The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report Tunnel T-6

(Sta. 10+080L to 10+200L)

Geocres No. 40J6-44

September / 2012

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

1.2 Report Introduction

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

This report presents the geotechnical design of Tunnel T-6 (Highway 401 Todd-Cabana Tunnel, between Stations 10+080L and 10+200L), located in the LaSalle sector of the Windsor-Essex Parkway (WEP) project. The proposed 120.3 m long, 2 span Tunnel T-6 structure will carry Parkland landscape and local traffic along Todd Lane / Cabana Road over Highway 401 between Sta. 10+100L and Sta. 10+130L. Highway 3 runs parallel to the Tunnel T-6 and is located north of tunnel. Trails are located to the north and south and run parallel to the tunnel. Secondary trails cross over the tunnel. As for all other tunnels at this project, Tunnel T-6 will be a cut-and-cover structure. The proposed structural solution incorporates structural deck on concrete girders supported on semi-integral abutments and centre pier on piles.

The report includes the results of the additional geotechnical investigation carried out to support the design and other relevant background information and addresses review comments from peer reviews and MTO on the 90% design report. 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 proposal in June 2010 (ref. R-40)[1] which was recognized as 30% design. The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines as well as the Parkway Infrastructure Constructors (PIC).

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.

[1] References are listed in Section 9.

The design complies with the requirements of the executable version of the Project Agreement Schedule 15-2 Part 2, Article 5.

2 Background Information

2.1 Geological Setting

The WEP project site is located within the Essex Clay Plain (a part of the St. Clair Clay Plain physiographic region) (ref. R-13, R-15 and R-24). 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-24) summarized the overburden geology in Windsor as consisting of the following strata: desiccated lacustrine clay, normally consolidated lacustrine clay, silty Tavistock till, glaciolacustrine clay and coarse Catfish Creek till. A distinct change in overburden deposits occurs in the east-west direction along a boundary located generally along the Huron-Church Road. Whereas, the eastern part of Windsor is underlain by firm to stiff glaciolacustrine silts and clays with upper deposits of stiff sandy to silty weathered clay and hard to stiff lacustrine clay-silt crust, the western part of Windsor is characterized by a thin surficial granular deposit underlain by thin crust layer underlain by soft to firm glaciolacustrine silts and clays.

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

The Windsor area, referred to as the Essex Domain (with respect to bedrock geology), is located in the Grenville Front Tectonic Zone (GFTZ) (ref. R-24). 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-7) 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-18), 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 (ref. R-18) 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-6 site is situated near the north extent of the LaSalle segment of the Parkway. The topography of the area immediately adjacent to Tunnel T-6 is generally flat with elevations ranging from approximately 180.5[2] to 181.6. Adjacent land use is typically residential to the north and commercial to the south.

The tunnel structure will be constructed under WEP Phase 1 development and will be used to carry Parkland and local traffic on Todd Lane and Cabana Road over Highway 401. Highway 3 in the vicinity of Tunnel T-6 is located on the north side of the proposed depressed Highway 401. Highway 401 at this location will be constructed within permanent cut. The finished grades along the tunnel walls will be raised up to approximately 2 m above the existing grades.

2.4 Frost Depth

In accordance with MTO–SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-35) and OPSD 3090.101, the frost depth below the ground surface in Windsor area is estimated at 1.0 m[3]. This estimate is considered applicable to 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-13 to R-20) 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-6 comprised a total of 3 boreholes, 2 Nilcon vane tests, 2 CPT, 1 DMT (flat blade dilatometer probe) and 1 test pit.

Table 3-1 lists the test holes put down at or in the close proximity to the tunnel site during both the previous and the current geotechnical investigations.

Table 3-1: Test Holes At and Around Tunnel T-6 Site

Reference	Boreholes	Nilcon Vane Tests	CPT	DMT	Test Pit
This Investigation (2011)	BH T6-1 / HGMW-7		CPT 36-RW	DMT T6-1	TP-7
	BH T6-2	NIL T6-2	CPT 37-RW		
	BH T6-3	NIL T6-3			
	BH 12-RW				
Previous Studies (2007-09)	BH-129	NIL-129	CPT-11		
	BH-129A		BH/CPT-324		

Drawing 285380-04-090-WIP1-2602 shows the locations of the test holes and an interpreted soil stratigraphic profile along the WEP centreline for the general area at and around Tunnel T-6 (i.e., from Sta. 14+700W to Sta. 10+400L). The test hole locations and stratigraphic sections at the tunnel location are illustrated on Drawings 285380-04-090-WIP1-2603 and 285380-04-091-WIP1-2604.

3.2 Fieldwork for Additional Investigation

The boreholes were advanced using track-mounted CME55 auger rigs owned and operated by Marathon Drilling Co. Ltd. under contract to AMICO and under technical supervision by AMEC engineers and technicians. Boreholes were generally advanced using 215 mm OD hollow stem augers, followed by wash boring with NW casing. The depth at which the drilling methods transition occurred is noted on the borehole logs.

Soil sampling was generally carried out using a 50 mm diameter split spoon sampler. Thin-walled Shelby tube (70 mm diameter x 600 mm long) samples were also recovered in the cohesive soil deposits below the upper crust layer. Soil sampling was carried out generally at 0.75 m depth interval in the top 7 to 8 m and at 1.5 m depth intervals thereafter. All samples were identified and placed in airtight containers and transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing[4]. Rock coring of the bedrock was carried out using 1.5 m long NQ or HQ 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 test listed in Table 3-1 was performed adjacent to the corresponding borehole. Table 3-2 summarizes the depths of overburden penetration and rock coring as well as the list of instruments and the accompanying Nilcon vane tests. The Nilcon vane tests listed in Table 3-2 were carried out typically adjacent the boreholes.

Borehole logs illustrating the interpreted soil conditions, field test results and laboratory index test results are included in Appendix A and B. Laboratory test results are presented on figures included in Appendix C.

Table 3-2: Overburden Thickness and Instrumentation in Boreholes

Borehole	Location	Overburden Thickness, m	Test Name & Elevation					
			Rock Coring	Nilcon Vane	S-Piez.	VWP	MSG	IN
BH T6-1 / HGMW-7	N 4,679,627 E 332,067	33.7	147.2 to 145.4		178.0 to 179.5	169.5 & 148.9	169.6 & 159.2	
BH T6-2	N 4,679,660 E 332,019	32.6	148.3 to 146.1	176.9 to 152.9		169.4 & 162.6		180.9 to 146.1
BH T6-3	N 4,679,578 E 332,079	34.7	146.9 to 145.3	176.6 to 154.6				
BH 12-RW	N 4,679,718 E 332,038	> 6.6 (BTEO)						
BH-129	N 4,679,625 E 332,110	29.3	147.2 to 143.8	176.1 to 162.1	148.0 to 149.5			
BH-129A	N 4,679,625 E 332,110	> 9.6 (BTEO)				171.8		

Legend: S-Piez. Standpipe Piezometer (Screen elevations)
VWP Vibrating Wire Piezometer (Sensor elevations)
MSG Spider Magnet Heave/Settlement Gauge
IN Inclinometer Casing
BTEO Borehole Terminated Early in Overburden

Note: Location coordinates and elevations are in UTM-NAD 83 (zone 14) and geodetic datum

Rock cores were examined in the field, and then transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing. The photographs of the rock cores are provided in Appendix H. 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. 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.

[4] Advanced lab tests (consolidation and direct shear tests) will be carried out in AMEC's Scarborough lab

[5] American Society for Testing and Materials

The CPT cone was pushed at a constant rate into the ground using hydraulic ram system of the drill rig (ASTM D5778). CPT 36-RW, CPT 37-RW and CPT-11 were advanced to refusal at around elevations 149, 154 and 155 respectively. CPT-324 was terminated earlier at around elevation 155. A pore pressure dissipation test was carried out in CPT 37-RW at 19.5 m below ground surface.

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 vane tests were conducted in accordance with ASTM D2573-01.

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

Test Pit TP-7 was located near the centre of the proposed Tunnel T-6 as shown in drawing 285380-04-090-WIP1-2602. The test pit was advanced using a J Deere 470G LC excavator and extended approximately 10 m below ground surface. A test pit log illustrating the interpreted soil conditions is included in Appendix A and laboratory test results are presented on figures included in Appendix C.

The locations of the test holes executed during the previous pre-bid and additional investigations, as also the inferred soil profile along the WEP alignment, are shown on Drawing 285380-04-090-WIP1-2603. Borehole, DMT, Nilcon, CPT and test pit logs from the additional 2011 investigation are included in Appendix A. Relevant borehole logs from the previous investigation are included in Appendix B.

3.3 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 one-dimensional consolidation test and 1 direct shear test).

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

The results of geotechnical and 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 and test pit logs.

3.4 Instrumentation

Geotechnical instruments (Stand pipe piezometer – S-Piez, Vibrating wire piezometers – VWP, Spider magnets heave/settlement gauges – MHSG and inclinometer casings – IN) were installed at selected locations on completion of boreholes to monitor pore water pressure and deformation behaviour of the soil strata during and after construction. A brief description follows.

Standpipe Piezometers: These piezometers comprise 1.5 m long 10 mil slotted intake screen located at selected depths and extended to the ground surface using 52 mm diameter, flush-joint, threaded, schedule 40 PVC riser pipe. A silica sand filter pack was placed between the intake screen and the wall of the borehole and extended approximately 0.3 m above the top of the well screen. Bentonite-cement grout was used to fill the annular space in the holes 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 at selected depths and electrical wires extended to the monitoring station located at the ground surface (outside the parkway footprint area). The borehole was filled with a bentonite-cement mixture designed to match, as near as practical, the permeability and strength-deformation characteristics of the native soils. Sensor elevation and details of installations are provided in Table 3-2 and applicable borehole logs.

Magnetic Settlement/Heave Gauges: Spider magnets (RST, Model SSMM100 mechanical release spider target for 25 mm pipe) were installed in Borehole T6-1/HRMW-7 at selected locations and depths to permit future measurement of heave and settlement. Each magnetic torus was placed around a 25 mm diameter pipe, which was extended to above the ground surface. The spider legs 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.

Inclinometers: An inclinometer casing was installed in Borehole T6-2. 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 2.2 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).

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 arc failure analyses of embankment failures suggest correction by multiplying the field vane data by 1.05 to 1.10 for soils with plasticity index of about 15 (ref. R-4 and R-28). However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), the Canadian Foundations Manual suggests that the vane test data for clays with PI<20 should not be corrected (ref. R-1and R-40, Figure 3-2). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI.

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

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

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

Where:

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

Q_t is the corrected total cone tip resistance;

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

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

The CPT based S_u profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles. In this regard, the N_{kt} factor values used to calibrate the CPT strength profiles varied slightly for different segments of the WEP and the soil strata. Thus, an N_{kt} factor of 14 was used to estimate the undrained shear strength of the clay crust and transition layers. The N_{kt} factors used for the underlying grey silty clay to clayey silt stratum and the lower clayey silt stratum were 16 and 12, respectively.

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-28). The following relationship was used to compute the pre-consolidation pressures:

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

Where:

S_u is the undrained shear strength;

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

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

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

OCR is the overconsolidation ratio; and

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

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

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

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-25), 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 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, CPT and DMT carried out in the vicinity of Tunnel T-6 are presented in Figure 3-3. Interestingly, the undrained shear strength (Su) profiles inferred from the DMT show a more uniform variation with depth than the Nilcon vane test results with values near the average values from Nilcon tets. Also included on these figures are $0.18 \times \sigma_{vo}'$ curve (representing undrained strength profile for OCR=1 condition) and simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units. The constant 0.18 for S_u/σ_{vo}' for OCR=1 curve is based on average plasticity index of the silty clay to clayey silt stratum and Chandler 1988 relationship (Figure 3-1) (ref. R-9).

4 Subsurface Conditions

The general soil stratigraphy at the borehole locations consists of the following successive strata: surficial layers of occasional fills, topsoil and upper granular deposit; an extensive cohesive clayey silt to silty clay deposit below about elevation 181, and a lower granular deposit below about elevation 150, overlying limestone and dolostone bedrock below about elevation 147. The thickness of the Clayey Silt to Silty Clay deposit varies between about 28.7 m and 30.3 m. The lower granular deposit (sandy silt / silty sand / sand and gravel) varied in thickness between 2.0 to 3.6 m. The bedrock was encountered at depths ranging from about 32.6 m to 33.7 m below the ground surface.

4.1 Surficial Fills, Topsoil and Upper Granular Deposit

Surficial fills were encountered in all except Boreholes BH T6-2, BH/CPT-324, CPT 36-RW. The thickness of the fill layer ranged from 0.2 to 3.0 m. All boreholes, except Boreholes BH-129, T6-1/HG-MW-7, T6-3 and CPT 12-RW 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. A layer of granular material was encountered below the topsoil in Boreholes BH T6-3 and CPT 36-RW. The thickness of the granular layer ranged from 0.2 to 0.7 m.

4.2 Silty Clay to Clayey Silt Stratum

The cohesive silty clay to clayey silt stratum was encountered directly underlying the surficial topsoil or fill/granular deposit. The encountered depth below existing ground surface was from 0.2 to 3.0 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 clayey silt to silty clay deposit (referred to hereafter as upper and lower silty clay), and then a lower grey clayey silt deposit (referred to as upper and 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 Silty Clay to Clayey Silt Stratum

Property	Clay Crust	Transition	Upper Silty Clay	Lower Silty Clay	Upper Clayey Silt	Lower Clayey Silt
Average Elevation Range, m	181 to 177	177 to 175	175 to 166	166 to 163	163 to 160	160 to 150
Natural Water Content, w_N, %	10 to 33	11 to 28	15 to 38	15 to 30	15 to 20	13 to 35
Liquid Limit, w_L	32 to 39	31 to 35	27 to 40	25 to 35	23 to 27	28 to 41
Plastic Limit, w_P	19 to 20	16 to 18	15 to 17	12 to 19	14 to 15	13 to 21
Plasticity Index, PI	12 to 20	15 to 17	10 to 23	13 to 18	9 to 13	11 to 20
Liquidity Index, LI	0.05 to 0.17	0.06 to 0.09	0.19 to 0.95	0.08 to 0.98	0.09 to 0.47	0.09 to 0.62
Unit Weight, γ, kN/m^3	-	-	18.6 to 20.3	21.4	21.4 to 21.8	20.8 to 21.1

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

- Clay Crust layer: > 80±20 kPa
- Clay Transition layer: 80±20 kPa to 60±10 kPa
- Upper silty clay: 60±10 kPa to 45±10 kPa to 50±10
- Lower clayey silt: 50±10 kPa to 65±20 kPa to > 65 kPa.

The stress-strain properties and the effective shear strength properties of the silty clay to clayey silt soils were based on published correlations (Kulhawy and Mayne, 1990, ref. R-27, Leroueil et al, 2001, ref. R-32 and Terzaghi et al.1990, ref. R-39) as well as on the tests reported in Golder's Subsurface Condition Interpretation Report (ref. R-16) and the tests performed during the additional geotechnical investigation carried out as part of the detailed design development for the entire WEP length.

The stress-strain relationships are correlated to natural water content (w_N, expressed as percent) as illustrated in Figure 4-1 and Figure 4-2 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-6 site are summarized in Table 4-2.

Table 4-2: Summary of Interpreted Compressibility Properties

Property	Clay Crust	Transition	Upper Silty Clay	Lower Silty Clay	Upper Clayey Silt	Lower Clayey Silt
Average Natural Water Content, w_N, %	19	23	25	20	17	22
Virgin Compression Index, C_c	0.16	0.19	0.20	0.16	0.14	0.18
Recompression Index, C_r	0.017	0.020	0.022	0.018	0.015	0.020
Swelling Index, C_s	0.039	0.046	0.051	0.041	0.034	0.045
Secondary Compression Index, C_α	0.0044	0.0052	0.0057	0.0046	0.0038	0.0051

Oedometer testing carried out on a sample in the upper grey silty clay from Borehole BH T6-1 TW12 (12.5 m depth) indicated the following compression indices: $C_c = 0.151$, $C_r = 0.017$, and $C_s = 0.044$.

The effective shear strength properties applicable to the silty clay to clayey silt stratum were determined from triaxial and direct shear tests performed during the pre-bid geotechnical investigation (Figure 4-3), the available results of the triaxial and direct shear tests carried out along the WEP, and supported also by published PI versus ø' relationships (ref. R-26, R-31 and R-39, Figure 4-4), and are summarized as follows:

Apparent cohesion, c' 0 kPa

Angle of internal friction, Φ 30°

Friction angle at critical state, Φ_c 25-26° (*)

(*) Based on Triaxial tests (ref. R-16)

Direct shear testing carried out on a sample in the transition between the upper and lower grey silty clay from Borehole BH T6-3 TW15 (17.1 m depth) indicated a residual internal friction angle of 30°.

The modulus of elasticity has been correlated with the undrained shear strength of the material, published information (ref.R-37) and local experience (ref.R-16). For the unweathered portion of the silty clay stratum the empirical relationship were used based on average shear strength profiles for the material, as shown in Table 4-3.

$E_u = 300\ S_u$

$E' = 0.9E_u$

Table 4-3: 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	20		18	
Upper Silty Clay	16		14	
Lower Silty Clay	15		14	
Upper Clayey Silt	17		15*	
Lower Clayey Silt	20		18*	

Note: (*) A higher drained elastic modulus of 23 MPa was used for the clayey silt in the stress deformation analysis based on the results determined using the Modified Cam Clay model.
(**) Assumed values

4.3 Lower Granular Deposit

Underlying the silty clay to clayey silt stratum and overlying the bedrock, a heterogeneous non-cohesive material deposit (varying from cobbles and boulders, sand and gravel, to sandy silts) was encountered. Based on SPT N-values ranging generally from 20 to greater than 100, this material is considered to be in a compact to very dense state of compactness. This layer was up to 3.6 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 146.9 to 148.3 in the vicinity of Tunnel T-6. The Rock Quality Designation (RQD) of the recovered rock cores varied on average between 33 to 100 per cent, indicating a poor 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 latter system. With the exception of Borehole BH-314, rock quality generally improved with depth. Photographs of rock cores recovered from the additional investigation are provided in Appendix E.

It was found during the preliminary investigations reported in Golder's Subsurface Condition Interpretation Report (ref. R-16) 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 tested for density and unit weight, while 16 were tested 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). 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	24.5	85.5
Standard Deviation	96	0.9	25.4
Minimum Value	2340	23.0	35.5
Maximum Value	2660	26.1	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 to measure the water levels within overburden and bedrock, respectively (Table 3-2).

As indicated on Table 4-5, the piezometric levels within the silty clay overburden and the granular deposit overlying the bedrock varied generally from 179.0 to 180.8 and 178.8 to 179.9, respectively. An exception to this trend was observed in the shallow piezometer of Borehole BH T6-1/HGMW-07 installed at elevation 169.5 within the upper silt clay overburden. Piezometric heads recorded at piezometer ranged from elevation 182.2 to 181.1 which is above the existing ground surface. Apart from the exception noted above, the highest piezometric levels within the overburden and the bedrock were recorded at elevations 180.8 and 179.0, respectively. These observations suggest a slight downward gradient between the overburden and the bedrock. However, given the experience in the Windsor area, occurrence of localised artesian conditions in bedrock cannot be ruled out.

Table 4-5: Summary of Measured Water Levels

Borehole	Surface El., m	Piezo. Type	Screen / Sensor El., m	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	El., m
BH T6-1/HG-MW-07	180.9	VWP	169.5	Silty Clay	2011-07-23	182.2
		VWP	148.9	Lower Granular	2011-07-23	178.8
		S-Piez	178 to 179.5	Fill	2011-08-29	180.5
BH T6-2	180.8	VWP	169.4	Silty Clay	2011-07-23	180.8
		VWP	162.6	Silty Clay	2011-07-23	180.6
BH-129	180.8	S-Piez	148 to 149.5	Lower Granular	2011-07-10	179.0
BH-129A	180.8	S-Piez	171.5 to 172.1	Silty Clay	2008-07-22	179.0

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

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

4.6 Subsurface Gases

The groundwater in the project area, especially within the lower granular deposit and bedrock, is known to contain dissolved hydrogen sulphide (H_2S) and methane (CH_4) gases that are liberated from the water on exposure to atmospheric pressure. The H_2S gas can frequently be detected by odour at concentrations on the order of 0.5 mg/L and can be corrosive at concentrations of about 2 mg/L to 3 mg/L in the groundwater. The gas odour was not detected during the drilling at the Tunnel T-6 site.

Although the presence of the H_2S and CH_4 gases was not observed during the 2011 geotechnical investigation at Tunnel T-6 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. Of these tests, TOW-2, located north of Tunnel T-6, indicated a concentration of 20.0 mg/L of H_2S gas and at TOW-3, located south of Tunnel T-6, indicated a concentration of 7.0 mg/L of H_2S gas . As Tunnel T-6 is located between TOW-2 and TOW-3, H_2S gas may be present in this area. A summary of the results of these pumping tests is provided in Table 4-6.

Table 4-6: 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

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-21 and R-38). It is, therefore, recommended that the design and construction methodologies should be developed in consideration of the potential presence of these gases (ref. R-12).

Air quality and subgrade pore pressure monitoring should be carried out during construction. The equipment operating in confined spaces should be selected to safely operate in a potentially gaseous environment. Excavation layers should be decided in consideration of the pore pressure monitoring data and the potential ground softening.

5 Development of Geotechnical Design

5.1 Tunnel Configuration

Tunnel T-6 (Todd-Cabana) will be constructed along the below-grade section of the WEP between Sta. 10+080L and Sta. 10+200L, and will accommodate the below-grade traffic of Highway 401 (Drawing 285380-03-060-WIP1-2601). Cabana Road meets Highway 3 at grade north of the tunnel alignment. Apart from the local road ways over the tunnel, it has been assumed that a 1 m soil cover will be placed over the tunnel. The proposed Tunnel T-6 is 120.3 m long and its width is 56.1 m.

As shown on Drawing 285380-03-060-WIP1-2601, Tunnel T-6 is a two-span deck-on-girder structure incorporating semi-integral abutments and centre piers. For geotechnical analyses purposes the deck elevations were estimated for each design section location using the elevations of work points WP#1, WP #2 and #3 and the grades shown on Drawing 285380-03-060-WIP1-2601 which was up to date as of July 27th, 2012. The abutments consist of 1.70m wide × 1.5 high pile cap founded on deep end-bearing HP 310×110 steel piles (Drawing 285380-03-061-WIP1-2605) to bedrock. The center piers include 3.2 m wide x 1.5 m high pile caps supported on vertical and batter piles (1H:6V) to bedrock as shown on Drawing 285380-03-061-WIP1-2605.

Geotechnical design incorporating false abutments using Reinforced Soil System (RSS) wall with zones of approved regular backfill, ultra-lightweight fill (LWF), and EPS have been developed as illustrated in Appendix I. The false abutments will be constructed using RSS walls founded on reinforced Granular Matt (RGM), which in turn will be founded over undisturbed native silty clay subgrade. Table 5-1 provides a summary of control elevations at the tunnel abutments used for the geotechnical design development.

Table 5-1: Summary of Interpreted Elevations at Abutments

Location	Existing Ground Surface*	Top of Finished Grade El.**, m	Top of Deck El., m	Top of Pile Cap El., m	Hwy 401 Pavement Subgrade El.*, m
North Wall - Centerline Tunnel Sta. 10+140 (WP#1)	180.8	182.3	181.3 (WP#1)	178.9	172.4
Centerline Tunnel & Hwy # 401 Sta. 10+140 (WP#2)	180.8	182.8	181.8 (WP #2)	172.65[(a)]	173.0
South Wall - Centerline Tunnel Sta. 10+140 (WP#3)	181.0	183.0	182.0 (WP #3)	179.6	173.6
North Wall –10+080	181.1	182.0	181.0	178.6	172.1
North Wall – 10+125	181.0	181.8	181.2	178.8	172.3
North Wall – 10+170	180.9	182.4	181.4	179.0	172.5
South Wall – 10+080	181.0	182.8	181.8	179.4	173.4
South Wall – 10+100	180.8	182.5	181.9	179.4	173.4
South Wall – 10+150	181.1	182.4	182.1	179.7	173.6

(a) Approximate elevation of Top of Pile Cap at the central Pier.

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, including the revision dated January 23, 2012. 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-6 and CHBDC, R-7).

Working Stress Design (WS Method) was employed for global stability of the earthworks, soil mass containing earth retaining structures and the external stability (bearing, sliding and overturning) of the RSS structures. The stability of the soil mass containing the false abutments and wing-wall is checked for all potential surfaces of sliding and has a minimum factor of safety of 1.3.

Tunnel T-6 construction is expected to involve the following sequence of earthwork, design elements and loading stages:

- Temporary excavations to about 6.8 to 9.0 m depths below existing grade;
- Installation of a 1.5 m thick Reinforced Granular Mats (RGM) foundation at the north and south abutments including base drain. Void forms may be considered within the RGM to accommodate later pile installation through RGM:;
- Temporary trenches at the pier location;
- Installation of piles (HP310×110) for all tunnel supports;
- Completion of the pier footings;
- Installation of 500 mm diameter Corrugated Steel Pipe (CSP) around the abutment pile stickup;
- Construction of the RSS structures and associated permanent subdrainage works, and approved backfill behind the RSS structure;
- Concrete fill placement within CSP;
- Construction of the pile caps, abutment stems, piers and tunnel deck;
- Completion of the toe berm in front of the RSS wall;
- Completion of final stage of backfill behind the semi-integral abutments;
- Completion of the final topsoil placement and trail materials; and
- Completion of the pavement structure over the Highway 401, Highway 3 and Todd-Cabana Road.

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 inferred from the CPT, DMT and Nilcon tests advanced around Tunnel T-6 and the design values obtained from these profiles are shown in Figure 3-3. Selected design values obtained from the profiles are summarized in Table 5-2.

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

Clay Substratum	Elevation, m	Undrained Shear Strength, S_u, kPa	Effective Stress Parameters	Pre-consolidation Pressure, σ_p', kPa	Over Consolidation Ratio
Clay Crust	>177	75 (*)	Cohesion, c' = 0 Friction Angle, ǿ = 30°	550	> 9
Transition	177 to 175	75 to 60		550 to 350	7
Upper Silty Clay	175 to 166	60 to 45		350 to 230	2.8
Lower Silty Clay	166 to 163	45 to 50		230 to 260	1.3
Upper Clayey Silt	163 to 160	50 to 65		260 to 400	1.5
Lower Clayey Silt	160 to 150	65		400	1.4

(*) Applicable for global stability verifications

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 hydraulic conductivity in horizontal direction (k_h) and the hydraulic conductivity anisotropy ratio (A = k_h/k_v) and in-situ void ratios required for the analysis of stress-deformation and seepage analyses are provided in Table 5-3. These 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, k_h cm/sec	Anisotropy ratio, k_h/k_v (*)	Initial Void Ratio, e_0
Clay Crust	6.8×10^{-7}	1	0.54
Transition	3.9×10^{-7}	2	0.63
Upper Silty Clay	1.1×10^{-7}		0.69
Lower Silty Clay			0.56
Upper Clayey Silt			0.47
Lower Clayey Silt			0.62
Lower Granular	1.1×10^{-6}	1	0.32

(*) Assumed

For design purposes the initial groundwater level in the overburden was considered at elevation 181.

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

Excavations are expected to encounter surficial fills, topsoil and water bearing granular soils and will be extended 8.1 to 9.0, 6.8 to 7.9 and approximately 10.5 m below existing grade (elevation 180.8 to 181.8) to about elevation 172.1 to 172.9, 173.3 to 174.5 and 170.5 m into the native firm silty clay for the north abutment, south abutment walls and pier, respectively.

Basal stability against hydrostatic uplift was verified considering the highest recorded piezometer level in the lower granular and bedrock (179) and the weight of the silty clay layer between the invert of excavation and the top of the lower granular and bedrock unit (elevation 151.5).

Basal hydrostatic uplift at the abutments was calculated based on the anticipated deepest abutment excavation depth (RGM base at elevation 172.1), and a silt-clay layer thickness of 20.6 m (Borehole BH-129) below the deepest excavation. The factor of safety (FS) against hydrostatic uplift was 1.6.

The approximate elevation of bottom of pile cap at the central pier is elevation 171.4. Accordingly, the factor of safety against hydrostatic uplift during temporary excavation for construction of central piers was 1.5.

As described in Section 4.6, presence of 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 (e.g., vibrating wire piezometers) should be installed prior to initiation of the major excavations. If warranted by the monitoring of the excavation progress performance, the excavation rates will have to be adjusted to allow sufficient time to dissipate the pore pressures to safe levels. The excavation guidelines can be revised based on on-site experience.

5.5 Pile Foundations

5.5.1 Resistance to Axial Loads

It is understood that HP310x110 steel H piles will be used at this project. The pile driving equipment and installation procedure should be established in the field 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 a Ultimate Limit States (ULS) axial geotechnical resistance in excess of 4000 kN. A factored ULS resistance of at least 2000 kN is anticipated.

The Servicability 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, piles may stop in very dense till (cobbles and boulders layer as in borehole BH T6-1) and the SLS resistance can decrease to not less than 1800 kN for estimated pile head settlement of 25 mm.

Based on the available borehole data at this structure, the bedrock surface elevation varies between 146.9 and 148.3, 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 lower granular deposit lying immediately above the bedrock, and/or a perceived risk of damaging the piles by overdriving is apparent, consideration should be given to supplementing the field testing to prove the actual mobilized resistance. If lower mobilized pile resistances are proven, options based on the most economical approaches may be considered (e.g., changes to the driving method and equipment, or addition of more piles).

