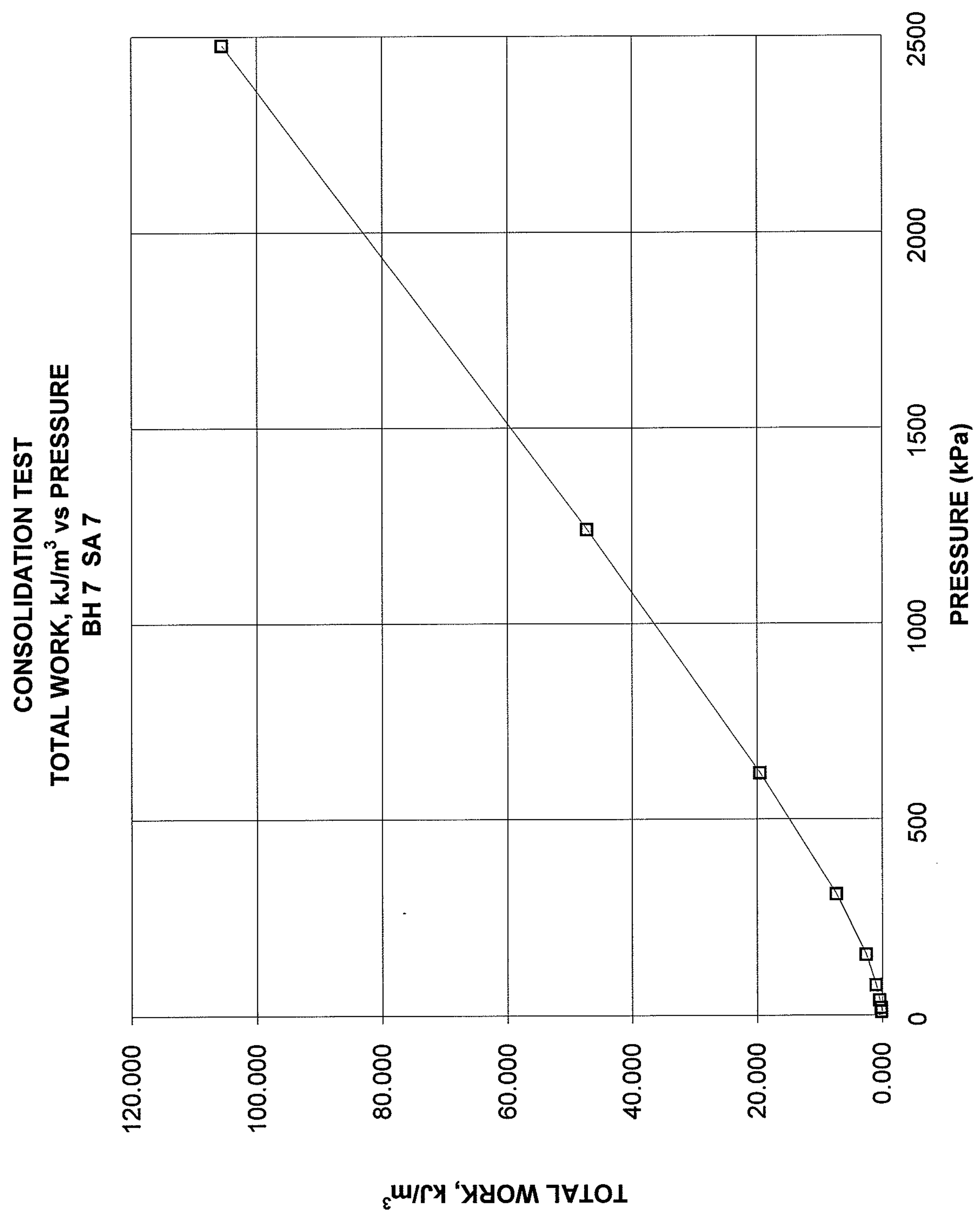


Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM



Project No. 04-1111-060

Golder Associates

Prepared By: LFG

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	11
Borehole Number	7	Sample Depth, m	14.6-15.0

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	11/13/2006		
Date Completed	11/24/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	20.68
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	16.99
Area, cm^2	31.65	Specific Gravity, measured	2.76
Volume, cm^3	60.13	Solids Height, cm	1.193
Water Content, %	21.71	Volume of Solids, cm ^{3}	37.75
Wet Mass, g	126.80	Volume of Voids, cm ^{3}	22.39
Dry Mass, g	104.18	Degree of Saturation, %	101.0

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.900	0.593	1.900				
4.83	1.897	0.591	1.899	8	9.55E-02	3.27E-04	3.06E-06
9.55	1.890	0.585	1.894	13	5.85E-02	7.81E-04	4.47E-06
19.51	1.876	0.573	1.883	23	3.27E-02	7.40E-04	2.37E-06
38.91	1.855	0.555	1.866	23	3.21E-02	5.70E-04	1.79E-06
77.57	1.826	0.531	1.841	28	2.56E-02	3.95E-04	9.92E-07
154.67	1.791	0.502	1.809	124	5.59E-03	2.39E-04	1.31E-07
309.92	1.732	0.452	1.762	271	2.43E-03	2.00E-04	4.76E-08
619.27	1.670	0.400	1.701	40	1.53E-02	1.05E-04	1.59E-07
1237.90	1.593	0.336	1.632	15	3.76E-02	6.55E-05	2.42E-07
2475.99	1.508	0.264	1.551	34	1.50E-02	3.61E-05	5.31E-08
1237.90	1.522	0.276	1.515				
309.92	1.549	0.299	1.536				
77.57	1.588	0.331	1.569				
19.51	1.634	0.370	1.611				
4.83	1.663	0.394	1.649				

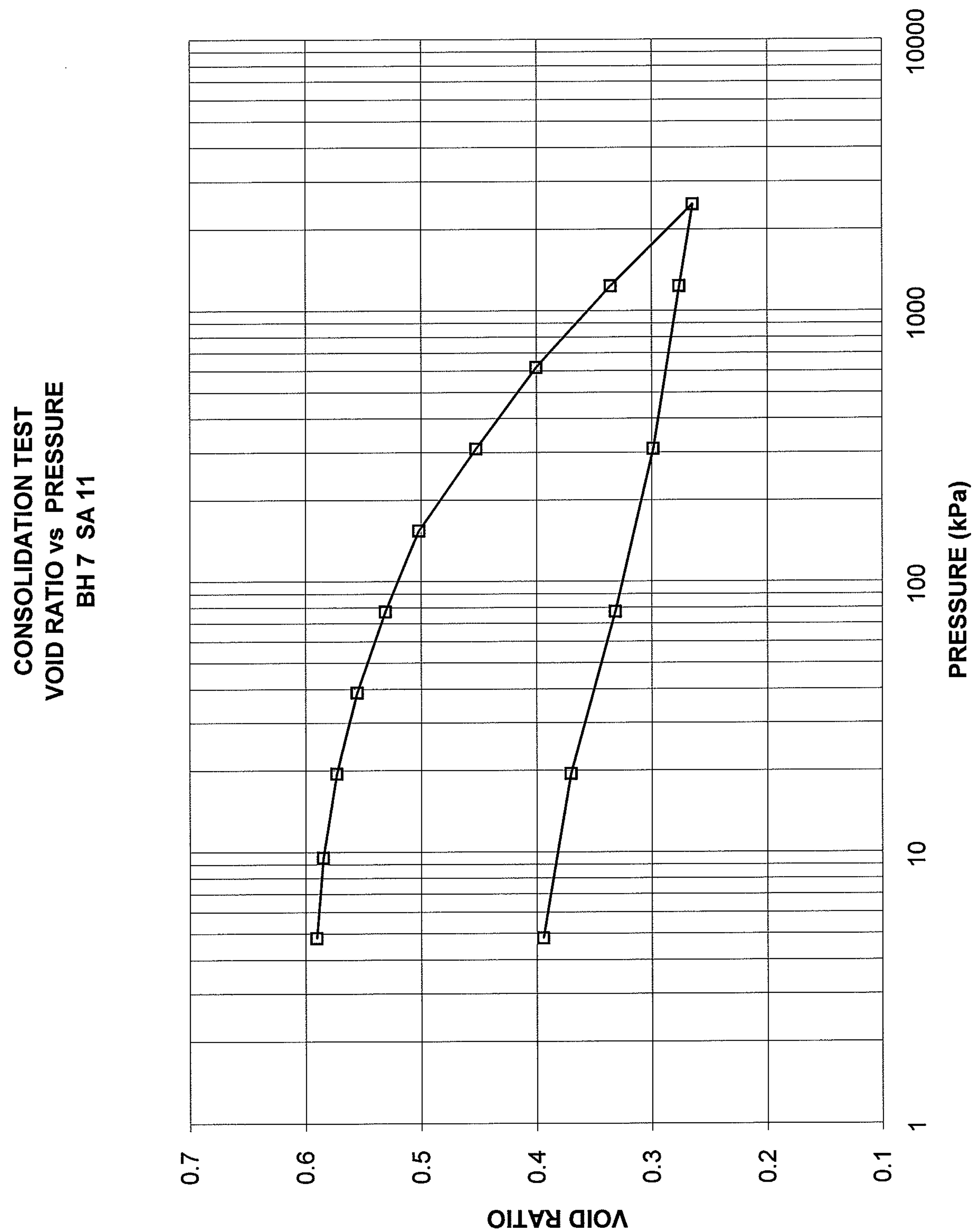
Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.66	Unit Weight, kN/m^3	22.90
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	19.41
Area, cm^2	31.65	Specific Gravity, measured	2.76
Volume, cm^3	52.63	Solids Height, cm	1.193
Water Content, %	18.00	Volume of Solids, cm ^{3}	37.75
Wet Mass, g	122.93	Volume of Voids, cm ^{3}	14.89
Dry Mass, g	104.18		

Prepared By: LFG **Golder Associates** Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

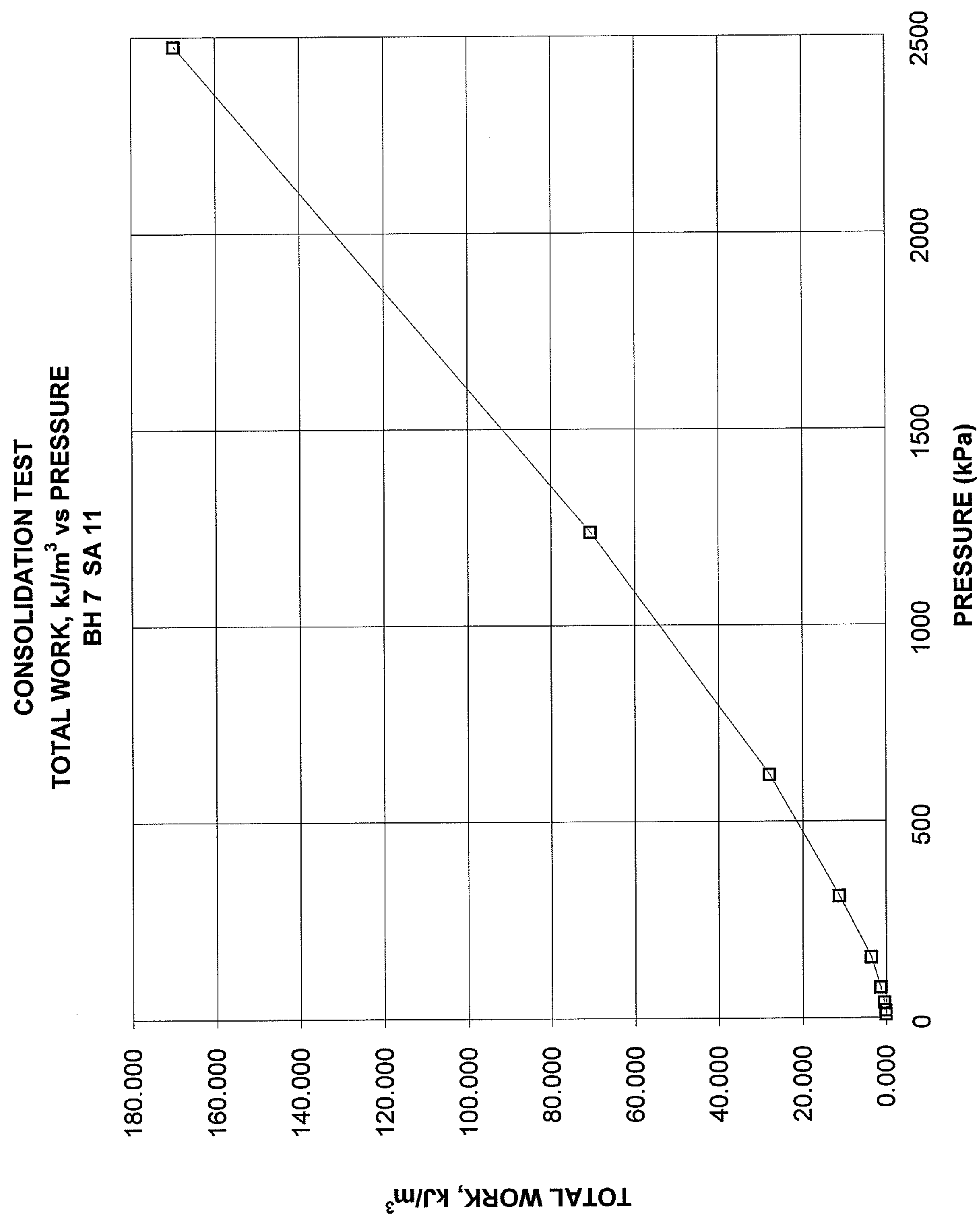


Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM



Project No. 04-1111-060

Golder Associates

Prepared By: LFG

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	20
Borehole Number	7	Sample Depth, m	25.9-26.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	11/14/2006		
Date Completed	11/29/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m^3	20.98
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	17.54
Area, cm^2	31.67	Specific Gravity, measured	2.74
Volume, cm^3	60.65	Solids Height, cm	1.250
Water Content, %	19.57	Volume of Solids, cm^3	39.60
Wet Mass, g	129.73	Volume of Voids, cm^3	21.05
Dry Mass, g	108.50	Degree of Saturation, %	100.9

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.915	0.532	1.915				
4.85	1.912	0.529	1.914	7	1.11E-01	3.23E-04	3.51E-06
9.50	1.907	0.525	1.910	19	4.07E-02	5.61E-04	2.24E-06
19.40	1.894	0.515	1.901	15	5.10E-02	6.86E-04	3.43E-06
38.64	1.876	0.500	1.885	20	3.77E-02	4.89E-04	1.80E-06
77.43	1.849	0.479	1.863	11	6.69E-02	3.63E-04	2.38E-06
154.57	1.801	0.440	1.825	12	5.88E-02	3.25E-04	1.87E-06
309.12	1.746	0.396	1.774	17	3.92E-02	1.86E-04	7.14E-07
618.28	1.678	0.342	1.712	68	9.14E-03	1.15E-04	1.03E-07
1236.63	1.609	0.287	1.644	158	3.62E-03	5.83E-05	2.07E-08
2474.00	1.533	0.226	1.571	84	6.23E-03	3.21E-05	1.96E-08
1236.63	1.543	0.234	1.538				
309.12	1.570	0.256	1.557				
77.43	1.611	0.288	1.591				
19.40	1.656	0.324	1.634				
4.85	1.692	0.353	1.674				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

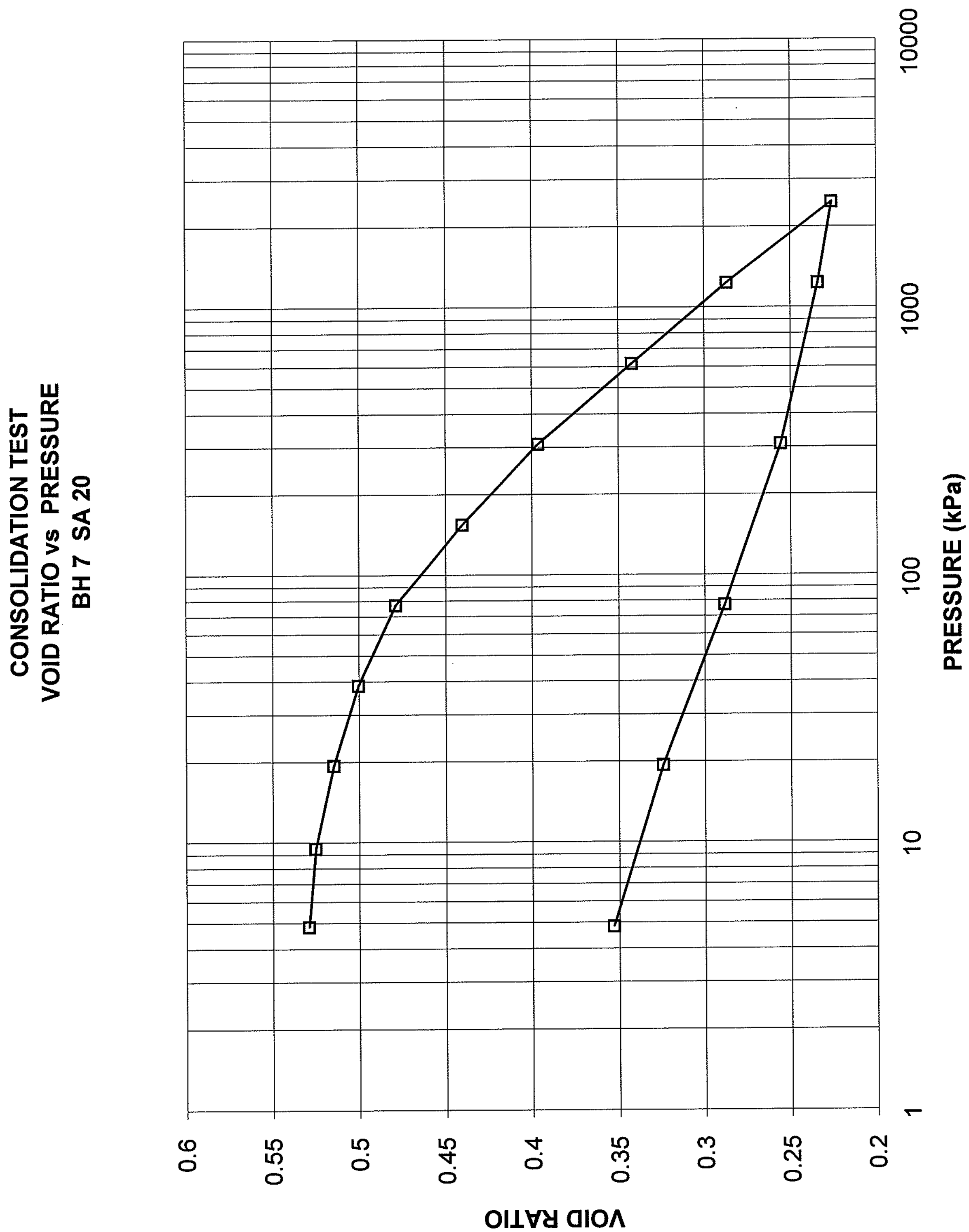
Sample Height, cm	1.69	Unit Weight, kN/m^3	23.20
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	19.86
Area, cm^2	31.67	Specific Gravity, measured	2.74
Volume, cm^3	53.58	Solids Height, cm	1.250
Water Content, %	16.81	Volume of Solids, cm^3	39.60
Wet Mass, g	126.74	Volume of Voids, cm^3	13.99
Dry Mass, g	108.5		

Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

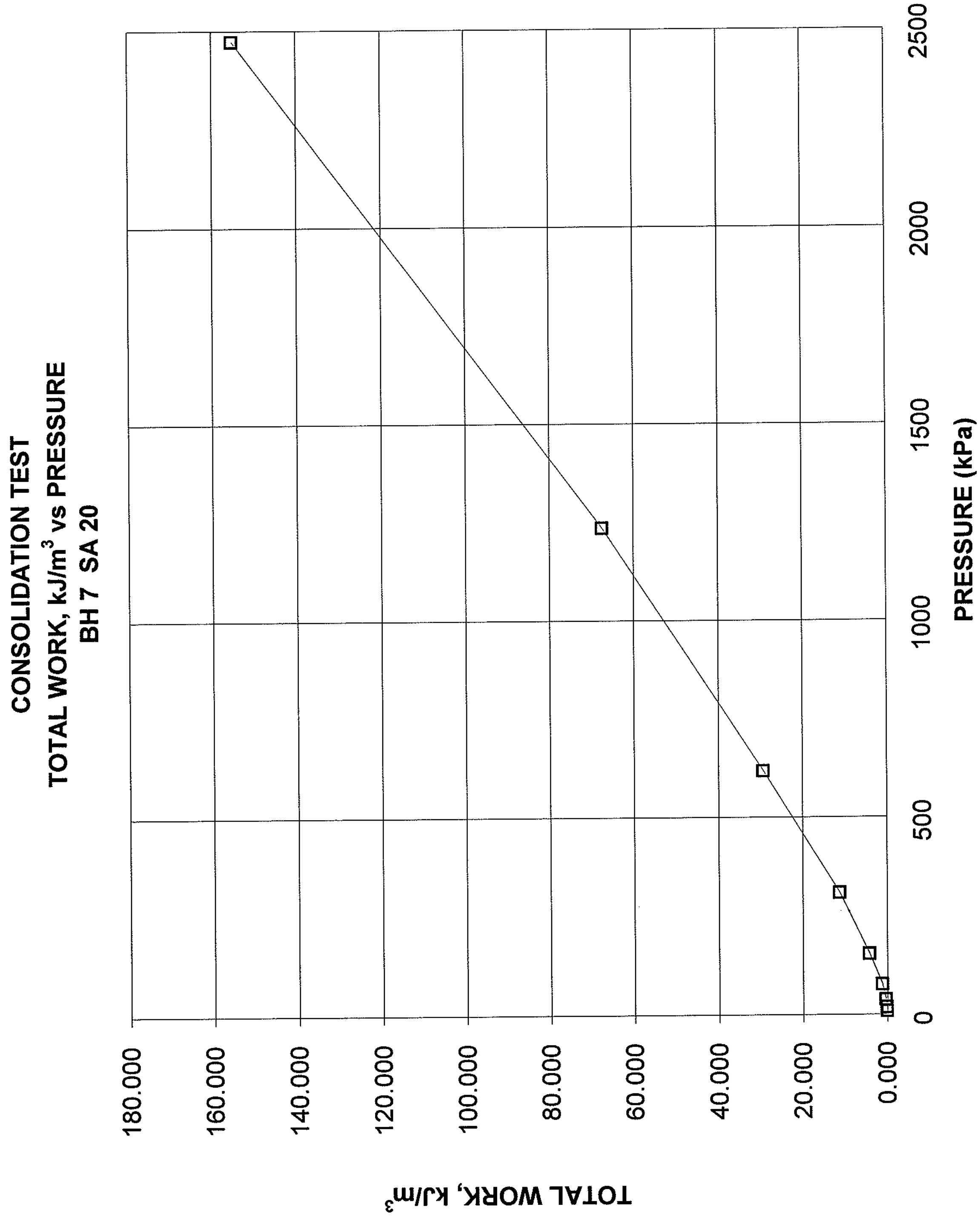


Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM



Project No. 04-1111-060

Golder Associates

Prepared By: LFG

Checked By: MM

Appendix D Analytical Laboratory Test Results

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

Date Received: 25-JUL-11
Report Date: 29-JUL-11 20:52 (MT)
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1035570

Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 112831

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1035570-1				
	Description	SOIL				
	Sampled Date	22-JUL-11				
	Sampled Time					
	Client ID	T8-1,SS6@12.5', SILTY CLAY,GREY				
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	12.5				
	pH (pH units)	7.84				
	Redox Potential (mV)	100				
	Resistivity (ohm cm)	4670				
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20				
Anions and Nutrients	Sulphate (mg/kg)	112				

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:

112831

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: L1035570   Report Date: 29-JUL-11

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

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential							
	1	22-JUL-11	27-JUL-11 14:12	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 L1035570 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.

112831

C of C # 00000

60 NORTHLAND ROAD, UNIT 1
WATERLOO, ON N2V 2B8
Phone: (519) 886-6910
Fax: (519) 886-9047
Toll Free: 1-800-668-9878

ALS Environmental

CHAIN OF CUSTODY / ANALYTICAL SERVICES REQUEST FORM

Page ___ of ___

Note: all TAT Quoted material is in business days which exclude statutory holidays and weekends. TAT samples received past 3:00 pm or Saturday/Sunday begin the next day.

Specify date required	Service requested		
	5 day (regular)	✓	*2 day TAT (50%)*
			Next day TAT (100%)
	3-4 day (25%)		*Same day TAT (200%)*

COMPANY NAME: Amec E+I
OFFICE: Windsor
PROJECT MANAGER: Shane Macleod
PROJECT #: SWE601.1004.101
PHONE: 519-735-2499
FAX: 519-735-9609 (6990)
ACCOUNT #:
QUOTATION #: Q26643
PO #:

CRITERIA — Criteria on report YES___ NO___
Reg 153/04 ☐ Reg 511/09 ☐
Table 1 2 3 4 5 6 7 8 9
TCLP _____ MISA _____ PWQO _____
ODWS _____ OTHER _____

REPORT FORMAT/DISTRIBUTION
EMAIL ✓ FAX _____ BOTH _____
SELECT: PDF _____ DIGITAL _____ BOTH _____
EMAIL 1 Shane.Macleod@Amec.com
EMAIL 2

ANALYSIS REQUEST: Corrosion Package

PLEASE INDICATE FILTERED, PRESERVED OR BOTH <---- (F, P, F/P)
SUBMISSION #: L1035570
ENTERED BY: P Stastny
DATE/TIME ENTERED: 25-July-11
BIN #:

SAMPLING INFORMATION

Date (dd-mm-yy)	Time (24hr) (hh:mm)	COMP	GRAB	WATER	SOIL	OTHER	SAMPLE DESCRIPTION TO APPEAR ON REPORT	NUMBER OF CONTAINERS	Corrosion Package	COMMENTS	LAB ID
22/07/11					X		TB-1, SS6 @ 12.5', silty clay, grey	1	X		-1

SPECIAL INSTRUCTIONS/COMMENTS

THE QUESTIONS BELOW MUST BE ANSWERED FOR WATER SAMPLES (CHECK Yes OR No)
Are any samples taken from a regulated DW System? Yes ☐ No ☐
If yes, an authorized drinking water COC MUST be used for this submission.
Is the water sampled intended to be potable for human consumption? Yes ☐ No ☐

SAMPLE CONDITION
FROZEN ☐
COLD ☐
COOLING INITIATED ☐
AMBIENT ☐
MEAN TEMP: 22.4°C
OBSERVATIONS: Yes ☐ No ☐
If yes add SIF
INIT: NG

	DATE & TIME	RECEIVED BY:	DATE & TIME
SAMPLED BY:			
RELINQUISHED BY:		RECEIVED AT LAB BY: [signature]	25-July-11 10:00

Notes
1. Quote number must be provided to ensure proper pricing
2. TAT may vary dependent on complexity of analysis and lab workload at time of submission. Please contact the lab to confirm TATs.
3. Any known or suspected hazards relating to a sample must be noted on the chain of custody in comments section.