The actual mobilized resistance of the production piles should be confirmed by dynamic testing using 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 at the pier locations may be based on a conventional SLS resistance of 70 kN along the strong axis and 50 kN along the weak axis of the HP310x110. This conventional SLS resistance represents the lateral shear force applied on a free-head pile that causes a lateral deflection of 10 mm measured at the ground surface. In the case of pile shaft encapsulated within the 500 mm diameter concrete filled CSP within the RSS abutment, the conventional SLS resistance for a free-head pile increases to about 110 kN and 50 kN for the strong direction and weak direction, respectively.

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 265 kN and 100 kN along the strong axis and weak axis, respectively. 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.

The above estimates were based on a pile model assumed to be embedded within firm to stiff silty clay below elevation 173.7. The above resistances were estimated using the "p-y" model (LPile 5.0 model Ensoft 2010). The "p-y" curves were generated using the Reese method described in the technical manual for LPILE, using the Reese "Stiff-Clay without free water" model in conjunction with the following soil parameters defined in Table 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 177	22	75	0.007
Native Transition Clay	177 to 175	21	Decreases linearly with depth from 75 to 60	0.007
Upper Silty Clay - 1	175 to 166	20	Decreases linearly with depth from 60 to 45	0.007 to 0.010
Upper Silty Clay – 2	166 to 163	21	Increases linearly with depths from 45 to 50	0.010 to 0.007
Native Lower Clayey Silt - 1	163 to 160	22	Increases linearly with depth from 50 to 65	0.007
Lower Clayey Silt - 2	160 to 150	21	65	0.007

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

Table 5-5: Fill Properties for Pile Interaction Assessment

Material	Soil Model in L-Pile	Design Bulk Unit Weight, kN/m^3	$\acute{\phi}°$	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).

Significant lateral loads in excess of the preliminary values previously cited should be resisted fully or partially by the use of battered piles. In this regard, batter piles are considered to be more effective in resisting horizontal loads, as a part of lateral load is converted into axial load and consequently the induced bending moments are less. For ease of constructability and to provide hammer energy sufficient for pile driving, batters are usually limited to 1H:5V.

The stress-deformation analysis of the piles to lateral loads may be carried out using the following approaches:

Horizontal Subgrade Reaction Method:

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

k_h	$= n_h\left(\frac{z}{d}\right)$	for cohesionless soils, and
	$= 67\left(\frac{Su}{d}\right)$	for cohesive soils.

Where:

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

The recommended ranges of soil parameters are tabulated Tables 5-4 and 5-5.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action may be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor indicated in Table 5.6. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed in this table.

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 may be considered using the nonlinear "p-y" interaction method (discussed in the next section) or elastic continuum theory as discussed in the Canadian Foundation Engineering Manual (ref. R-6).

The p-y curve represents the total lateral soil reaction pressure 'p' (kPa) to the pile lateral deflection 'y' (m) relative to the surrounding soil mass at a particular section of the pile shaft in contact with the surrounding soils. 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 the 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 $\boldsymbol{B_m}$ = 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:

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

where :

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

Table 5-7: 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	$0.64(s/d)^{0.34} \le 1$
Leading pile in Line (first pile in line parallel to the load direction)	≤ 4	$0.70(s/d)^{0.26} \le 1$
Trailing piles in line (piles behind the leading pile)	≤ 7	$0.48(s/d)^{0.38} \le 1$

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) estimated to occur during and following excavation of the overburden of up to 8 m to accommodate the future depressed highways, followed by backfilling to construct the tunnel abutments.

Soil stress-deformation analyses described later in Section 5.8.2 were conducted using the SIGMA/W software. The estimated ground vertical movements (settlement/heave) and vertical effective stresses after excavation in the vicinity of the pile shaft are represented in Figures G.10 and G.12, respectively, at the following representative stages:

- After RSS completion, including the backfill behind the RSS wall but before placement of general backfill above RSS wall and placements of Highway 3 and Highway 401 pavements (Short-term Condition during Construction);
- After completion of the top backfill against the tunnel diaphragm, Highway 3 fills and with Highway 401 pavement in place (End of Construction - EC) ; and
- Long-term (LT) when excess pore water pressures generated by construction activity have dissipated.

The analyses indicated the following:

- Ground settlements are expected to occur along the pile shaft during the construction of the RSS wall, Tunnel T-6 and the associated backfill and continue for approximately one year after completion of construction.

- Ground rebound is expected to occur along Highway 401 and at abutments shortly (approximately within 1 to 6 months) after substantial completion of the ground surface loadings.

Considering the construction staging, the anticipated settlement-rebound of the soils and the transient nature of the downdrag at the site, , as well as the presence of low compressibility dense silty sand deposit below elevation 152, the recommended dead load and downdrag load combinations are as follows:

a) Maximum transient downdrag of 600 kN plus structural dead load only (pile cap and tunnel deck) occurring during completion of the backfilling against the tunnel diaphragm.

b) Residual (long-term) downdrag of 400 kN plus total design dead loads (structural and topsoil/landscape materials over tunnel roof) after the completion of construction.

In all cases the reported maximum downdrag forces in the pile occur near the pile tip and decreases along the pile shaft with increasing elevation above the pile tip.

The recommendations herein assume that the placement of the soil fill over the tunnel roof occurs after substantial completion of the final grading along the tunnel sides. A pile load of 1300 kN was used in these analyses.

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

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

No downdrag is anticipated at the pier location.

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 filled with concrete around the pile shaft) within the RSS wall and the RGM. Below the RGM, the pile section was HP section.
- The pile head was assumed to be a free head.
- The ground lateral movement (Figure G.11) 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 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 maximum unfactored bending moment in the shaft was estimated at 55 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 40 kN. The calculated maximum pile deflection at the underside of the RSS wall base was 5 mm.

These results should be considered in the structural design of the piles and in the design of RSS structural components. These bending moments, shear forces and deflections are in addition to those caused by bridge loads applied to the piles.

Assuming a simplified and conservative load combination including a shear force at the pile head equal to the SLS resistance (110 kN) along with the anticipated ground lateral displacement discussed above, and ignoring the restraint by the pile cap straps, the calculated maximum bending moment in the pile shaft is145 kN-m which is significantly less than the yield moment of the pile section. The location of section of this maximum bending is at about 2 m below the invert of the pile cap.

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

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 ground 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. Based on these analyses, the maximum bending moment caused by ground movement on batter piles was calculated to be 15 and 11 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) is recommended. 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:

$p_h = K_r K_a \sigma_v + \Delta\sigma_H$ (FHWA-NHI-10-024, ref. R-36)

where:

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

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

Regular Backfill:	21 kN/m^3
LWF:	12 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.

All the property values discussed above are unfactored.

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

ELL = 10 kN/m	(earth pressure from Live Loads (LL=9 kPa and 16 kPa) on trails and roadways, respectively)
EDS = 30 to 45 kN/m	(earth pressure from Dead Surcharge load above the pile cap)
EB = 10 kN/m	(earth pressure due to regular 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

Geotechnical design configurations (typical arrangements) for Tunnel T-6 North and South abutments have been developed based on the global stability and foundation bearing considerations. The proposed configurations are shown in Appendix I (Figure I.1) and are summarized in Section 5.6.6.

The abutments comprise RSS walls founded on an RGM foundation, and approved clay and granular backfill as well as EPS and LWF. The configurations and preliminary dimensions were developed at representative sections along the tunnels to verify the geotechnical design requirements with respect to (a) the global stability of the soil mass containing the structure (b) the anticipated deformations, and (c) the external stability based on the principles of Working Stress Design.

The design assessments were based on (a) assumed strength and deformation properties of the proprietary components (RSS, RGM, LWF and EPS), which will have to be provided by proprietary suppliers, and (b) the assumed external loads and backfill properties. The final design of the abutment may require adjustments based on the proprietary components and structural design. 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 assumed in the geotechnical analyses are summarized in Table 5-8.

Table 5-8: Assumed Proprietary Product Properties

Material	Unit weight, kN/m^3	Limit Equilibrium Analyses			Stress Deformation Analyses	
		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
RSS with Lightweight Fill	12	50	35	50	40	0.35
RGM	21	40	35	40	60	0.35
EPS	0.5	15	0	15	10	0.20

The properties assumed for the backfill materials are given in Table 5-9.

Table 5-9: 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	20	0.35
Granular Backfill	21	N/A	30	22.5	0.35

Preliminary dimensions of the abutments are listed in Table 5-13.
* φ'= 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-18 illustrate the stability models for the north and south abutments. The global stability analyses have been carried out for both short-term (undrained soil properties) and long-term (drained soil properties) loading conditions. The analysis using undrained soil properties was carried out with and without the pavement structure over the subgrade at the toe of the slope (referred to as "Short-Term during Construction" and "Short-term at End of Construction", respectively). The drained analyses assumed that all the components of the system are present. The presence of the piles was not considered in the stability models (somewhat conservative approach). Surcharges of 9 and 12 kPa for short-term and long-term model were applied at the top of ground surface in the vicinity of trails and Highway 3, respectively, while tension crack was assumed for short-term only.

As indicated earlier, the abutment configurations were determined in consideration of the global stability and geotechnical bearing of the false abutments using the applicable soil characteristics and the design undrained strength profiles. The calculated factors of safety (FS) generally exceed 1.3 for short-term condition and 1.5 for long-term condition against global instability of the abutments, as shown in Figures F-1 to F-18 and summarized in Table 5-10.

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

Abutment	Factor of Safety for Loading Condition			Figure
	Short-Term during Construction (1)	Short-term at End of Construction (2)	Long-term (Drained) (3)	
North Wall – 10+080	1.5 (1.3)	1.6 (1.4)	1.5 (1.5)	F-1 to F-3
North Wall – 10+125	1.4 (1.3)	1.5 (1.3)	1.5 (1.5)	F-4 to F-6
North Wall – 10+170	1.4 (1.3)	1.6 (1.3)	1.5 (1.5)	F-7 to F-9
South Wall – 10+080	1.5 (1.3)	1.7 (1.4)	1.6 (1.5)	F-10 to F-12
South Wall – 10+100	1.5 (1.3)	1.6 (1.3)	1.5 (1.5)	F-13 to F-15
South Wall – 10+150	1.5 (1.3)	1.7 (1.4)	1.5 (1.5)	F-16 to F-18

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

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 calculation model is presented in Figure G-1 considering the most loaded abutment due to the local roadway embankment. 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;

b) Bulk excavation to the subgrade level under the highway pavement;

c) Construction of the RGM and RSS structures, and the associated backfill;

d) Completion of the remaining fill above the RSS structure and Highway 3 pavement;

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

f) Dissipation of excess pore pressure leading to long-term steady state condition.

The stratigraphy and selection of the soil properties (except for the RSS structure and pavement box) was based on the design soil properties discussed in Sections 4 and 5.3. The RSS structure, RGM and pavement were assumed to comprise homogeneous elastic materials described in Table 5-9.

The SDA were carried out using an effective stress-based model. The phreatic surface was assumed to correspond to the initial groundwater level at approximately 181.0 (existing ground surface) and then follow the excavation and subgrade surfaces. Elastic-plastic Mohr-Coulomb models were used for all soil layers except the unweathered soft to firm silty clay, which was described by the Modified Cam-Clay model. Hydraulic conductivity properties described in Table 5-3 were assigned to the different soil layers.

The construction stages were represented by excavation, construction of the RGM and RSS and completion of the entire abutment and Highway 3 pavement followed by the placement of the pavement box. A period of 60 days was assumed for each construction stages of bulk excavation, completion of RGM+RSS and completion of entire abutment and Highway 3 pavement. Completion of Highway 401 pavement structure was assumed to occur rapidly (1 day stage).

After the completion of the entire construction (stage (b) to (e)), the model is allowed to dissipate the excess pore-pressures over a period of time until a steady-state pore pressure condition is achieved.

The SIGMA model was developed for the north abutment (Sta 10+200) where the height of the retained soils measured from the top of finished grade to the bottom of the RSS is 8.2 m high and the Highway 3 is in proximity to the RSS wall. The north abutment (Sta 10+200) model will provide the upper limits for the deformation estimates.

Figures G-1 and G-2 show the calculated vertical cumulative settlement/heave and lateral movement at the end of excavation, respectively[7]. Figures G-3 and G-4 show the calculated cumulative settlement/heave for the end of construction ("181 days") largely undrained conditions and the long-term drained loading conditions ("7,481 days"), respectively. 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 5-11 and Figures G-1 through G-12. The ground movements generated by the construction loads are anticipated to stabilize within approximately 4 to 9 years following completion of construction. Due to the relatively smooth changes in the geometry of the tunnel along Highway 401, the above settlement changes along Highway 401 are anticipated to be gradual in longitudinal profile.

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

[7] "Cumulative" deformations presented in this section represent the relative deformation with respect to the original (insitu) ground condition.

Table 5-11: Summary of Calculated Cumulative Deformations

Parameter	End of Excavation (60 day)	End of RSS Construction (120 days)	End of Construction (Undrained) (181 days)	Long-term (Drained) (7,481 days)	Figure(s)
Settlements on Top of Ground at Distances (m) from the Edge of Deck(†)					G-1 to G-6
0 m†	N/A	N/A	-15/-20 mm(*)	-15/-20 mm	
South Edge of Highway 3	N/A	N/A	-20/-25 mm(*)	-15/-20 mm	
Center of Highway 3	-35 mm	-40 mm	-40/-45 mm(*)	-40/-45 mm	
North Edge of Highway 3	-30 mm	-25 mm	-15/-20 mm (*)	-15/-20 mm	
50 m	-25 mm	-20 mm	- 10/-15 mm (*)	-10/-15 mm	
70 m	-20 mm	-15 mm	- 10/-15 mm (*)	- 5/-10 mm	
Settlement at the top of RSS facing	N/A	-50 mm (*)	-60 mm	-60 mm	G-7
Lateral displacement at the base of RSS facing (mm)	N/A	< 5 mm	6 mm	< 5 mm	G-9
Rotation of the RSS facing	N/A	< 0.002	0.002	< 0.002	
Maximum Heave (rebound) at Highway 401 (subgrade level)	60 mm	75 mm	70 mm (*)	95 mm	G-10

N/A Not Applicable – Area located within the temporary excavation.
(-) ve denotes settlements
(†) Distances measured perpendicular to the tunnel abutment.
(*) Indicates calculated settlement at top of wall / abutment backfill / top of pavement to be compensated during constructions
Note: The abutment design and soil properties assumed represent the north abutment configuration.

Figure G-6 shows cumulative ground surface settlements across Highway 3. Figure G-7 shows the cumulative settlement at the top of the RSS wall facing and Figure G-8 shows the lateral displacement along the RSS wall facing. Figure G-9 shows the cumulative settlement and heave along Highway 401. Figures G-10, G-11 and G-12 show soil settlement, lateral soil displacements and vertical effective stress along the pile line determined from SDA, respectively, which were used in pile calculations in Section 5.5.

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.

Bearing Capacity:

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

- Short-term (undrained): 280 kPa (based on average shear strength of 55 kPa).
- Long-term (drained): 460 kPa based on friction angle of 30°.

Base Sliding:

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

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

Where:

A' (m^2)	= effective contact area of the base;
c' (kPa)	= cohesion/adhesion at sliding interface;
δ (°)	= friction angle at sliding interface;
V (kN)	= vertical force (kN); and
H_f (kN)	= design horizontal load.

Allowance for buoyancy should be made, where applicable.

Based on Highway Flood Hazard Analysis (ref. R-23), it is understood that Tunnel T-6 will not be flooded in the occurrence of 1:100 year storm and regional storm. However, in the event of failure of Pump Station 2 which is located in the vicinity of Tunnel T-6, the estimated flood elevations for the 100-year flooding event and the regional storm event are 174.3 and 174.7, respectively. Therefore, partial flooding of the roadway in Tunnel T-6 is expected to occur. As the EPS and LWF incorporated in Tunnel T-6 abutments and wing walls are located above the base of the pile cap at elevations greater than 175.8, submergence of these materials is not anticipated to occur in the area of Tunnel T-6.

The following soil properties (Table 5-12) can be used in the design at the interfaces between the RSS, RGM and silty clay subgrade:

Table 5-12: Soil Properties for use for Sliding Resistance

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

5.6.5 RGM – Loads and Preliminary Design

RGM foundation was considered under the RSS false abutment walls to improve the foundation load distribution and satisfy the ULS 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 5-13) were estimated to act on top of the RGM on the basis of conventional calculation of the bearing pressures under gravity retaining walls.

For both north and south abutments, an RGM is required along with select use of expanded polystrene (EPS) material. Conceptual wall configurations established to meet the external stability requirements are shown in Appendix I and in Table 5-14.

Table 5-13: Estimated Unfactored Loads on RGM

Abutment Location	Maximum Unfactored Edge Bearing Pressure below RSS wall, kPa	Average Unfactored Bearing Pressure below RSS Wall, kPa
North Wall	130	125
South Wall	155	130
North Extension Walls	200	160

Based on the above, a maximum unfactored horizontal tensile load of 36 kN per meter of RGM was estimated across the entire height of 1.5 m at the north and south abutments. For preliminary cost estimates, this tensile load can be accommodated by 2 layers of UX1100HS, or equivalent.

The above loads are for the use by the RGM suppliers to assist in the RGM's internal design. The bearing resistance of the subgrade soils under the RGM are provided in Section 5.6.4.

5.6.6 Abutment Configurations

Based on geotechnical analyses discussed in Section 5.2 to 5.6, abutment configurations and dimensions summarized in Table 5-14 were determined with respect to bearing and sliding modes of failure. The abutment configurations and dimensions indicated in these analyses are preliminary (e.g., the indicated width of the RSS is the minimum width) and are to be finalized by proprietary suppliers. The final design of the abutments is to be developed in consultation with the proprietary component suppliers.

Table 5-14: Abutment Dimensions(5)

Abutment Location	Assumed Total Height(1), m	RSS Structure Size (Width x Height)(3), m	RGM(2) Size (Thickness x Min. Width at Base), m	EPS(2) Volume, m^3/m
North Wall – 10+080	8.2	6.0 × 3.3	1.5 × 9.0	6.0
North Wall – 10+125	7.8	6.0 × 3.3	1.5 × 9.0	6.0
North Wall – 10+170	8.2	6.0 × 3.3	1.5 × 9.0	6.0
South Wall – 10+080	7.9	6.0 × 3.1	1.5 × 8.25	4.0
South Wall – 10+100	7.6	6.0 x 2.8	1.5 x 8.25	4.0
South Wall – 10+150	7.4	6.0 x 2.5	1.5 x 8.25	4.0

(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.

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

5.7 Wing Walls

Similar to the RSS walls at the abutments, the RSS Wing Walls have been checked for bearing capacity and resistances against sliding. Light weight fill (LWF) was required for north abutment RSS extended walls and south abutment RSS return walls.

The global stability analyses have been carried out for the extension wall, return wall and the highest RSS tapered wall. For drainage purposes, RGM must be extended to the full length of Tapered wing walls. 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. Figures F-19 to F-27 show the configuration assumed for RSS extended walls, RSS tapered walls outside the tunnel and the RSS Return walls. Table 5-15 summarizes the results of slope stability analyses carried out for the RSS wing walls.

Table 5-15: Summary of the Results of North Wing wall Slope Stability Analyses

Wing Wall Components	Factor of Safety for Loading Condition			Figure
	Short-Term during Construction (1)	Short-term at End of Construction (2)	Long-term (Drained)	
Extension Wing Walls – North	1.4 (1.3)	1.6 (1.4)	1.7 (1.6)	F-19 to F-21
Tapered Walls – South	1.6 (1.5)	1.8 (1.6)	1.6 (1.5)	F-22 & F-24
Return Walls - South	1.5 (1.4)	1.7 (1.6)	2.0 (1.8)	F-25 to F-27

(*) Values within parentheses refer to non-circular failure surfaces
(1) Undrained response without pavement box over Hwy 401 subgrade
(2) Undrained response with pavement box over Hwy 401 subgrade

Based on geotechnical analyses discussed in Section 5.2 to 5.6, tentative wing wall configurations and dimensions summarized in Table 5-16 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. The design of the abutments is to be developed in consultation with the proprietary component suppliers.

Table 5-16: Wingwall Dimensions(2)

Wing Wall	RSS Structure (Width × Height)(1), m	Quantity of Lightweight Fill, m^3/m
Extension North Wall (Highest section)	8.5 × 8.5	63.8
Tapered South Walls (Highest section)	5.0 × 3.9	-
Return South Wall (Highest Section)	4.5 x 4.9	22.0

(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.

5.8 Local Roads

The tunnel is crossed by Cabana Road to the north and Todd Lane to the south. The intersection of Cabana Road is at grade with Highway 3. The crossing will be lower than the grade of the Parkland landscape on top of the tunnel. No additional dead loads were anticipated over the abutment fills.

5.9 Backfilling

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

The granular backfill should be compacted in maximum 200 mm thick loose lifts in accordance with OPSS 501. Longitudinal drains and weep holes should be installed, as required, to ensure positive drainage of the backfill.

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

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 total lateral pressure due to soil weight and compactive effort should not be less than 12 kPa in any section of the wall.

Earth pressures on abutments and wing walls may be calculated on the basis of the parameters listed in Table 5-17. Compactable Group III soils may be used as general backfill within approved areas.

Table 5-17: 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).

In the case of sloping backfill surface, the coefficients in this table should be modified based on the following equations:

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

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

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

Where: ǿ = Friction angle of backfill material,

β = Slope of the backfill surface.

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 of 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 distribution 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.

A simplified steady-state model (Appendix H) was used to estimate seepage rate associated with the long-term drawdown of the groundwater along a typical cross-section of the north abutment of Tunnel T6. SEEP/W 2007 software was used for this analysis. The initial groundwater table was assumed at approximately elevation 181 (existing ground surface elevation). Groundwater recharge from infiltrations from ground surface sources was also considered. The rate of recharge was 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 considered 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 5.5 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 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.

All surface water should be directed away from all 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 prevent damage during excavation to the 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.
- 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 existing nearby structures should take place. Monitoring should consist of a precondition survey along with regular surveying conducted of the nearby utilities, residences, etc.
- Rip/rap, or otherwise coarse rockfill cover are considered to have half the insulation effect as offered by soil deposits/cover, and therefore, the depth of frost penetration will have to be increased proportionally.
- Air quality and subgrade pore pressure monitoring should be carried out during construction. The equipment operating in confined spaces should be selected to safely operate in a potentially gaseous environment. Excavation layers should be decided in consideration of the pore pressure monitoring data and the potential ground softening.

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.

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, verify compliance with the PA, 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 T6-1/HGMW-7 (Sample 10), BH T6-2 (Sample 11), BH T6-3 (Sample 10) and BH 12-RW (Sample 4). Table 6-1 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 T6-1 / HGMW-7 (Sample 10)	171.7	8.11	111	2600	<0.2	436
Borehole BH T6-2 (Sample 11)	170.2	8.00	105	2820	<0.2	451
Borehole BH T6-2 (Sample 10)	172.5	8.14	111	2310	<0.2	374
Borehole BH 12-RW (Sample 4)	178.2	7.62	131	5750	<0.2	69

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-6, 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 (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-6 was prepared by Mr. Ganan Nadarajah, P.Eng. and checked by Dr. Dan Dimitriu, P.Eng. The project was executed under the technical direction of Dr. Narendra S. Verma, P.Eng., who also provided the senior review of the report. Mr. Matt Oldewening, P.Eng., managed the geotechnical investigation and Mr. Brian Lapos, P.Eng., is the project manager.

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
TODD–CABANA TUNNEL T–6
GENERAL ARRANGEMENT

SHEET S2601

Phase 1

90% Sub

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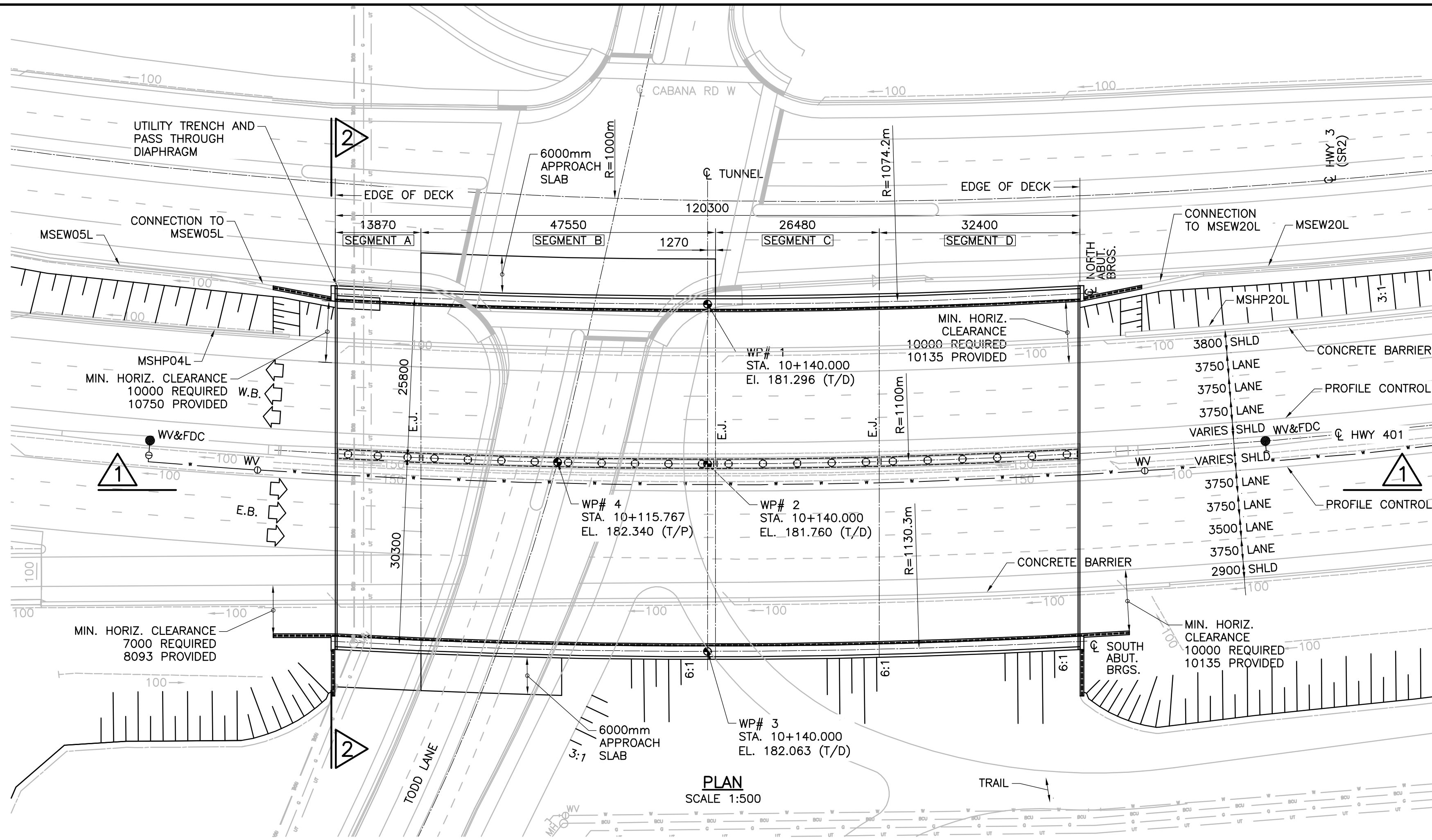

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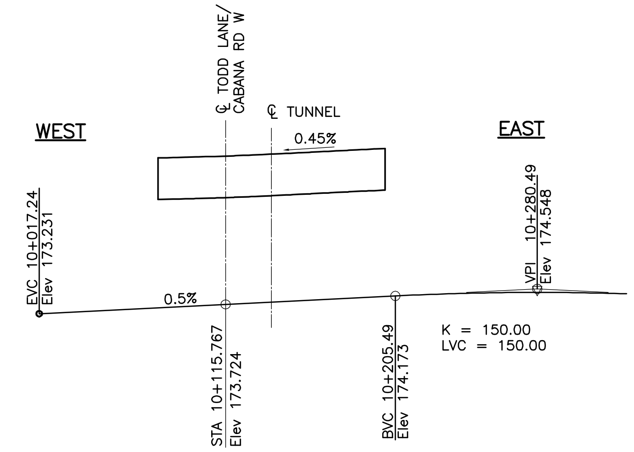
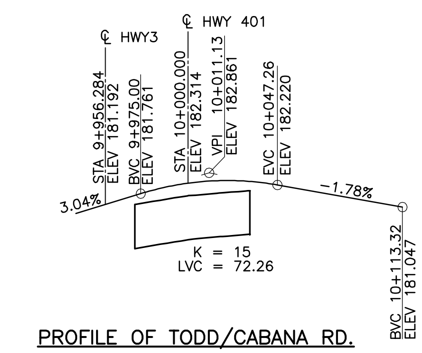


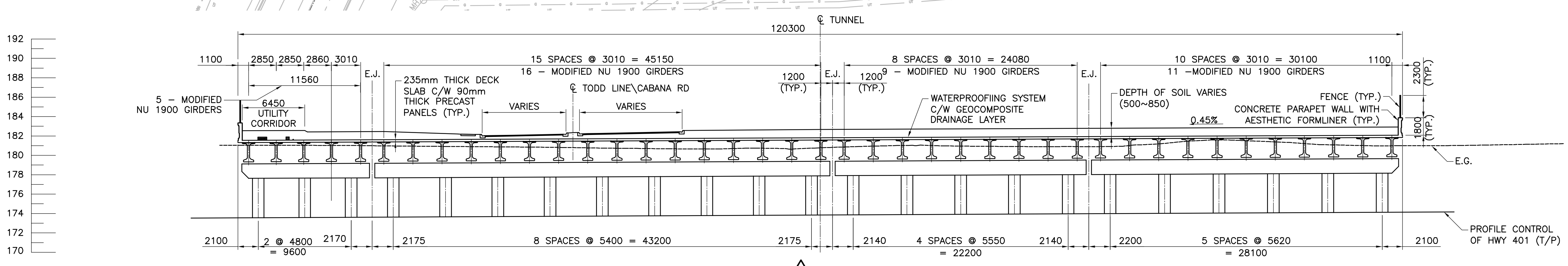
PROFILE OF HWY 401
N.T.S.



PROFILE OF TODD/CABANA RD.
N.T.S.

NOTES:

1. FOR GENERAL NOTES SEE SHEET S2602.



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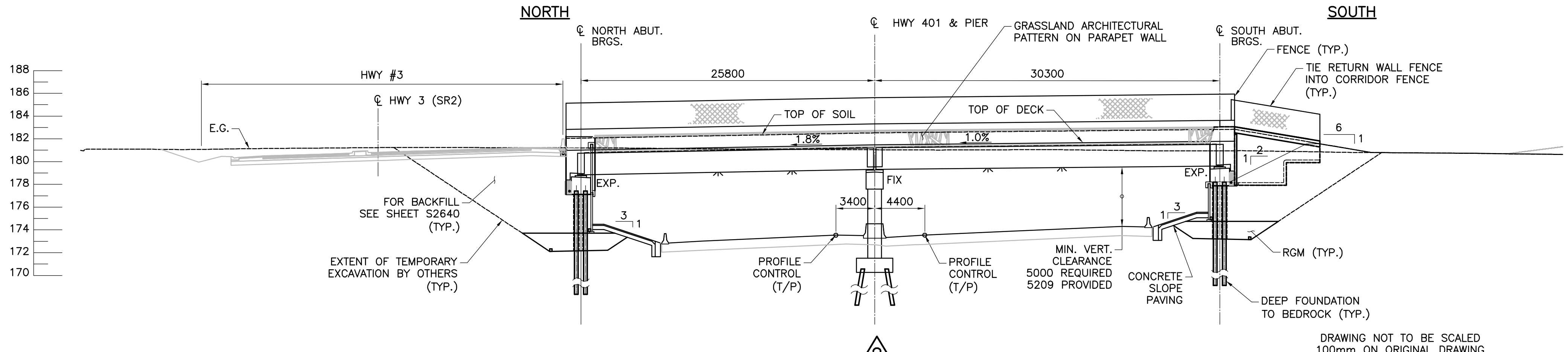

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DRAWING NOT TO BE SCALED
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REVISIONS				
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04–MAY–12	A	SFY	60% MTO SUBMISSION	
DATE	REV.	BY	DESCRIPTION	

DESIGN SFY	CHK BR	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN DM	CHK MAS	SITE 6–706	DATE 30–AUG–11

DOC: 285380–03–060–WIP1–2601

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MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

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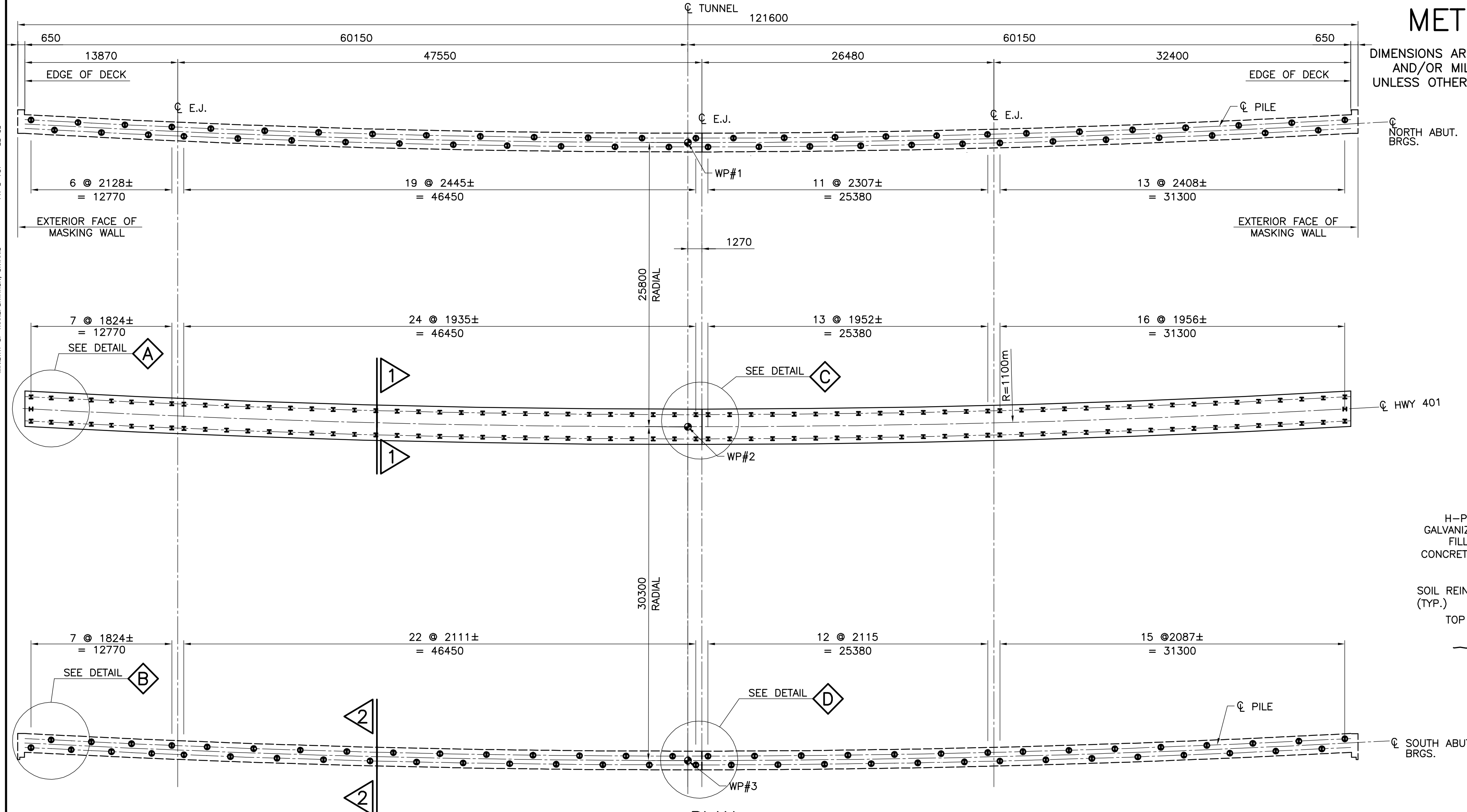
Parkway Infrastructure Engineers | Hatch Mott MacDonald

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

NEW CONSTRUCTION
HWY 401
TODD–CABANA TUNNEL T–6
FOUNDATION LAYOUT

SHEET S2605

Phase 1
90% Sub



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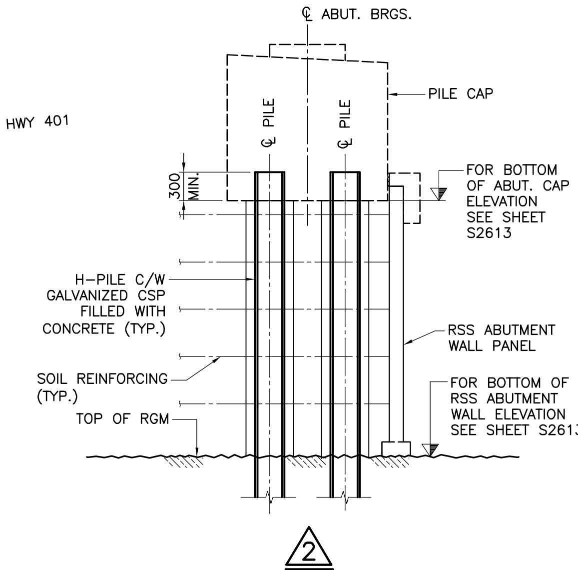


2 SCALE 1:40

NOTES:

1. FOR GENERAL NOTES SEE SHEET S2602.
2. THIS DRAWING TO BE READ IN CONJUNCTION WITH THE RETAINED SOIL SYSTEM WALLS AND ABUTMENT LAYOUT DRAWINGS.

PILE NOTES:

1. PILE LENGTHS SHOWN ARE ESTIMATED LENGTHS FROM THE CUT–OFF TO THE ESTIMATED BEDROCK / REFUSAL SURFACE.
2. ALL PILES ARE HP310X110 STEEL H–PILES INSTALLED AS PER OPSS 903.
3. ALL PILES SHALL BE FITTED WITH TYPE I DRIVING SHOE PER OPSD 3000.100 OR APPROVED EQUIVALENT.
4. PILE SPLICES SHALL BE BUTT WELDED AS PER OPSD 3000.150 AND OPSS903. SPLICE PLATES ARE NOT PERMITTED.
5. ALL PILES ARE TO BE DRIVEN TO BEDROCK OR TO REFUSAL IN THE VERY DENSE COHESIONLESS DEPOSIT OVERLYING BEDROCK IN ACCORDANCE WITH SS103–11 TO DEVELOP AN ULTIMATE GEOTECHNICAL RESISTANCE OF 4000 KN, GIVING A DESIGN FACTORED ULS RESISTANCE OF 2000 KN.
6. THE PILE ULTIMATE GEOTECHNICAL RESISTANCE AND REFUSAL CRITERIA SHALL BE CONFIRMED ON AT LEAST 3% OF THE PILES BY PDA METHOD SUPPLEMENTED WITH STATIC LOAD TESTS IN THE AREA OF THE STRUCTURE.
7. PILE DRIVING EQUIPMENT SHALL BE APPROPRIATE TO THE DRIVING CONDITIONS TO DEVELOP THE ULTIMATE GEOTECHNICAL RESISTANCE, AND PREVENT DAMAGES TO THE PILES DURING DRIVING. CONSIDERATION SHOULD BE GIVEN TO POTENTIAL DRIVING DIFFICULTIES DUE TO THE PRESENCE OF COBBLES OR BOULDERS.
8. HAMMER DETAILS (HAMMER TYPE AND MODEL, RATED ENERGY, HELMET AND CUSHION DETAILS) SHALL BE SUBMITTED 10 DAYS PRIOR TO THE EQUIPMENT MOBILIZATION TO THE SITE.
9. SURVEY ALL PILE HEAD ELEVATIONS AT END OF DRIVING AND JUST PRIOR TO FORMING OF PILE CAP. RE–TAP PILES WHERE UPLIFT >5 MM OR AS DIRECTED BY THE ENGINEER.
10. DURING PILE DRIVING THE CONTRACT SHALL IMPLEMENT APPROPRIATE MITIGATION MEASURES AGAINST ANY SEEPAGE OF NATURAL GAS AND GROUNDWATER THAT MIGHT CAUSE LOSS OF BEARING RESISTANCE.
11. THE CONTRACTOR SHALL MONITOR VIBRATIONS AT STRATEGIC LOCATIONS (E.G. TEMPORARY SLOPES, UTILITIES AND STRUCTURES) AND ESTABLISH APPROPRIATE FREQUENCY BASED LIMITS ON PEAK PARTICLE VELOCITIES IN ORDER TO PREVENT DAMAGE CAUSED BY PILE DRIVING.

CONSTRUCTION SEQUENCE – ABUTMENTS:

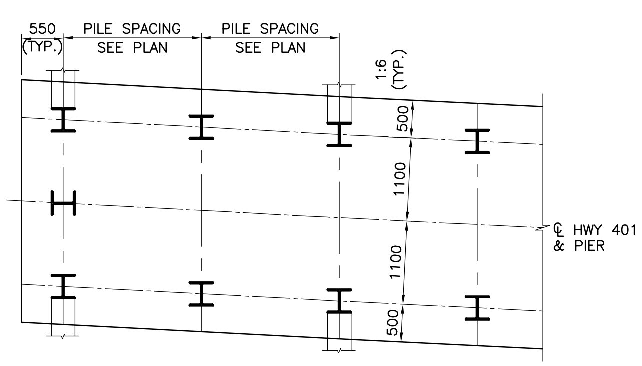
1. EXCAVATE FOR TUNNEL OPENING.
2. PROVIDE SUITABLE PROTECTION DURING CONSTRUCTION.
3. EXCAVATE TO UNDERSIDE OF RGM.
4. INSTALL RGM.
5. DRIVE PILES.
6. PLACE 500mm DIA. GALVANIZED CSP PIPES AND PROVIDE BLOCKING OF PILES IN CSP PIPES.
7. CONSTRUCT RETAINED SOIL SYSTEM WALLS AND BACKFILL TO UNDERSIDE OF ABUTMENTS AND PLACE STRAPS FOR RESTRAINING OF ABUTMENT, SEE SHEET S2640.
8. FILL 500mm DIA. CSP PIPES WITH CONCRETE.
9. CONSTRUCT ABUTMENTS TO UNDERSIDE OF BEARING PEDESTALS.
10. CONTRACTOR TO PROVIDE SUITABLE STABILITY DURING CONSTRUCTION.
11. SEE ABUTMENT DETAILS DRAWING FOR CONTINUATION.

CONSTRUCTION SEQUENCE – PIER:

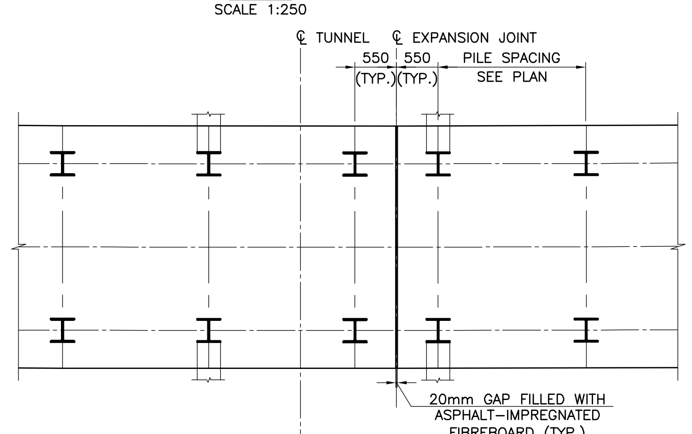
1. EXCAVATE TO UNDERSIDE OF FOOTING.
2. DRIVE PILES.
3. PLACE FOOTING PAD.
4. CONTRACTOR TO PROVIDE SUITABLE STABILITY DURING CONSTRUCTION.
5. CONSTRUCT PIER FOOTING TO TOP OF FOOTING
6. SEE PIER DETAILS DRAWING FOR CONTINUATION.

APPLICABLE STANDARD DRAWINGS:

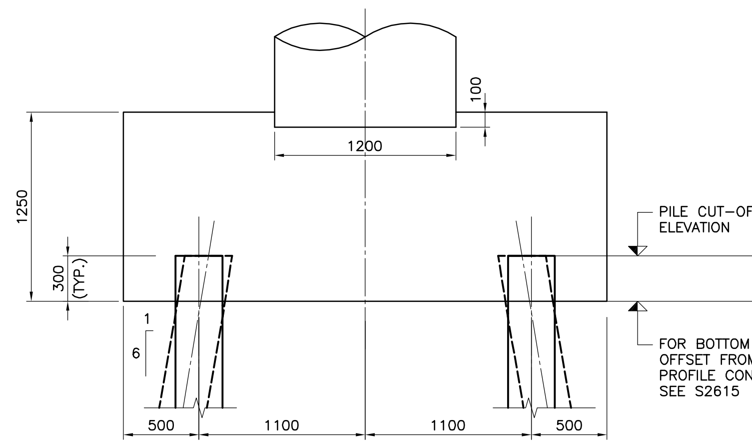
OPSD–3000.100 FOUNDATION PILES – STEEL H–PILE DRIVING SHOE
OPSD–3000.150 FOUNDATION PILES – STEEL H–PILE SPLICE



A SCALE 1:50



C SCALE 1:50 CAP BEAM PILE DETAIL



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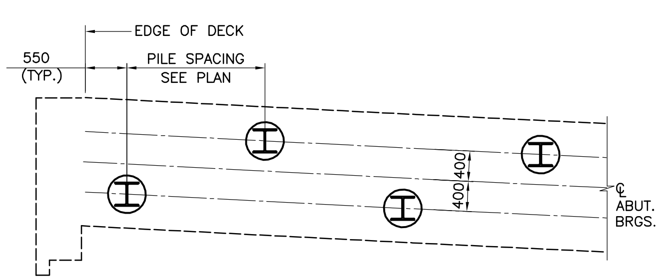

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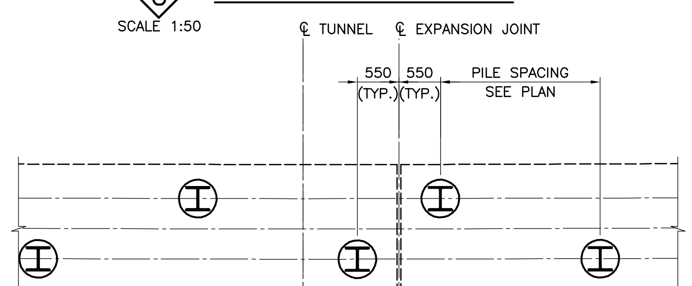

D SCALE 1:50 ABUTMENT PILE DETAIL

PILE DATA			
LOCATION	No. REQUIRED	ESTIMATED LENGTH (m)*	BATTER
N. ABUTMENT	53	30.9	VERTICAL
PIER	66	25.4	VERTICAL
	64	25.7	1:6
S. ABUTMENT	60	31.7	VERTICAL

*ESTIMATED AVERAGE PILE LENGTH

WORKING POINT DATA		
LOCATION	NORTHING	EASTING
WP #1	4 679 654.021	332 083.709
WP #2	4 679 639.281	332 062.534
WP #3	4 679 621.971	332 037.666

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

REVISIONS			
24–AUG–12	B	SFY	90% MTO SUBMISSION
04–MAY–12	A	SFY	60% MTO SUBMISSION
DATE	REV.	BY	DESCRIPTION

DESIGN	SFY	CHK	BR	CODE	CAN/CSA S6–06	LOAD	CL–625–ONT
DRAWN	DM	CHK	MAS	SITE	6–706	DATE	30–AUG–11

DOC: 285380–03–061–WIP1–2605

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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

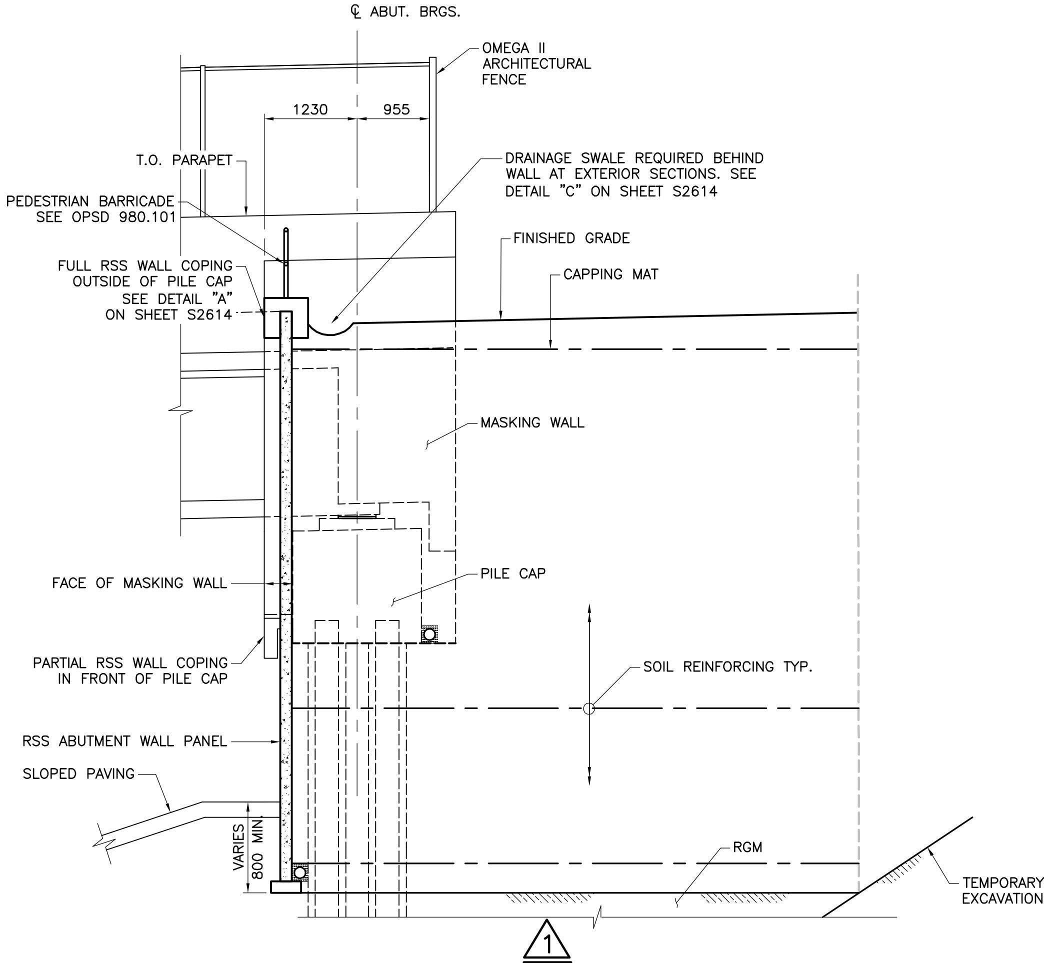
Windsor–Essex Parkway Project RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
TODD–CABANA TUNNEL T–6
WINGWALL DETAILS

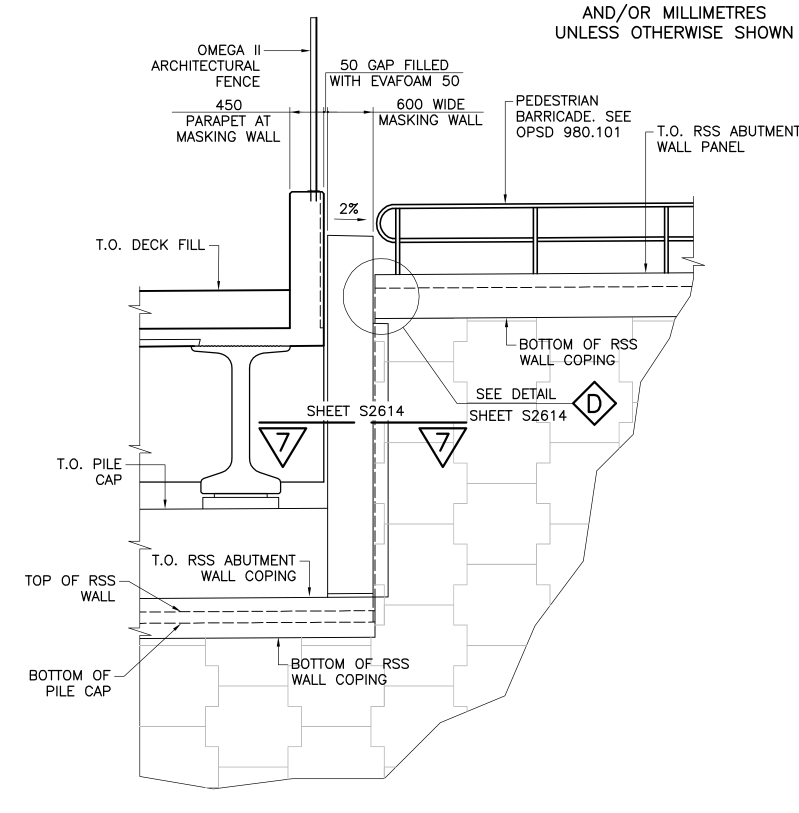
SHEET S2612

Phase 1

90% Sub



1
SCALE 1:50
SHEET S2613



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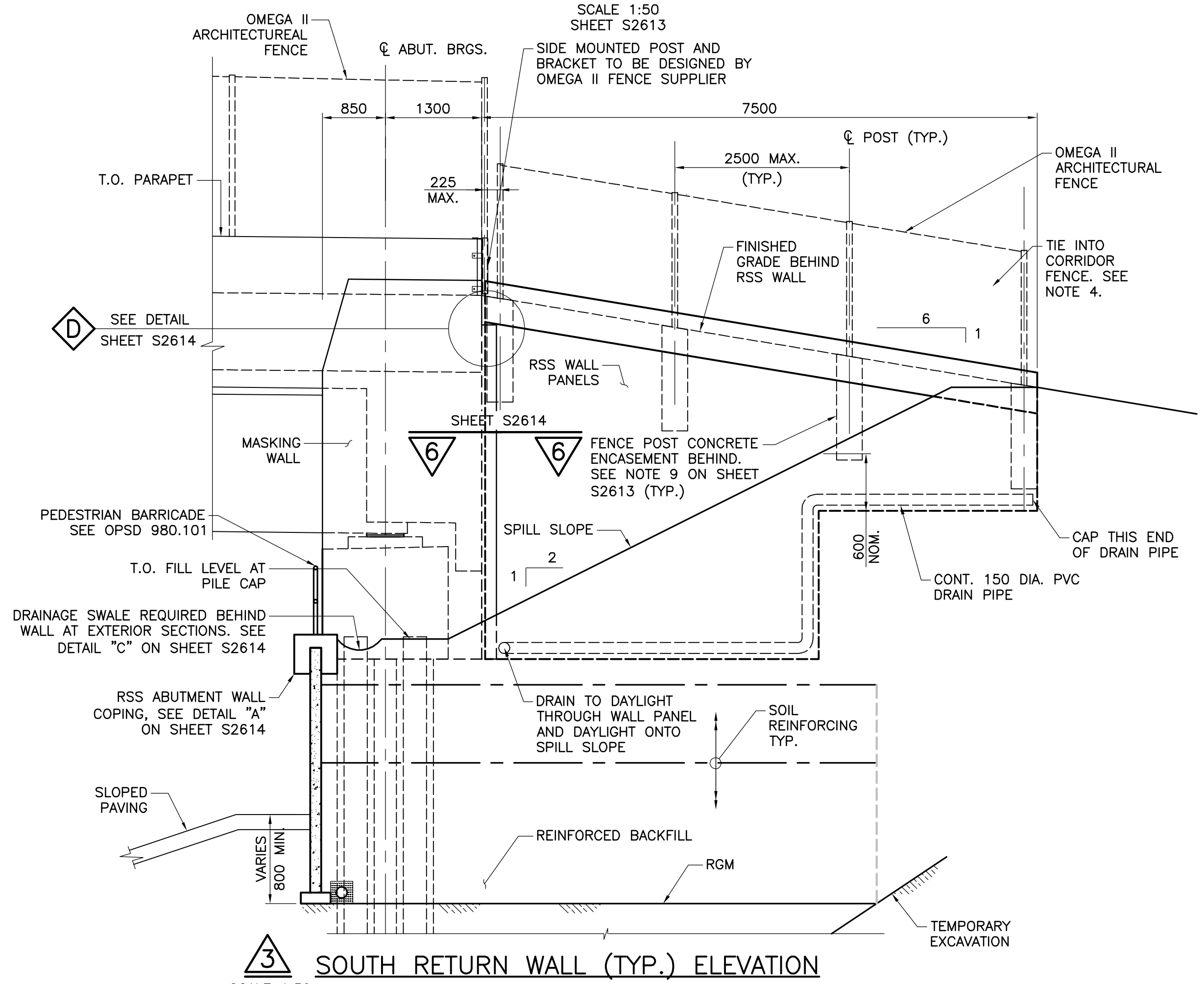
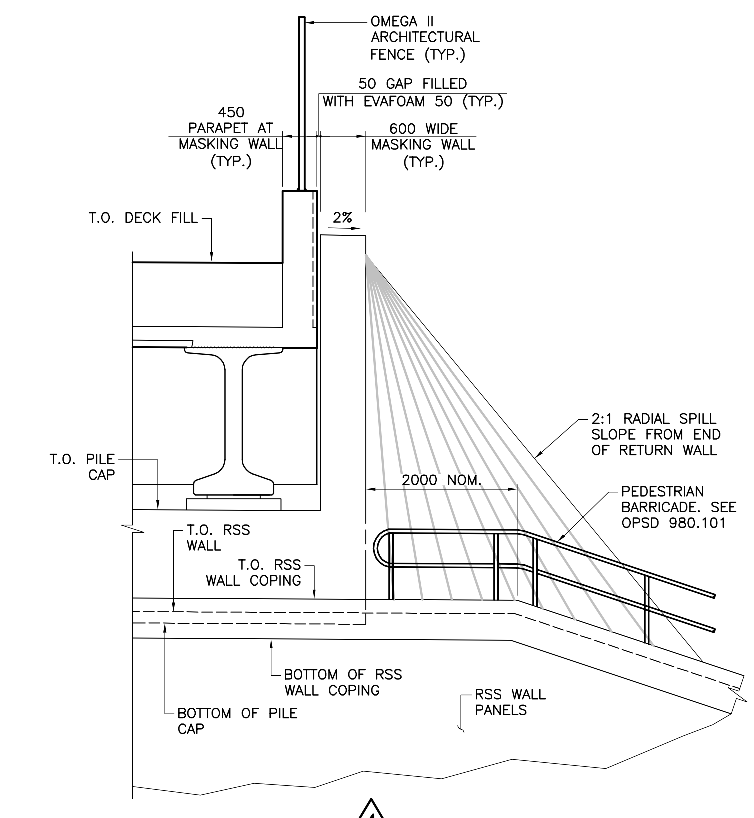
SHEET S2613



3 SOUTH RETURN WALL (TYP.) ELEVATION
SCALE 1:50
SHEET S2613



4
SCALE 1:50
SHEET S2613

NOTES:

1. READ THIS DRAWING IN CONJUNCTION WITH SHEETS S2613, S2614, S2611.
2. SEE SHEET S2613 FOR RSS WALL NOTES.
3. THE MIN 600mm DEPTH FOR RSS WALL LEVELING PAD TO BE CONFIRMED BY RSS MANUFACTURER.
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.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

REVISIONS

DATE	REV.	BY	DESCRIPTION
24–AUG–12	B	SFY	90% MTO SUBMISSION
04–MAY–12	A	SFY	60% MTO SUBMISSION

DESIGN SFY	CHK BR	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN DM	CHK MAC	SITE 6–706	DATE 30–AUG–11

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METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | Hatch Mott MacDonald