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

Date Received: 13-JUL-11
Report Date: 19-JUL-11 13:51 (MT)
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #:	**L1030731**
Project P.O. #:	NOT SUBMITTED
Job Reference:	SW8801.1004.101
Legal Site Desc:	
C of C Numbers:	092959

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1030731-1				
	Description	SOIL				
	Sampled Date	09-JUL-11				
	Sampled Time					
	Client ID	TB6-1 SA#10				
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	14.5				
	pH (pH units)	7.86				
	Redox Potential (mV)	125				
	Resistivity (ohm cm)	3700				
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20				
Anions and Nutrients	Sulphate (mg/kg)	100				

Reference Information

Test Method References:

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

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

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

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

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

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

Chain of Custody Numbers:

092959

GLOSSARY OF REPORT TERMS
Surrogate - A compound that is similar in behaviour to target analyte(s), but that does not occur naturally in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery.
mg/kg - milligrams per kilogram based on dry weight of sample.
mg/kg wwt - milligrams per kilogram based on wet weight of sample.
mg/kg lwt - milligrams per kilogram based on lipid-adjusted weight of sample.
mg/L - milligrams per litre.
< - Less than.
D.L. - The reported Detection Limit, also known as the Limit of Reporting (LOR).
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Quality Control Report

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

Contact: SHANE MACLEOD

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT	**Soil**							
Batch R2218341								
WG1311854-3 DUP		**L1030731-1**						
% Moisture		14.5	14.4		%	0.49	30	13-JUL-11
WG1311854-2 LCS								
% Moisture			93		%		70-130	13-JUL-11
WG1311854-1 MB								
% Moisture			<0.10		%		0.1	13-JUL-11
PH-WT	**Soil**							
Batch R2220797								
WG1315023-1 CVS								
pH			99		%		80-120	19-JUL-11
RESISTIVITY-WT	**Soil**							
Batch R2220855								
WG1315028-1 CVS								
Resistivity			99		%		70-130	19-JUL-11
SO4-WT	**Soil**							
Batch R2219765								
WG1312668-3 LCS								
Sulphate			103		%		60-140	15-JUL-11
WG1312668-1 MB								
Sulphate			<20		mg/kg		20	15-JUL-11
SULPHIDE-WT	**Soil**							
Batch R2218729								
WG1312664-1 CVS								
Sulphide			106		%		50-120	14-JUL-11
WG1312662-1 MB								
Sulphide			<0.20		mg/kg		0.2	14-JUL-11

Quality Control Report

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential							
	1	09-JUL-11	19-JUL-11 14:12	24	242	hours	EHTR
Resistivity							
	1	09-JUL-11	19-JUL-11 14:32	7	10	days	EHT

Legend & Qualifier Definitions:

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

Notes*:
Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1030731 were received on 13-JUL-11 10:30.

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

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

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

60 NORTHLAND ROAD, UNIT 1
WATERLOO, ON N2V 2B8
Phone: (519) 886-6910
Fax: (519) 886-9047
CANADA TOLL FREE: 1-800-668-9878

ALS Environmental

CHAIN OF CUSTODY / ANALYTICAL SERVICES REQUEST FORM

C of C # 092959
PAGE ___ OF ___

Note: all TAT Quoted material is in business days which exclude statutory holidays and weekends. TAT samples received past 3:00 pm or Saturday/Sunday begin the next day.

Specify date required	Service requested		
	5 day (Regular) ✓	2 day TAT (50%)	
	3-4 day TAT (25%)	Next day TAT (100%)	
		Same day TAT (200%)	

COMPANY NAME: Amec E+I
OFFICE: Windsor
PROJECT MANAGER: Shane Macleod
PROJECT #: SW8801.1004.101
PHONE: 519 735-2499 FAX: 519 735-9669
ACCOUNT #:
QUOTATION #: Q28643 PO#:

CRITERIA — Criteria on report Yes ☐ No ☐
Reg 153/04
Table 1 2 3
TCLP ___ MISA ___ PWQO ___
ODWS ___ OTHER ___

REPORT FORMAT / DISTRIBUTION
EMAIL ✓ FAX ___ BOTH ___
SELECT: PDF ___ DIGITAL ___ BOTH ___
EMAIL1 Shane.Macleod@Amec.com
EMAIL2

ANALYSIS REQUEST: Corrosion Package

PLEASE INDICATE FILTERED, PRESERVED OR BOTH (F, P, F/P)
SUBMISSION #: L1030731
ENTERED BY: KB
DATE/TIME ENTERED: 13 July 11
BIN #

Date (dd-mm-yy)	Time (24 hr) (hh:mm)	COMP	GRAB	WATER	SOIL	OTHER	SAMPLE DESCRIPTION TO APPEAR ON REPORT	NUMBER OF CONTAINERS	Corrosion Package	COMMENTS	LAB ID
July 8, 11					X		TB6-1 SA#10		✓		-1

SPECIAL INSTRUCTIONS/COMMENTS

THE QUESTIONS BELOW MUST BE ANSWERED FOR WATER SAMPLES (CHECK Yes OR No)
Are any samples taken from a regulated DW System? Yes ☐ No ☒
If yes, an authorized drinking water COC MUST be used for this submission.
Is the water sampled intended to be potable for human consumption? Yes ☐ No ☒

SAMPLE CONDITION
FROZEN ☐
COLD ☐
COOLING INITIATED ☐
AMBIENT ☐
MEAN TEMP: 18.3°C
OBSERVATIONS Yes ☐ No ☐ If yes add SIF
INIT

SAMPLED BY: DATE & TIME:
RELINQUISHED BY: DATE & TIME:
RECEIVED BY: DATE & TIME:
RECEIVED AT LAB BY: Ben Paul DATE & TIME: 13-Jul-11 10:30

NOTES AND CONDITIONS:
1. Quote number must be provided to ensure proper pricing.
2. TAT may vary dependent on complexity of analysis and lab workload at time of submission. Please contact the lab to confirm TATs.
3. Any known or suspected hazards relating to a sample must be noted on the chain of custody in comments section.

White - Report copy YELLOW - File copy PINK - Customer copy

Rev COC Rev#4.09

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

Date Received: 16-SEP-11
Report Date: 23-SEP-11 06:20 (MT)
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1059696

Project P.O. #:	NOT SUBMITTED
Job Reference:	SW8801.1004.101
C of C Numbers:	112774
Legal Site Desc:	

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1059696-1				
	Description	SOIL				
	Sampled Date	25-AUG-11				
	Sampled Time					
	Client ID	PS5-1,SS23@90',GREY SILTY CLAY				
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	16.6				
	pH (pH units)	7.90				
	Redox Potential (mV)	230				
	Resistivity (ohm cm)	2580				
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20				
Anions and Nutrients	Sulphate (mg/kg)	486				

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:

112774

GLOSSARY OF REPORT TERMS

Surrogate - A compound that is similar in behaviour to target analyte(s), but that does not occur naturally in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery.
mg/kg - milligrams per kilogram based on dry weight of sample.
mg/kg wwt - milligrams per kilogram based on wet weight of sample.
mg/kg lwt - milligrams per kilogram based on lipid-adjusted weight of sample.
mg/L - milligrams per litre.
< - Less than.
D.L. - The reported Detection Limit, also known as the Limit of Reporting (LOR).
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Quality Control Report

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

Contact: SHANE MACLEOD

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT	**Soil**							
Batch R2254382								
WG1351428-2 LCS								
% Moisture			94		%		70-130	19-SEP-11
WG1351428-1 MB								
% Moisture			<0.10		%		0.1	19-SEP-11
PH-WT	**Soil**							
Batch R2254003								
WG1351581-1 CVS								
pH			101		%		80-120	19-SEP-11
RESISTIVITY-WT	**Soil**							
Batch R2255410								
WG1353108-1 CVS								
Resistivity			102		%		70-130	21-SEP-11
SO4-WT	**Soil**							
Batch R2255430								
WG1352527-3 LCS								
Sulphate			101		%		60-140	20-SEP-11
WG1352527-1 MB								
Sulphate			<20		mg/kg		20	20-SEP-11
SULPHIDE-WT	**Soil**							
Batch R2254650								
WG1352442-1 CVS								
Sulphide			107		%		50-120	20-SEP-11
WG1352431-1 MB								
Sulphide			<0.20		mg/kg		0.2	20-SEP-11

Quality Control Report

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
% Moisture							
	1	25-AUG-11	19-SEP-11 10:40	14	25	days	EHTR
Redox Potential							
	1	25-AUG-11	21-SEP-11	24	651	hours	EHTR
Resistivity							
	1	25-AUG-11	21-SEP-11	7	27	days	EHTR
Leachable Anions & Nutrients							
Sulphide							
	1	25-AUG-11	20-SEP-11 13:10	7	26	days	EHTR

Legend & Qualifier Definitions:

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

Notes*:
Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1059696 were received on 16-SEP-11 09:00.

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.

112774

C of C # 00000

60 NORTHLAND ROAD, UNIT 1
WATERLOO, ON N2V 2B8
Phone: (519) 886-6910
Fax: (519) 886-9047
Toll Free: 1-800-668-9878

ALS ENVIRONMENTAL

CHAIN OF CUSTODY / ANALYTICAL SERVICES REQUEST FORM

Page 1 of 1

Note: all TAT Quoted material is in business days which exclude statutory holidays and weekends. TAT samples received past 3:00 pm or Saturday/Sunday begin the next day.

Specify date required	Service requested			
	5 day (regular)	✓	2 day TAT (50%)	
	3-4 day (25%)		Next day TAT (100%)	
			Same day TAT (200%)	

COMPANY NAME: AMEC E & I
OFFICE: Windsor
PROJECT MANAGER: Shane Macleod
PROJECT #: SW880L 1004.101
PHONE: (519) 735-2499
FAX: (519) 735-9669
ACCOUNT #:
QUOTATION #: Q28643
PO #:

CRITERIA — Criteria on report YES___ NO___
Reg 153/04 ☐ Reg 511/09 ☐
Table 1 2 3 4 5 6 7 8 9
TCLP ______ MISA ______ PWQO ______
ODWS ______ OTHER ______

REPORT FORMAT/DISTRIBUTION
EMAIL ✓ FAX ______ BOTH ______
SELECT: PDF ______ DIGITAL ______ BOTH ______
EMAIL 1: shane.macleod@amec.com
EMAIL 2 ______

ANALYSIS REQUEST: Corrosion Package

PLEASE INDICATE FILTERED, PRESERVED OR BOTH <---- (F, P, F/P)
SUBMISSION #: L1059696
ENTERED BY: [illegible]
DATE/TIME ENTERED: 16-Sept.
BIN #: 159

SAMPLING INFORMATION

Date (dd-mm-yy)	Time (24hr) (hh:mm)	COMP	GRAB	WATER	SOIL	OTHER	SAMPLE DESCRIPTION TO APPEAR ON REPORT	NUMBER OF CONTAINERS	Corrosion Package	COMMENTS	LAB ID
Aug 25/11					✓		PS5-1, SS23@ 90', gr. silty clay		✓		

SPECIAL INSTRUCTIONS/COMMENTS

THE QUESTIONS BELOW MUST BE ANSWERED FOR WATER SAMPLES (CHECK Yes OR No)
Are any samples taken from a regulated DW System? Yes ☐ No ☐
If yes, an authorized drinking water COC MUST be used for this submission.
Is the water sampled intended to be potable for human consumption? Yes ☐ No ☐

SAMPLE CONDITION
FROZEN ☐
COLD ☐
COOLING INITIATED ☐
AMBIENT ☐
MEAN TEMP: 9.2
OBSERVATIONS Yes ☐ No ☐
If yes add SIF

SAMPLED BY:	DATE & TIME	RECEIVED BY:	DATE & TIME
RELINQUISHED BY:	DATE & TIME	RECEIVED AT LAB BY: [signature]	DATE & TIME: 9/16/11 900

Notes
1. Quote number must be provided to ensure proper pricing
2. TAT may vary dependent on complexity of analysis and lab workload at time of submission. Please contact the lab to confirm TATs.
3. Any known or suspected hazards relating to a sample must be noted on the chain of custody in comments section.

Appendix E Core Photographs

Photograph E-1: Borehole T8-1 – Rock Core Elevation 150.2 to 148.1 m

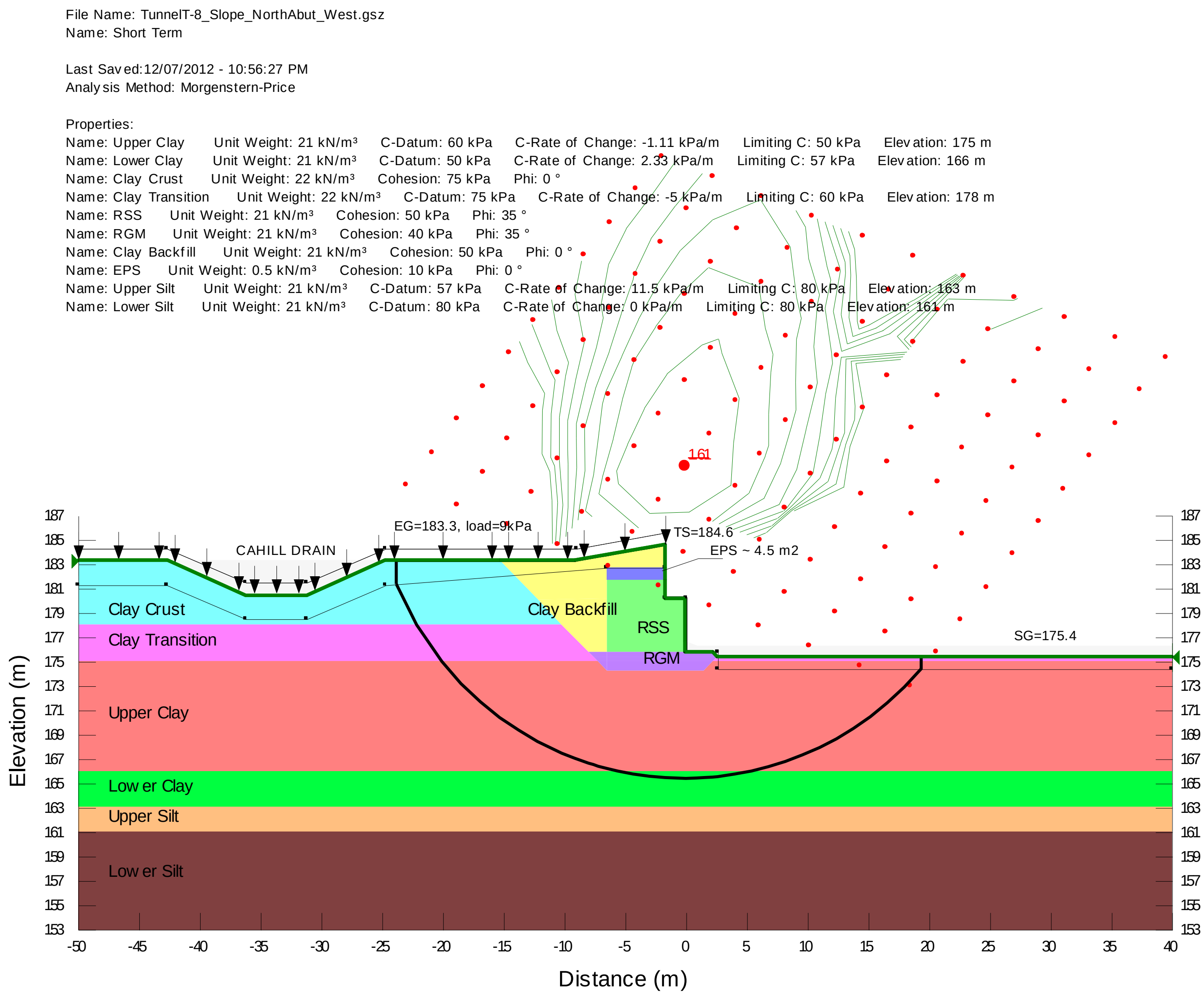
Photograph E-2: Borehole PS5-1 – Rock Core Elevation 150.1 to 149.3 m

Appendix F Slope Stability Analyses Results

Parkway Infrastructure Engineers — amec — Hatch Mott MacDonald

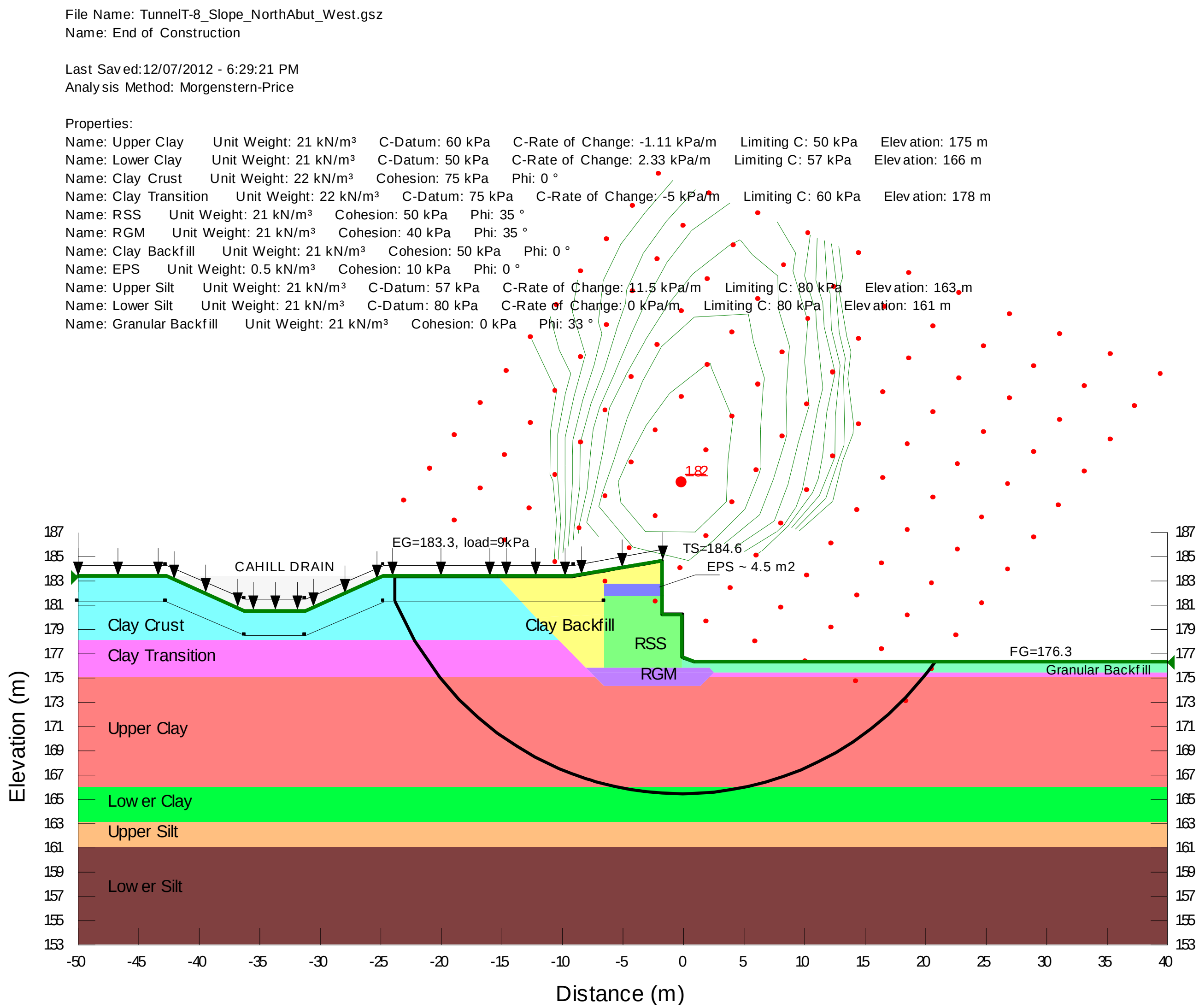
PARKWAY INFRASTRUCTURE CONSTRUCTORS — acciona Infraestructuras — DRAGADOS — FLUOR

Figure F-1: Global Stability Result – North Abutment (West Segment)– Short Term (Undrained) Loading



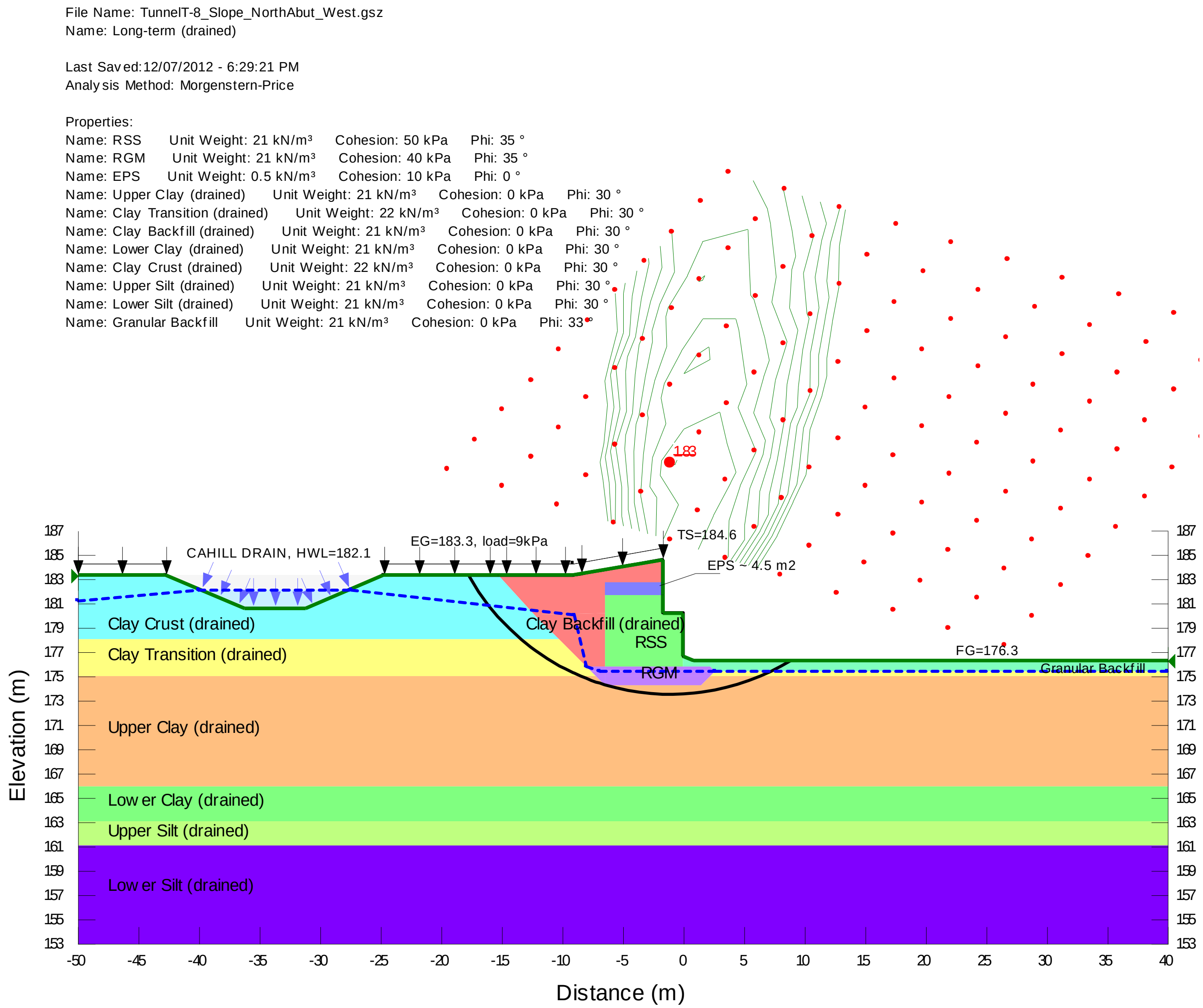
Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 1 of 27

Figure F-2: Global Stability Result – North Abutment (West Segment)– End of Construction (Undrained) Loading



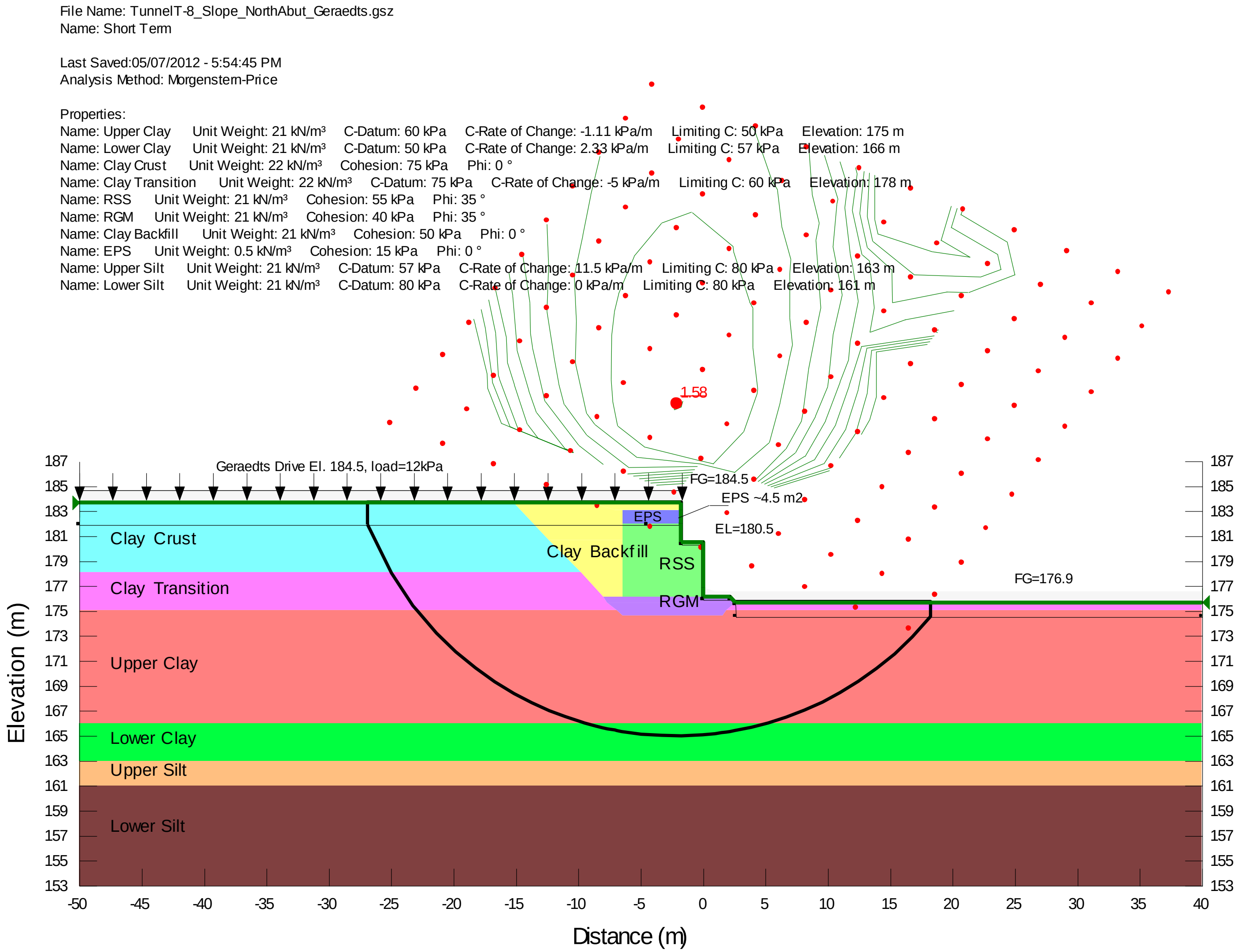
Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 2 of 27