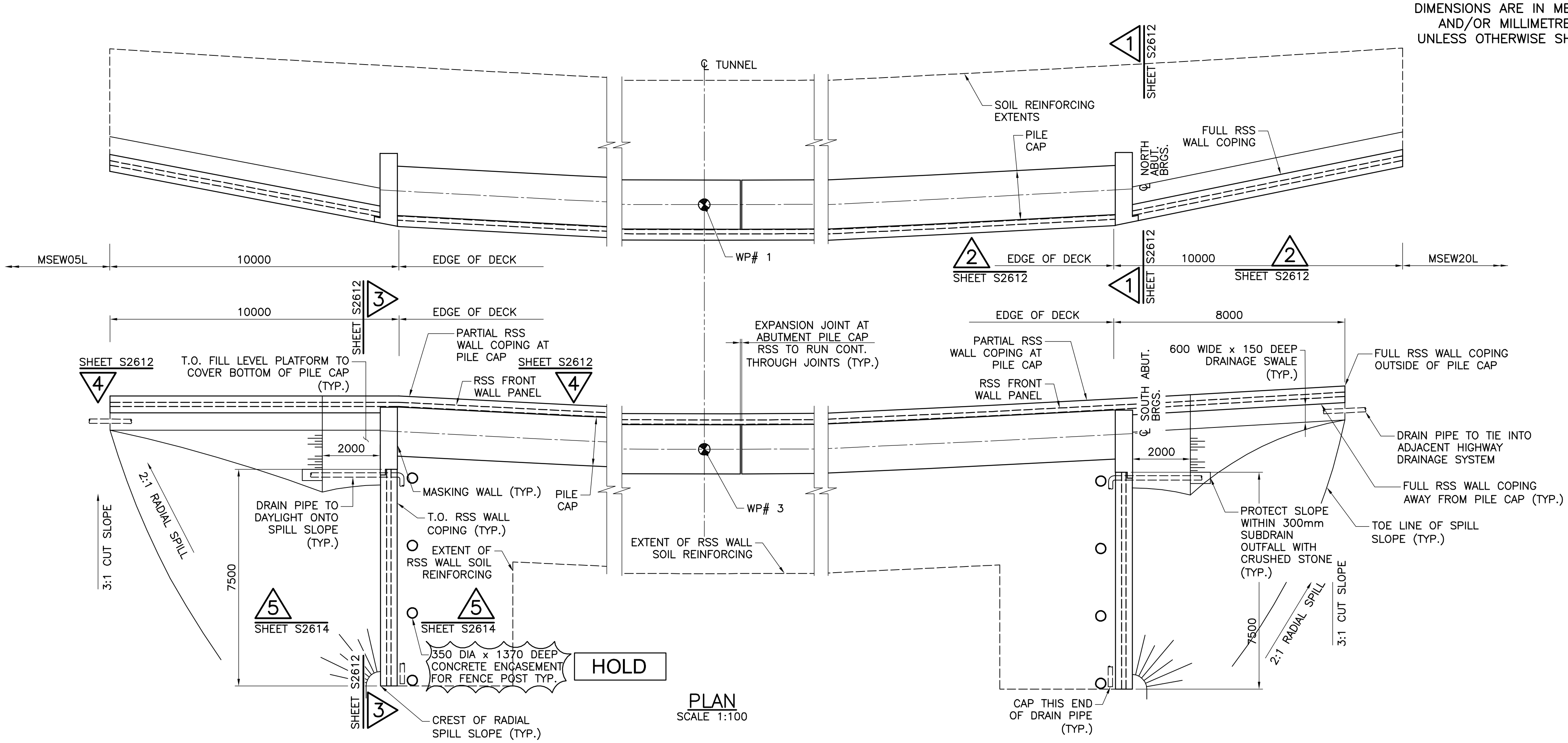
Windsor–Essex Parkway Project
RFP No. 09–54–1007

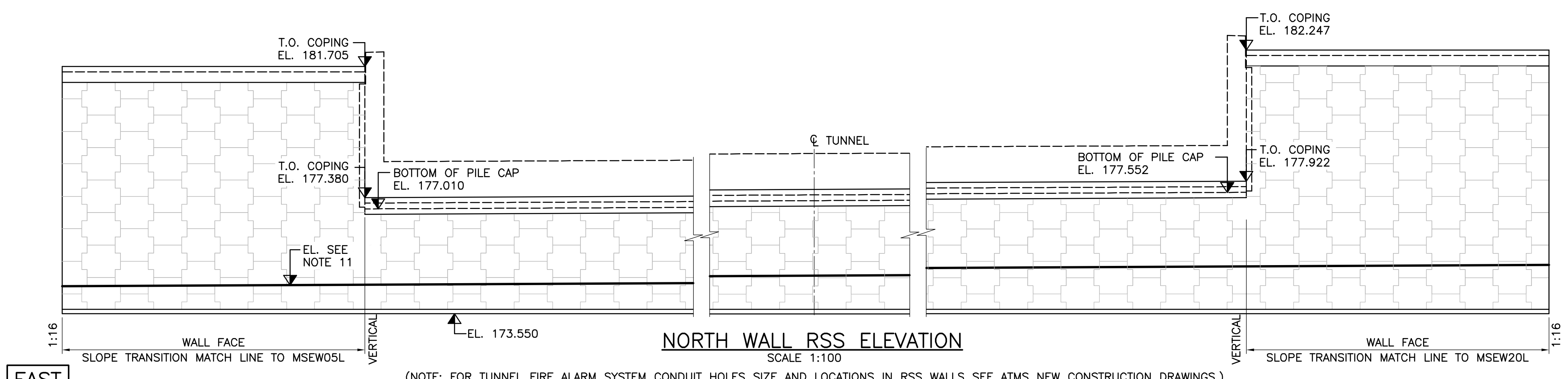
NEW CONSTRUCTION
HWY 401
TODD–CABANA TUNNEL T–6
RSS WALLS LAYOUT

SHEET S2613

Phase 1

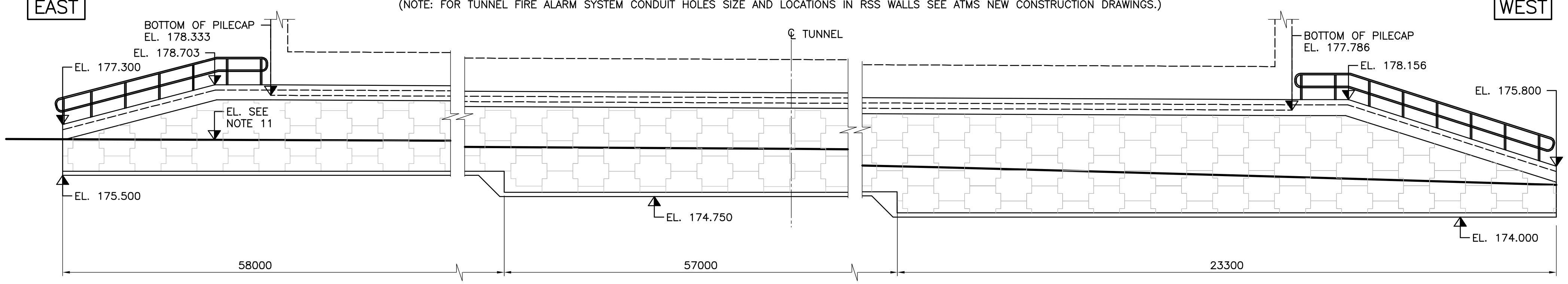
90% Sub





EAST

WEST



SOUTH RSS WALL ELEVATION
SCALE 1:100

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

NORTH FOR CONSTRUCTION

NOTES:

1. READ THIS DRAWING TOGETHER WITH SHEETS S2605, S2607 TO S2609, S2612, & S2614.
2. SEE SHEET S2607 FOR SOIL REINFORCING STRIP 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
   PERFORMANCE: HIGH
   APPEARANCE: 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 FILL.
11. SEE SHEET S2631 FOR SLOPE PAVING ELEVATION.
12. THERE ARE SPECIAL PATTERNS ON THE NORTH RSS WALL FOR AN AESTHETIC PURPOSE FOR DETAILS, SEE RELEVANT DRAWINGS.

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS			
DATE	REV.	BY	DESCRIPTION
24–AUG–12	B	SFY	90% MTO SUBMISSION
04–MAY–12	A	SFY	60% MTO SUBMISSION

DESIGN SFY	CHK BR	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN DM	CHK MAC	SITE 6–706	DATE 30–AUG–11

DOC: 285380–03–061–WIP1–2613

METRIC

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

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

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

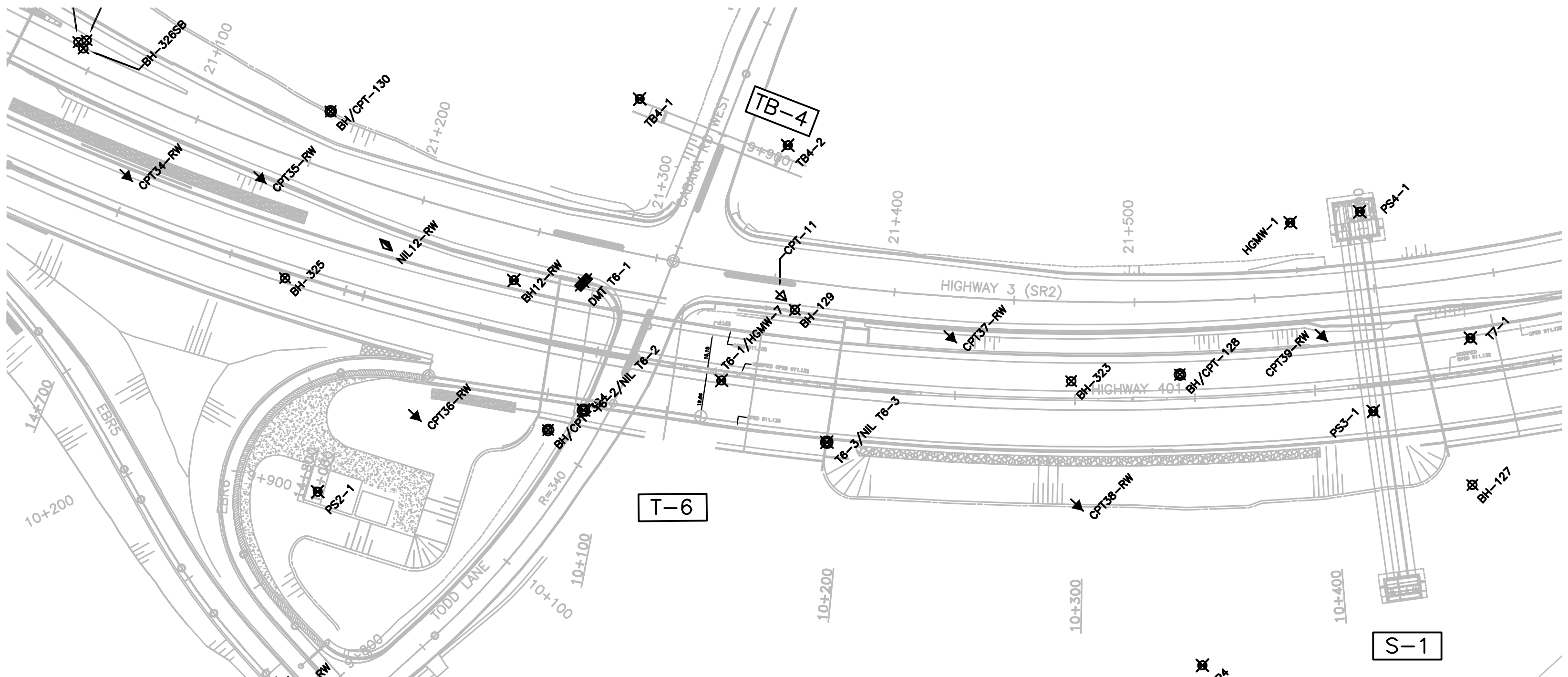
DESIGN JF | APR NSV | DATE 15–JUL–11

LOCATION PLAN & INTERPRETED STRATIGRAPHIC PROFILE
STA 14+700W TO STA 10+400L

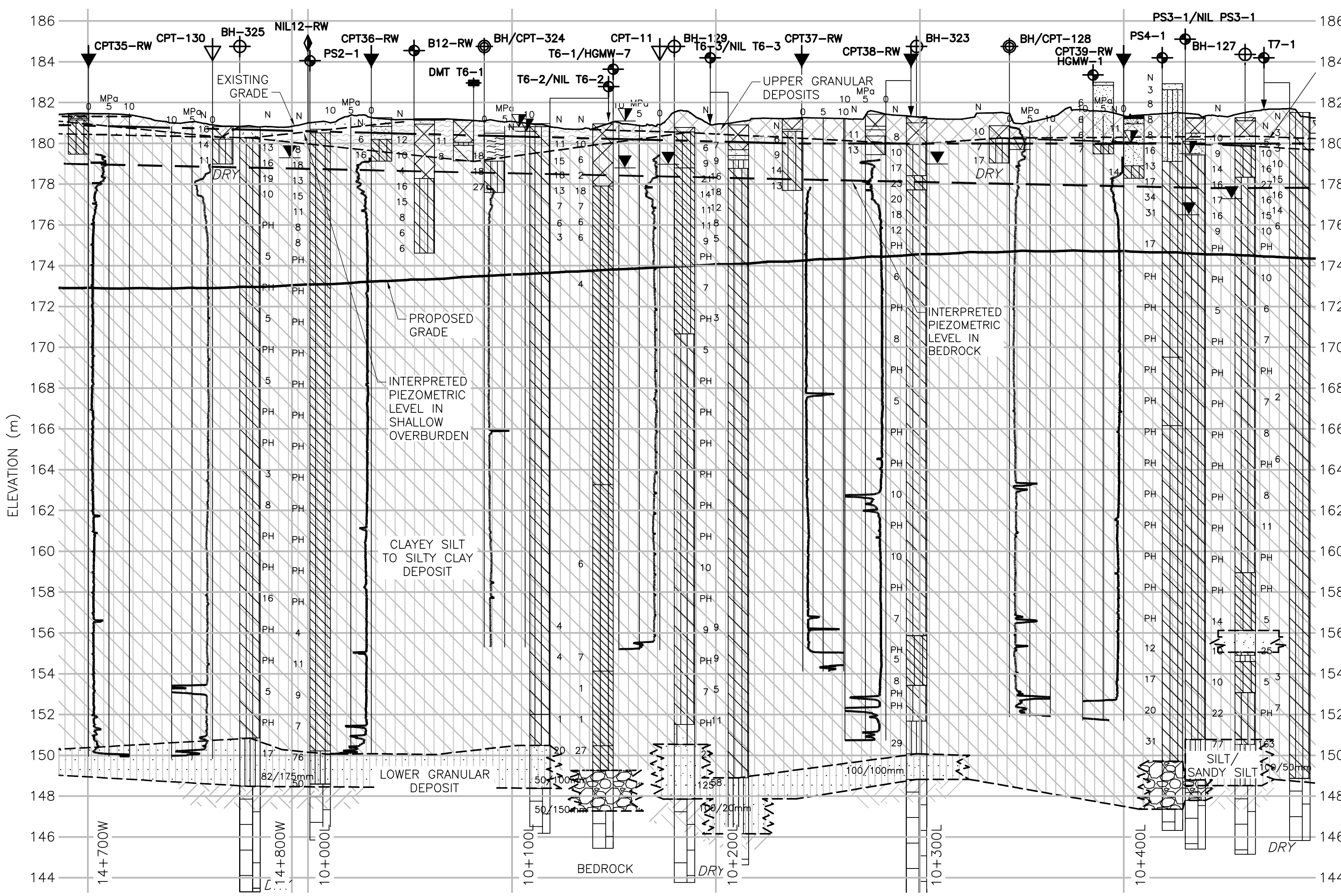
SHEET G2602

Phase 1

IFC



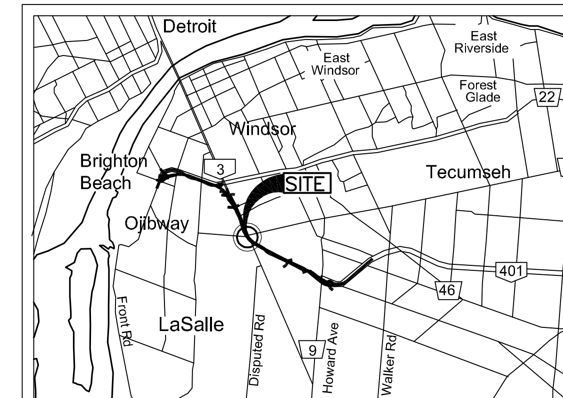
PLAN
SCALE 1:1500



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

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

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 WINDSOR (W) AND LASALLE (L) SECTORS.

SCALES
15 0 30m Horizontal
1.5 0 3 Vertical

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METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

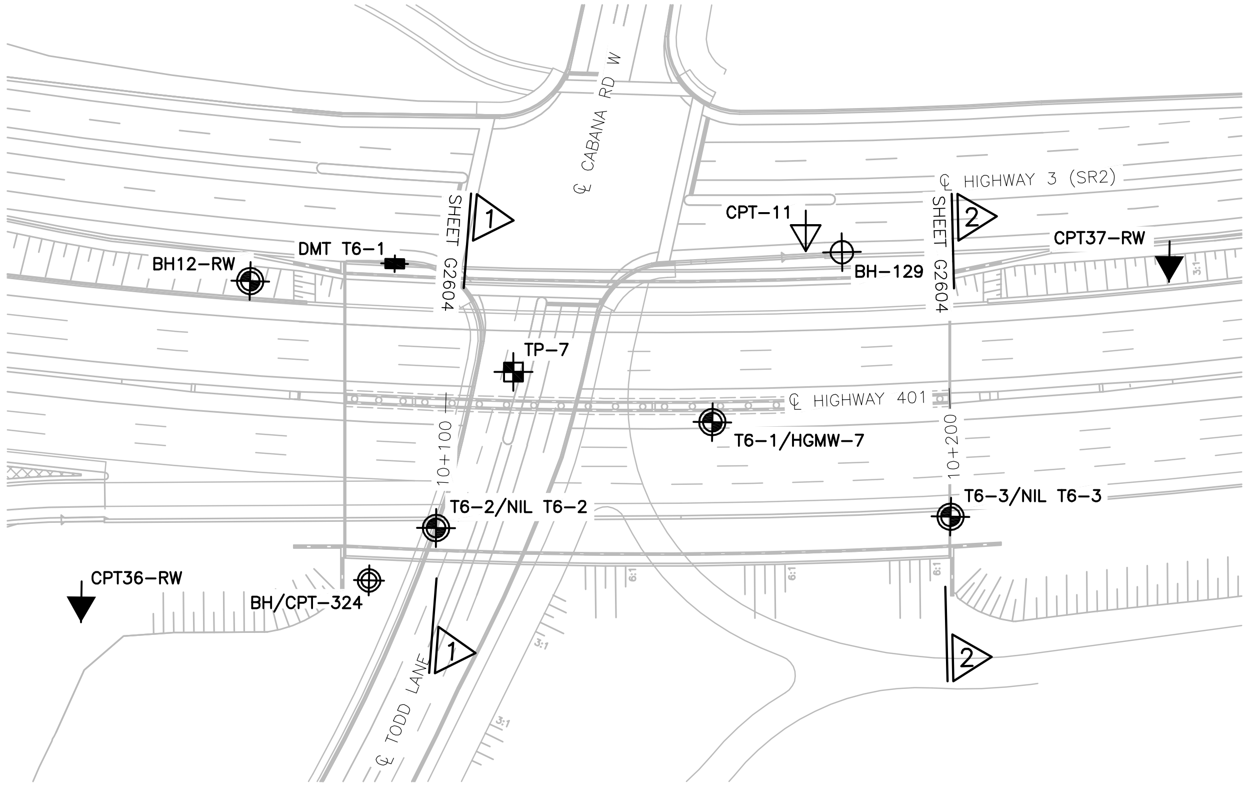
Windsor–Essex Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
TODD–CABANA TUNNEL T–6
BOREHOLE LOCATIONS & SOIL STRATA

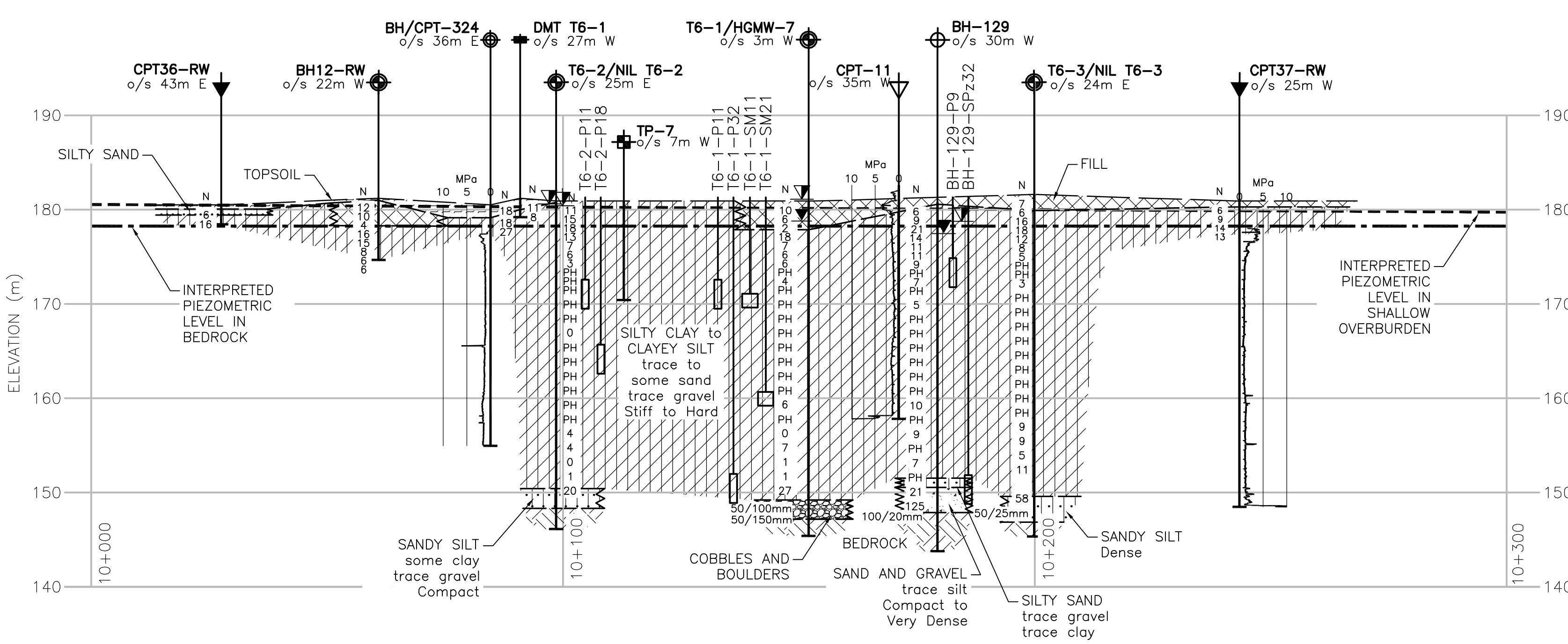
SHEET G2603

Phase 1

IFC

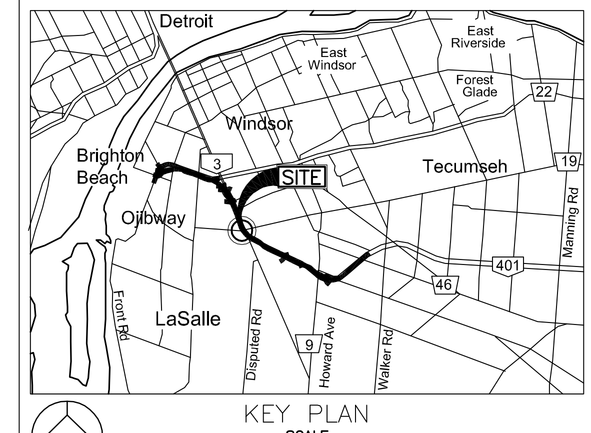


PLAN
SCALE 1:750



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

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING



KEY PLAN
SCALE
1 0 2 4Km

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

No.	ELEVATION	CO–ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC BOREHOLES			
BH12–RW	181.2	4679718.1	332037.9
DMT T6–1	181.2	4679696.6	332057.3
T6–1/HGMW–7	180.9	4679627.0	332067.4
T6–2/NIL T6–2	180.8	4679659.9	332018.8
T6–3/NIL T6–3	181.6	4679577.5	332079.1
CPT36–RW	180.5	4679710.0	331968.8
CPT37–RW	180.9	4679571.4	332146.2
TP–7	180.9	4679665.0	332053.0
PREVIOUS BOREHOLES			
CPT–11	180.91	4679634.0	332110.0
BH–129	180.78	4679625.1	332109.7
BH/CPT–324	180.85	4679664.9	332002.7

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)
SPz – STANDPIPE PIEZOMETER
DRY BOREHOLE DRY DURING DRILLING
WATER LEVEL DURING DRILLING
WATER LEVEL (SHALLOW PIEZO)
WATER LEVEL (DEEP PIEZO)
MPa 0 5 10 CPT–qc

NOTES

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

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

DESIGN JF	CHK NSV	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN MM	CHK DD	SITE 6–706	DATE 15–NOV–11

DOC: 285380–04–090–WIP1–2603

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

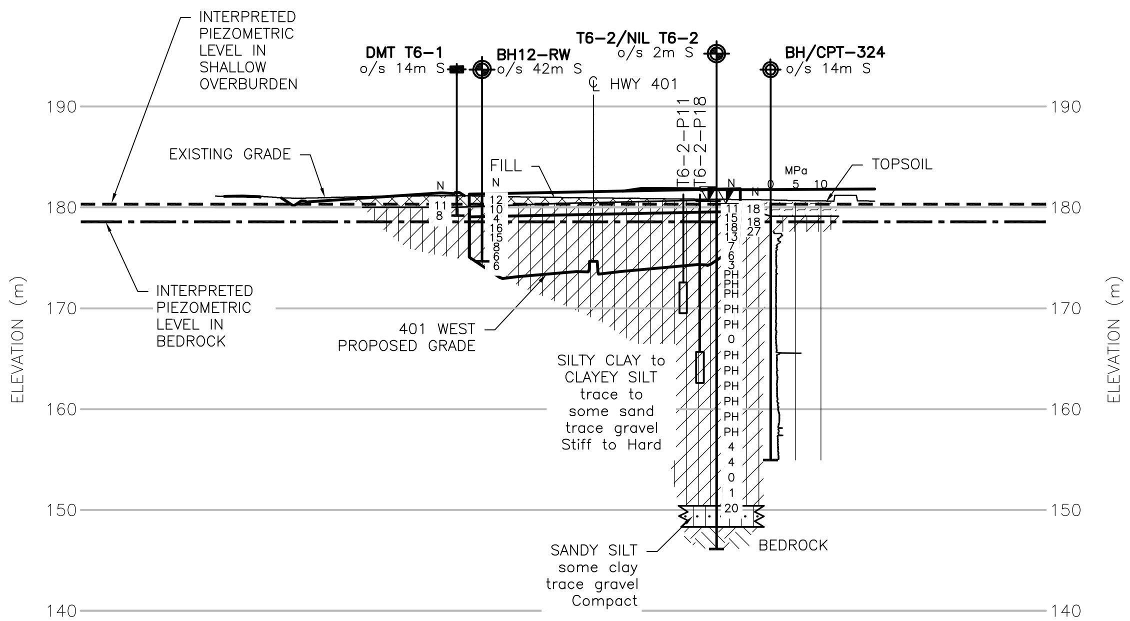
Windsor–Essex
Parkway Project
RFP No. 09–54–1007

NEW CONSTRUCTION
HWY 401
TODD–CABANA TUNNEL T–6
SOIL STRATIGRAPHY

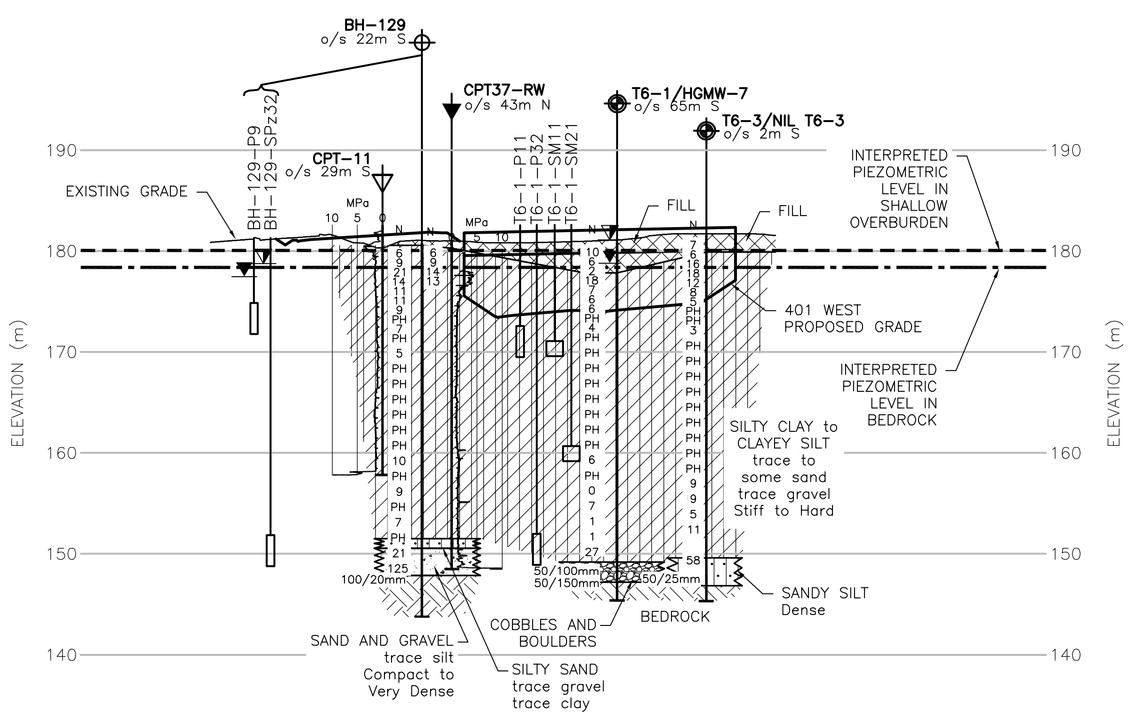
SHEET G2604

Phase 1

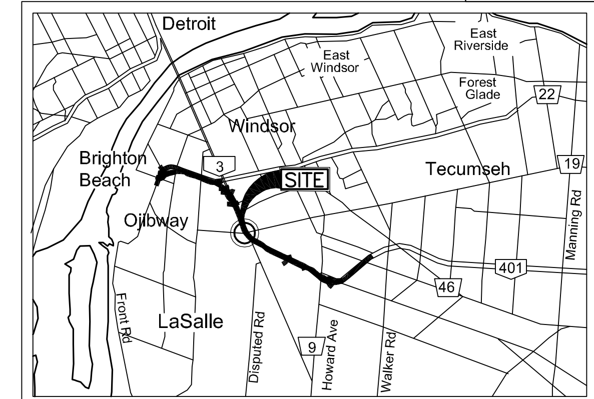
IFC



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

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

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

NOTES

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3. ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