Figure F-3: Global Stability Result – North Abutment (West Segment)– Long-term (Drained) Loading



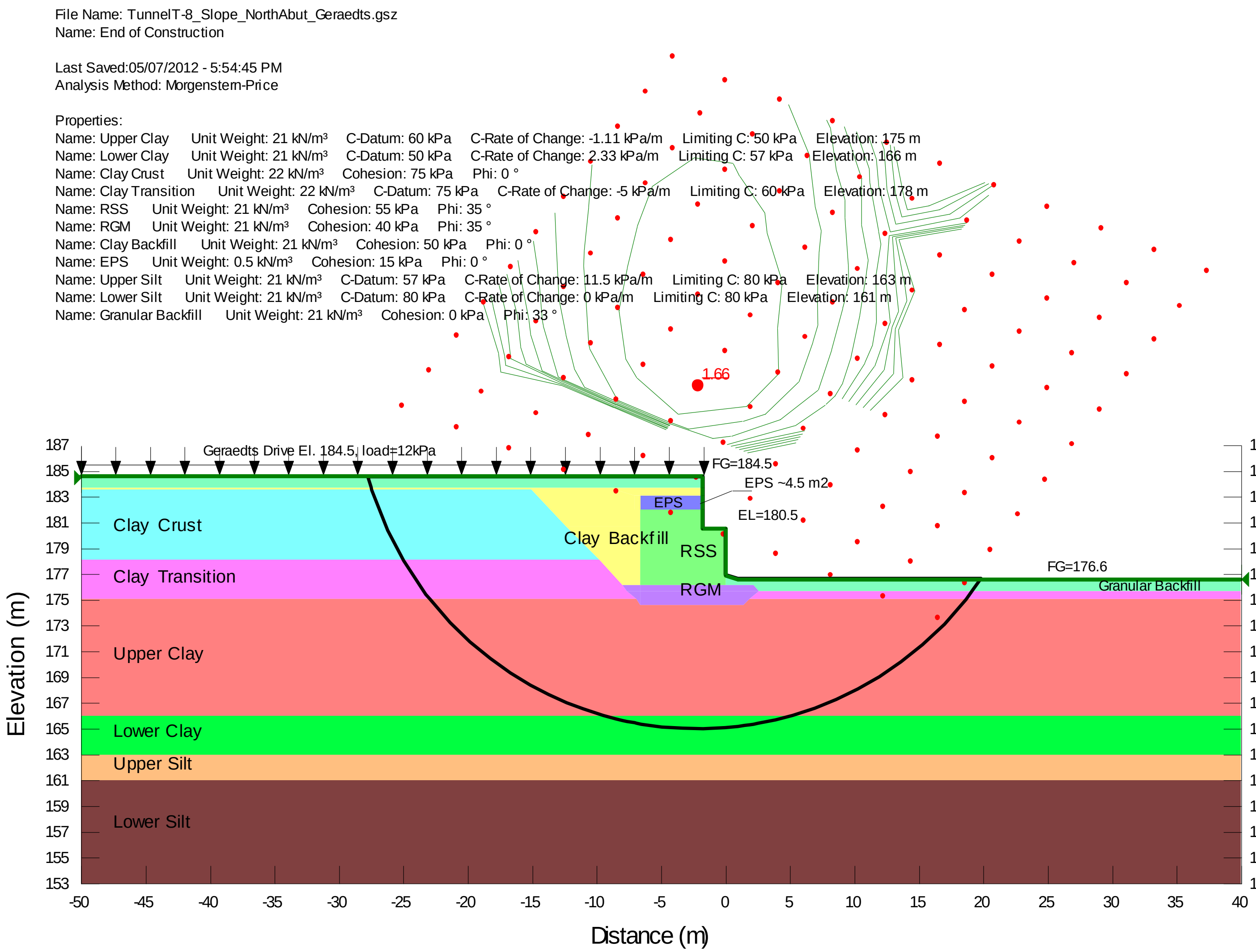
Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 3 of 27

Figure F-4: Global Stability Result – North Abutment (Geraedts Drive) – Short Term (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 4 of 27

Figure F-5: Global Stability Result – North Abutment (Geraedts Drive) – End of Construction (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 5 of 27

Figure F-6: Global Stability Result – North Abutment (Geraedts Drive) – Long Term (Drained) Loading

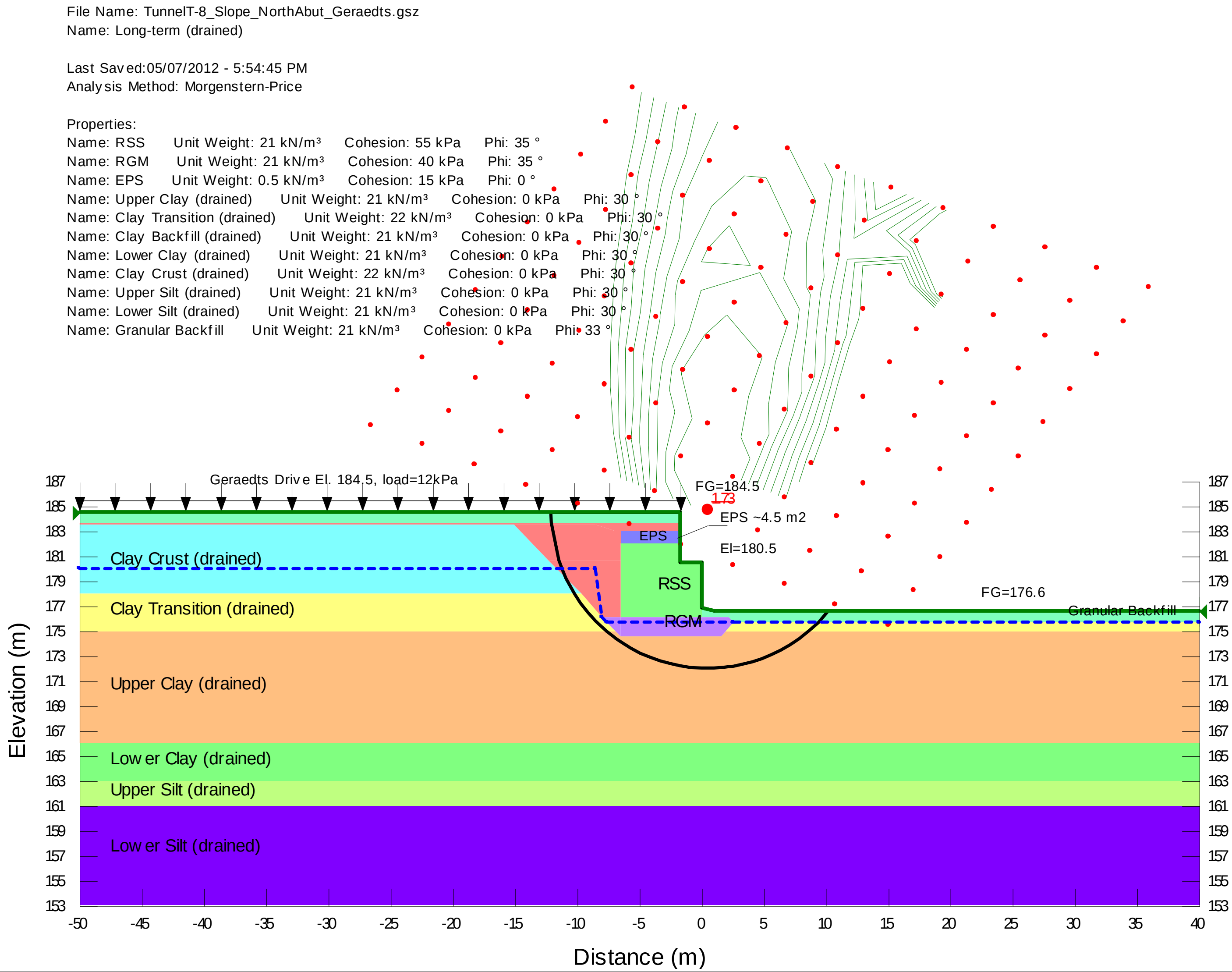
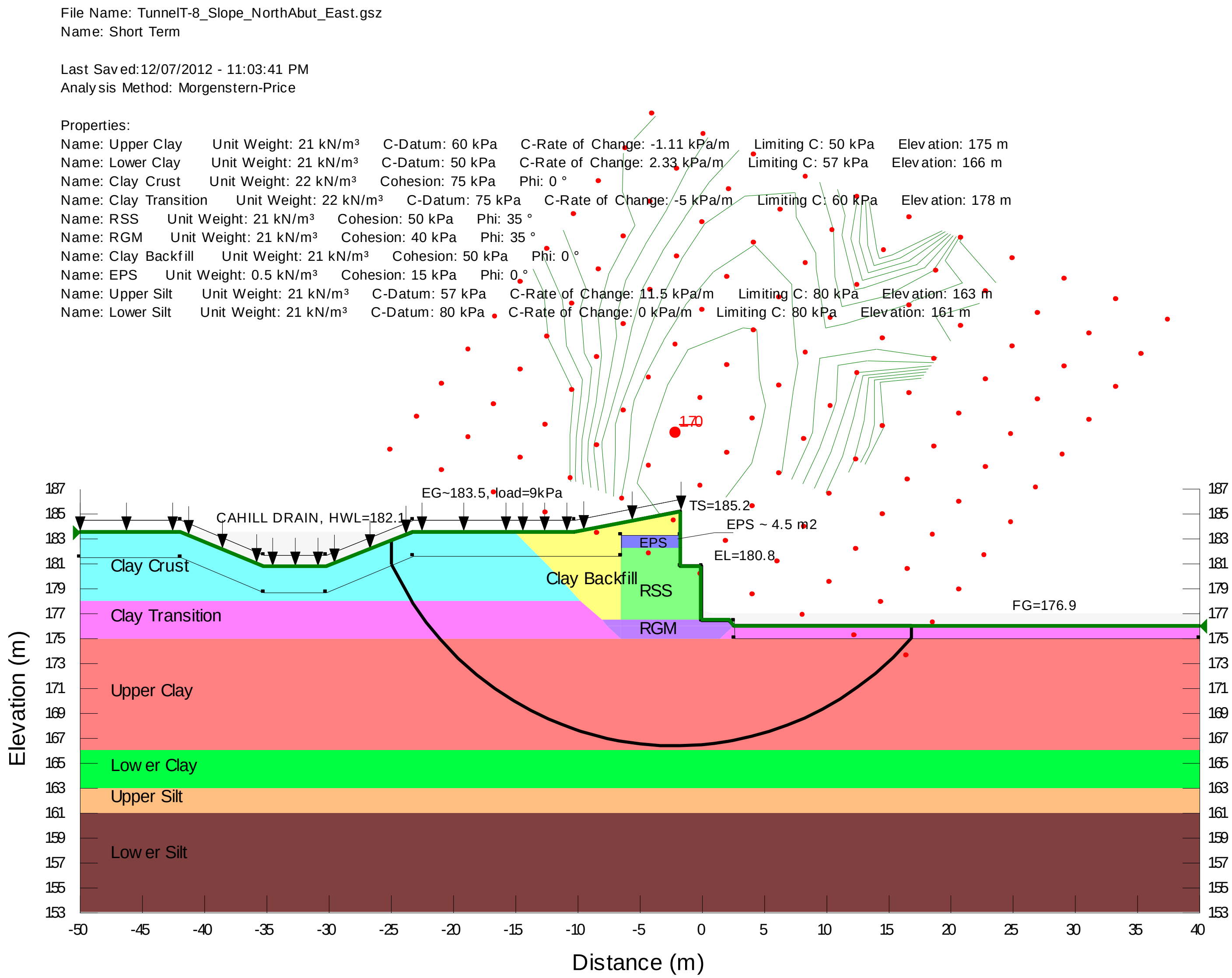
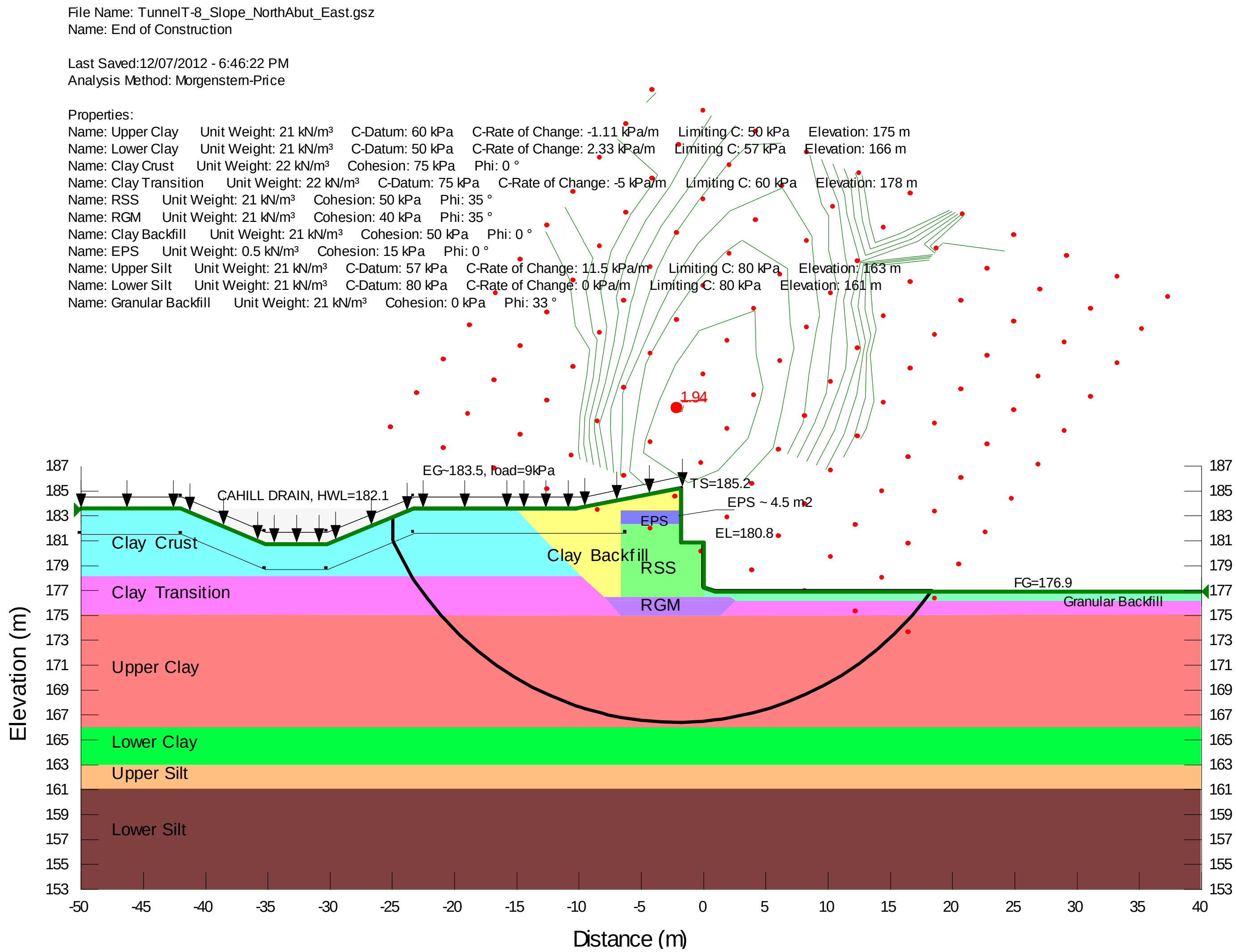


Figure F-7: Global Stability Result – North Abutment (East Segment) – Short Term (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 7 of 27

Figure F-8: Global Stability Result – North Abutment (East Segment) – End of Construction (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 8 of 27

Figure F-9: Global Stability Result – North Abutment (East Segment) – Long Term (Drained) Loading

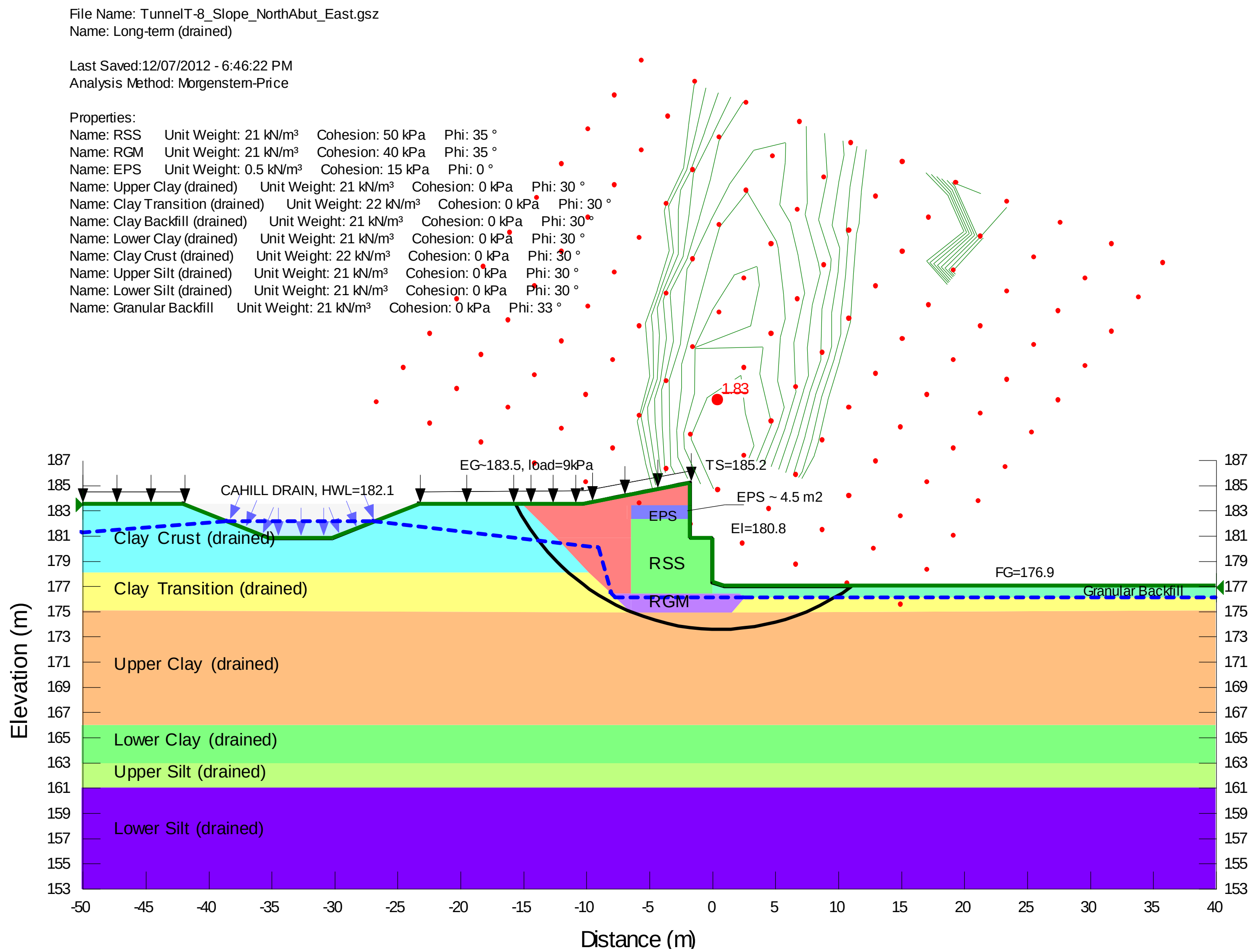
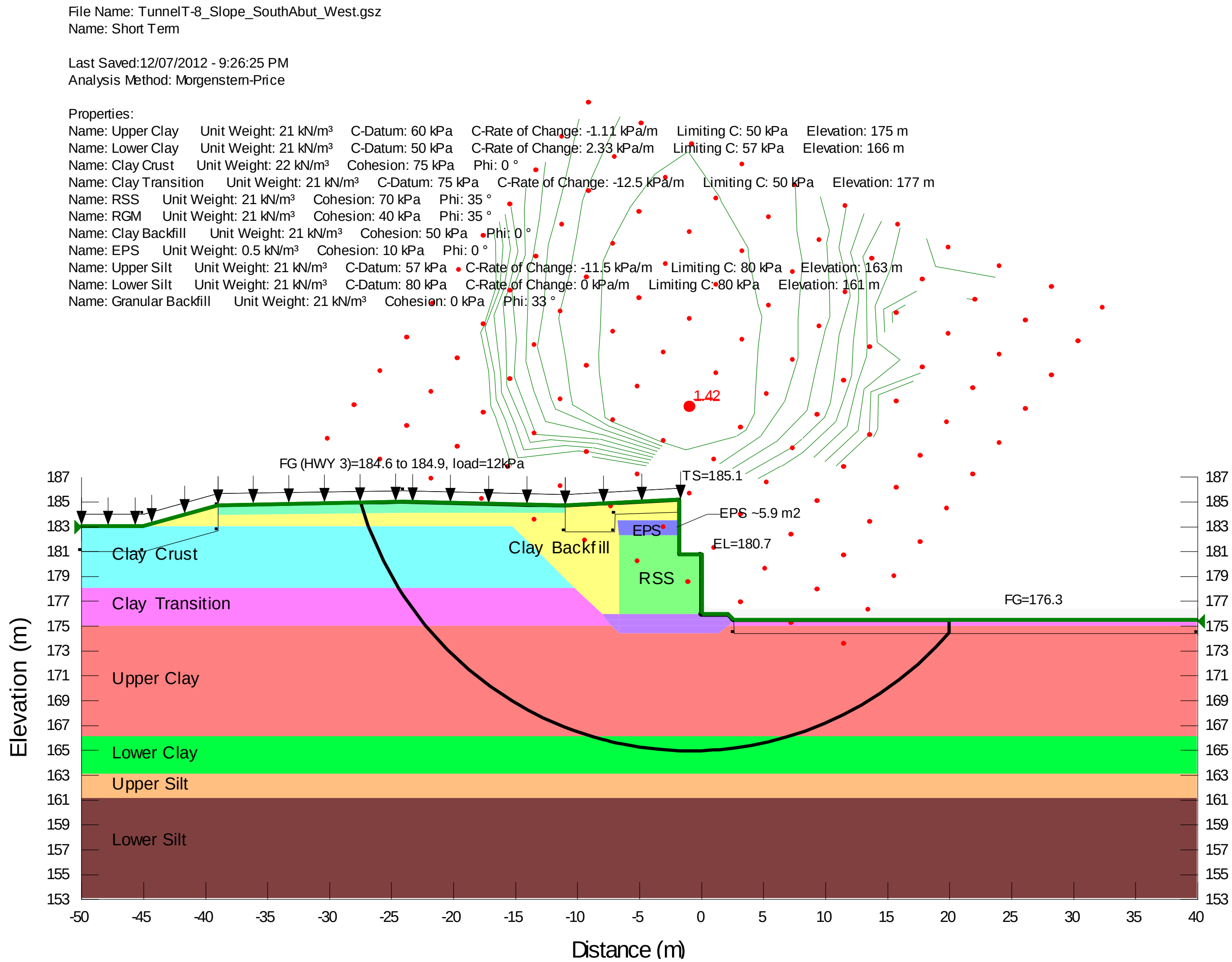
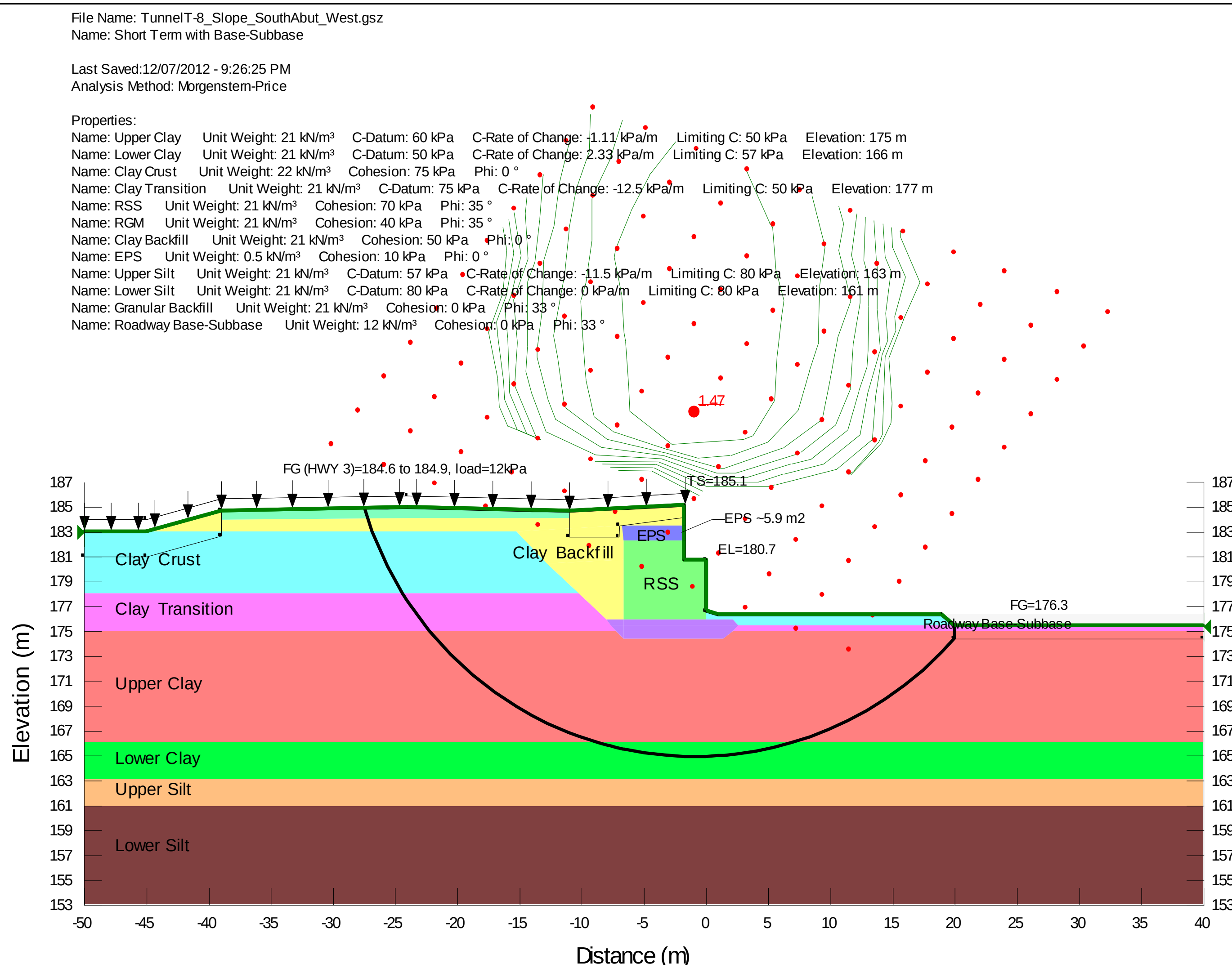


Figure F-10: Global Stability Result – South Abutment (West Segment) – Short Term (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 10 of 27

Figure F-11: Global Stability Result - South Abutment (West Segment) - Short Term (Undrained) Loading with Sub-base



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 11 of 27

Figure F-12: Global Stability Result – South Abutment (West Segment) – End of Construction (Undrained) Loading

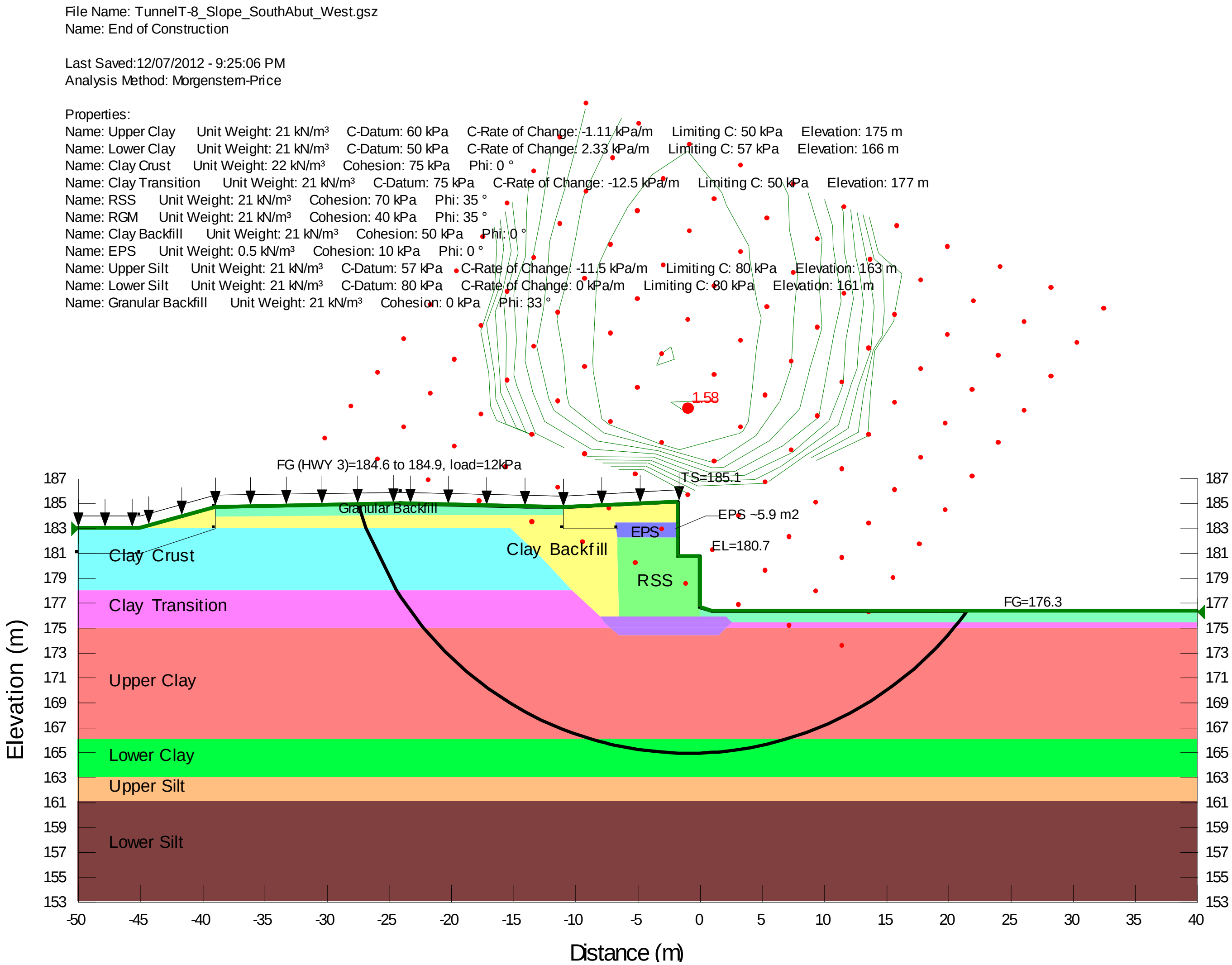


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Figure F-13: Global Stability Result – South Abutment (East Segment) – Long Term (Drained) Loading

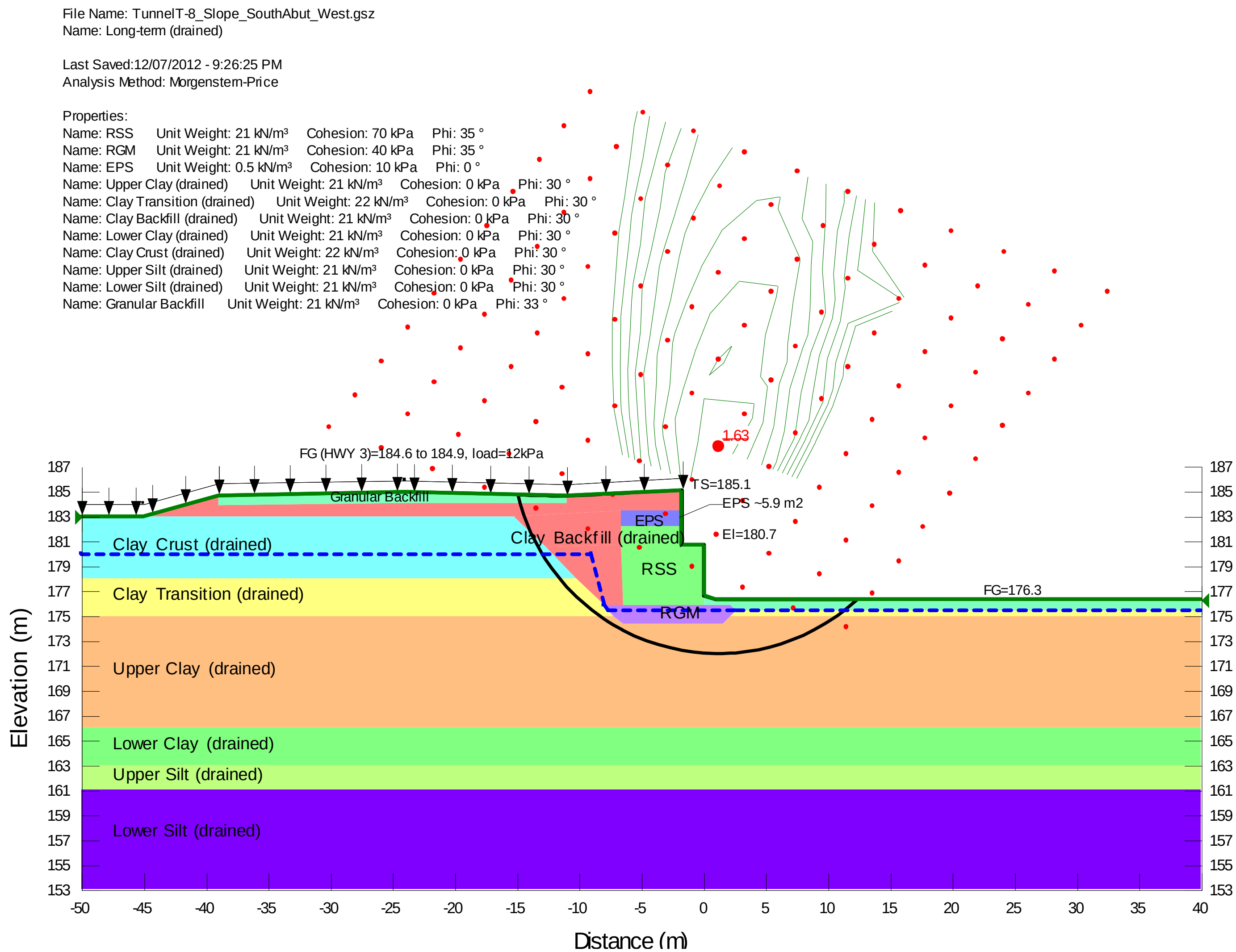


Figure F-14: Global Stability Result – South Abutment (Geraedts Drive) – Short Term (Undrained) Loading

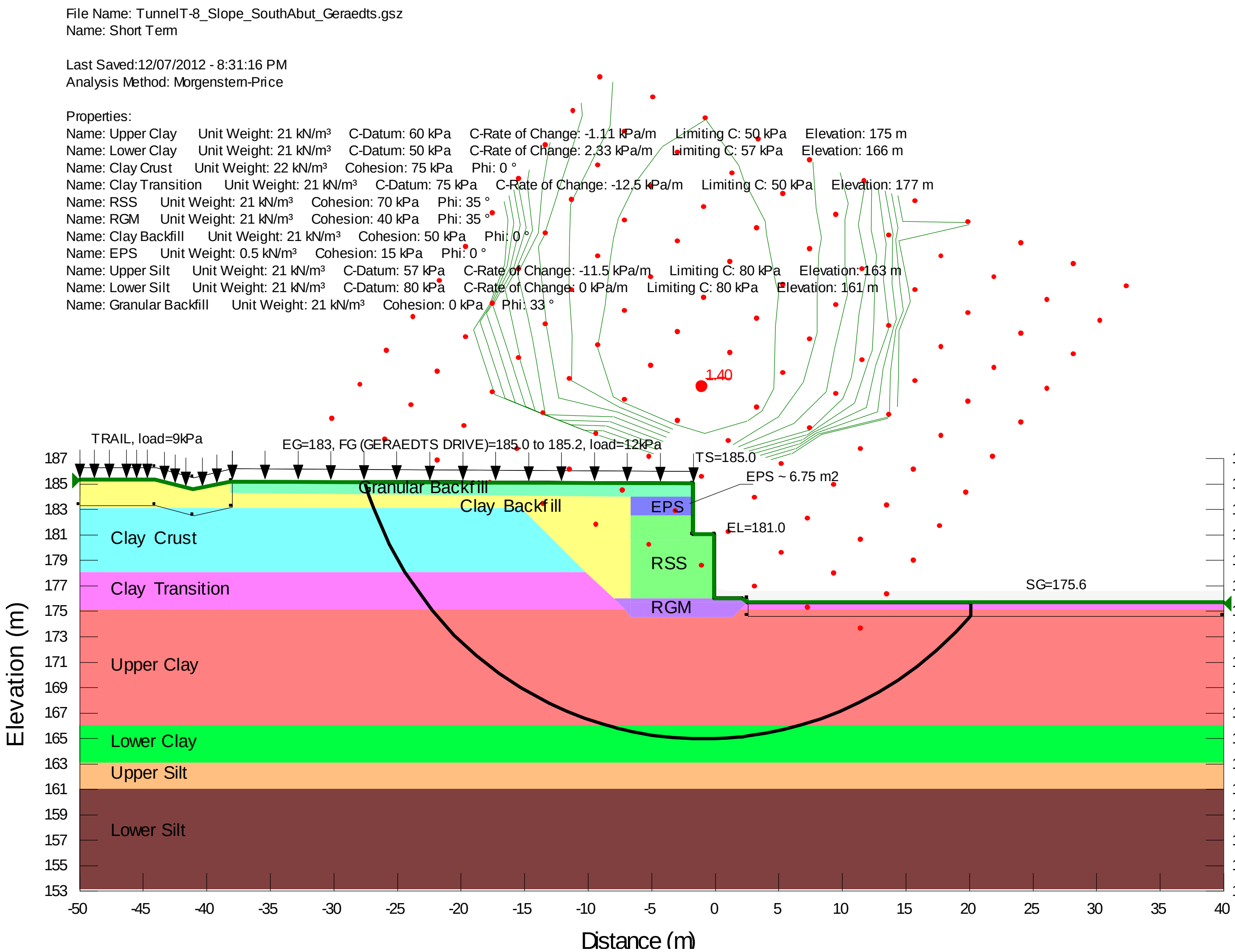
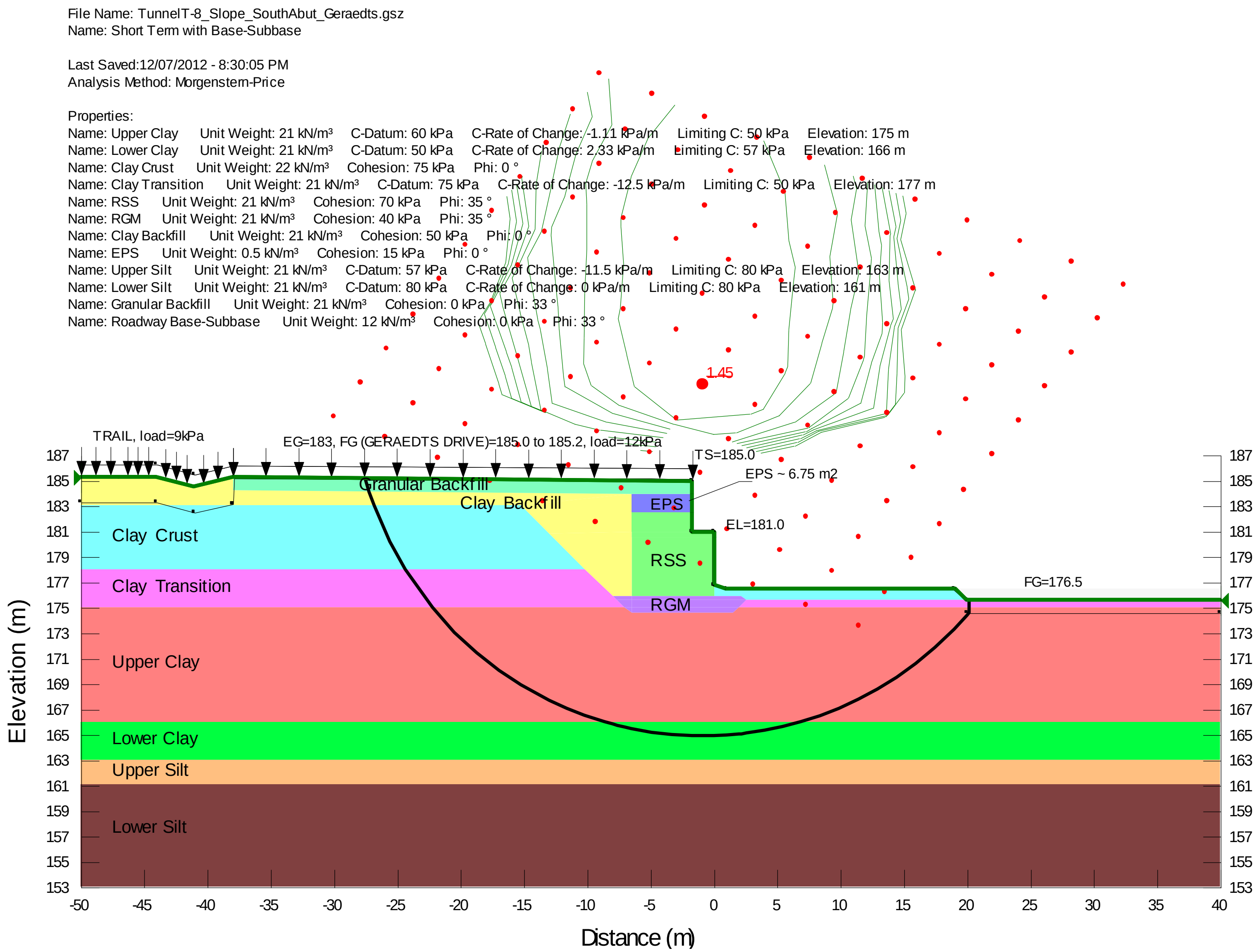


Figure F-15: Global Stability Result – South Abutment (Geraedts Drive) – Short Term (Undrained) Loading with Sub-base



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
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Figure F-16: Global Stability Result – South Abutment (Geraedts Drive) – End of Construction (Undrained) Loading

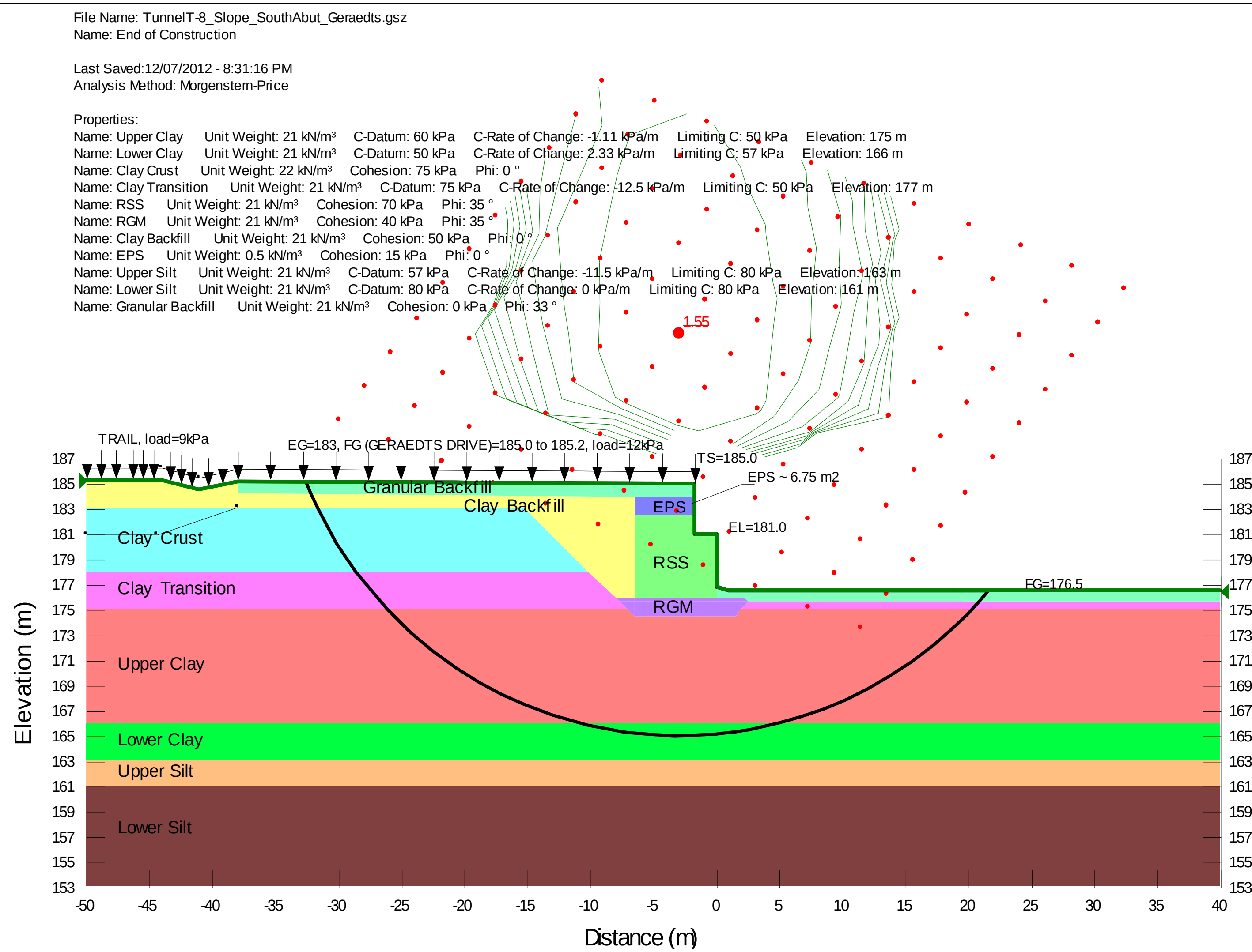
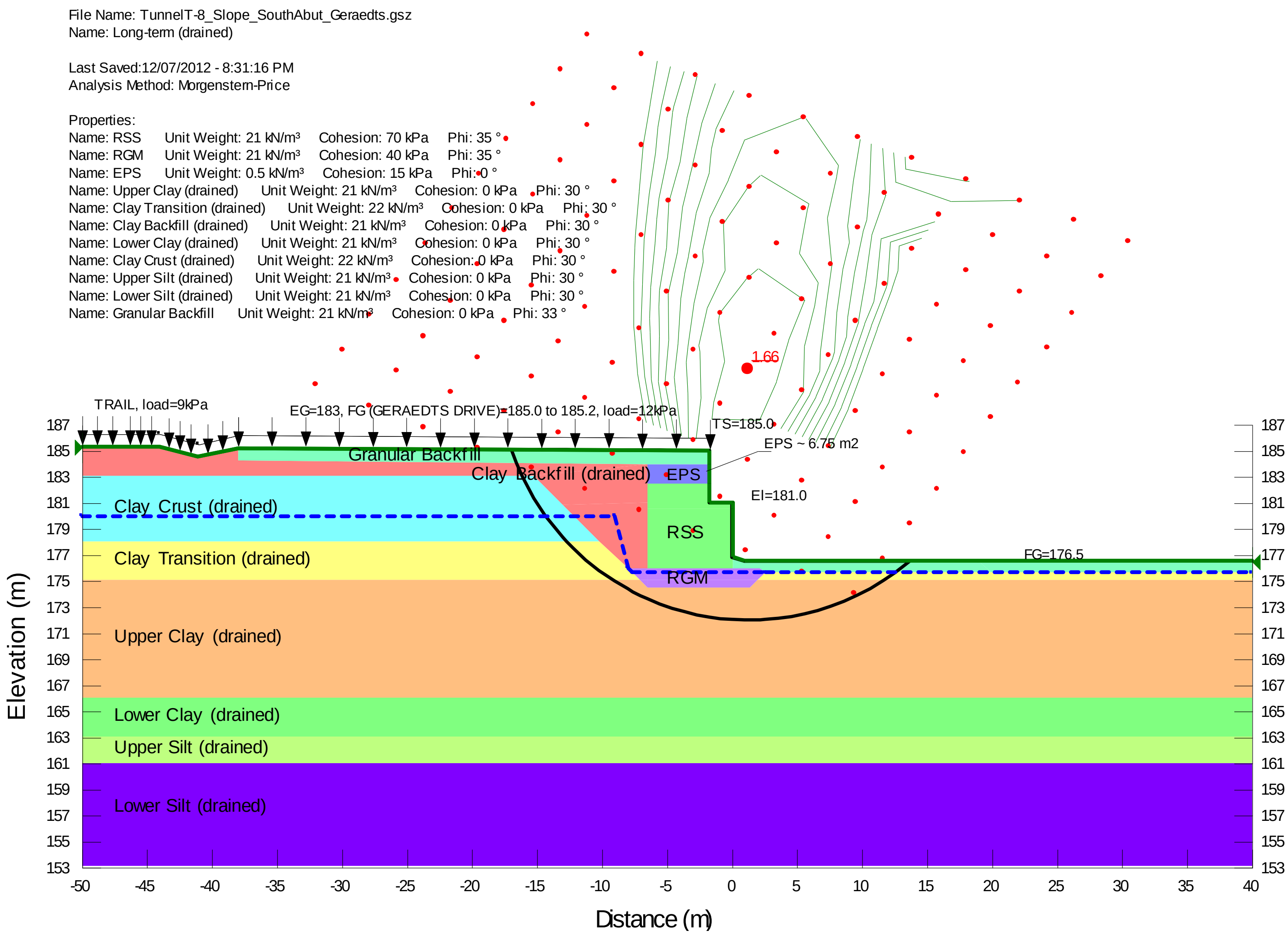
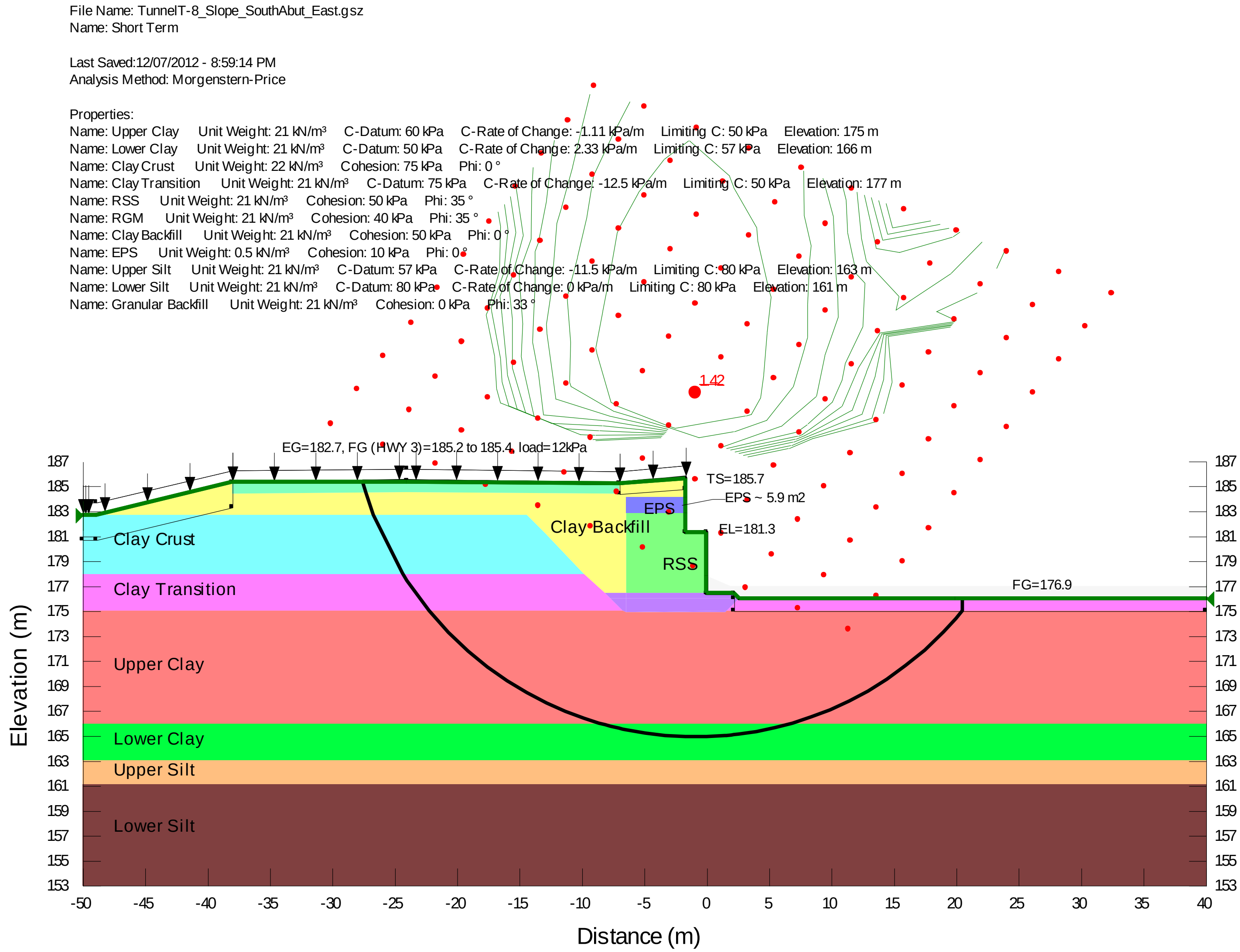