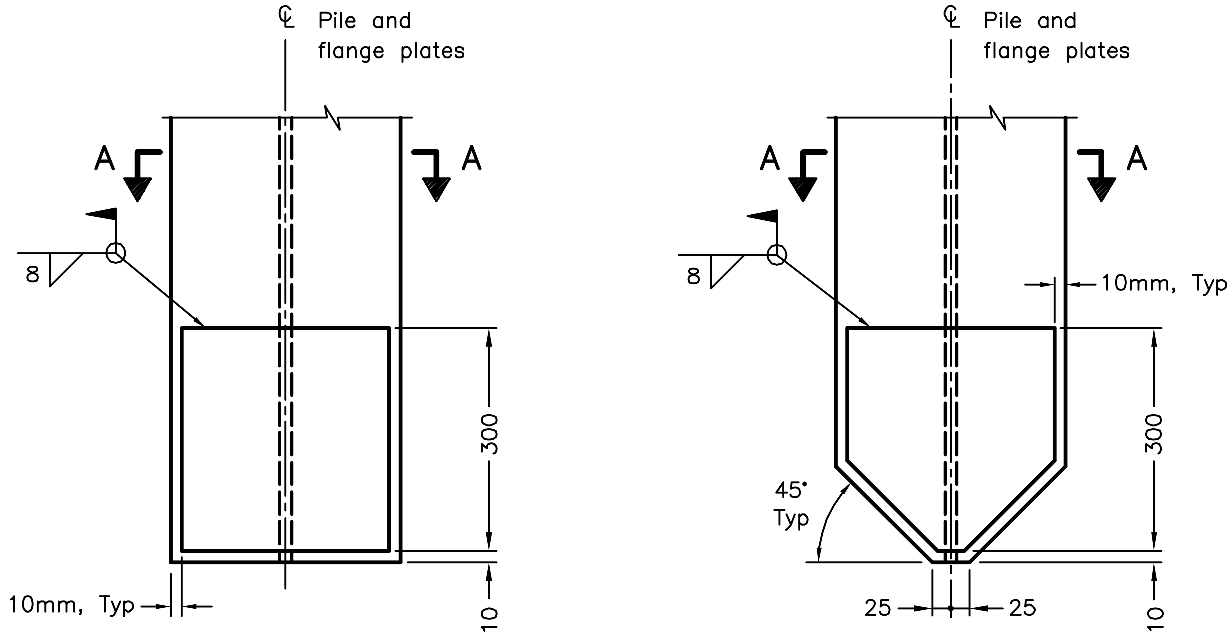
REVISIONS

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

DESIGN JF	CHK NSV	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN KT	CHK DD	SITE 6–706	DATE 15–NOV–11

DOC: 285380–04–091–WIP1–2604

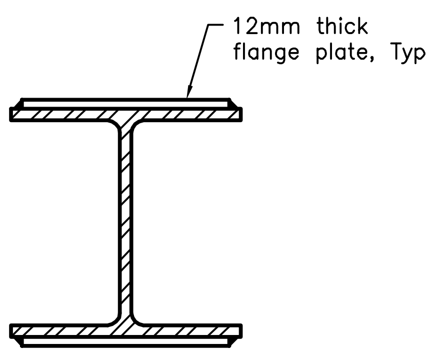
Applicable OPSDs



TYPE I

TYPE II

ELEVATION



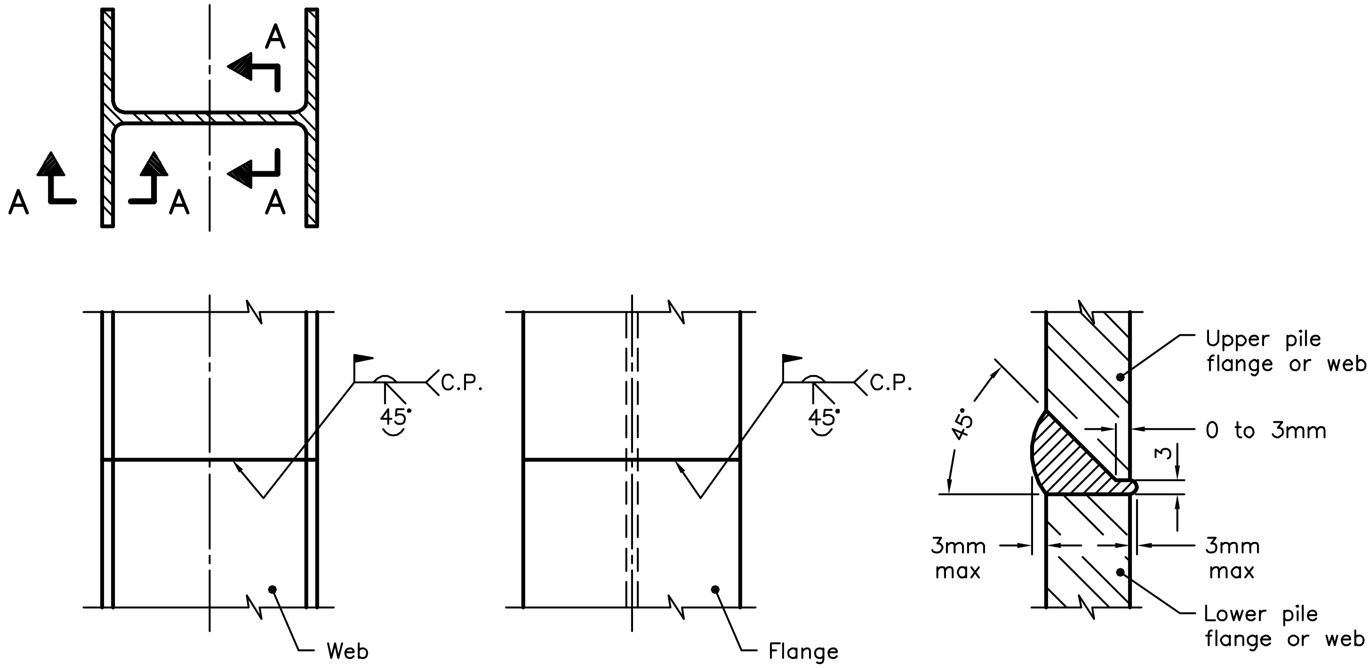
PILE DRIVING SHOE

SECTION A–A

NOTES:

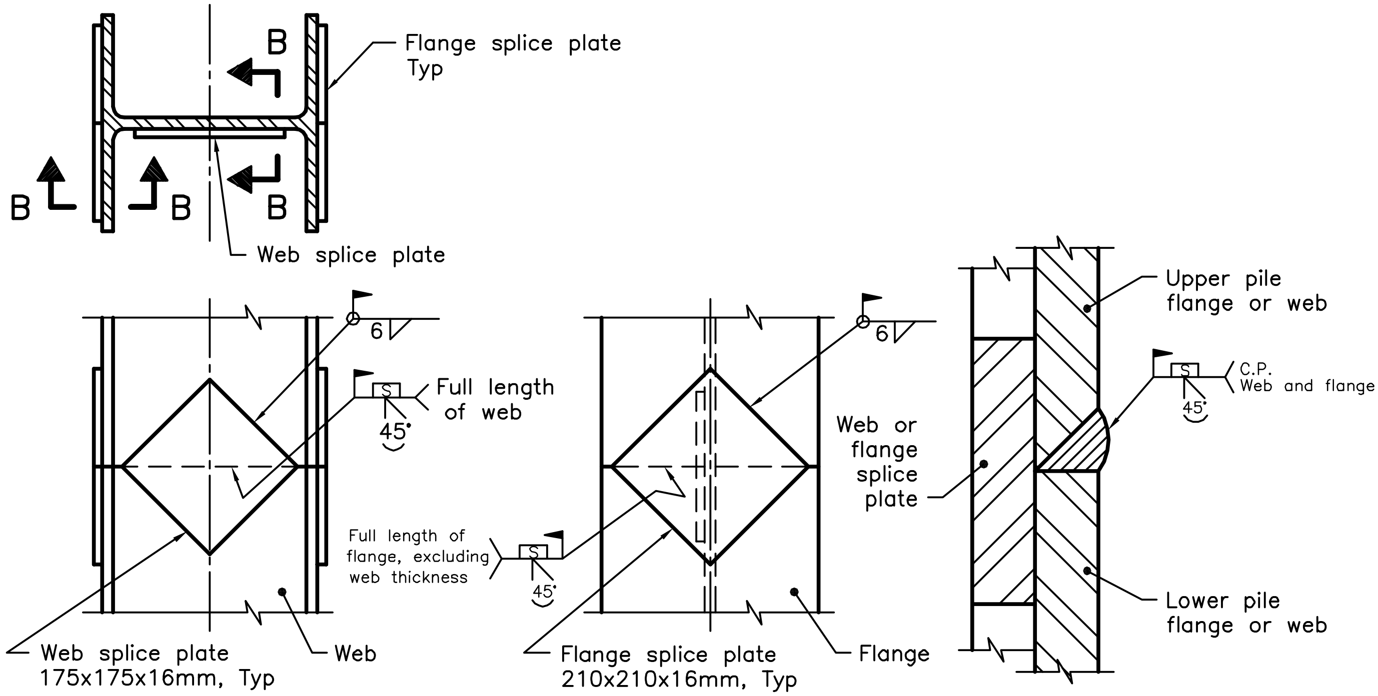
A Flange plates shall be according to CSA G40.20/G40.21, Grade 300W.
B Welding shall be according to CSA W59.
C Driving shoe Type I shall be used unless Type II is specified.
D All dimensions are in millimetres unless otherwise shown.

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



BUTT WELD

SECTION A–A



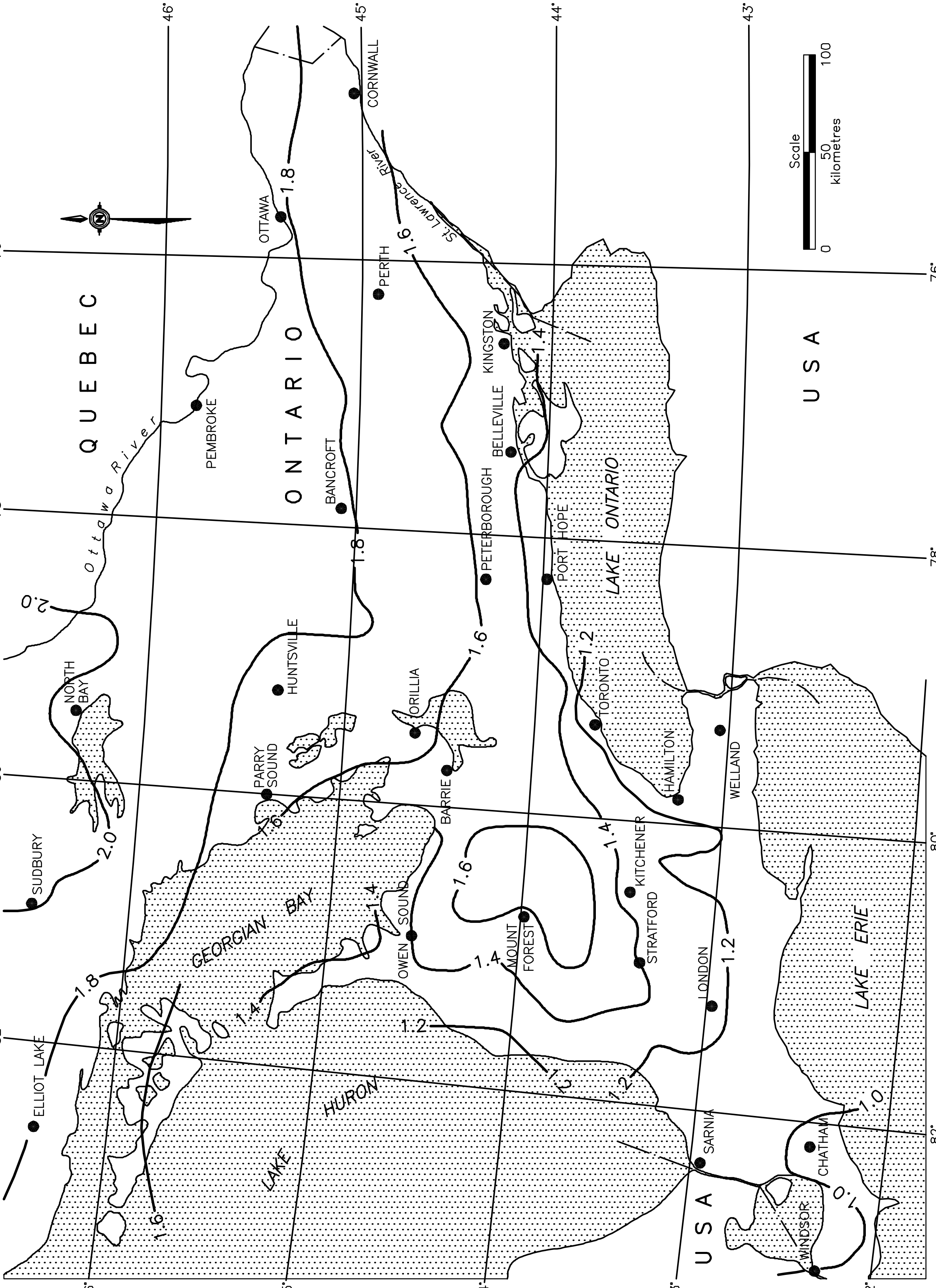
BUTT WELD WITH SPLICE PLATES

SECTION B–B

NOTES:

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

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



NOTES:

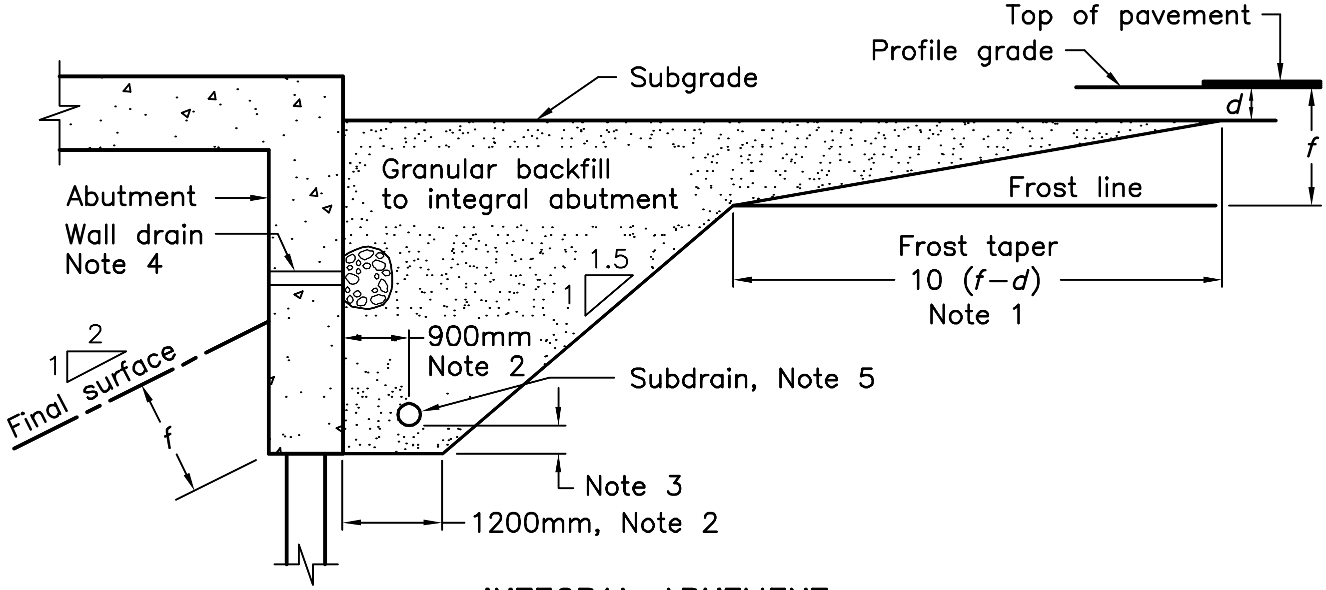
A These values are approximate and should only be used where the recommendations of a geotechnical engineer are not available.

B This information is based on the Ministry of Transportation and Communications Research Publication RR225 "Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures" dated December 1981.

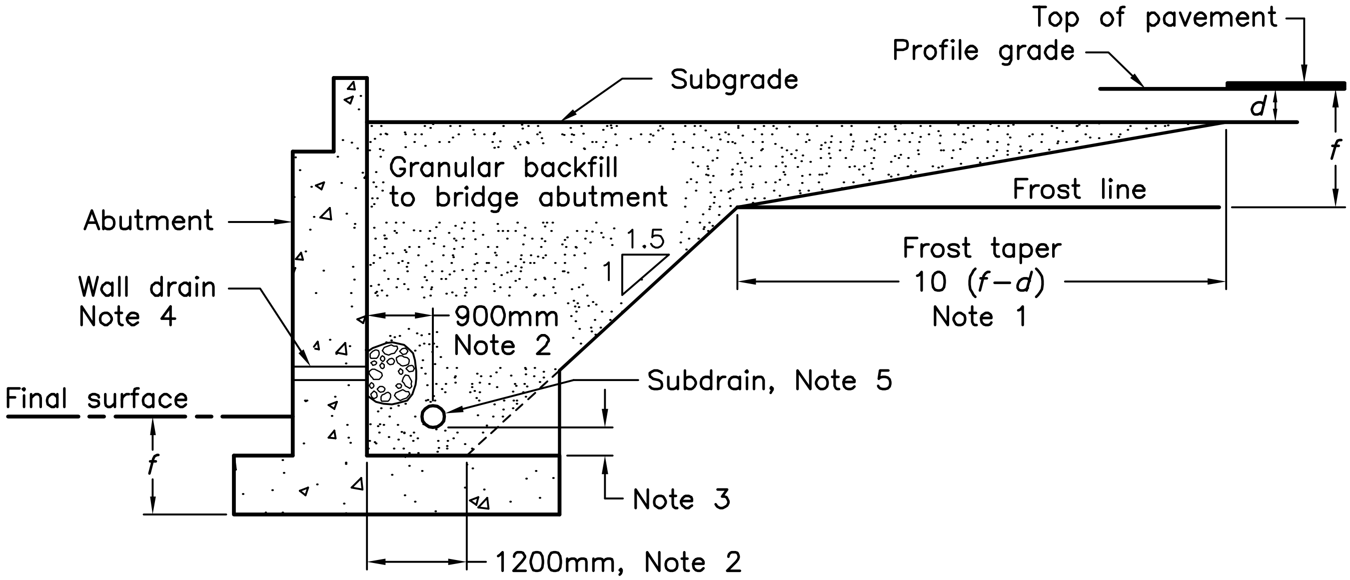
C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.

D Frost penetration depths are in metres.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
FOUNDATION FROST PENETRATION DEPTHS FOR SOUTHERN ONTARIO	OPSD 3090.101	



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

1 d = depth of combined base and subbase courses
  f = frost penetration depth as specified
2 Dimensions perpendicular to back face of abutment.
3 Height to be consistent with positive drainage of subdrain as specified.
4 Where specified, wall drains shall be installed according to OPSD 3190.100.
5 150mm dia perforated pipe subdrain wrapped with geotextile.
A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the backfill unless interrupted by the retaining wall or wingwall.
B Sections shown are parallel to centreline of roadway.
C Subdrain shall be installed with a 2% gradient behind wall.
D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
WALLS ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT	OPSD 3101.150	

Figures

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

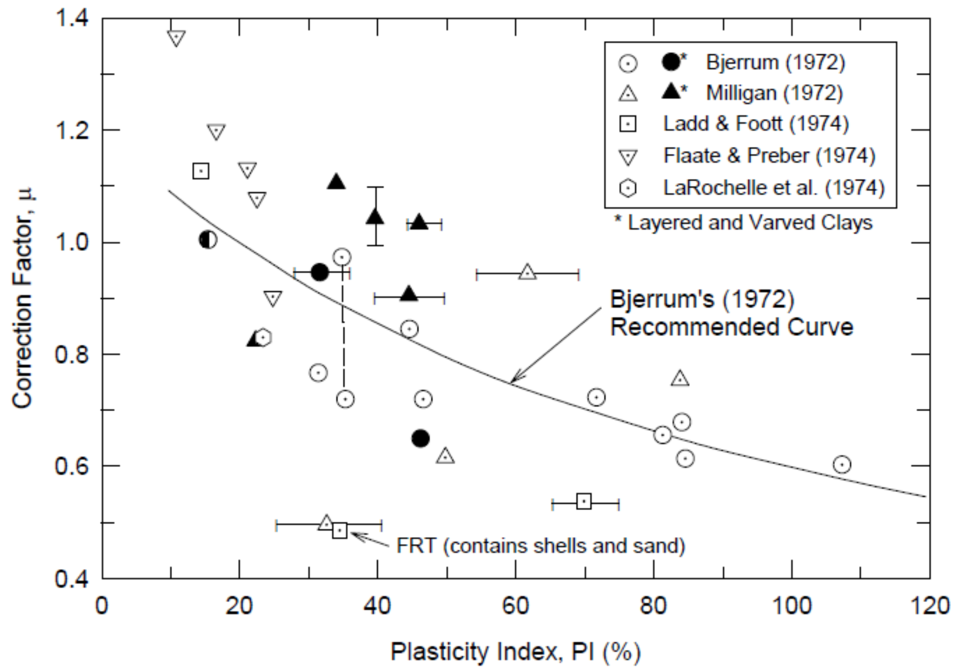


Figure 3-2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays
(Ladd & DeGroot, 2004)

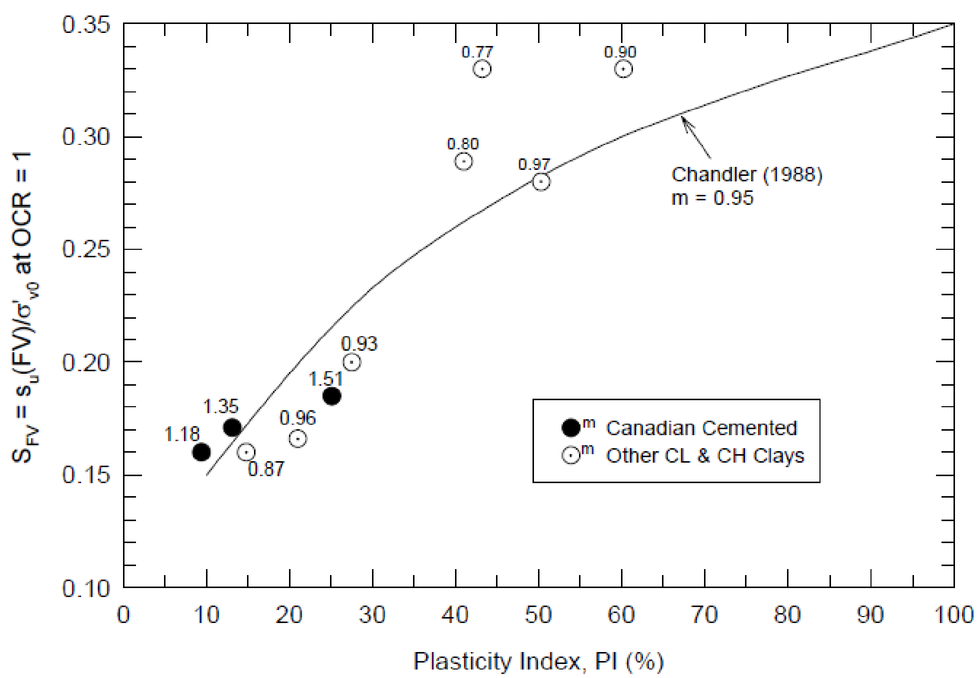
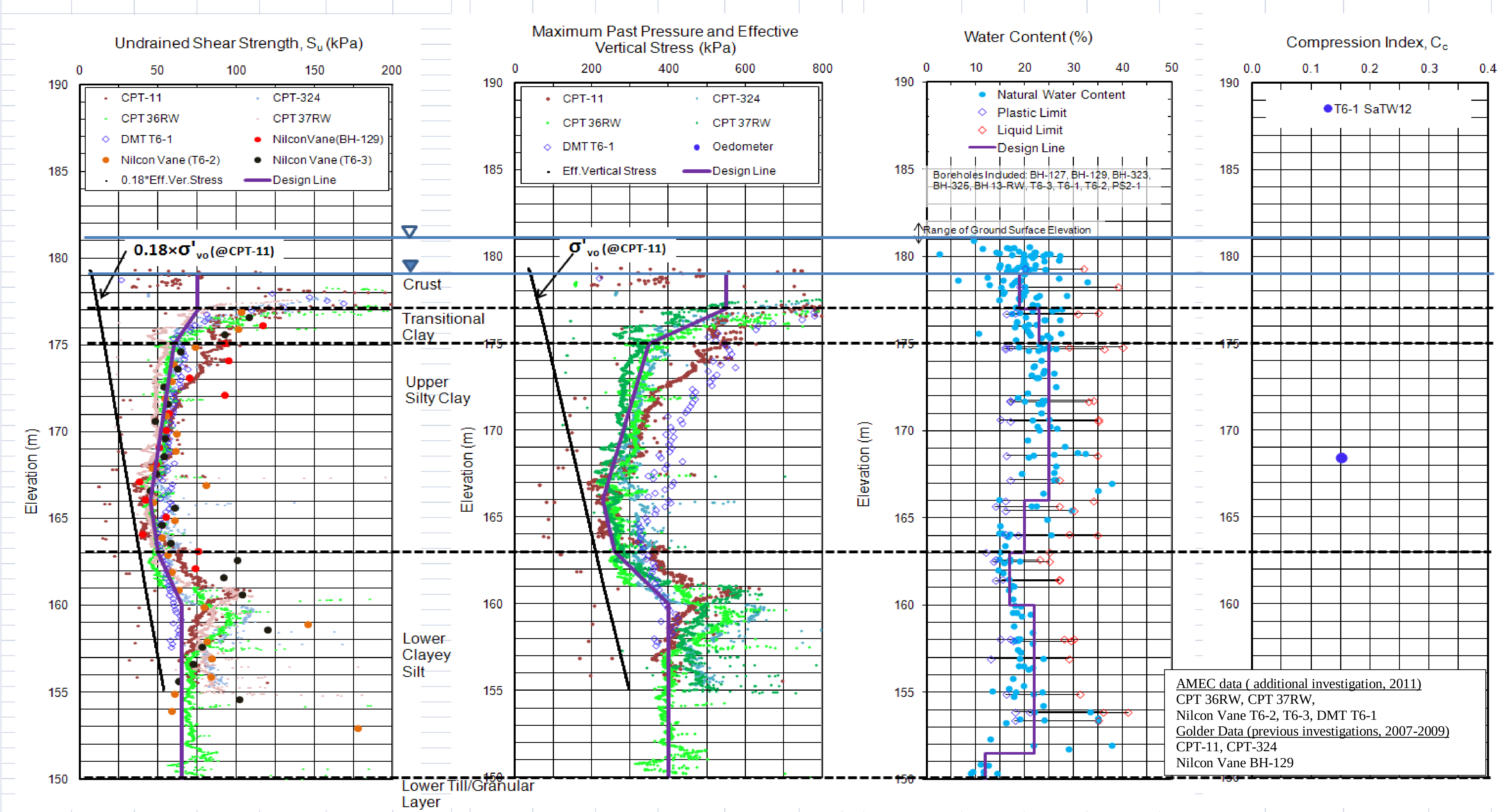


Figure 3-3: Soil Property Profiles for Tunnel T-6



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)