Figure F-17: Global Stability Result – South Abutment (Geraedts Drive) – Long Term (Drained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 17 of 27

Figure F-18: Global Stability Result – South Abutment (East Segment) – Short Term (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 18 of 27

Figure F-19: Global Stability Result – South Abutment (East Segment) – Short Term (Undrained) Loading with Sub-base

File Name: TunnelT-8_Slope_SouthAbut_East.gsz
Name: Short Term with Base-Subbase

Last Saved:12/07/2012 - 8:59:14 PM
Analysis Method: Morgenstern-Price

Properties:
Name: Upper Clay Unit Weight: 21 kN/m³ C-Datum: 60 kPa C-Rate of Change: -1.11 kPa/m Limiting C: 50 kPa Elevation: 175 m
Name: Lower Clay Unit Weight: 21 kN/m³ C-Datum: 50 kPa C-Rate of Change: 2.33 kPa/m Limiting C: 57 kPa Elevation: 166 m
Name: Clay Crust Unit Weight: 22 kN/m³ Cohesion: 75 kPa Phi: 0 °
Name: Clay Transition Unit Weight: 21 kN/m³ C-Datum: 75 kPa C-Rate of Change: -12.5 kPa/m Limiting C: 50 kPa Elevation: 177 m
Name: RSS Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 35 °
Name: RGM Unit Weight: 21 kN/m³ Cohesion: 40 kPa Phi: 35 °
Name: Clay Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 0 °
Name: EPS Unit Weight: 0.5 kN/m³ Cohesion: 10 kPa Phi: 0 °
Name: Upper Silt Unit Weight: 21 kN/m³ C-Datum: 57 kPa C-Rate of Change: -11.5 kPa/m Limiting C: 80 kPa Elevation: 163 m
Name: Lower Silt Unit Weight: 21 kN/m³ C-Datum: 80 kPa C-Rate of Change: 0 kPa/m Limiting C: 80 kPa Elevation: 161 m
Name: Granular Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 33 °
Name: Roadway Base-Subbase Unit Weight: 12 kN/m³ Cohesion: 0 kPa Phi: 33 °

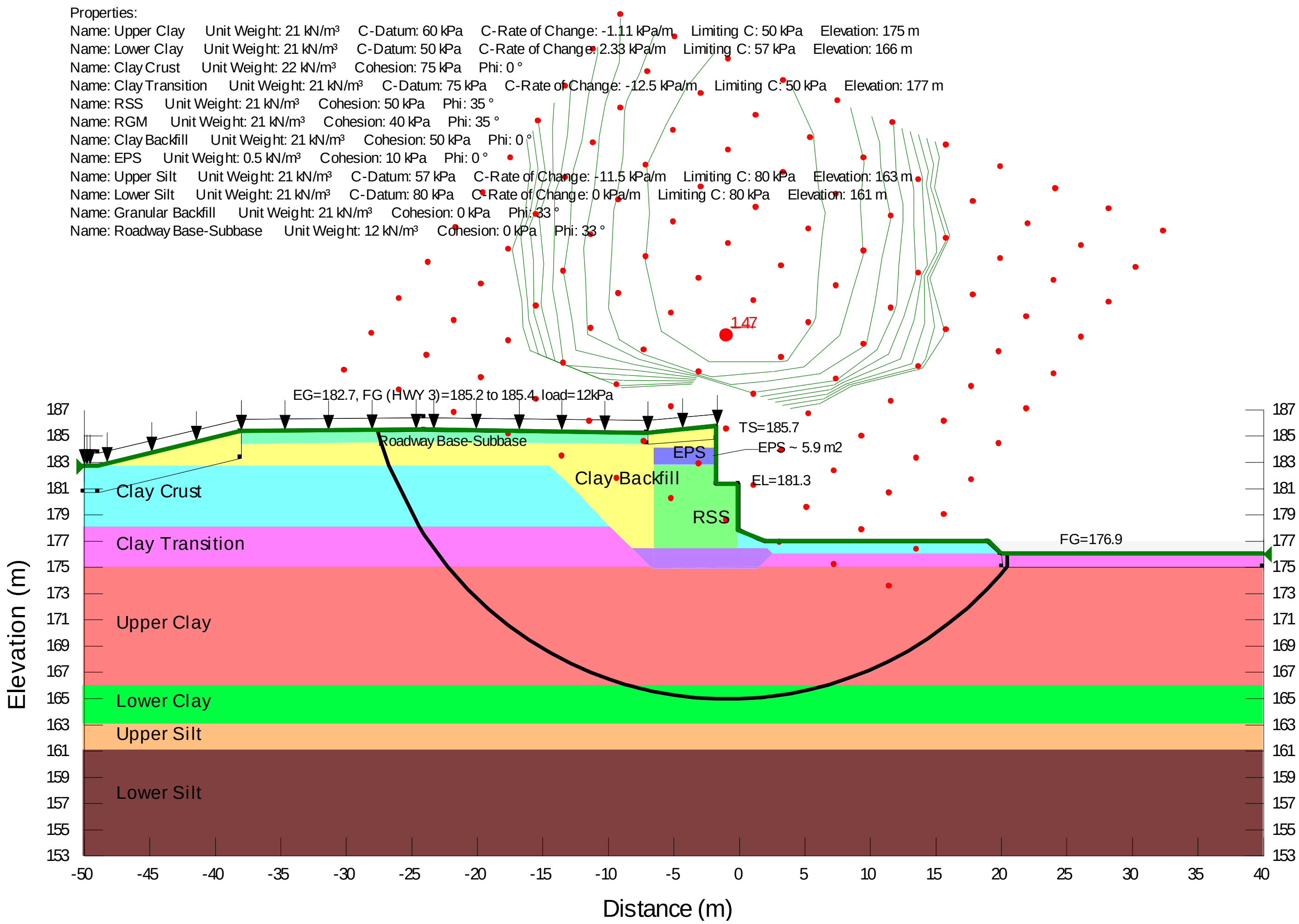


Figure F-20: Global Stability Result – South Abutment (East Segment) – End of Construction (Undrained) Loading

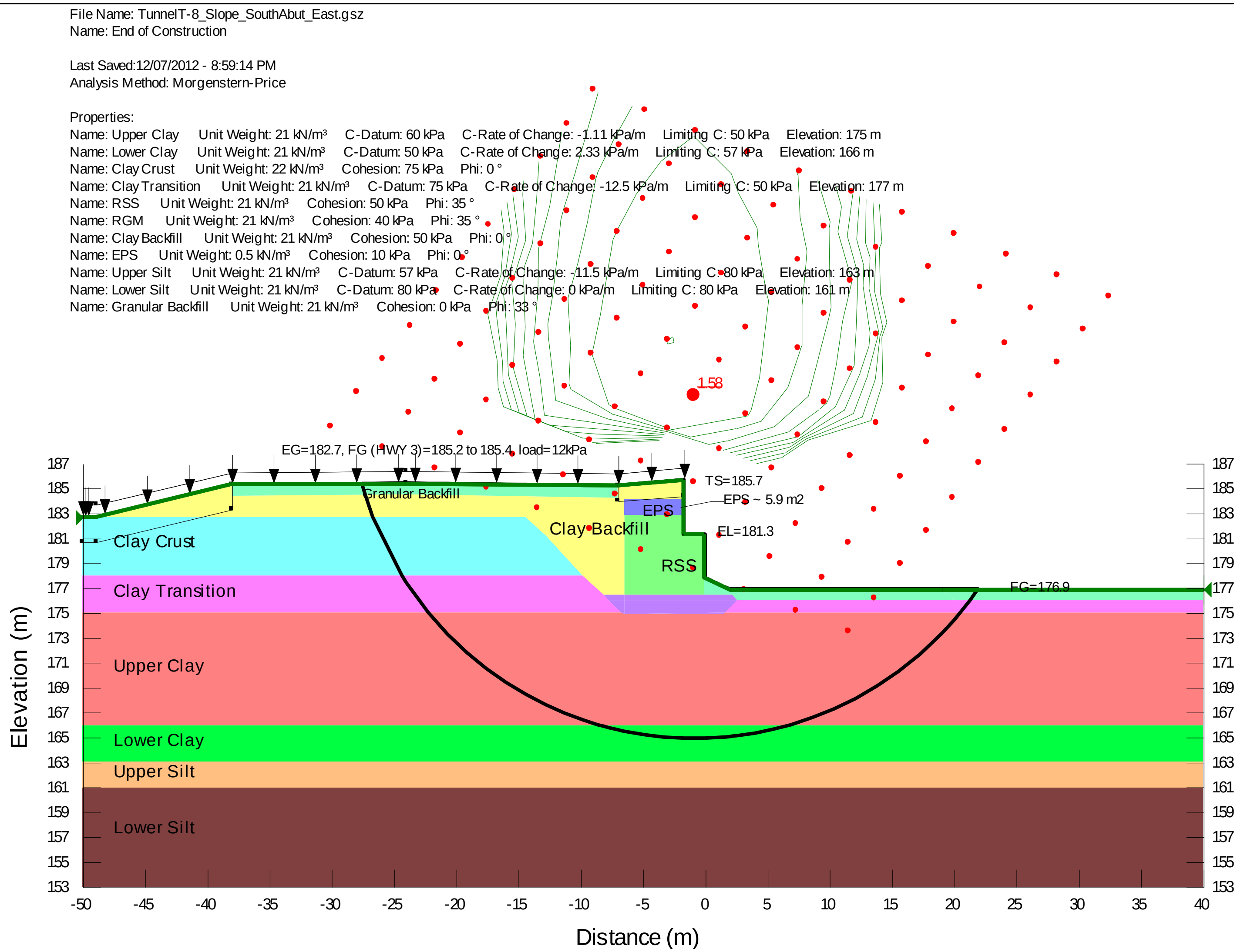
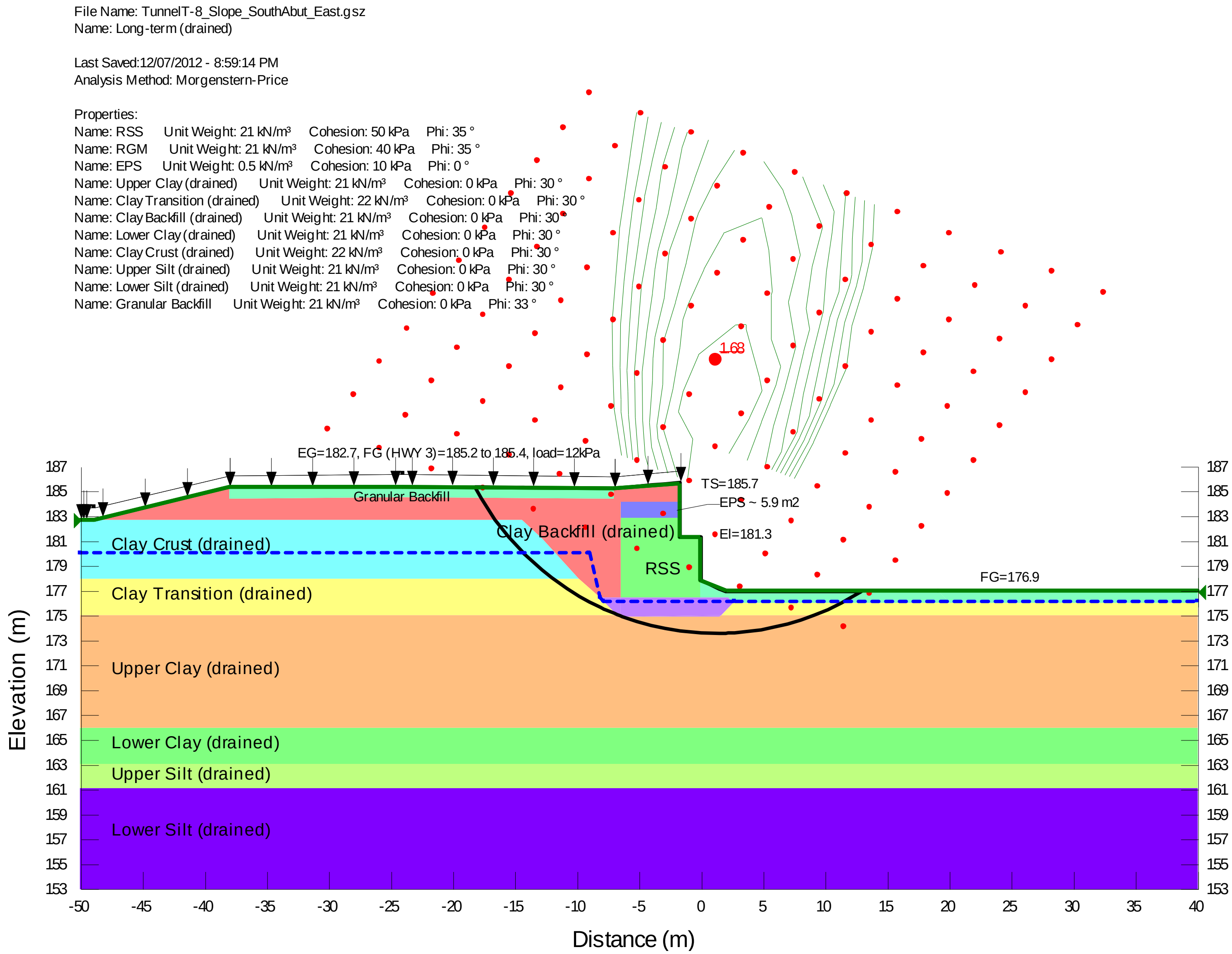


Figure F-21: Global Stability Result - South Abutment (East Segment) - Long Term (Drained) Loading



Project: Windsor-Essex Parkway
Date: September 2012
Document: Geotechnical Investigation and Design Report
Tunnel T-8 (Sta. 11+600L to 11+720L)
Rev: 0
Doc No.: 285380-04-119-0032 (Geocres No. 40J3-16)

Figure F-22: Global Stability Result – Tapered Wingwall South (West Segment)– Short Term (Undrained) Loading

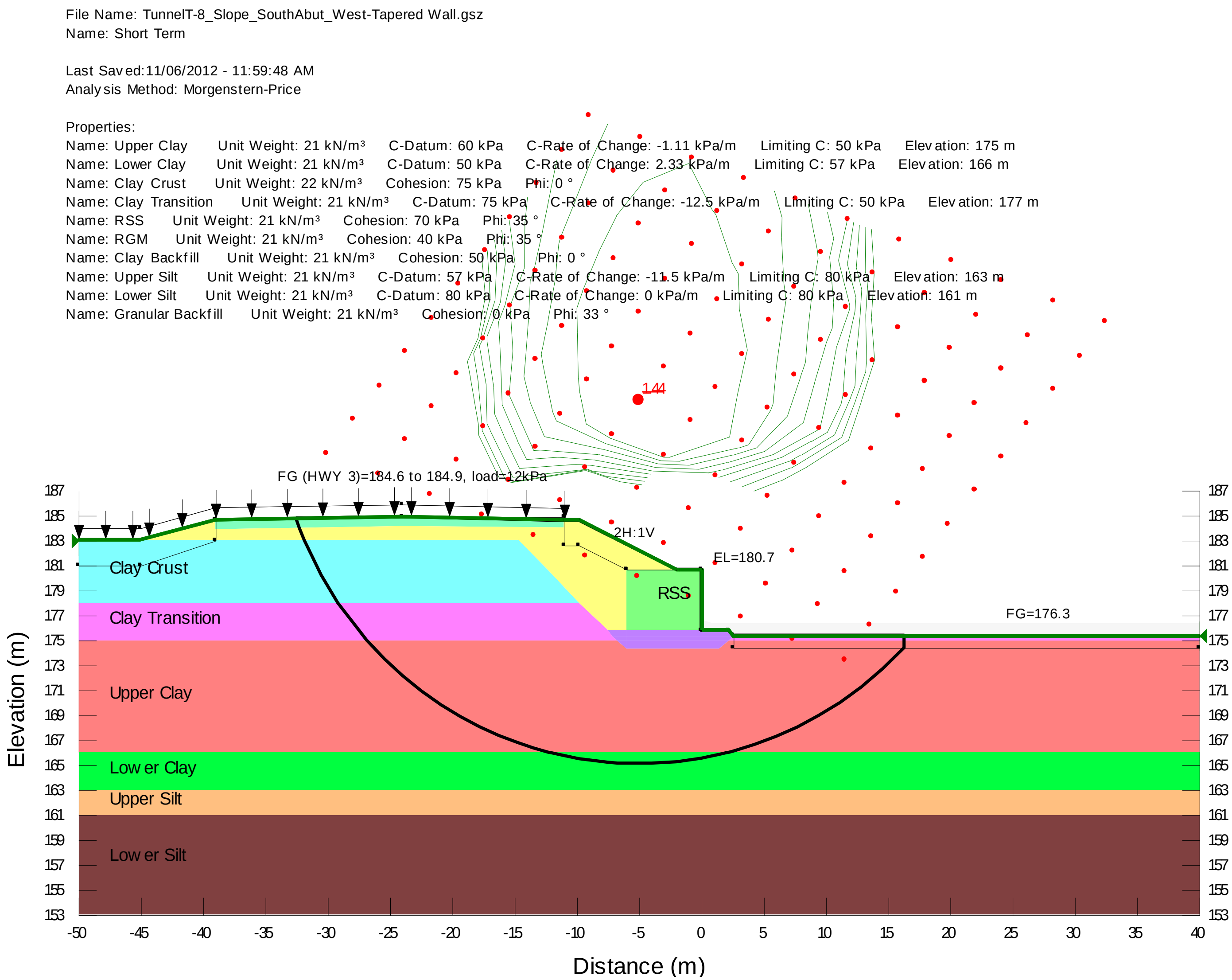


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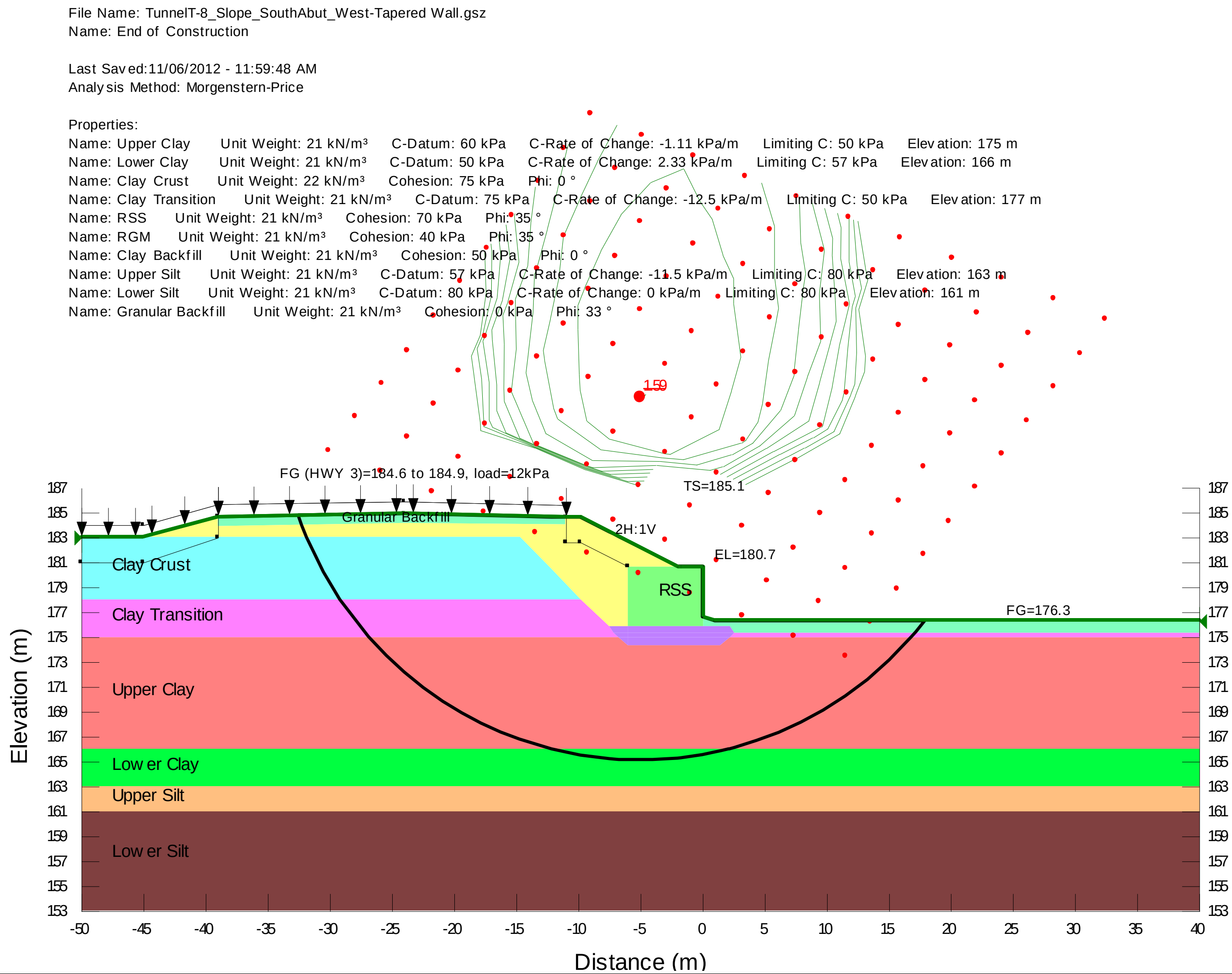
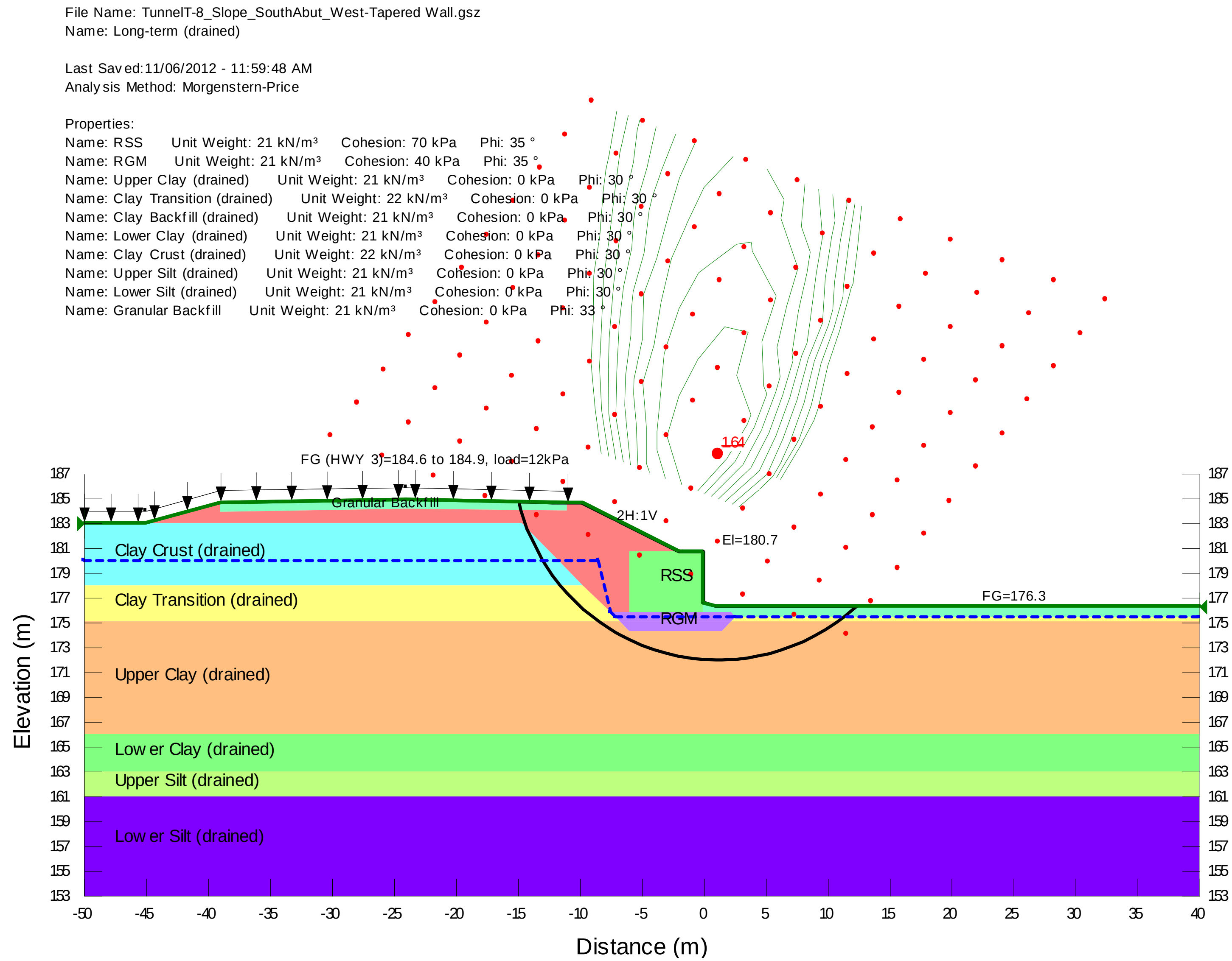
Figure F-23: Global Stability Result – Tapered Wingwall South (West Segment)– End of Construction (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 23 of 27