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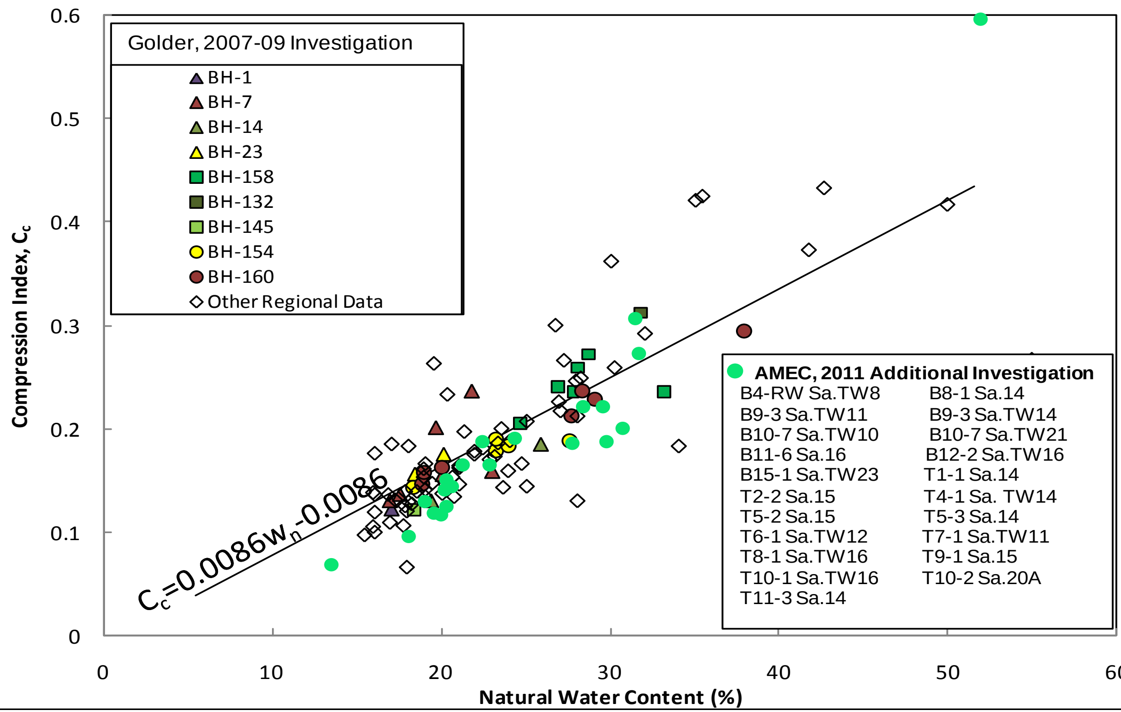
PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	SOIL PROPERTIES PROFILES TUNNEL T-6
CLIENT:	
DATE: Jul 2012	JOB NO.: SW8801.1002
CAD FILE:	FIGURE NO.:
REV:	

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona Infraestructuras | DRAGADOS | FLUOR

Figure 4-1: Compressibility Parameters at WEP



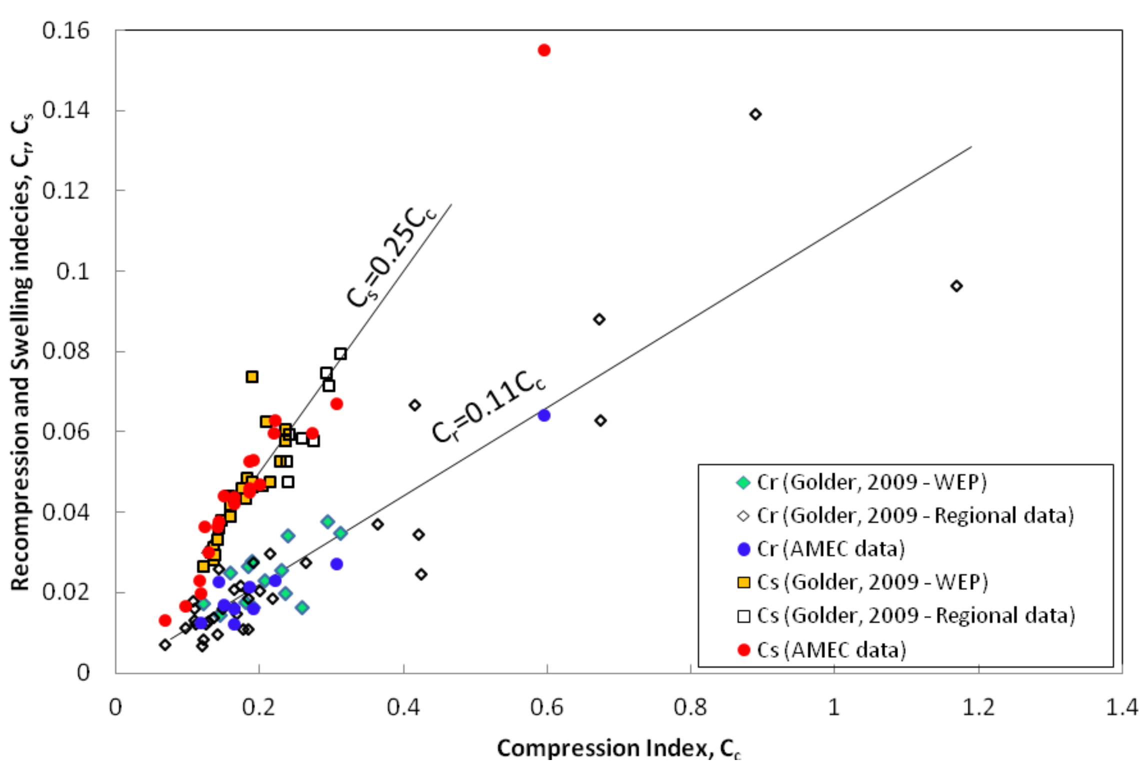


Figure 4-2: C_c versus C_α Relationship at WEP

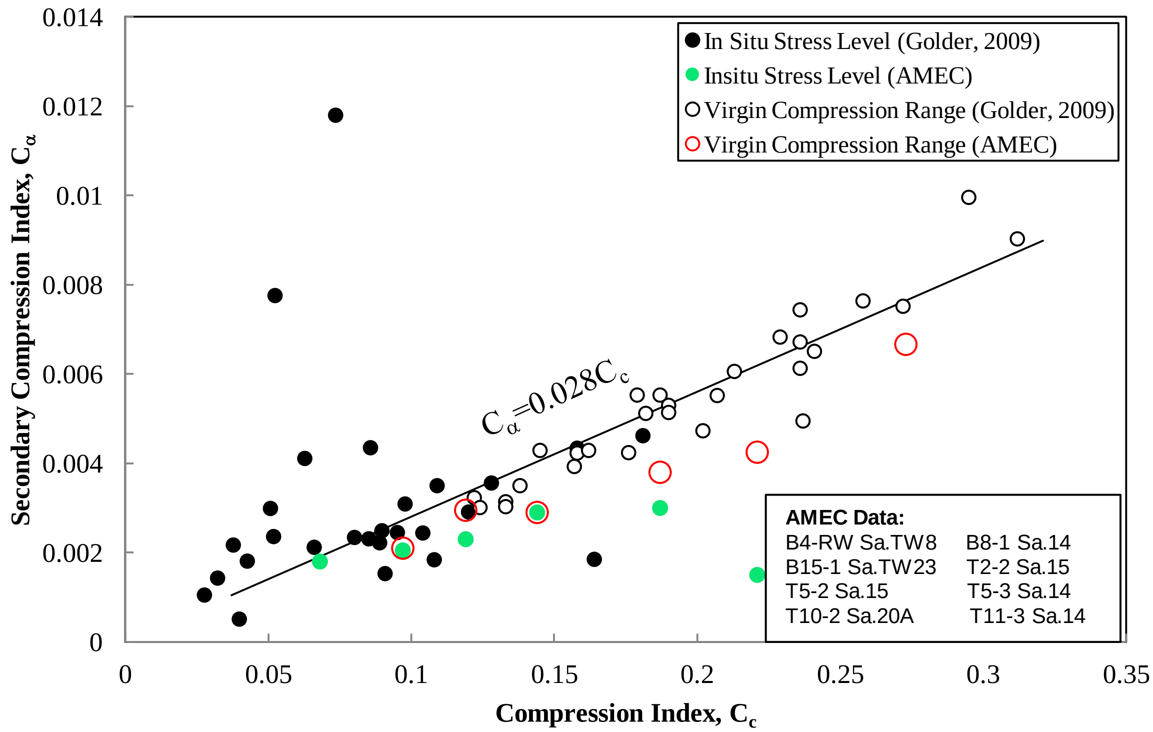
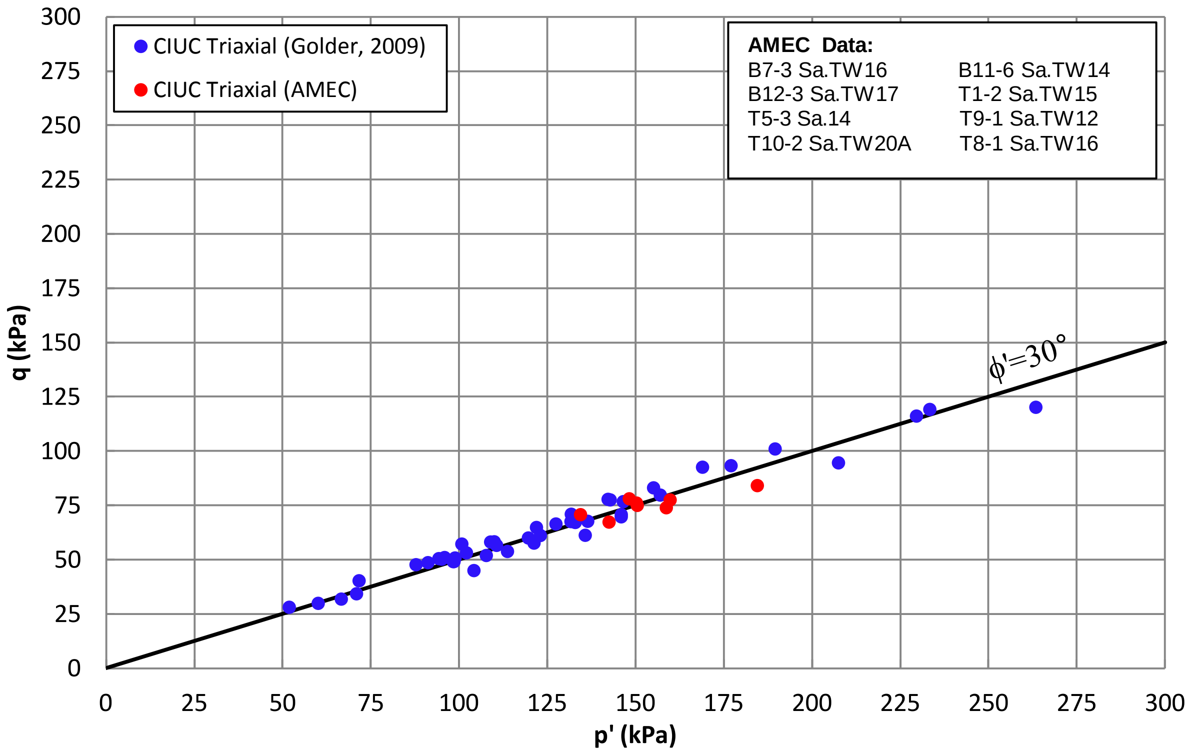


Figure 4-3: Effective Friction Angle (ϕ’) for Silty Clay to Clayey Silt Stratum at WEP



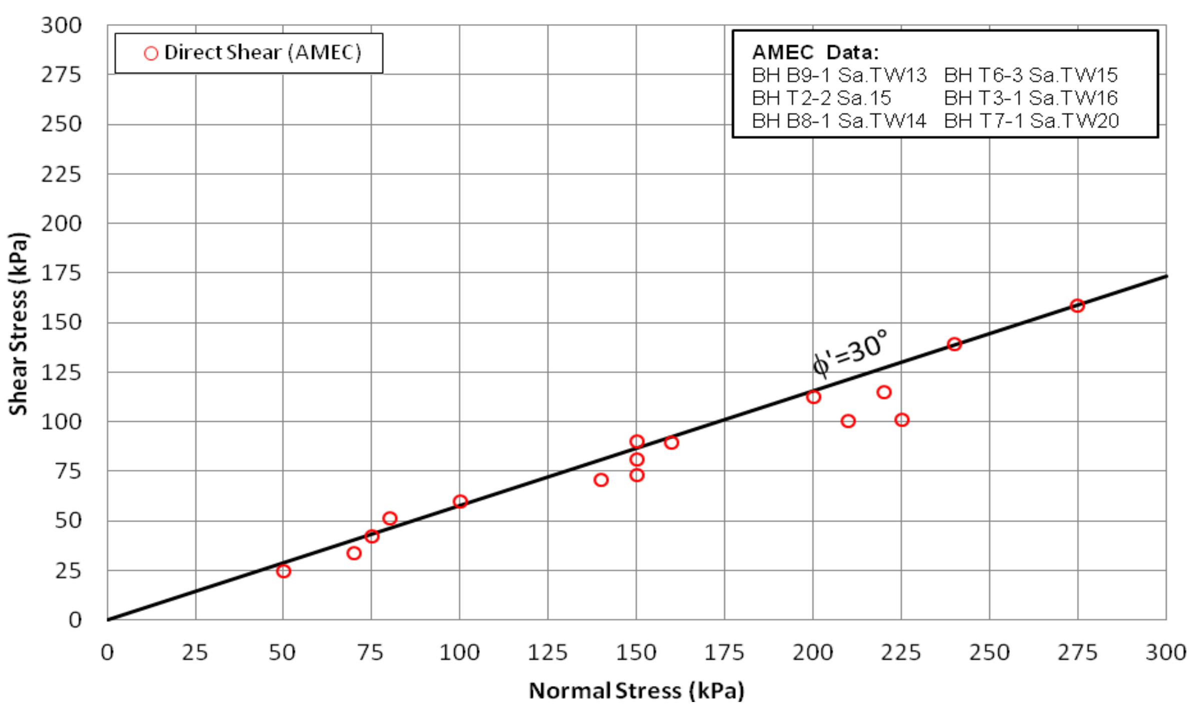


Figure 4-4: Relationship between sin ϕ' and Plasticity Index for Normally Consolidated Soils

(Kenney, 1959)

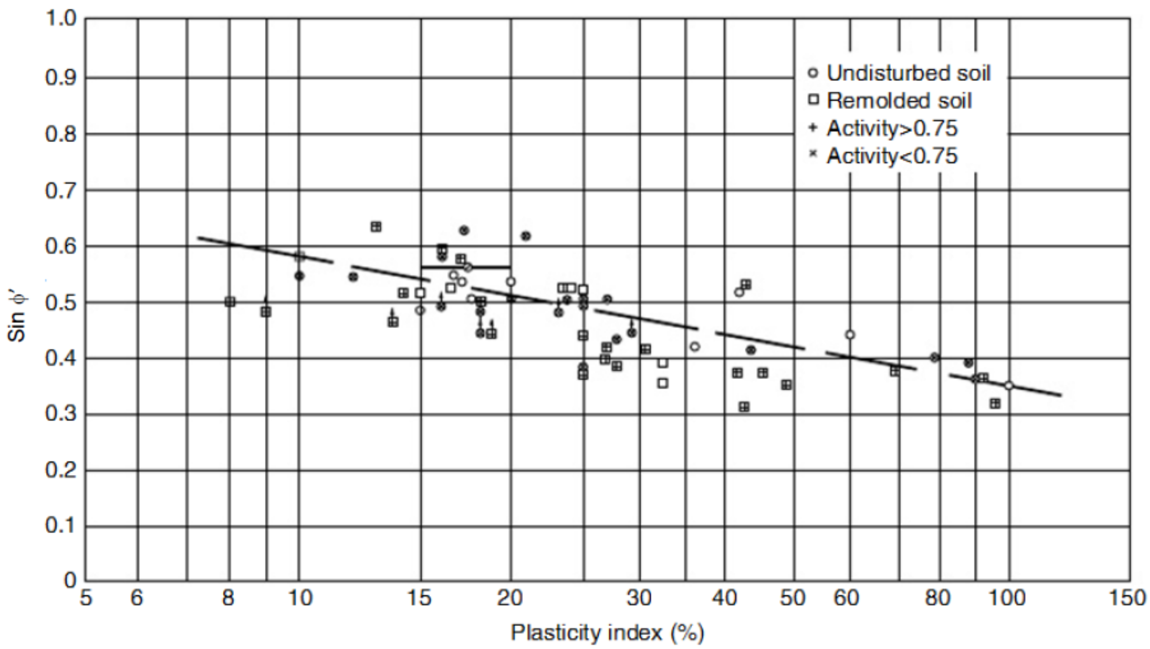
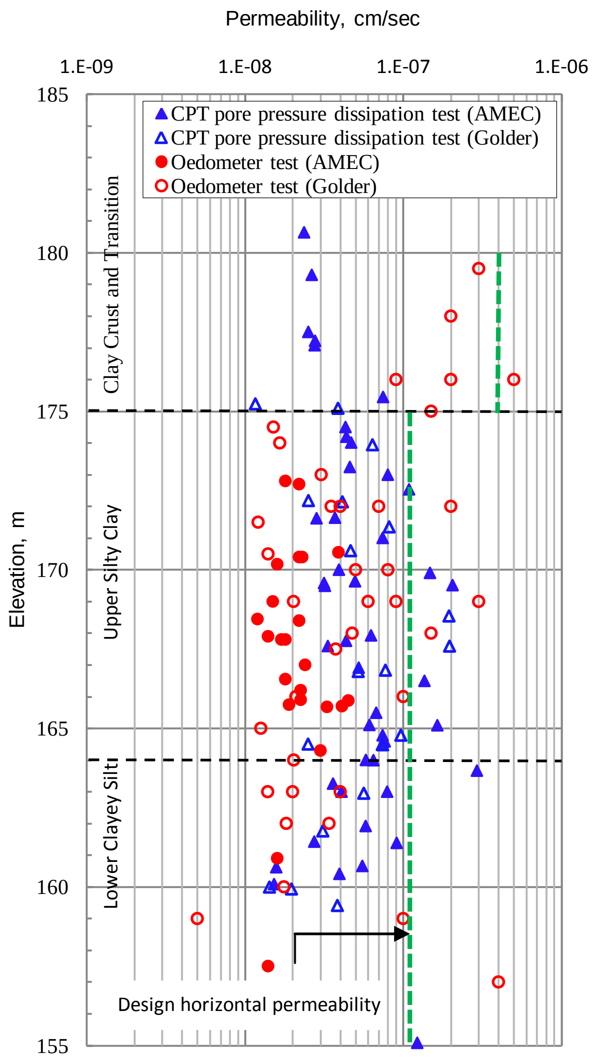
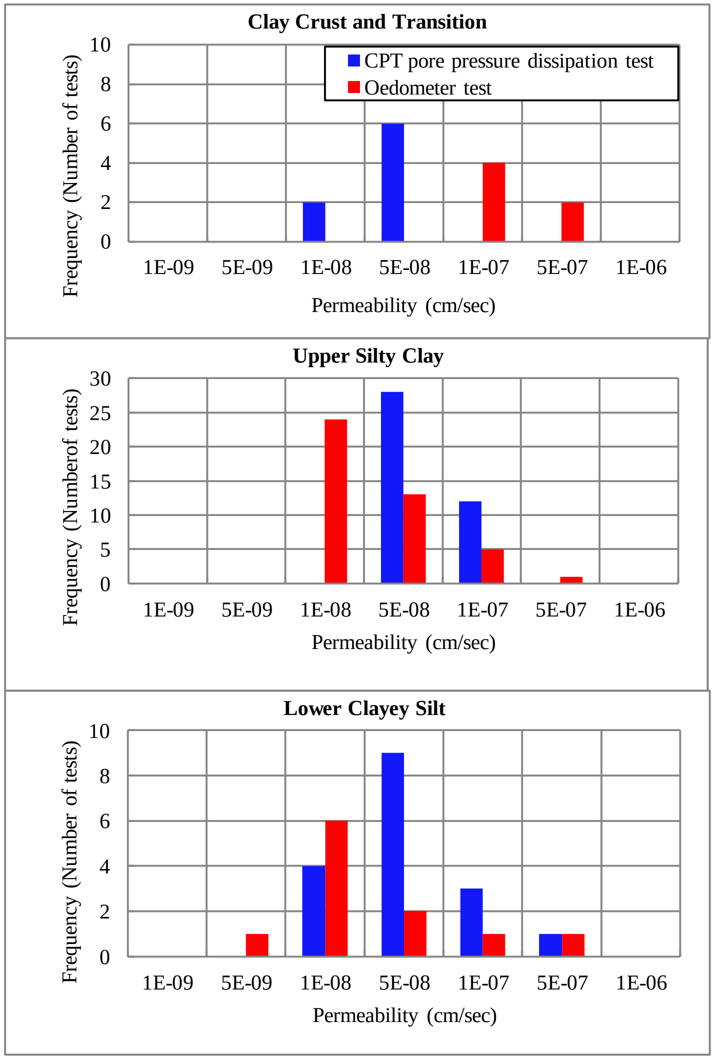


Figure 4-5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests





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

EXPLANATION OF BOREHOLE LOG

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

GENERAL INFORMATION

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

SOIL LITHOLOGY

Elevation and Depth

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

Lithology Plot

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

Description

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

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

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

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

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

Soil Sampling

Sample types are abbreviated as follows:

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

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

Field and Laboratory Testing

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

Instrumentation Installation

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

Comments

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

BEDROCK DESCRIPTION

STRENGTH CLASSIFICATION

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

JOINT SPACING CLASSIFICATION

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

ROCK QUALITY CLASSIFICATION

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

Reference: Deere et al, 1967

WEATHERING CLASSIFICATION

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

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

TERMINOLOGY

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

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

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

MTC SOIL CLASSIFICATION
Based on MTC Soil Classification Manual

Ontario

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

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

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

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

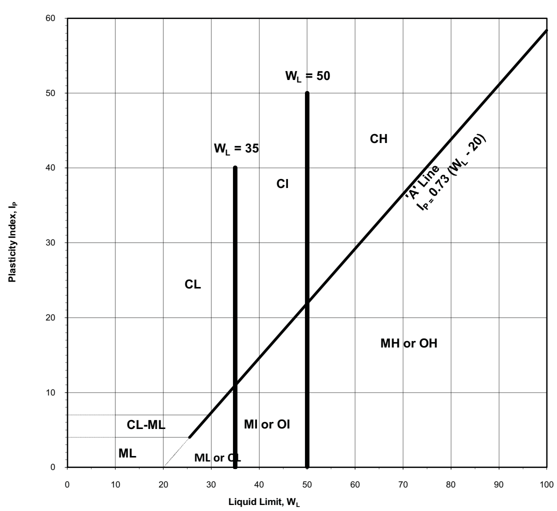
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75μm) — SILT AND CLAYS — IDENTIFICATION PROCEDURE ON FRACTION SMALLER THAN 425μm

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

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

OVERSIZED MATERIAL

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



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

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

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MTC SOIL CLASSIFICATION MANUAL

ENGINEERING PROPERTIES OF SOIL

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

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RECORD OF BOREHOLE No T6-1/HG-MW-07

METRIC

W.P. RFP No. 09-54-1007	LOCATION	N4679627.0, E332067.4	ORIGINATED BY DG
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE	14 Jul 11 - 15 Jul 11	CHECKED BY MSO

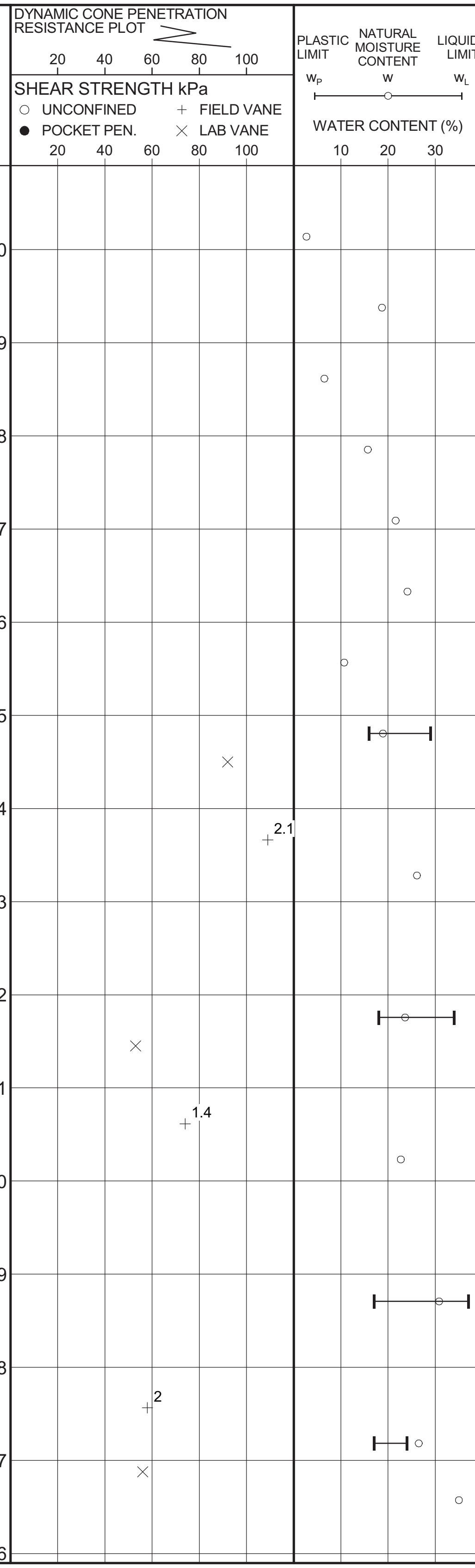
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
180.9	Ground Surface
0.0	**FILL** Sand and gravel, some silty clay clumps Brown
177.9 / 3.0	**CLAYEY SILT** Trace sand, trace fine-medium gravel Very stiff to very soft trace pink nodules Grey

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ (kN/m³)	GR	SA	SI	CL
1	SS	10					
2	SS	6					
3	SS	2					
4	SS	18					
5	SS	7					
6	SS	6					
7	SS	6					
8	TW	PH	20.2	1	19	41	39
	VT						
9	SS	4					
10	TW	PH	19.8	3	19	35	43
	VT						
11	TW	PH					
12	TW	PH		1	6	55	38
	VT						
13	TW	PH	19.6	1	14	51	34



REMARKS & GRAIN SIZE DISTRIBUTION (%)

-Vibrating Wire Piezometers (VWP) installed in sampled borehole
-Observation Well installed in adjacent boring at (coordinates)
-Spider magnets (MG) installed in adjacent boring at (coordinates)

-no recovery with shelby tube; sample retrieved by pushing split spoon
-end of drilling July 14; continued July 15
-corrosivity sample

-switched to wash boring at 9.6m below ground surface (EL. 171.3m)

-MG T6-1-SM11 installed at 11.28m below ground surface (El. 169.6 m)
-VWP T6-1-P11 installed at 11.43m (El. 169.5 m)

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

RECORD OF BOREHOLE No T6-1/HG-MW-07

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4679627.0, E332067.4	ORIGINATED BY	DG
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	14 Jul 11 - 15 Jul 11	CHECKED BY	MSO

SOIL PROFILE: ELEV / DEPTH; DESCRIPTION; STRAT PLOT

CLAYEY SILT
Trace sand, trace fine-medium gravel
Very stiff to very soft
trace pink nodules
Grey *(continued)*

SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
14	TW	PH						
	VT							
15	TW	PH						
16	TW	PH	21.6	3	26	45	25	
								-MG T6-1-SM21 installed at 21.73m below ground surface (El. 159.2 m)
	VT							
17	TW	PH						
18	SS	6						-no recovery with shelby tube; sample retrieved by pushing split spoon
	VT							
19	TW	PH	20.9	3	22	44	30	
20	SS	0						-no blow counts, weight of rods pushed 0.45m
21	SS	7						
22	SS	1						
23	SS	1						

GROUND WATER CONDITIONS

ELEVATION SCALE: 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153, 152, 151

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

Field vane sensitivity values: 1.8; 1.9; >> 2.5

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

Continued Next Page

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

RECORD OF BOREHOLE No T6-1/HG-MW-07

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4679627.0, E332067.4		ORIGINATED BY DG
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 14 Jul 11 - 15 Jul 11		CHECKED BY MSO

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT Trace sand, trace fine-medium gravel Very stiff to very soft trace pink nodules Grey *(continued)*	24	SS	27		150	
149.2 / 31.7	**COBBLES AND BOULDERS** Weathered Limestone (inferred from rock fragments retrieved by split spoon)	25	SS	50/ 100mm		149	-installed VWP T6-1-P32 at 32m (El. El. 148.9 m)
						148	
147.2 / 33.7	**LIMESTONE** Fine Grained, laminated Non-calcareous black colour inclusions, calcite mineralization is visible, stylolites present Fractured at location between 32.7m-32.9m and 35.3m-35.5m. Fractures are running parallel to the core length Brown	26	SS	50/ 150mm		147	
		27	RC			146	RQD = 77% TCR = 100% SCR = 80%
145.4 / 35.5	**END OF BOREHOLE**					145	

Plot column 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; STRAT PLOT.