Figure F-24: Global Stability Result – Tapered Wingwall South (West Segment)– Long Term (Drained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 24 of 27

Figure F-25: Global Stability Result – Return Wingwall South (West Segment)– Short Term (Undrained) Loading

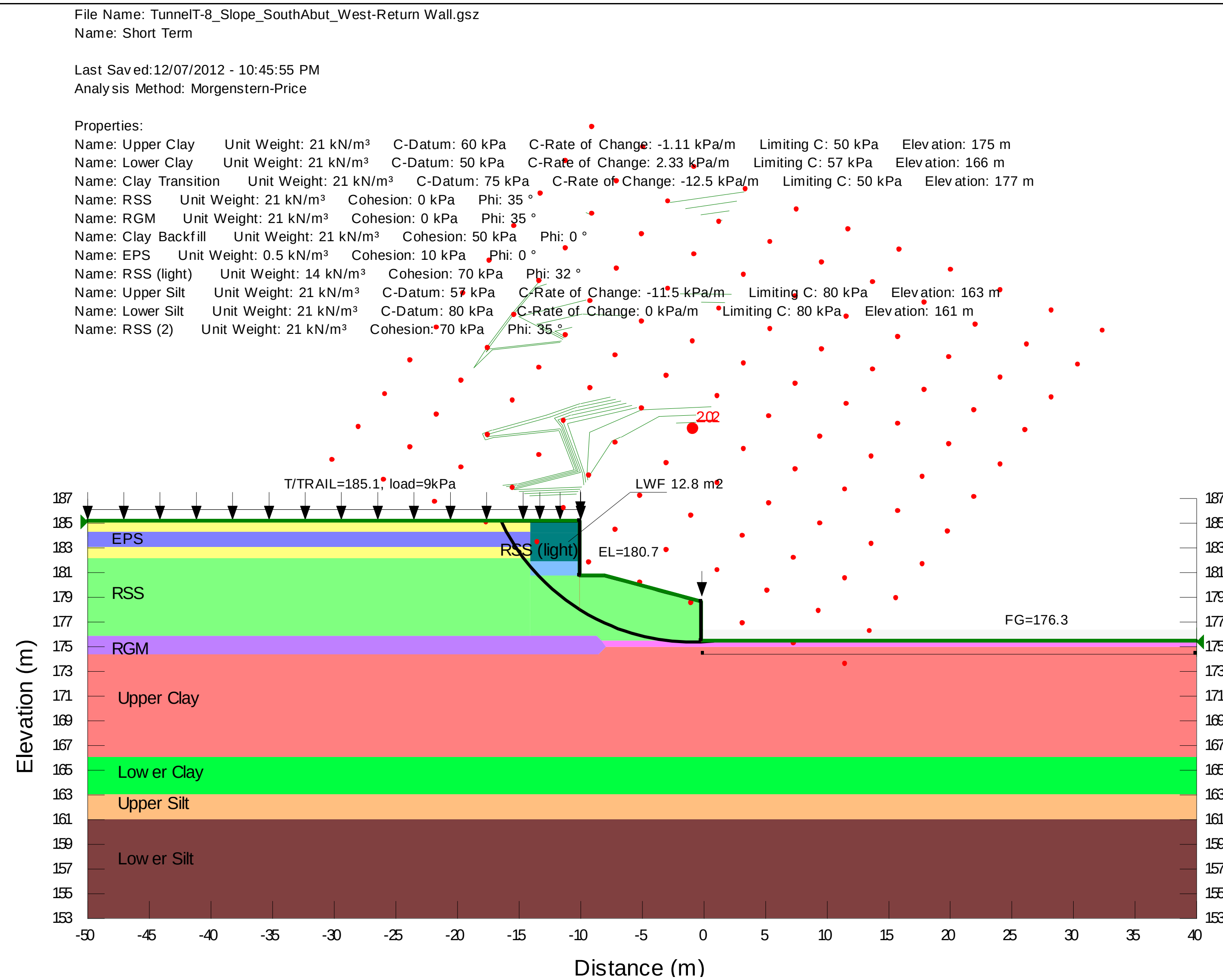
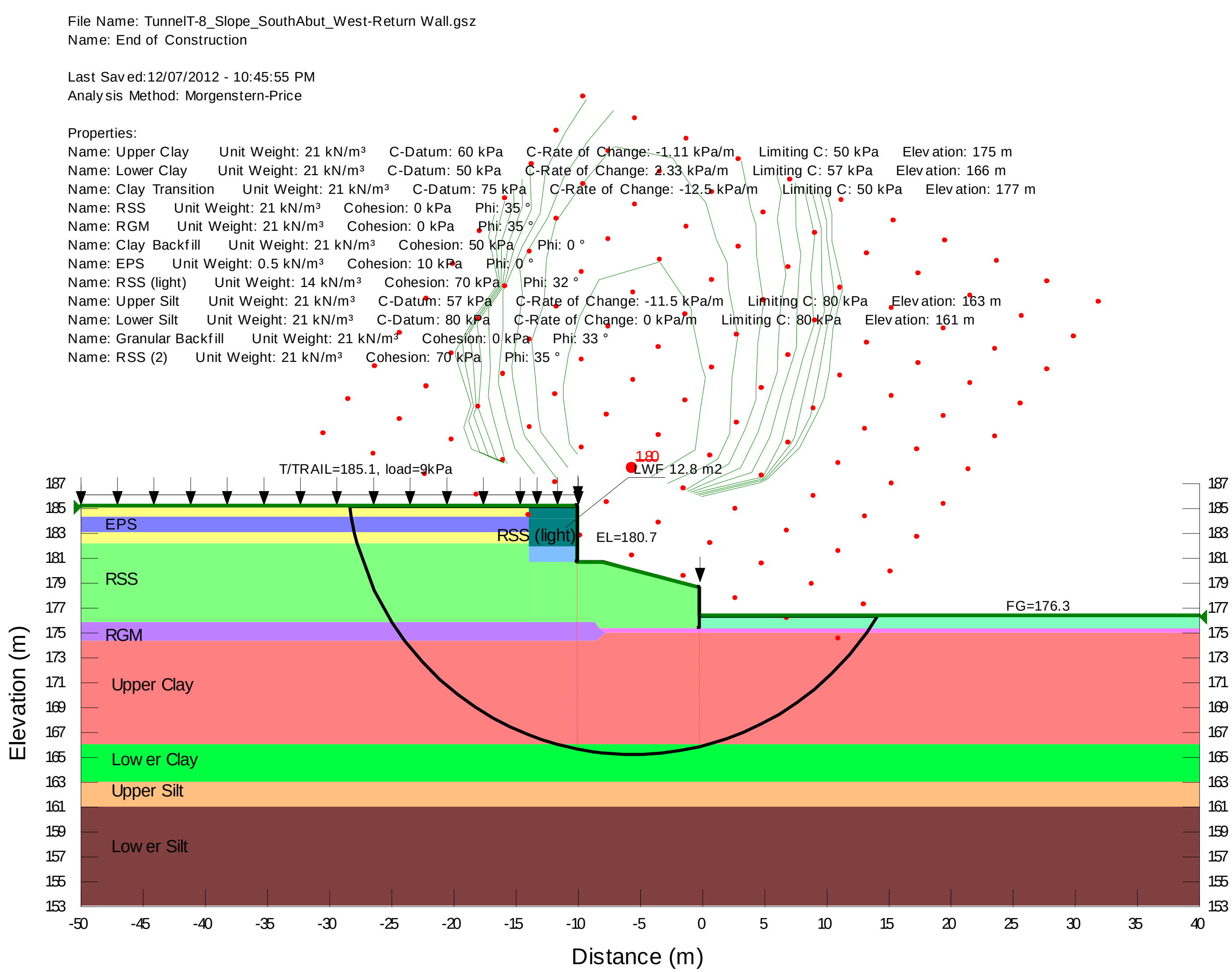
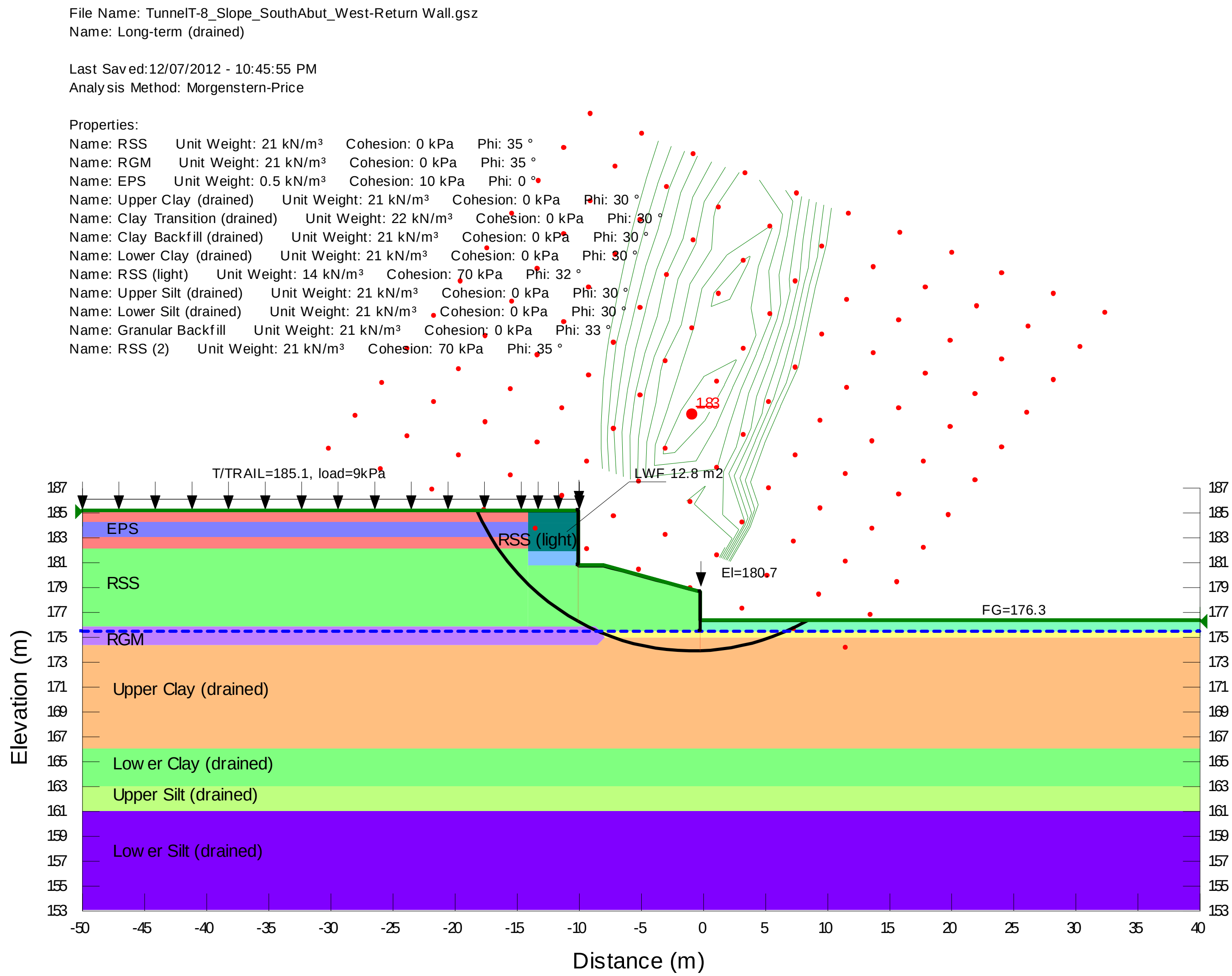


Figure F-26: Global Stability Result – Return Wingwall South (West Segment)– End of Construction (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 26 of 27

Figure F-27: Global Stability Result – Return Wingwall South (East Segment)– Long Term (Drained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September 2012
Document:	Geotechnical Investigation and Design Report Tunnel T-8 (Sta. 11+600L to 11+720L)	**Rev:**	0
Doc No.:	285380-04-119-0032 (Geocres No. 40J3-16)	**Page No.:**	Appendix F 27 of 27

Appendix G Stress-Deformation Analysis Results

Figure G-1: Cumulative Heave/Settlement - End of Excavation

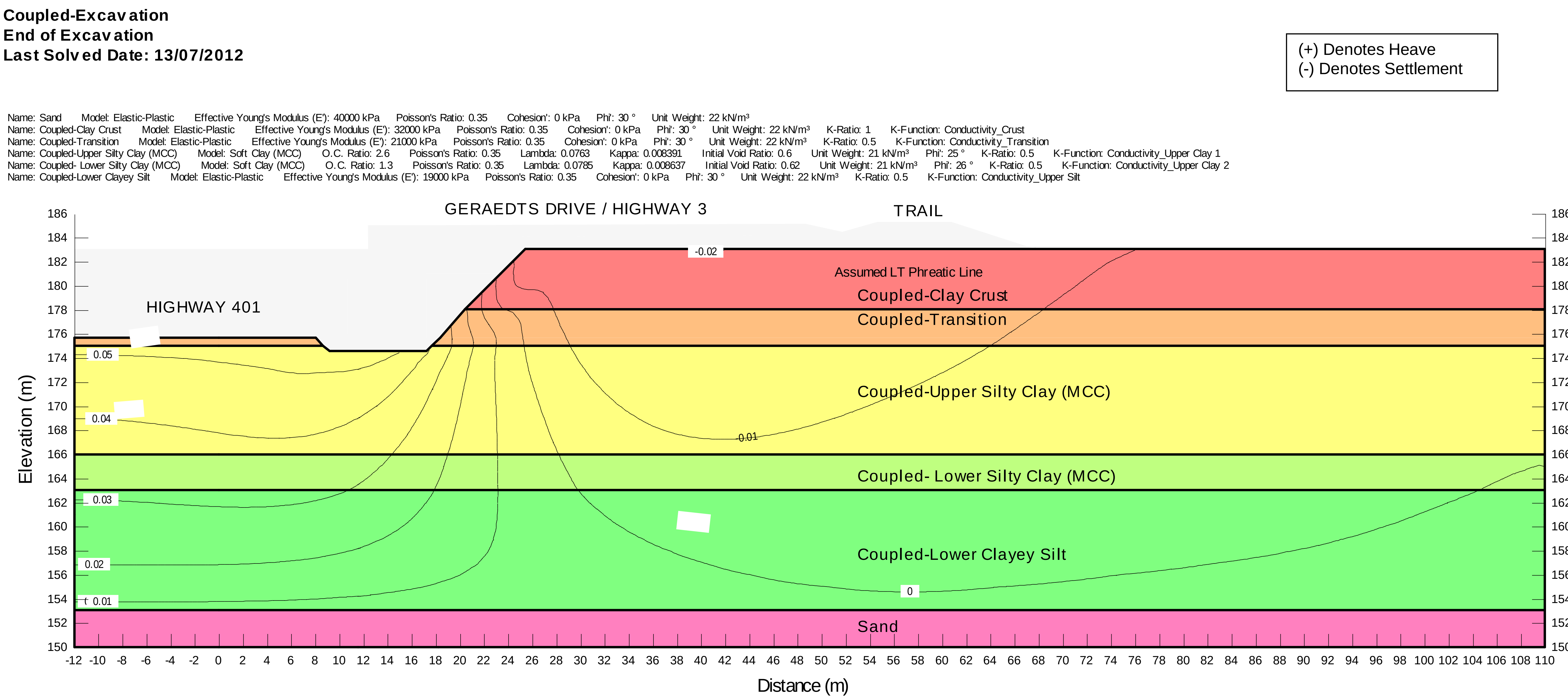


Figure G-2: Cumulative Lateral Movement - End of Excavation

Coupled-Excavation
End of Excavation
Last Solved Date: 13/07/2012

Name: Sand Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Coupled-Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Coupled-Transition Model: Elastic-Plastic Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Transition
Name: Coupled-Upper Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 2.6 Poisson's Ratio: 0.35 Lambda: 0.0763 Kappa: 0.008391 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 25 ° K-Ratio: 0.5 K-Function: Conductivity_Upper Clay 1
Name: Coupled- Lower Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 1.3 Poisson's Ratio: 0.35 Lambda: 0.0785 Kappa: 0.008637 Initial Void Ratio: 0.62 Unit Weight: 21 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Upper Clay 2
Name: Coupled-Lower Clayey Silt Model: Elastic-Plastic Effective Young's Modulus (E'): 19000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Upper Silt

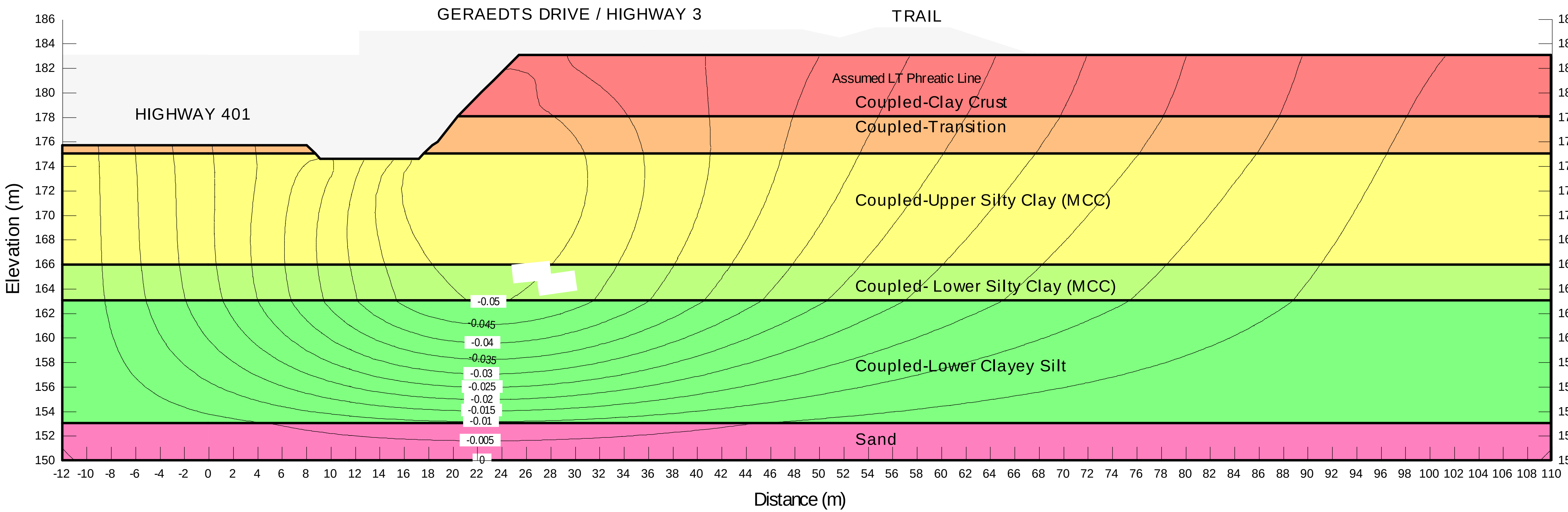


Figure G-3: Cumulative Heave/Settlement – End of Construction

Coupled-Roadway Backfill
End of Construction
Last Solved Date: 13/07/2012

(+) Denotes Heave
(-) Denotes Settlement

Name: General Backfill Model: Elastic-Plastic Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³
Name: Pavement Model: Linear Elastic Young's Modulus (E): 54000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Sand Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: RGM Model: Linear Elastic Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: EPS Model: Linear Elastic Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m³ Poisson's Ratio: 0.2
Name: RSS Backfill Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Coupled-Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Coupled-Transition Model: Elastic-Plastic Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Transition
Name: Coupled-Upper Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 2.6 Poisson's Ratio: 0.35 Lambda: 0.0763 Kappa: 0.008391 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 25 ° K-Ratio: 0.5 K-Function: Conductivity_Upper Clay 1
Name: Coupled- Lower Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 1.3 Poisson's Ratio: 0.35 Lambda: 0.0785 Kappa: 0.008637 Initial Void Ratio: 0.62 Unit Weight: 21 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Upper Clay 2
Name: Coupled-Lower Clayey Silt Model: Elastic-Plastic Effective Young's Modulus (E'): 19000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Upper Silt

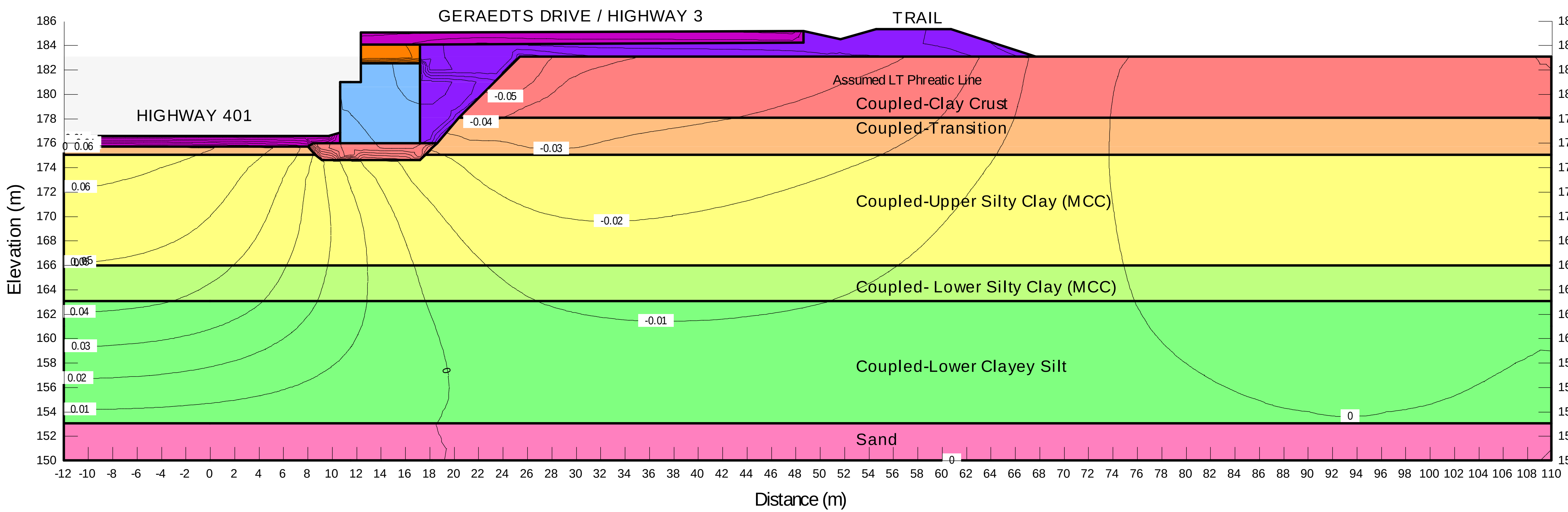


Figure G-4: Cumulative Heave/Settlement – Long-term (Drained)

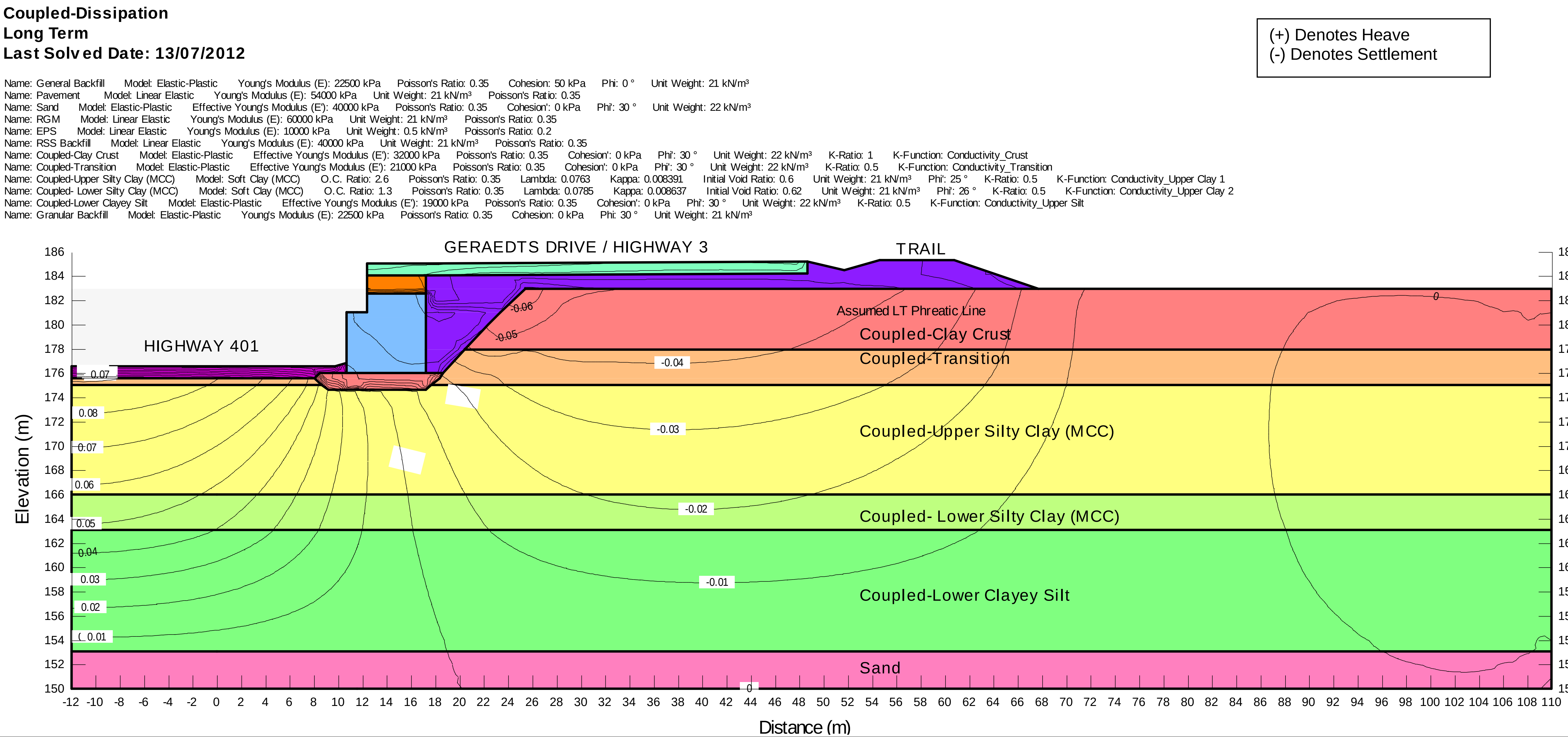


Figure G-5: Stabilized Porewater Pressure Contours - Long-term (Drained)

Coupled-Dissipation
Long Term
Last Solved Date: 13/07/2012

Name: General Backfill Model: Elastic-Plastic Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³
Name: Pavement Model: Linear Elastic Young's Modulus (E): 54000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Sand Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: RGM Model: Linear Elastic Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: EPS Model: Linear Elastic Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m³ Poisson's Ratio: 0.2
Name: RSS Backfill Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Coupled-Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Coupled-Transition Model: Elastic-Plastic Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Transition
Name: Coupled-Upper Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 2.6 Poisson's Ratio: 0.35 Lambda: 0.0763 Kappa: 0.008391 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 25 ° K-Ratio: 0.5 K-Function: Conductivity_Upper Clay 1
Name: Coupled- Lower Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 1.3 Poisson's Ratio: 0.35 Lambda: 0.0785 Kappa: 0.008637 Initial Void Ratio: 0.62 Unit Weight: 21 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Upper Clay 2
Name: Coupled-Lower Clayey Silt Model: Elastic-Plastic Effective Young's Modulus (E'): 19000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Upper Silt
Name: Granular Backfill Model: Elastic-Plastic Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³

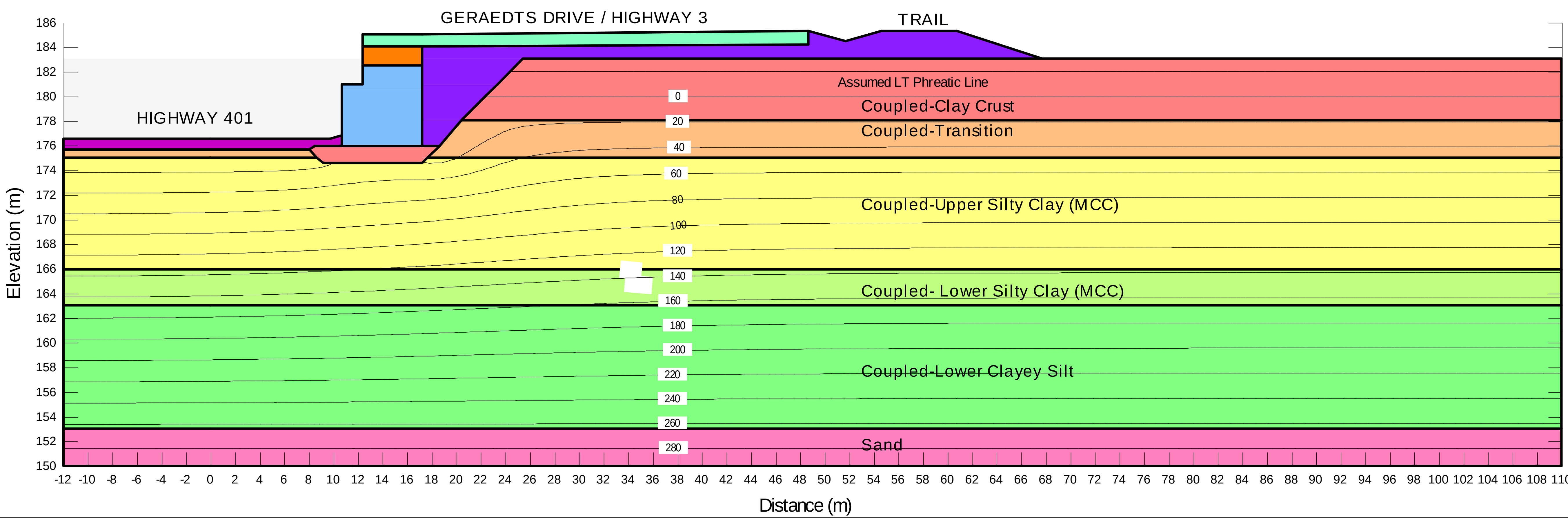
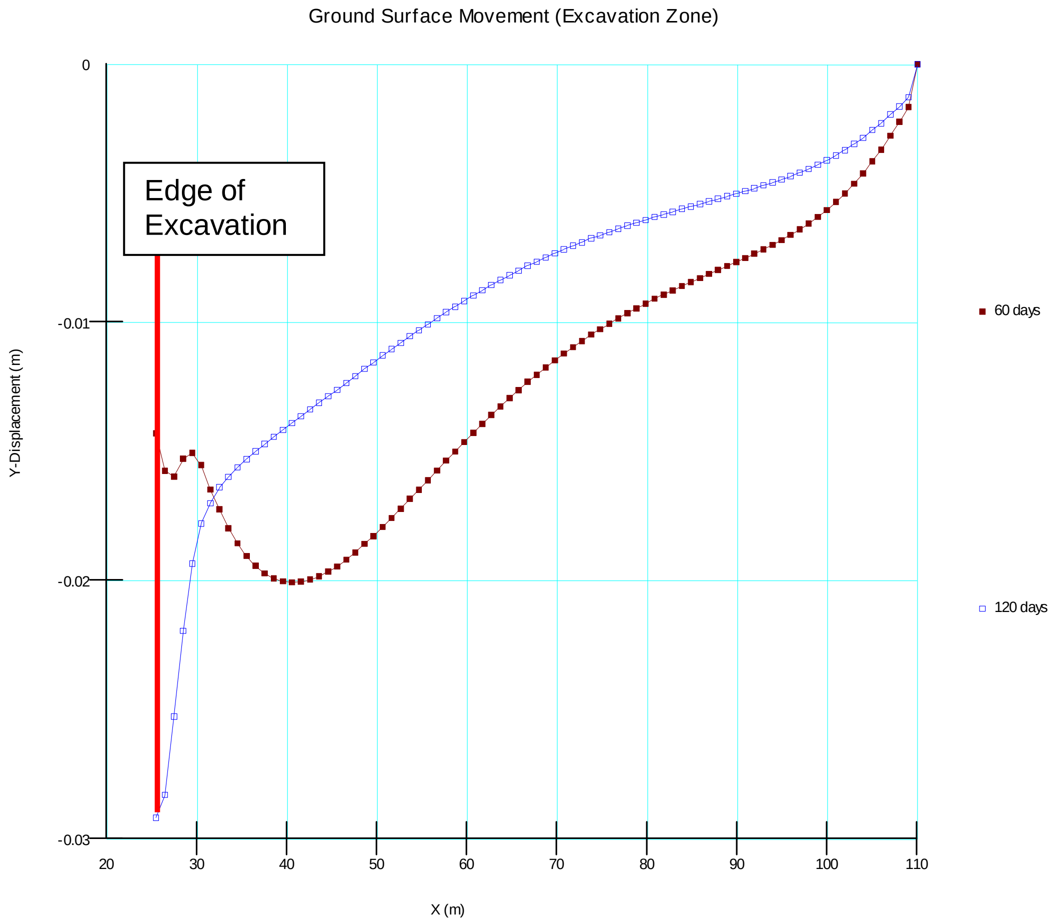


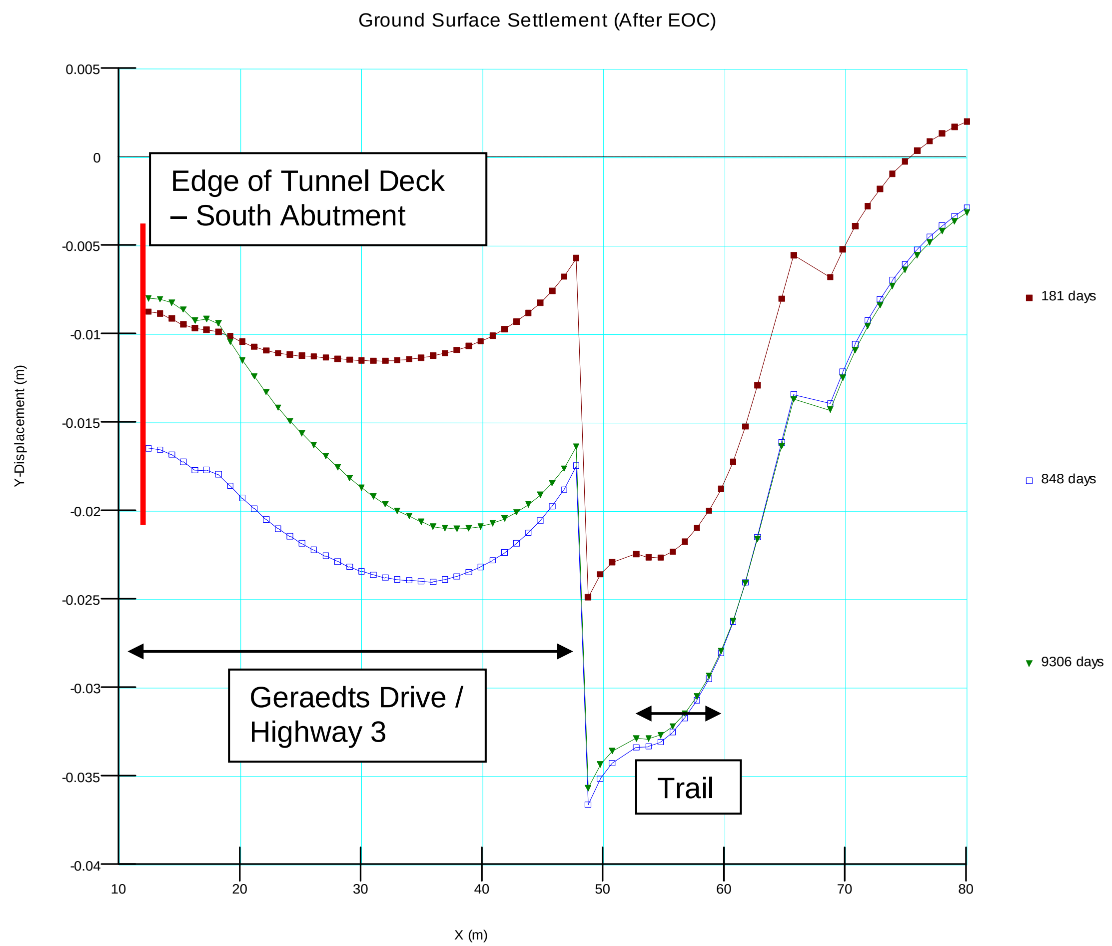
Figure G-6: Cumulative Ground Settlement at Approachway (End of Excavation and RSS Construction)



60 days = End of Excavation
120 days = RSS Construction

(+) Denotes Heave
(-) Denotes Settlement

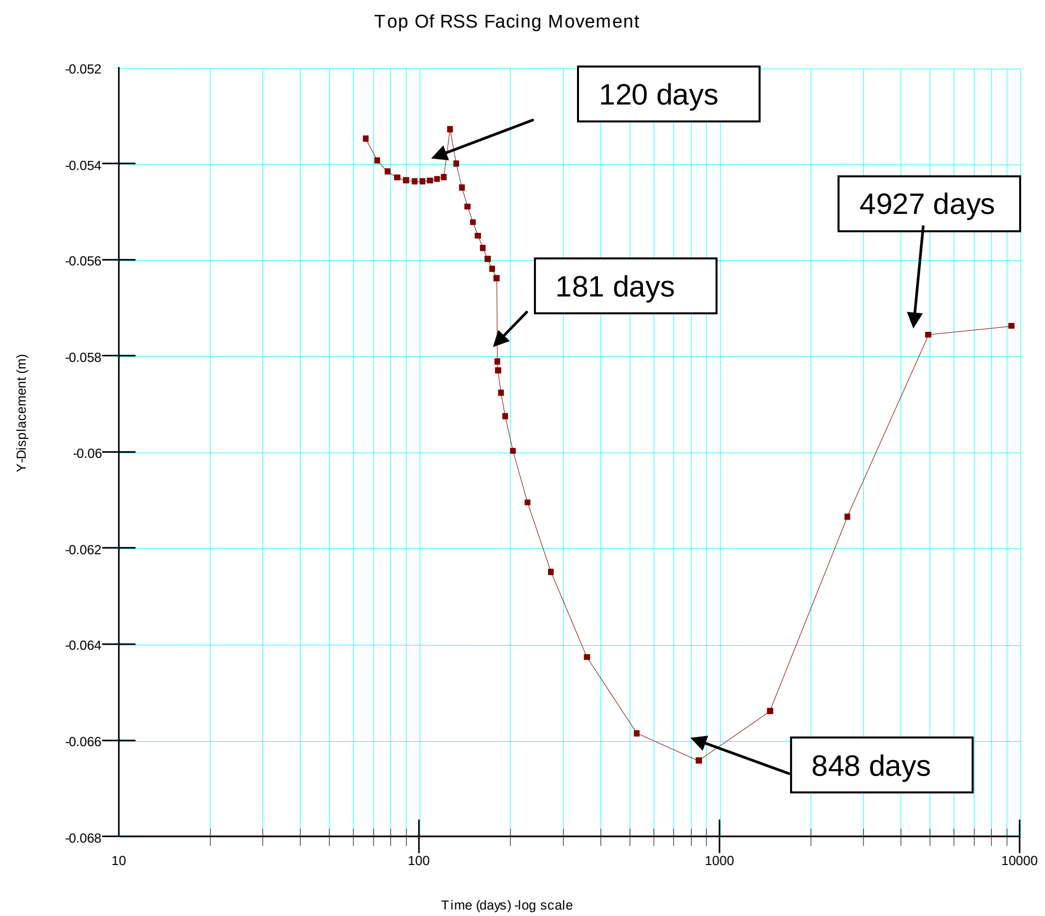
Figure G-7: Cumulative Ground Settlement at Approachway (End of Construction and Long-term Condition)



181 days = End of Construction
848 days = Estimated Start of Rebound
9306 days = Long Term Condition

(+) Denotes Heave
(-) Denotes Settlement

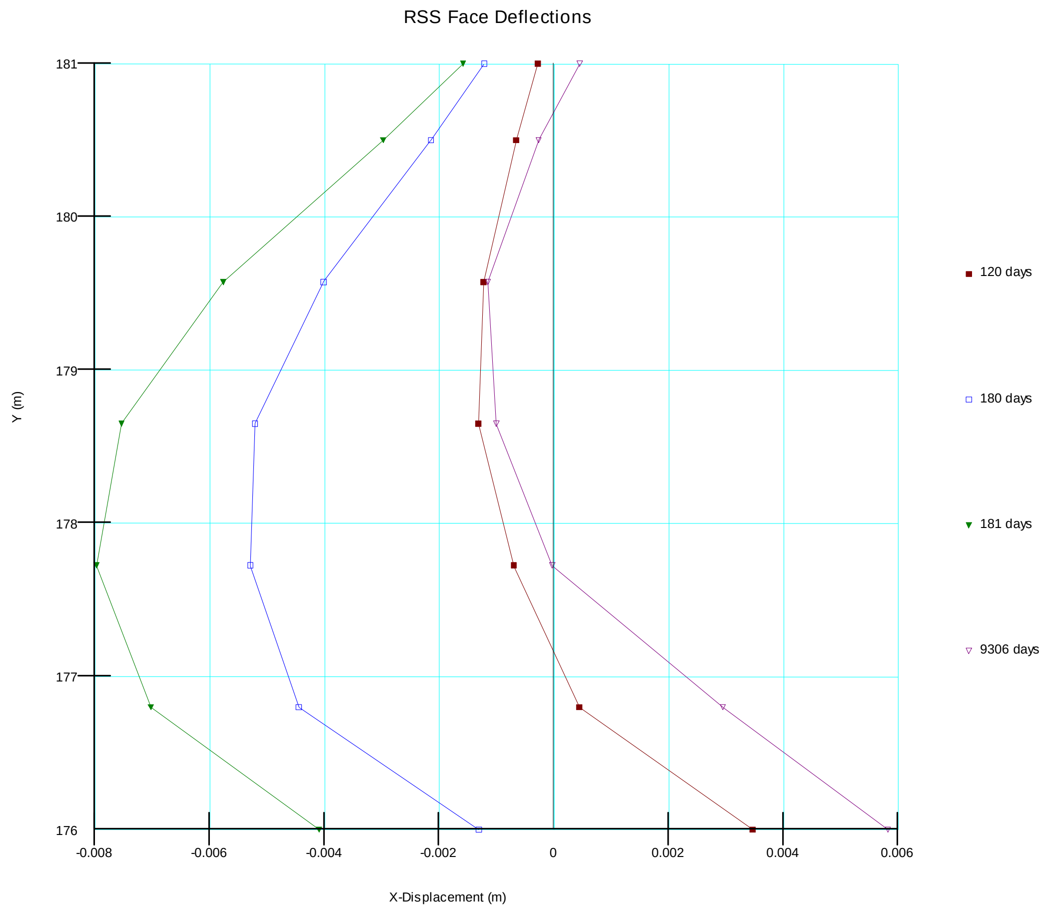
Figure G-8: Cumulative Settlement at Top of RSS Wall Facing



60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
848 days = Estimated Start of Rebound
9306 days = Long-term Condition

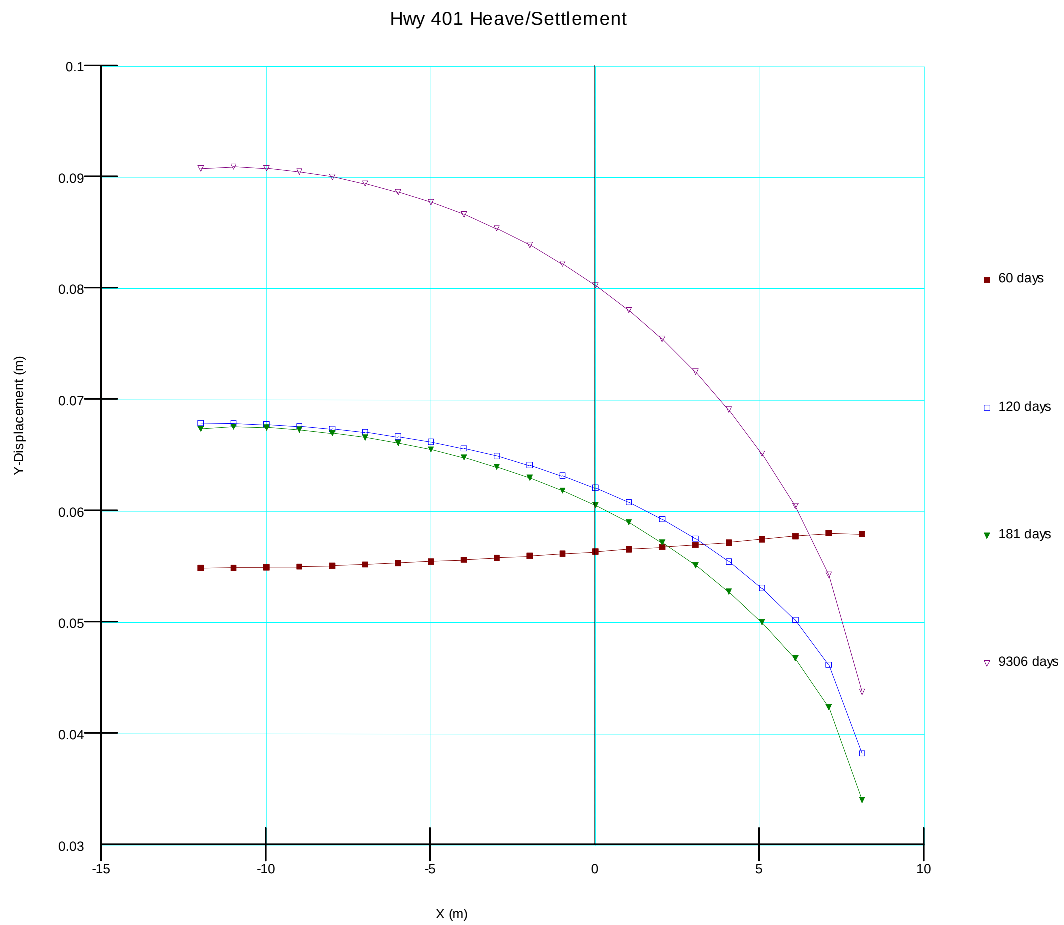
(+) Denotes Heave
(-) Denotes Settlement

Figure G-9: Cumulative Lateral Deflection of RSS Wall



120 days = RSS Completion
180 days = Abutment Completion
181 days = End of Construction
9306 days = Long-term Condition

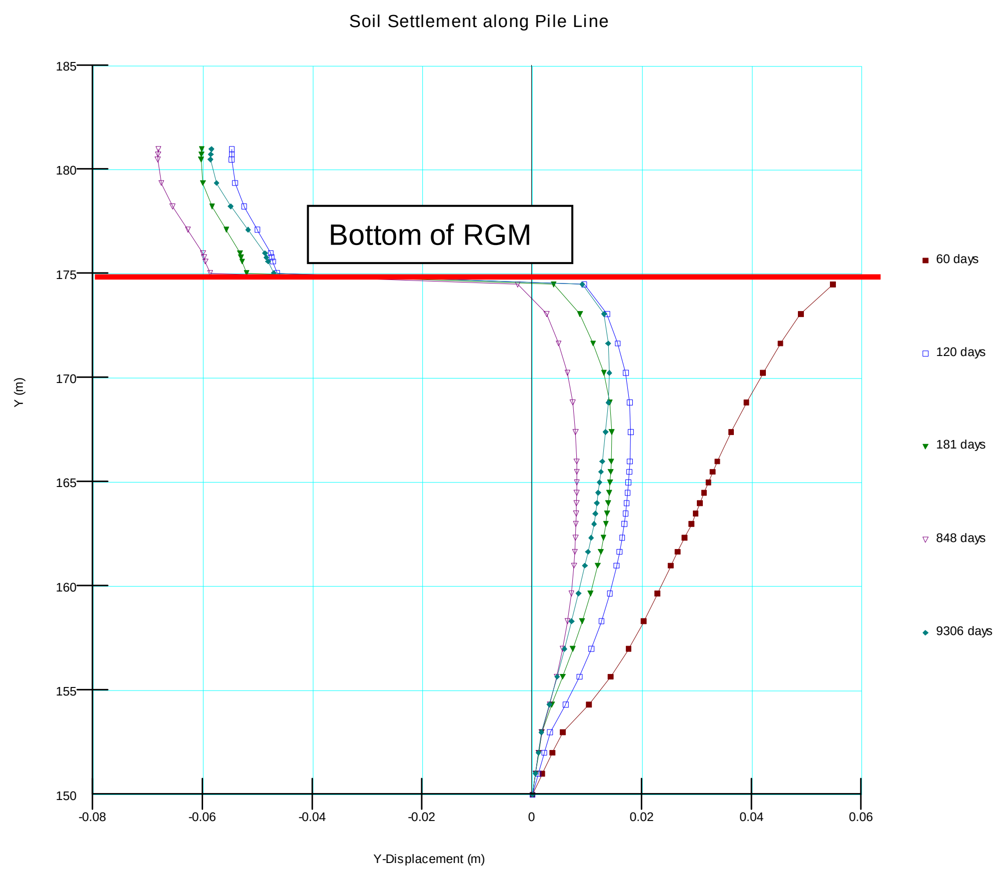
Figure G-10: Cumulative Highway 401 Settlement/Heave (Subgrade Level)



60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
9306 days = Long-term Condition

(+) Denotes Heave
(-) Denotes Settlement

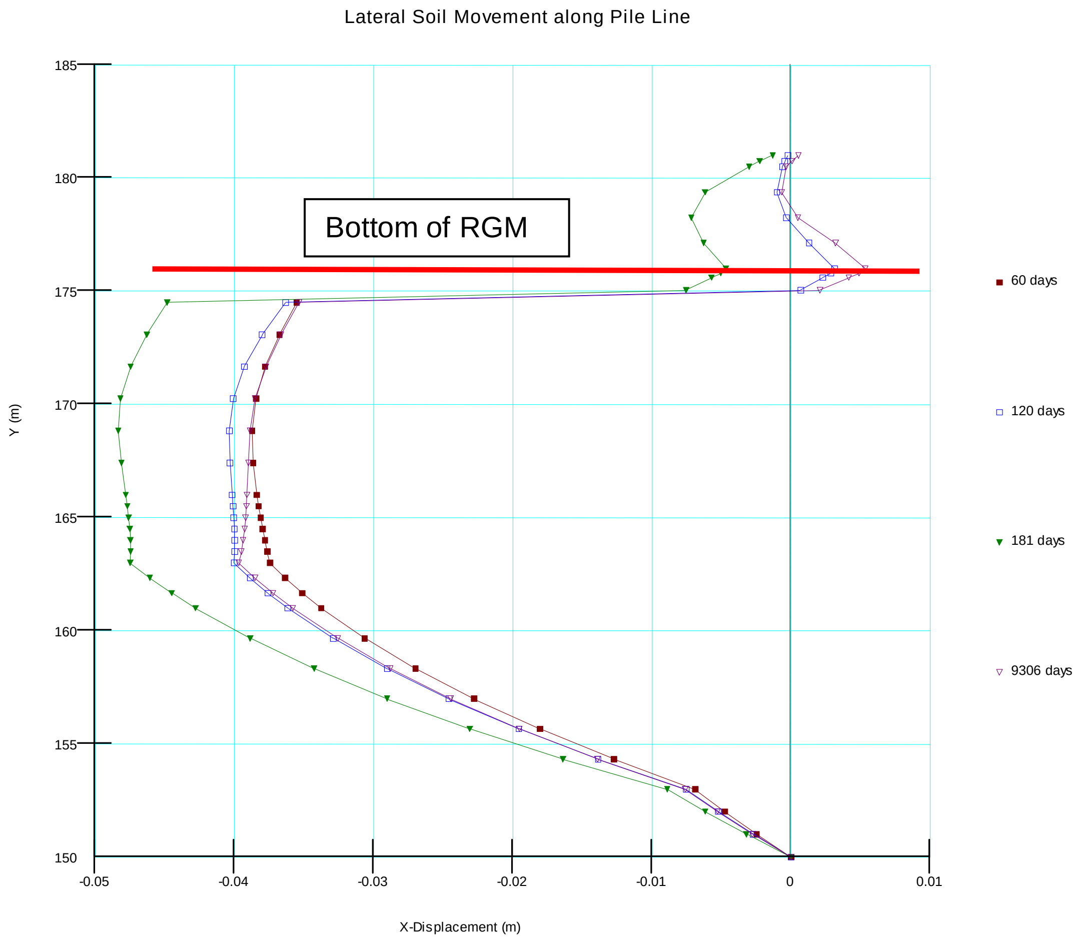
Figure G-11: Cumulative Soil Settlement Profile along Pile Line