Elevation scale: 150, 149, 148, 147, 146, 145, 144, 143, 142, 141, 140, 139, 138, 137, 136

No groundwater observed during drilling due to wash boring

Observation Well was dry on July 23, 2011
Water level measured in Observation Well at elevation 180.0m on July 29, 2011
Water level measured in Observation Well at elevation 180.2m on August 6, 2011
Water level measured in Observation Well at elevation 180.5m on August. 29, 2011

Water level measured in Piezometer VWP T6-1-P11 at elevation 182.2m on July 23, 2011
Water level measured in Piezometer VWP T6-1-P11 at elevation 182.0m on July 29, 2011
Water level measured in Piezometer VWP T6-1-P11 at elevation 181.6m on August 6, 2011
Water level measured in Piezometer VWP T6-1-P11 at elevation 181.1m on August 29, 2011

Water level measured in Piezometer VWP T6-1-P32 at elevation 178.8m on July 23, 2011
Water level measured in Piezometer VWP T6-1-P32 at elevation 178.7m on July 29, 2011
Water level measured in Piezometer VWP T6-1-P32 at elevation 178.7m on August 6, 2011
Water level measured in Piezometer VWP T6-1-P32 at elevation 178.8m on August 29, 2011

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

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RECORD OF BOREHOLE No T6-2

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4679659.9, E332018.8	ORIGINATED BY DG
DIST ____ HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 18 Jul 11 - 19 Jul 11	CHECKED BY MSO

SOIL PROFILE

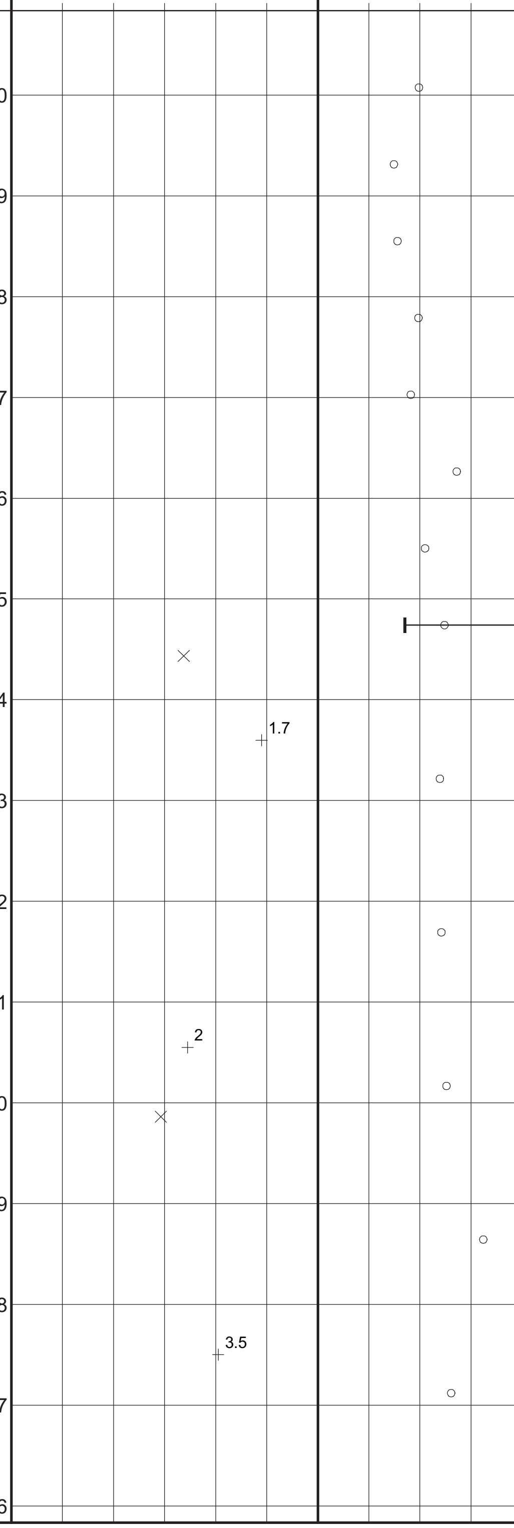
ELEV / DEPTH	DESCRIPTION
180.8	Ground Surface
180.7 / 0.0	152mm **TOPSOIL**
0.2	**CLAYEY SILT** Trace sand, trace fine-medium gravel, trace pink nodules Soft to very stiff Mottled brown and grey Grey
169.3 / 11.6	**CLAYEY SILT** Soft to firm Grey
166.2 / 14.6	

Continued Next Page

SAMPLES AND TEST RESULTS

NUMBER	TYPE	"N" VALUES	Unit Weight γ (kN/m³)	Grain Size Distribution (%) GR SA SI CL
1	SS	11		
2	SS	15		
3	SS	18		
4	SS	13		
5	SS	7		
6	SS	6		
7	SS	3		
8	TW	PH	20.3	1 17 36 46
	VT			
9	TW	PH		
10	TW	PH		
	VT			
11	TW	PH	20.2	
12	TW	PH		
	VT			
13	SS	0		

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (180 to 166); 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.



REMARKS

-Slope inclinometer casing installed in sampled borehole; Vibrating Wire Piezometers installed in adjacent boring at N4679627.0, E332068.1
Nilcon vane advanced adjacent to sampled borehole from 4 m to 28 m (El. 176.9 m to El. 152.9 m)
-drove 50.8mm spoon to get sample for PIC
-took 1.4 dioxane sample

-samples retained for environmental background

-switched to wash boring at 6.7m below ground surface (EL. 174.2m)

-VWP T6-2-P11 installed at 11.4m below ground surface (El. 169.5 m)

-sampler sank 0.45m under weight of rods
-lost shelby sample

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

RECORD OF BOREHOLE No T6-2

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679659.9, E332018.8 ORIGINATED BY DG

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

DATUM Geodetic DATE 18 Jul 11 - 19 Jul 11 CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
	CLAYEY SILT Some sand, trace gravel Stiff Pink-grey *(continued)*	14	TW	PH			21.4	2	26	42	30	
					165							
			VT			1.3						
164.2 16.6	**CLAYEY SILT** Some sand, trace gravel Very soft to very stiff Grey				164							
		15	TW	PH								
					163							
		16	TW	PH								-VWP T6-2-P18 installed at 18.3m below ground surface (El. 162.6 m)
					162							
			VT			2.1						
					161							
		17	TW	PH			21.4					
					160							
		18	TW	PH								
					159							
			VT			>> 2.2						
					158							
		19	TW	PH			20.8	6	23	41	30	
					157							
		20	SS	4								
					156							
					155							
		21	SS	4								
					154							
		22	SS	0								-sampler sank 0.45m under weight of rods
					153							
					152							
		23	SS	1								
					151							

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DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

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

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Foundation Design

RECORD OF BOREHOLE No T6-2

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679659.9, E332018.8 ORIGINATED BY DG

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

DATUM Geodetic DATE 18 Jul 11 - 19 Jul 11 CHECKED BY MSO

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS
150.4 / 30.5	**SANDY SILT** Trace fine-medium gravel, some clay Compact Grey Saturated	24	SS	20	end of driling July 18; continued July 19
148.3 / 32.6	**LIMESTONE** Fine Grained, rubble, dense, grey -Rock/boulder fragments and pieces	25	RC		
147.4 / 33.5	**LIMESTONE** Fine Grained, laminated, porous, stylolites, grey	26	RC		RQD = 87% TCR = 100% SCR = 90%
146.1 / 34.8	**END OF BOREHOLE**				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (150 to 136); 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

END OF BOREHOLE

No groundwater observed during drilling due to wash boring

Water level measured in Piezometer VWP T6-2-P11 at elevation 180.8m on July 23, 2011
Water level measured in Piezometer VWP T6-2-P11 at elevation 180.5m on August 6, 2011
Water level measured in Piezometer VWP T6-2-P11 at elevation 180.6m on August 29, 2011

Water level measured in Piezometer VWP T6-2-P18 at elevation 180.6 on July 23, 2011
Water level measured in Piezometer VWP T6-2-P18 at elevation 180.3m on August 6, 2011
Water level measured in Piezometer VWP T6-2-P18 at elevation 180.4 on August 29, 2011

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 07/08/12

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

RECORD OF BOREHOLE No T6-3

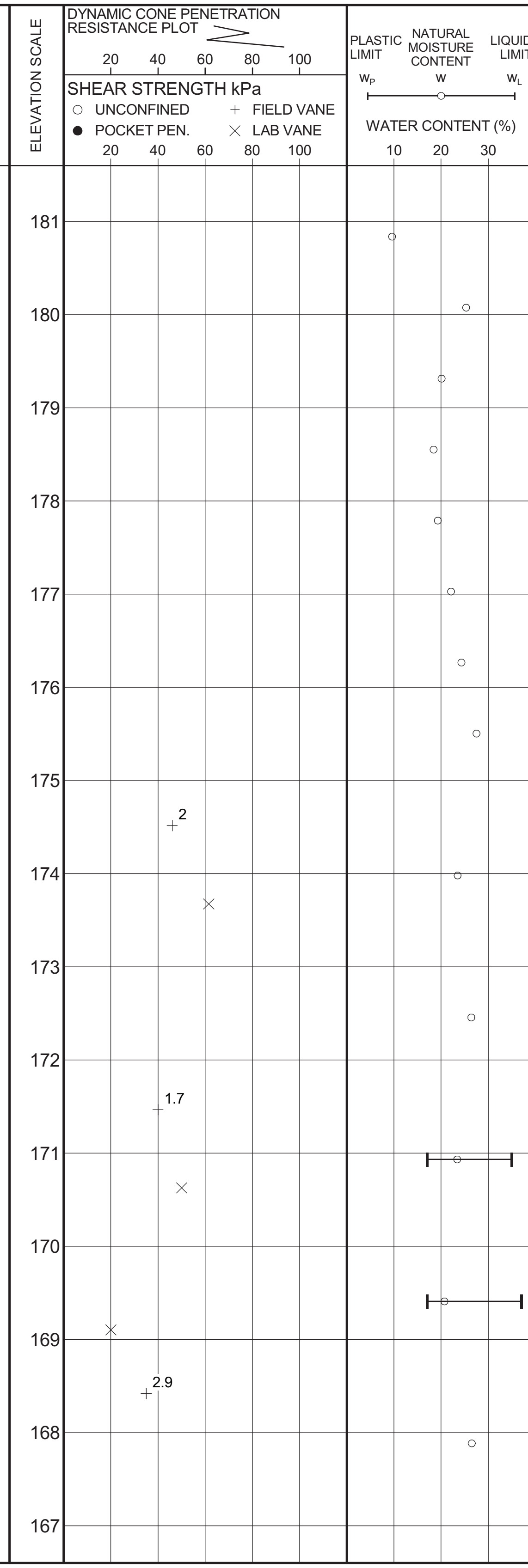
METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679577.5, E332079.1 ORIGINATED BY SD

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

DATUM Geodetic DATE 14 Jul 11 - 19 Jul 11 CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	REMARKS
181.6	Ground Surface						
0.0	**FILL** Topsoil/sand/silt mixture, trace gravel and bricks, brown to black	1	SS	7			Nilcon vane advnaced adjacent o sampled borehole from 5 m to 27 m depth (El. 176.6 m to El. 154.6 m)
180.4 / 1.2	**CONCRETE**						
180.1 / 1.5	**FINE SAND** Trace gravel Brown						
179.9 / 1.7	**CLAYEY SILT** Some sand, trace gravel Soft to stiff Mottled brown and grey Trace pink nodules and moist to wet below approx. 4 m Brown	2	SS	6			
		3	SS	16			
		4	SS	18			
	Grey	5	SS	12			
		6	SS	8			
		7	SS	5			
		8	TW	PH			
			VT				
		9	TW	PH	19.9		
	-Sandy pocket						
		10	SS	3			
			VT				-corrosivity sample
		11	TW	PH	20.3	0 15 48 37	
		12	TW	PH	21.2	1 6 54 39	
168.6 / 13.0	**CLAYEY SILT** Soft Trace black and pink inclusions, varved Grey						
			VT				
		13	TW	PH			



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

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RECORD OF BOREHOLE No T6-3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679577.5, E332079.1 ORIGINATED BY SD

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

DATUM Geodetic DATE 14 Jul 11 - 19 Jul 11 CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH / VANE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT Soft Trace black and pink inclusions, varved Grey *(continued)*	14	TW	PH	166		18.6	1 14 44 41
			VT			3.6		-end of augers at 15.2m -wash bore with NW casing
		15	TW	PH	165			2 15 41 42
163.9 17.7	**CLAYEY SILT** Some sand, trace gravel Soft to stiff Grey Moist to wet				164			
		16	TW	PH	163			
			VT		162	>> 1.8		
	-Sand pocket, wet	17	TW	PH	161		21.8	3 25 47 25
		18	TW	PH	160			
			VT		159	1.5		
		19	TW	PH	158		21.1	
		20	SS	9	157			
	-Some shale fragments	21	SS	9	156 / 155			
		22	SS	5	154 / 153			
		23	SS	11	152			-no recovery spoon blocked with gravel piece -end of drilling July 18; continue July 19

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

Column headers: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; 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; STRAT PLOT

RECORD OF BOREHOLE No T6-3

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4679577.5, E332079.1	ORIGINATED BY	SD
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	14 Jul 11 - 19 Jul 11	CHECKED BY	MSO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS
	CLAYEY SILT Some sand, trace gravel Soft to stiff Grey Moist to wet *(continued)*	24	SS		151		-N-Values not recorded
					150		
149.6 / 32.0	**SANDY GRAVELLY SILT** Dense Grey Moist	25	SS	58		○	-spoon blocked with gravel
					149		
	-Inferred cobbles	26	SS	50/ 25mm	148		-SPT refusal at 33.4m; Augers advanced to refusal at 34.7m
146.9 / 34.7	**LIMESTONE** Fine Grained, well crystallized and dense Grey-Brown	27	RC		147		RQD = 100% TCR = 100% SCR = 71%
146.5 / 35.1	**LIMESTONE** Well crystallized and dense Grey	28	RC		146		RQD = 100% TCR = 100% SCR = 82%
	LIMESTONE Fine Grained, microfractures throughout filled with solution activity Brown						
145.3 / 36.3	**END OF BOREHOLE** No groundwater observed during drilling due to wash boring				145		
					144		
					143		
					142		
					141		
					140		
					139		
					138		
					137		

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

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

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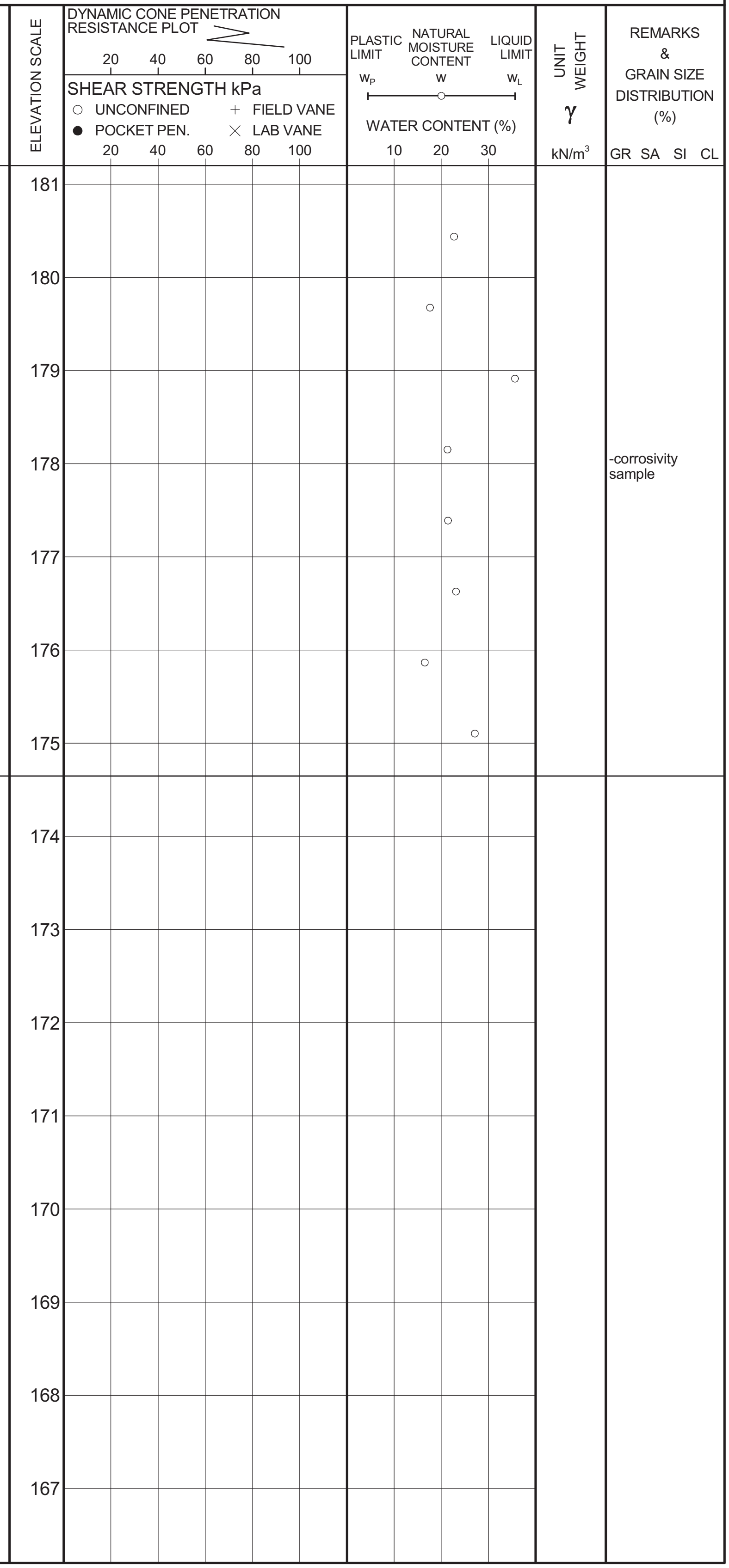
RECORD OF BOREHOLE No BH12-RW

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4679718.1, E332037.9		ORIGINATED BY SD
DIST	HWY WEP	BOREHOLE TYPE Truck Mounted Drill - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 16 Jul 11 - 16 Jul 11		CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
181.2	Fill Surface				
181.0 / 0.0	**FILL** Topsoil Black				
0.2	**FILL** Silty clay, some topsoil, brown-black to grey	1	SS	12	
		2	SS	10	
		3	SS	4	
178.3 / 2.9	**CLAYEY SILT** Some sand, trace gravel Firm to very stiff Brown to grey	4	SS	16	-corrosivity sample
		5	SS	15	
		6	SS	8	
		7	SS	6	
		8	SS	6	
174.6 / 6.6	**END OF BOREHOLE** (no refusal) Borehole dry on completion				



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

RECORD OF BOREHOLE No CPT36-RW

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4679710.0, E331968.8	ORIGINATED BY	TA
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	15 Aug 11 - 15 Aug 11	CHECKED BY	MSO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
180.5	Ground Surface			
0.0 / 180.1	**TOPSOIL**			
0.4 / 179.4	**SILTY SAND** Brown	1A, B	SS	6
1.1 / 178.4	**SILTY CLAY** Some sand, trace gravel, trace fissures Mottled brown and grey Brown	2	SS	16
2.1	**END OF SAMPLED BOREHOLE** Continued with CPT to refusal Borehole dry on completion			

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● POCKET PEN., × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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RECORD OF BOREHOLE No CPT37-RW

METRIC

W.P. RFP No. 09-54-1007 | LOCATION N4679571.4, E332146.2 | ORIGINATED BY TA

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

DATUM Geodetic | DATE 15 Jul 11 - 15 Jul 11 | CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%) (approx.)	REMARKS
180.9	Ground Surface					
0.0	**FILL** Crushed Limestone Grey					-faint hydrocarbon odour
180.4 / 0.5 (180.3 / 0.6)	**TOPSOIL**					
0.6	**SILTY CLAY** Some sand, trace gravel Mottled brown and grey	1	SS	6	○	
	Brown	2	SS	9	○ ○	
		3	SS	14	○	
		4	SS	13	○	
177.4 / 3.5	**END OF SAMPLED BOREHOLE** Continued with CPT from 3.3 m to refusal Borehole dry on completion					

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166); 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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No DMT T6-1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679696.6, E332057.3 ORIGINATED BY LC

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

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

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)
181.2	Ground Surface					
0.0	**FILL** Silty clay and topsoil. Some sand, trace gravel	1A, B	SS	11		
180.2 / 180.0	**ORGANIC CLAY** Black					
1.2	**SILTY CLAY** Some sand, trace gravel. Trace organic inclusion, weathered. Mottled brown and grey	2	SS	8		
179.2 / 2.0	**END OF SAMPLED BOREHOLE** Continued with DMT from 2.4 m to refusal at 23.4 m (El. 178.8 m to El. 157.8 m)					
	Borehole dry on completion					

SOIL PROFILE | SAMPLES | STRAT PLOT | ELEVATION SCALE (181–167)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m^3

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

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

TEST PIT STRATIGRAPHY LOG

Project Name: Geotechnical Test Pit	Contractor: Amico Infrastructure	Test Pit Designation: **TP 7**
Project No.: SW8801.1004.301		
Client: Parkway Infrastructure Engineers	Surface Elevation: **~ 180.9 m**	Date Completed: 08-15-11
Location: Windsor, ON	Test Pit Method: J Deere 470G LC	Supervisor: A Cevizli

Location: Easting 0332053, Northing 4679665

Depth *(m)* From	At	To	*Soil Symbol, Primary Component, Secondary Component, Relative Density/Consistency, Grain Size/Plasticity, Gradation/Structure, Colour, Moisture Content, Supplementary Descriptors*	*Sample No.*
			Surface: grass	
0		0.4 - 0.5	FILL clayey and TOPSOIL	
0.4 - 0.5		0.6	TOPSOIL - sandy, some roots, dark brown to black, oxidized (extended to 0.9 m on north side)	
0.6		1.0	CLAYEY SILT - mottled grey and brown (extended to 2.1 m on north side)	
1.0		3.4	SILTY CLAY - mottled brown and grey [1.5 m 26.1% MC, 41% LL, 19% PL, 22% PI] [2.5 m 22.6% MC, 36% LL, 17% PL, 19% PI]	
3.4		10.5	SILTY CLAY - with embedded sand and gravel, grey [4.2 m 23.9% MC, 42% LL, 19% PL, 23% PI] [10.5 m 26.1% MC, 36% LL, 18% PL, 18% PI]	
	10.5		End of Test Pit at ~ 170.9 m	
			bgs- below ground surface	

Photo: Looking North

NOTES:

Seepage depth: Moderate seepage on South Wall @ 1 m bgs	MC - Natural Moisture Content
Sidewall Stability: Stable	LL - Liquid Limit
Organic depth-thickness: 0.6 m of topsoil (mixed with backfill)	PL - Plastic Limit
Surface water run into excavation: No	PI - Plasticity Index
Ranges of Hand Vanes: @ 3m - >260 kPa; @4.2m - 220 kPa; @7m - 80 kPa; @8m - 62-76 kPa; @8.5m - 60-70 kPa; @10.5m - 44-56 kPa	

Completed By: P Neumann

Date: 9-20-2011

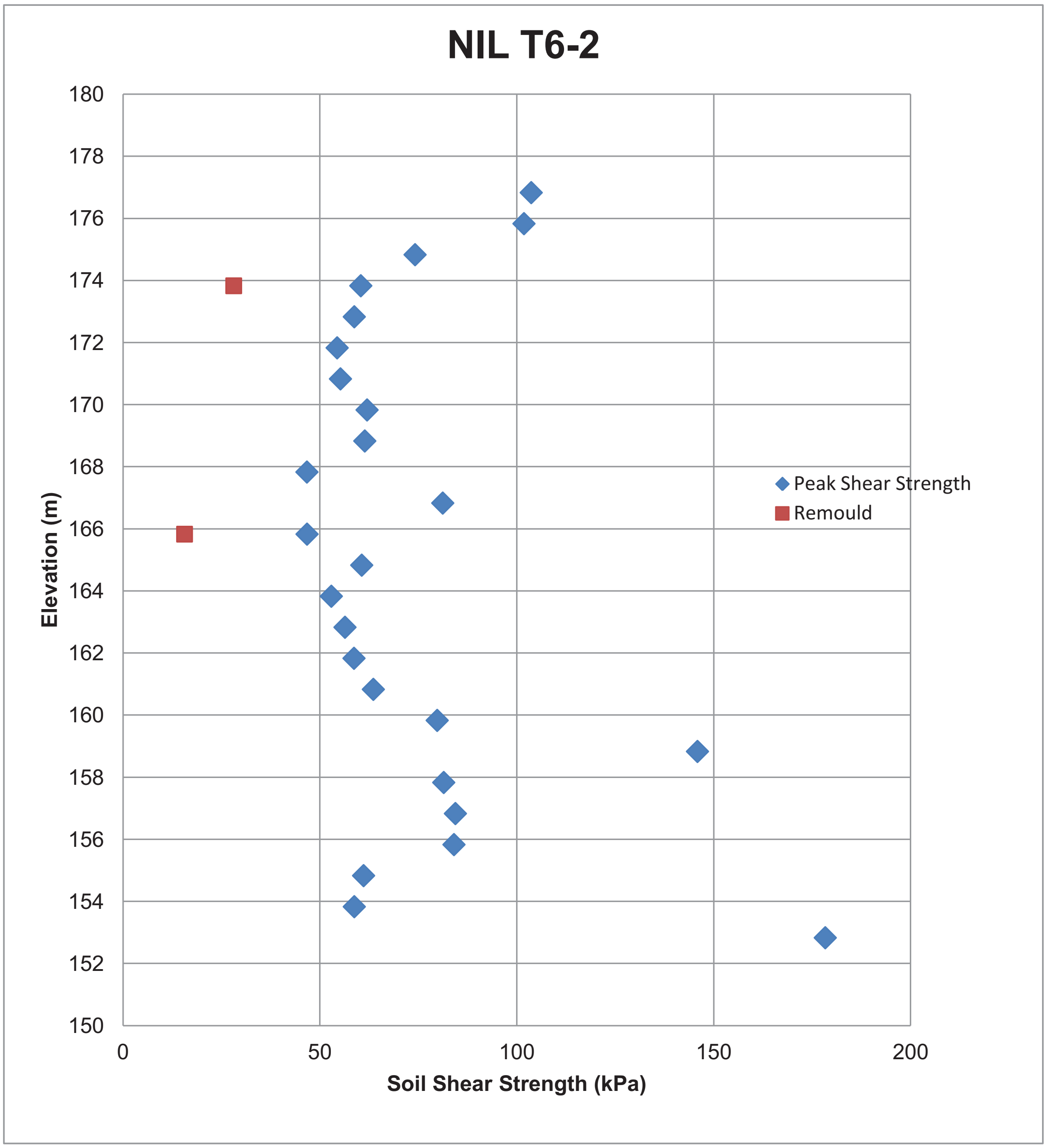
Foundation Design

RECORD OF NILCON VANE TEST NIL T6-2

Project : Windsor-Essex Parkway
Test Date: 8/12/2011
Sheet 1 of 1

Location: N4679661.8; E332020.5
Predrill Depth : 3.0 m
Datum Geodetic

Ground Surface Elevation: 180.8 m

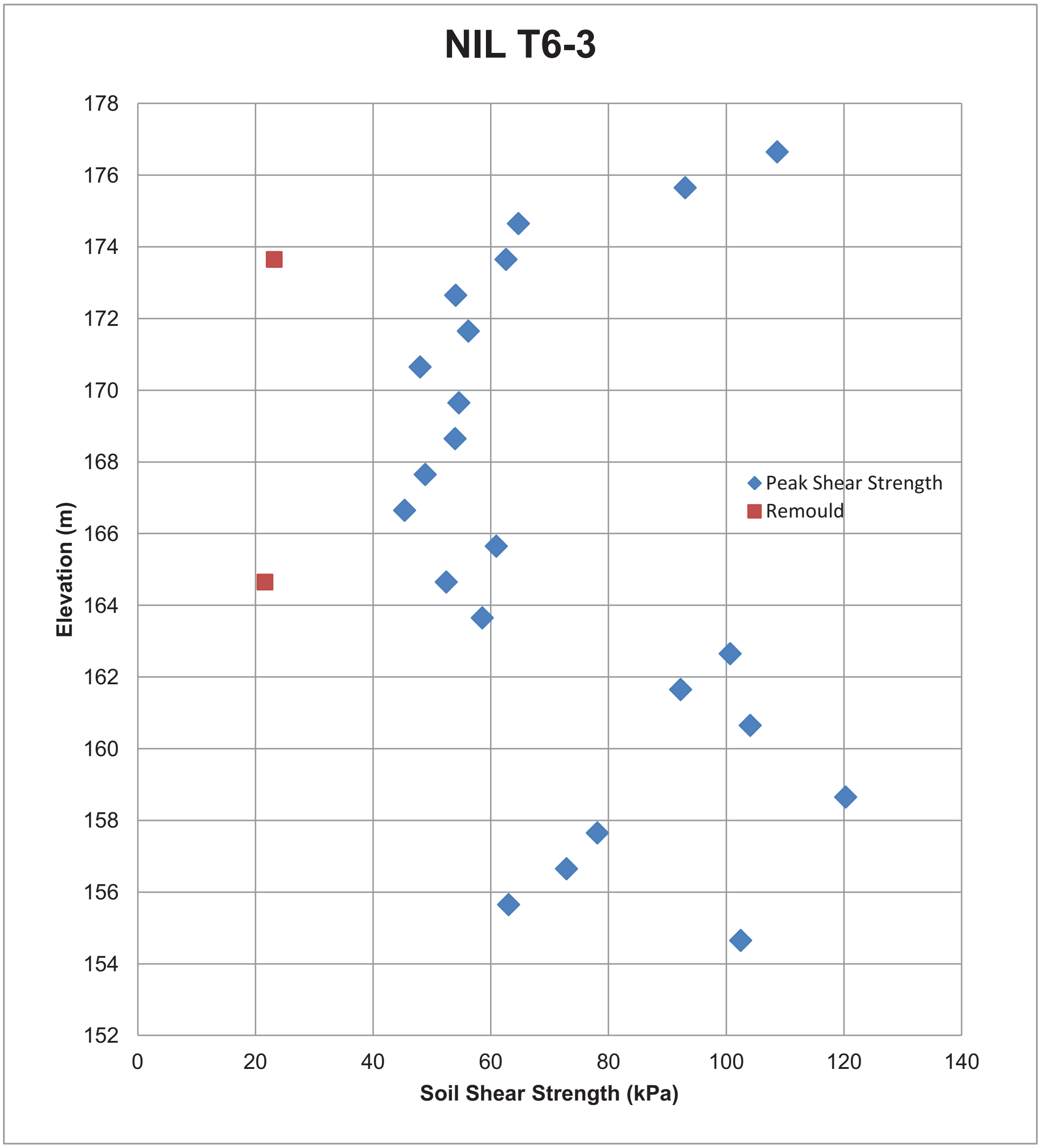


Operator: SD

Checked: DD

RECORD OF NILCON VANE TEST NIL T6-3

Project : Windsor-Essex Parkway
Test Date: 8/13/2011
Sheet 1 of 1
Location: N4679574.1; E332073.1
Predrill Depth : 4.6 m
Datum Geodetic
Ground Surface Elevation: 181.7 m