60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
848 days = Estimated Start of Rebound
9306 days = Long-term Condition

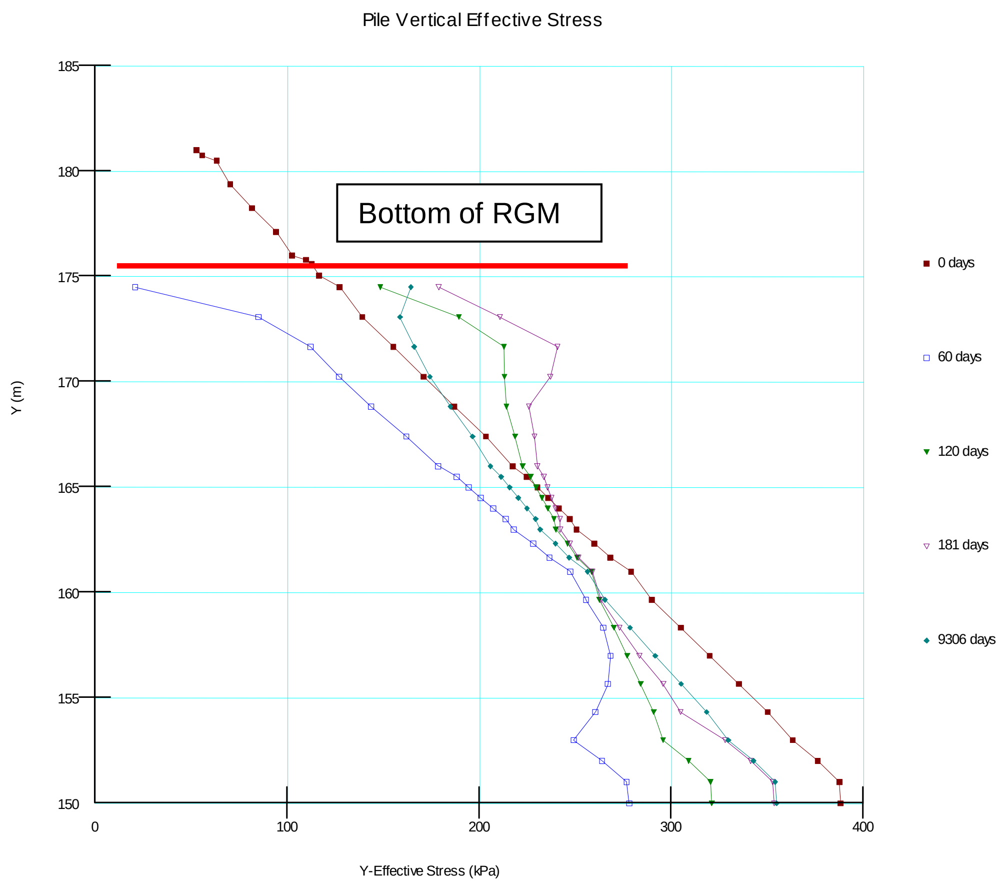
(+) Denotes Heave
(-) Denotes Settlement

Figure G-12: Cumulative Lateral Soil Displacement Profile along Pile Line



60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
9306 days = Long-term Condition

Figure G-13: Vertical Effective Stress Profile along Pile Line



0 days = Insitu Condition
60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
9306 days = Long-term Condition

Appendix H Seepage Analysis Results

Figure H-1: Tunnel T-8 North Abutment Section - Steady-State Seepage Analysis

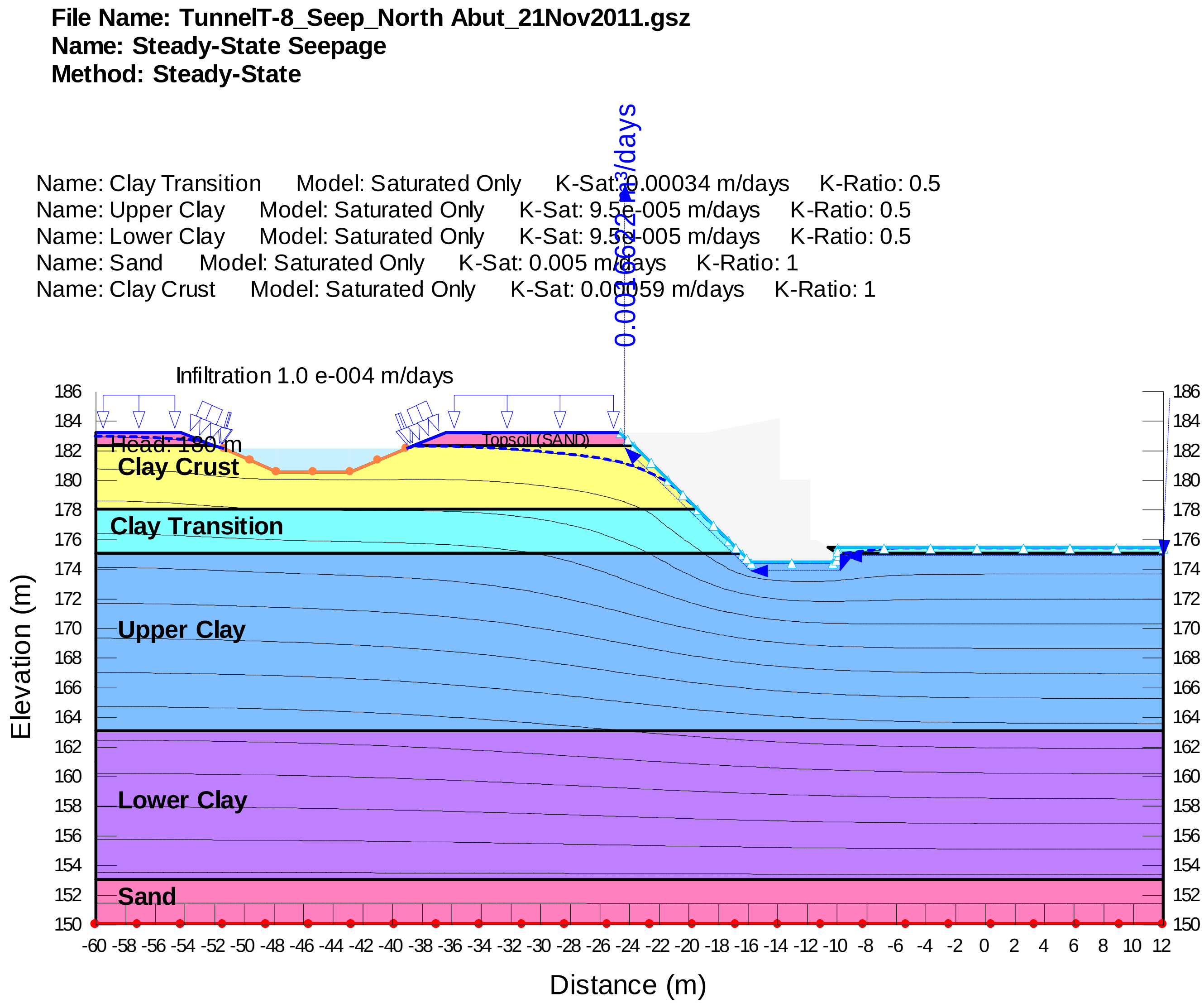
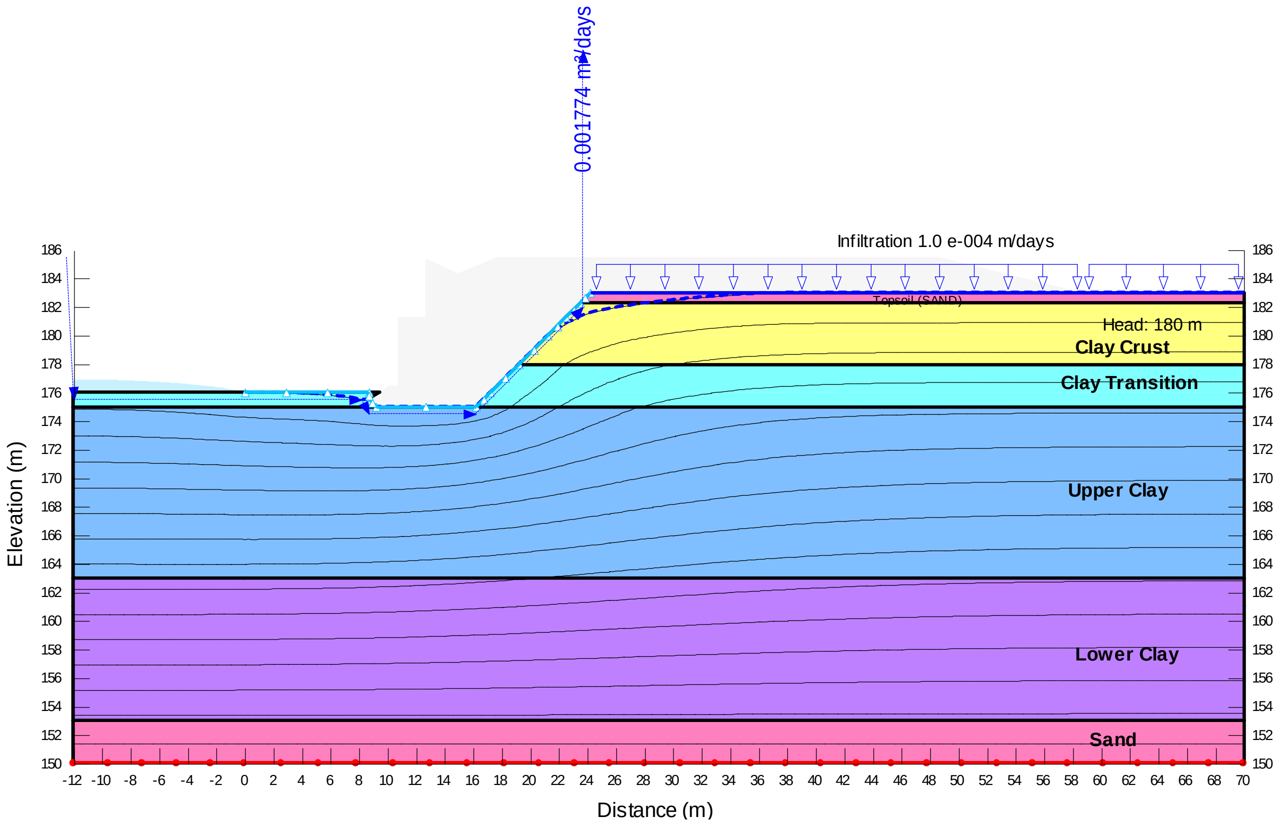


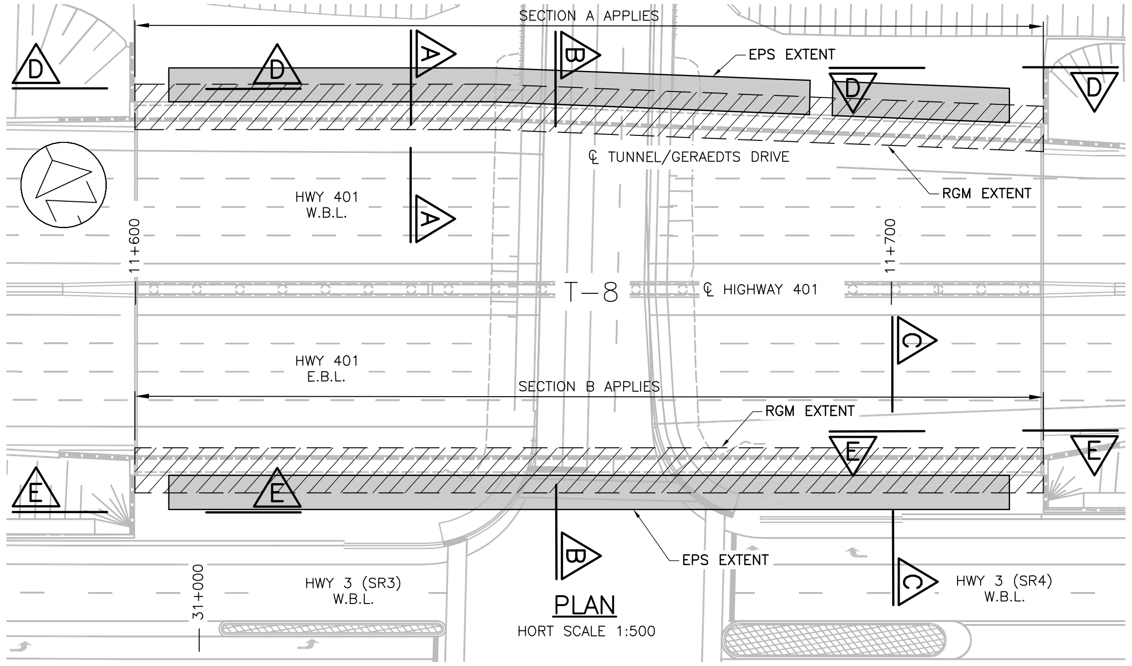
Figure H-2: Tunnel T-8 South Abutment Section - Steady-State Seepage Analysis

File Name: TunnelT-8_Seep_South Abut_21Nov2011.gsz
Name: Steady-State Seepage
Method: Steady-State

Name: Clay Transition Model: Saturated Only K-Sat: 0.00034 m/days K-Ratio: 0.5
Name: Upper Clay Model: Saturated Only K-Sat: 9.5e-005 m/days K-Ratio: 0.5
Name: Low er Clay Model: Saturated Only K-Sat: 9.5e-005 m/days K-Ratio: 0.5
Name: Sand Model: Saturated Only K-Sat: 0.005 m/days K-Ratio: 1
Name: Clay Crust Model: Saturated Only K-Sat: 0.00059 m/days K-Ratio: 1



Appendix I Conceptual Drawings



PLAN
HORT SCALE 1:500

NOTES:

1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
2. THIS DRAWING ILLUSTRATES THE GENERAL ARRANGEMENTS AT SELECTED REPRESENTATIVE LOCATIONS OF THE ABUTMENTS OF TUNNEL T-8 BASED ON GEOTECHNICAL DESIGN ANALYSES.
3. THE ILLUSTRATED RSS WALL WIDTH REPRESENTS THE MINIMUM WIDTH BASED ON GEOTECHNICAL REQUIREMENTS. THE DESIGN OF THE RSS WALL IS TO BE DEVELOPED BY OTHERS.
4. TUNNEL ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGNS WERE OBTAINED FROM STRUCTURAL DRAWINGS AVAILABLE IN MAY 2012. ABUTMENT ELEVATIONS VARY ALONG THE TUNNEL.
5. CLAY SUBGRADE IS SUSCEPTIBLE TO DISTURBANCE AND LOSS OF STRENGTH DUE TO WATER INFLOW/PONDING, CONSTRUCTION TRAFFIC AND THE LIKE. SUITABLE EXCAVATION METHODS, DEWATERING AND SUBGRADE PROTECTION MUST BE EXERCISED.
6. CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS. EXCAVATED CLAY SURFACES ARE SUSCEPTIBLE TO DETERIORATION AND EXPERIENCE DEFORMATIONS AND INSTABILITY; THEY ARE TO BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED AND TREATED AS REQUIRED.
7. AT THE SOUTH ABUTMENT, BACKFILL ABOVE THE RSS SHALL BE PLACED ONLY AFTER THE PAVEMENT SUBGRADE ADJACENT TO THE TOE OF THE RSS IS FULLY RESTORED AND COVERED BY THE GRANULAR BASE WITHIN AT LEAST 20m FROM RSS FACING.
8. RGM INTERNAL DESIGN TO BE DETERMINED BY SUPPLIER IN CONJUNCTION WITH LOADING CONDITIONS DESCRIBED IN GEOTECHNICAL REPORT.
9. SEE ACCOMPANYING DRAWINGS FOR APPLICABLE CONSTRUCTION NOTES.
10. RGM DRAIN ELEVATIONS AT LEAST 500mm BELOW TOP OF RGM

LEGEND:

RSS – REINFORCED SOIL STRUCTURE

GF – APPROVED GRANULAR FILL

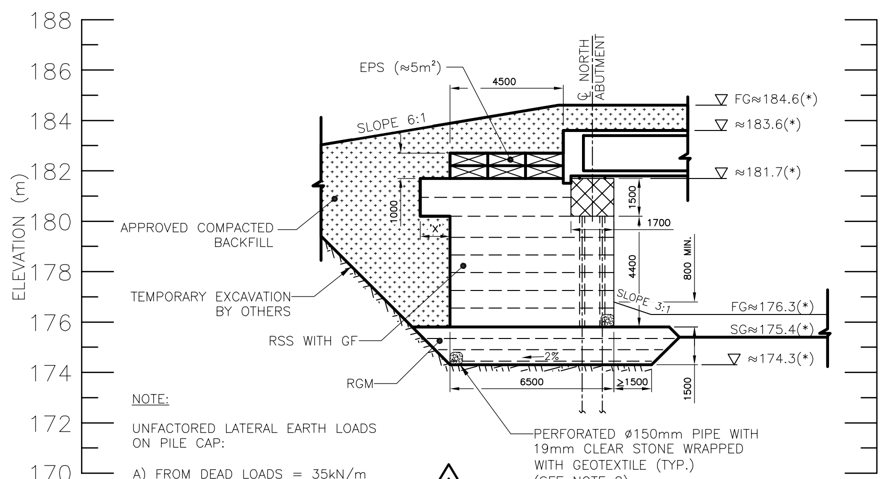
RGM – REINFORCED GRANULAR MAT (LONG-TERM ALLOWABLE LOAD CAPACITY OF GEOGRID SHALL BE MINIMUM 21kN/m)

EPS – EXPANDED POLYSTYRENE

LWF – LIGHT WEIGHT FILL (ULTRALIGHT WATER-COOLED IRON FURNACE SLAG)

'X' – LENGTH TO BE DETERMINED BY SUPPLIER

(*) – VARIES



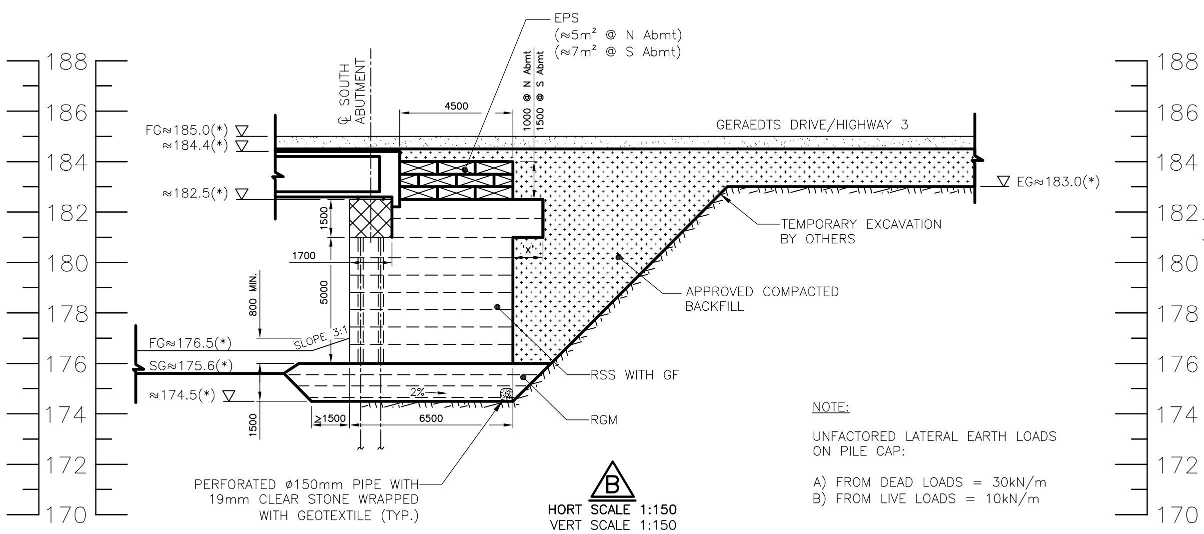
NOTE:

UNFACTORED LATERAL EARTH LOADS ON PILE CAP:

A) FROM DEAD LOADS = 35kN/m
B) FROM LIVE LOADS = 6kN/m

HORT SCALE 1:150
VERT SCALE 1:150

NORTH ABUTMENT SECTION A–A



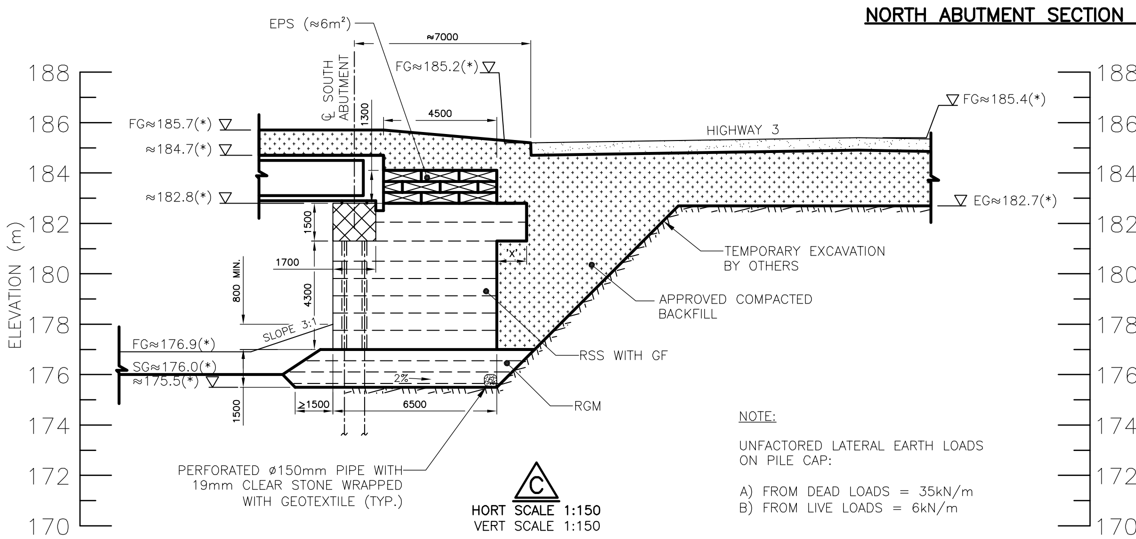
HORT SCALE 1:150
VERT SCALE 1:150

NOTE:

UNFACTORED LATERAL EARTH LOADS ON PILE CAP:

A) FROM DEAD LOADS = 30kN/m
B) FROM LIVE LOADS = 10kN/m

SOUTH ABUTMENT SECTION B–B
(NORTH ABUTMENT MIRRORED)



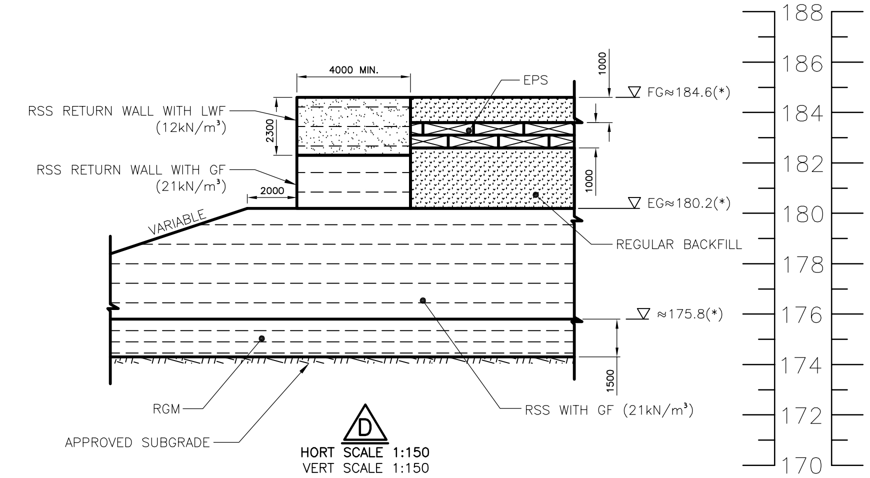
HORT SCALE 1:150
VERT SCALE 1:150

NOTE:

UNFACTORED LATERAL EARTH LOADS ON PILE CAP:

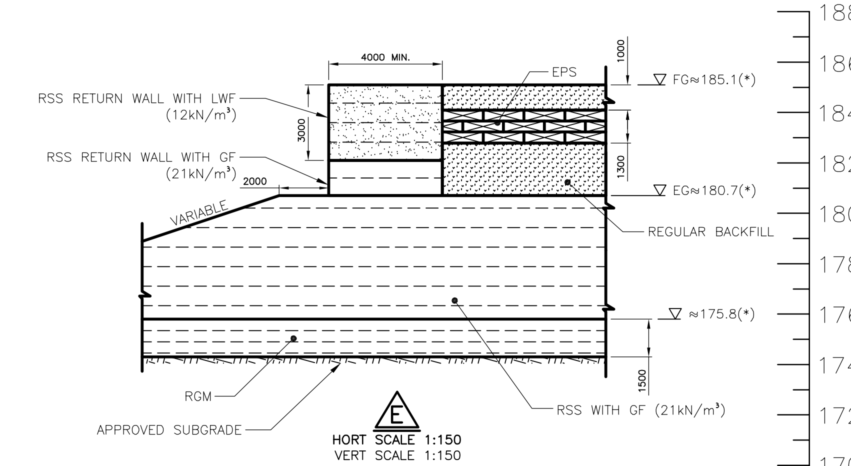
A) FROM DEAD LOADS = 35kN/m
B) FROM LIVE LOADS = 6kN/m

SOUTH ABUTMENT SECTION C–C



HORT SCALE 1:150
VERT SCALE 1:150

NORTH ABUTMENT SECTION D–D



HORT SCALE 1:150
VERT SCALE 1:150

SOUTH ABUTMENT SECTION E–E

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

TUNNEL T-8
HWY 401 GERAEDTS TUNNEL
PLAN AND SECTIONS OF ABUTMENT WALL

DWG. BY: SJL	CHK. BY: JF	FIGURE NO.: I.1
DATE: AUG-12	SHEET: 1 OF 1	

DOC: T-8 PLAN W ABMT SECTIONS_FIG I.1