Operator: SD

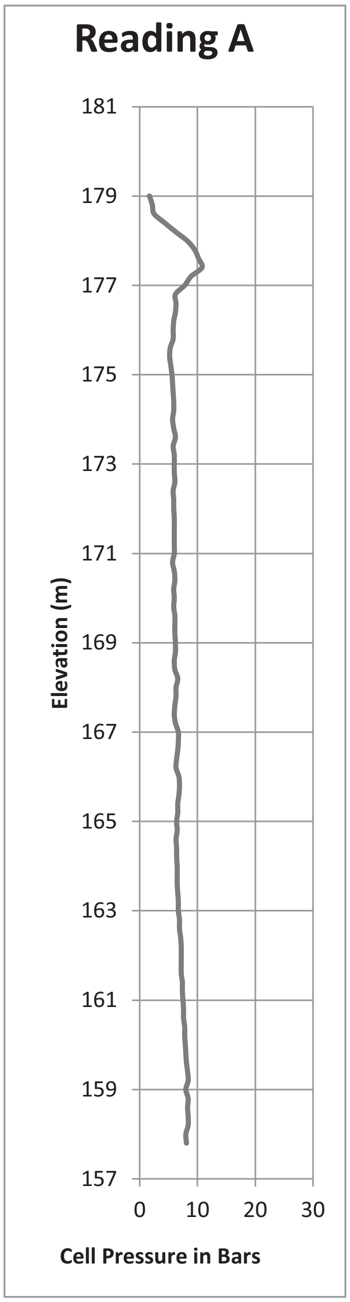
Checked: DD

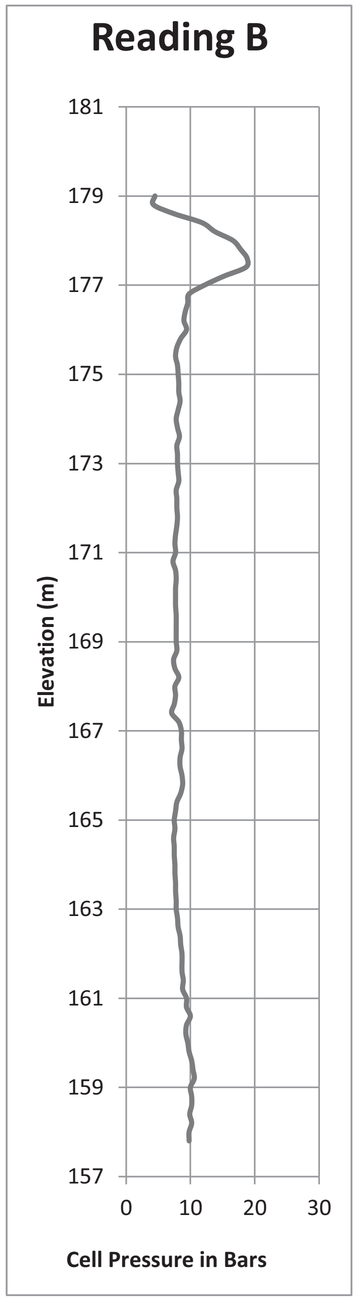
RECORD OF DILATOMETER TEST DMT T6-1

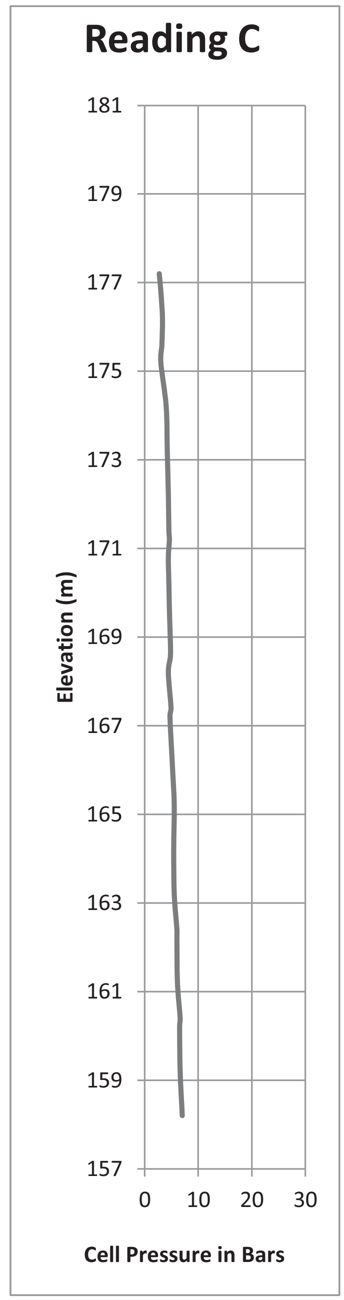
Project : Windsor-Essex Parkway
Location: N 4679696.6; E 332057.3
Ground Surface Elevation : 181.2

Test Date: 7/14/2011
Predrill Depth : 2.0 m
Delta A: 0.18 Bar

Datum Geodetic
Delta B: 0.22 Bar







Operator: LC

Checked: DD

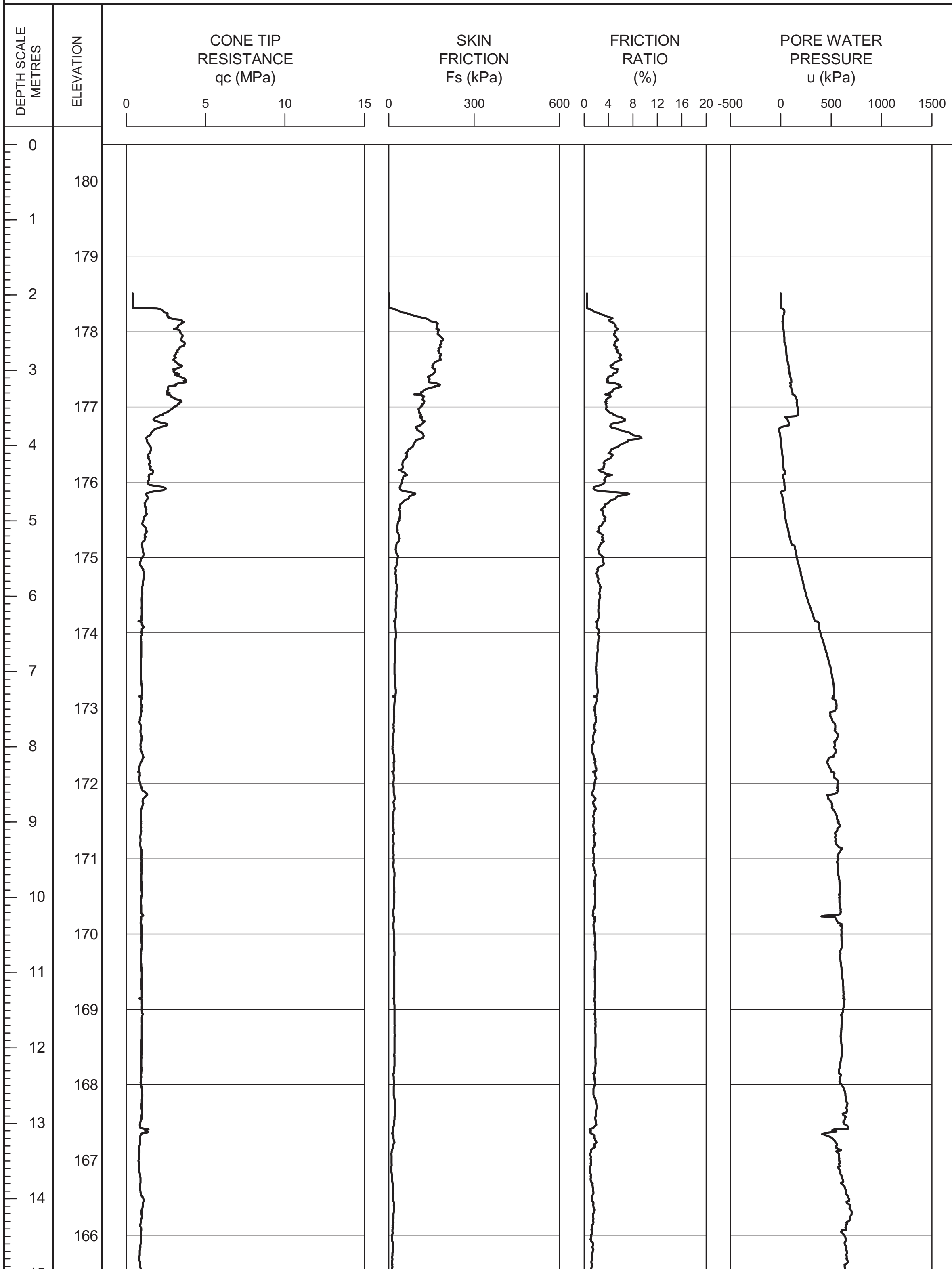
Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF CONE PENETRATION TEST CPT 36-RW

METRIC

PROJECT	Windsor-Essex Parkway	TEST DATE	8/15/2011 - 8/15/2011
LOCATION	N4679710.0; E331968.8		
SHEET	1 OF 3	DATUM	Geodetic

GROUND SURFACE ELEVATION: 180.5 PREDRILL DEPTH: 2.17 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

Continued Next Page

OPERATOR: TA

CHECKED: DD

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

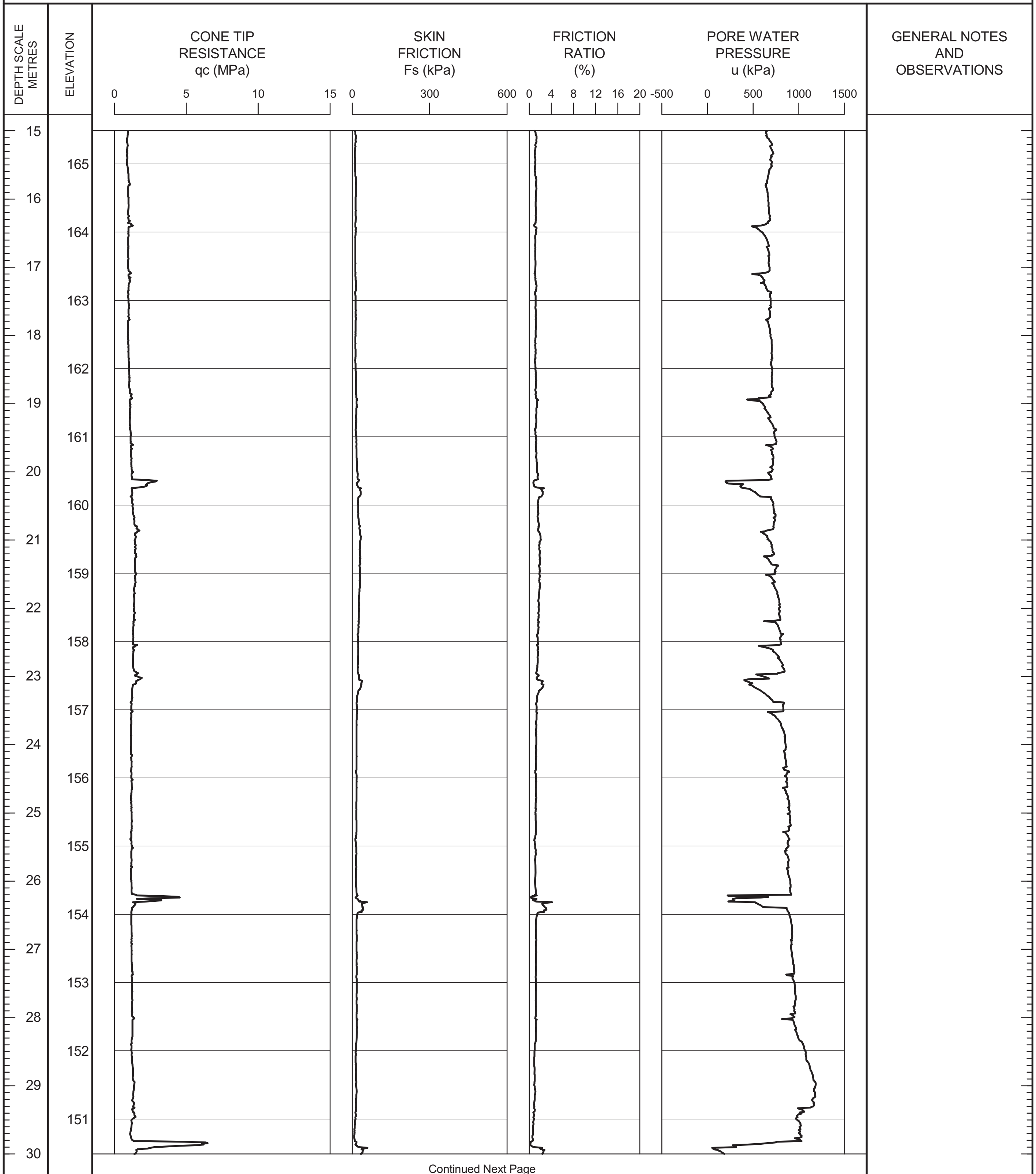
RECORD OF CONE PENETRATION TEST CPT 36-RW

METRIC

PROJECT Windsor-Essex Parkway | TEST DATE 8/15/2011 - 8/15/2011 | SHEET 2 OF 3

LOCATION N4679710.0; E331968.8 | DATUM Geodetic

GROUND SURFACE ELEVATION: 180.5 PREDRILL DEPTH: 2.17 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

CHECKED: DD

Foundation Design

RECORD OF CONE PENETRATION TEST CPT 36-RW

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 8/15/2011 - 8/15/2011

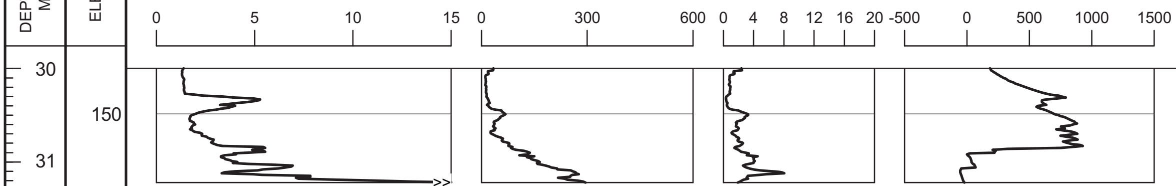
SHEET 3 OF 3

LOCATION N4679710.0; E331968.8

DATUM Geodetic

GROUND SURFACE ELEVATION: 180.5 PREDRILL DEPTH: 2.17 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0

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



OPERATOR: TA

CHECKED: DD

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

Foundation Design

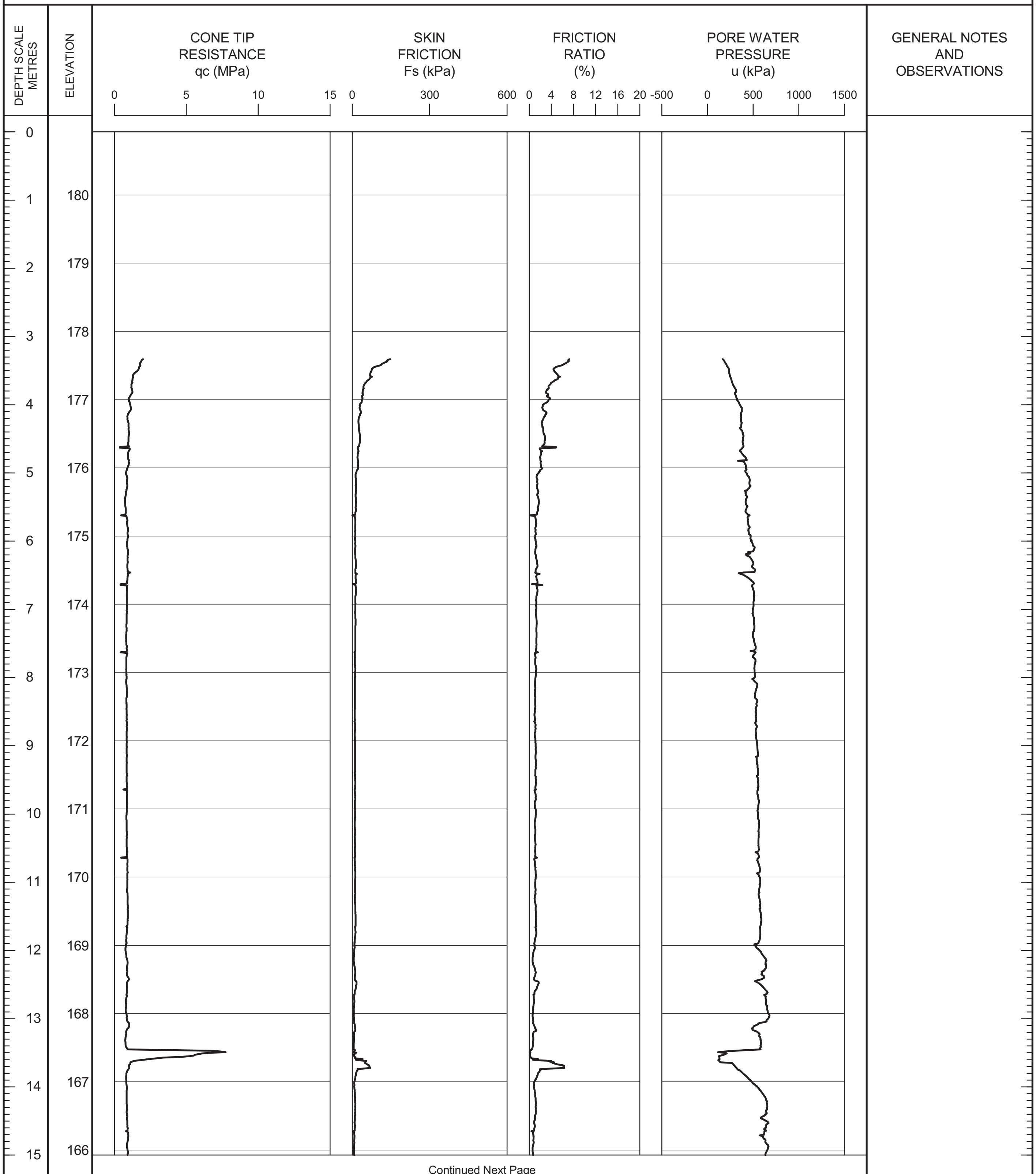
RECORD OF CONE PENETRATION TEST CPT 37-RW

METRIC

PROJECT Windsor-Essex Parkway   TEST DATE 7/27/2011 - 7/27/2011   SHEET 1 OF 2

LOCATION N4679571.4; E332146.2   DATUM Geodetic

GROUND SURFACE ELEVATION: 180.9   PREDRILL DEPTH: 2.98   CORRECTION FACTOR A: 0.8   CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 37-RW

METRIC

PROJECT Windsor-Essex Parkway

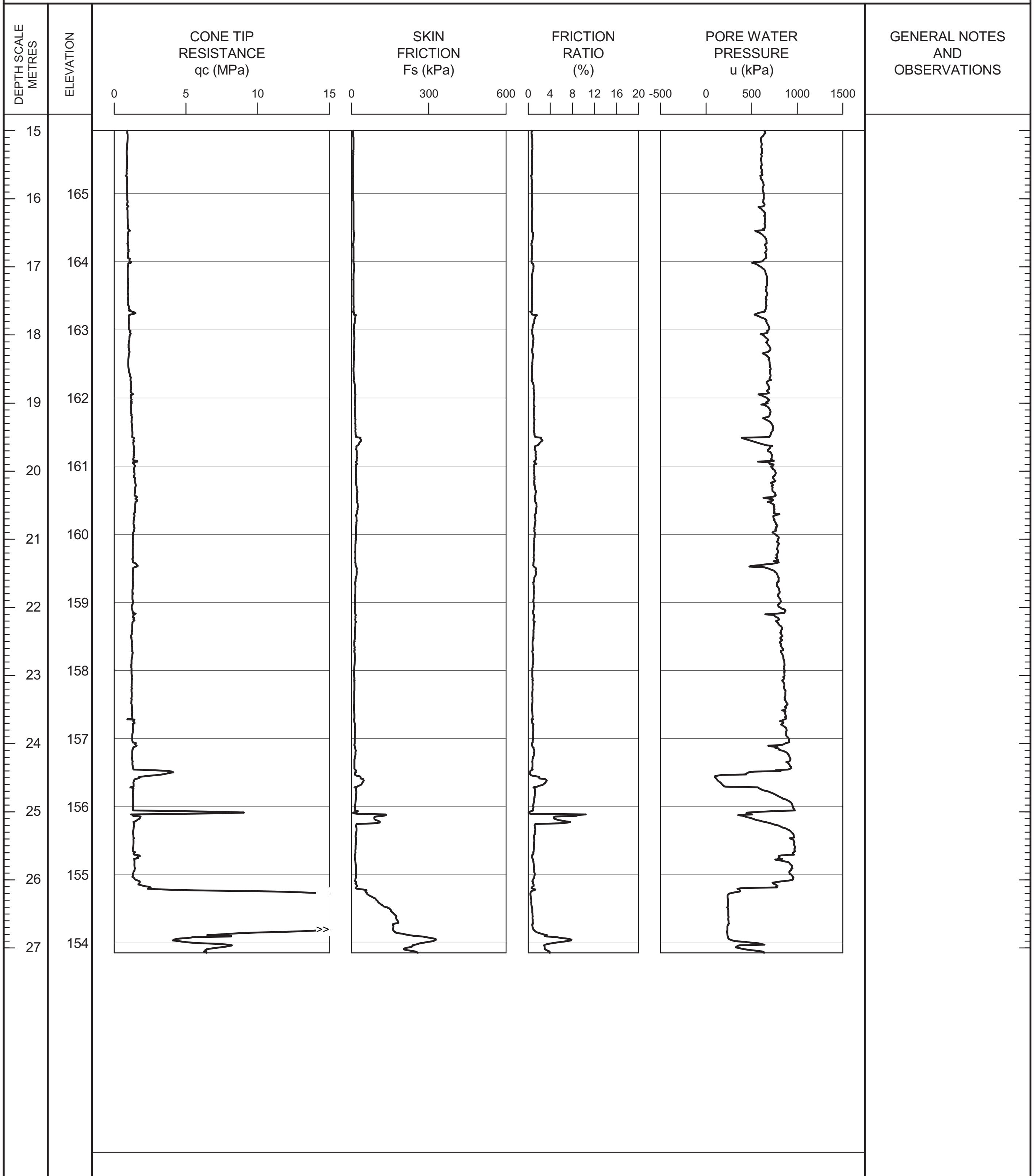
TEST DATE 7/27/2011 - 7/27/2011

SHEET 2 OF 2

LOCATION N4679571.4; E332146.2

DATUM Geodetic

GROUND SURFACE ELEVATION: 180.9 PREDRILL DEPTH: 2.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

Appendix B Borehole and CPT Logs from Previous Investigations

Golder Associates
London, Ontario

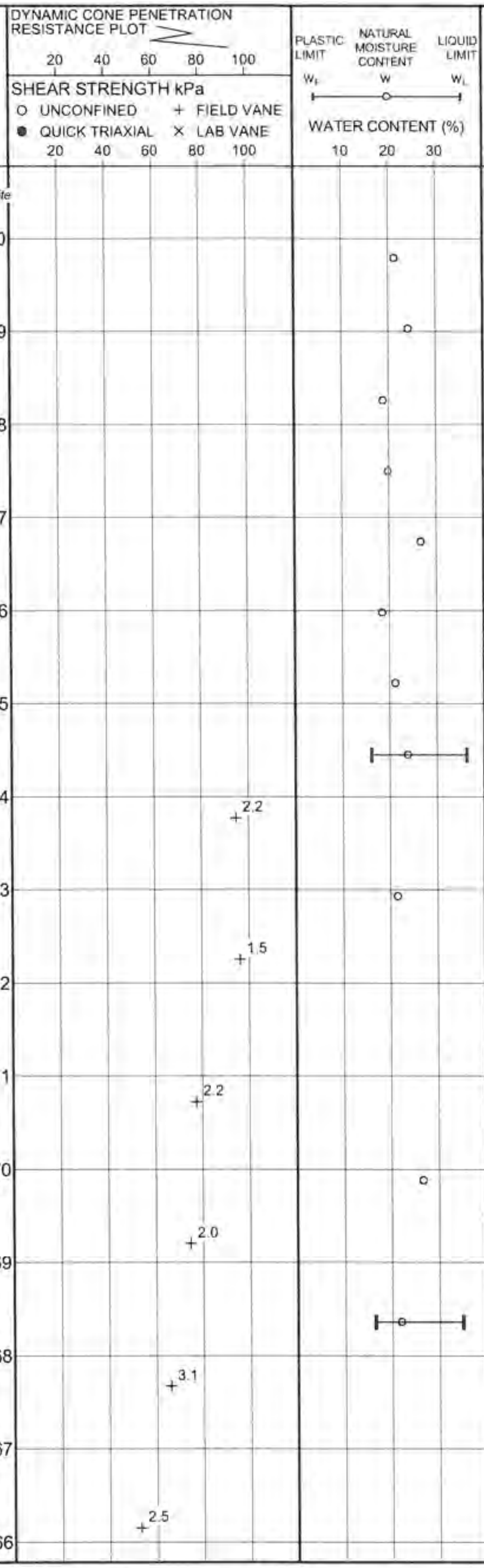
RECORD OF BOREHOLE No 129

 METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4679625.1 ;E 332109.7		ORIGINATED BY LZ/CC/MA/SM
DIST WEST	HWY 401/3	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC	COMPILED BY BRS
DATUM GEODETIC		DATE March 4, 2008 - March 10, 2008	CHECKED BY SJB

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.78	GROUND SURFACE					
0.00	FILL, clayey topsoil, with gravel Black					
0.20	SILTY CLAY, some sand Firm to stiff Mottled brown and grey				180	
		1	SS	6		
		2	SS	9	179	
178.80 1.98	SILTY CLAY, some sand, trace gravel Stiff to very stiff Mottled brown and grey becoming grey at about elev. 177.98m	3	SS	21	178	
		4	SS	14		
177.12 3.66	SILTY CLAY, some sand, trace gravel Firm to stiff Grey	5	SS	11	177	
		6	SS	11	176	
		7	SS	9	175	
		8	TO	PH		1 16 38 45 CIUC
					174	
		9	SS	7	173	
					172	
		10	TO	PH	171	
170.65 10.13	CLAYEY SILT, some sand, trace gravel Firm tto stiff Grey	11	SS	5	170	
					169	
		12	TO	PH		1 18 40 41 CIUC
					168	
		13	TO	PH	167	
					166	

Ground water conditions: Bentonite; Grout



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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 129

 METRIC

PROJECT 07-1130-207-0	LOCATION N 4679625.1 ;E 332109.7		ORIGINATED BY LZ/CC/MA/SM
W.P.	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY BRS
DIST WEST HWY 401/3			
DATUM GEODETIC	DATE March 4, 2008 - March 10, 2008		CHECKED BY SJS

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, some sand, trace gravel Firm tto stiff Grey		14	TO	PH						
							165	+ 2.1			
			15	TO	PH		164				
							163	+ 1.4			
			16	TO	PH						3 29 46 22 CIUC
							162	136.4 + 1.5			
			17	TO	PH		161				
							160	+ 1.3			
			18	SS	10						
							159				
			19	TO	PH		158				
							157				
						Grout					
			20	SS	9		156				
								+ 1.1			
			21	TO	PH		155				2 16 45 37 CIUC
							154				
			22	SS	7		153				
								+ 1.1			
151.48			23	TO	PH		152				
29.30	SILTY SAND, trace gravel, trace clay Grey						151				

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 129

 METRIC

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4679625.1 ;E 332109.7
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE March 4, 2008 - March 10, 2008
ORIGINATED BY LZ/CC/MA/SM
COMPILED BY BRS
CHECKED BY SB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	R.Q.D. (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
150.55									
30.23	SAND AND GRAVEL, medium to coarse, trace silt Compact to very dense Grey	24	SS	21		150			
					Bentonite				
						149			
		25	SS	125	Screen				
147.88						148			
32.90	DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous Grey to brown (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	26	SS	100/120mm	Bedrock soil interface				
		27	NQ RC			147	0	0	
						146			
		28	NQ RC		Sand		94	94	UC
		29	NQ RC			145	33	33	
		30	NQ RC			144	174	167	
143.78									
37.00	END OF BOREHOLE Borehole dry during drilling between March 4 and 10, 2008. Water level measured in deep piezometer at elev. 178.50m on July 22, 2008. Water level measured in deep piezometer at elev. 177.88m on August 11, 2008. Water level measured in deep piezometer at elev. 177.48m on September 19, 2008. Water level measured in deep piezometer at elev. 177.57m on November 11, 2008. Water level measured in deep piezometer at elev. 177.46m on January 28, 2009.								

Natural moisture content (water content %) plotted: sample 24 ≈ 14; sample 25 ≈ 12; sample 26 ≈ 14.

Column headings: SOIL PROFILE; STRAT PLOT; 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; GR SA SI CL

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

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 129

LOCATION: N 4679625.1 ;E 332109.7

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: March 4, 2008 - March 10, 2008

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	ELEVATION	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE	147.88				
33	MUD ROTARY / NO ROCK CORE	DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous, interbedded light and dark grey	32.90				
34				1	147		
		DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous, interbedded medium and dark brown, stylolites at 35.64 m	146.21 34.57		146		
35				2		JN,PL,SM CI	
36				3	145		
		DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous, stylolitic, interbedded light and dark grey	144.56 36.22	4	144	JN,IR,Ro CI	
37		END OF DRILLHOLE	143.78 37.00				

Depth scale: 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 129A

METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4679625.1 ;E 332109.7	ORIGINATED BY SM	
DIST WEST HWY 401/3	BOREHOLE TYPE POWER AUGER, HOLLOW STEM	COMPILED BY BRS	
DATUM GEODETIC	DATE March 4, 2008	CHECKED BY SJB	

ELEV DEPTH	DESCRIPTION (SOIL CONDITIONS INFERRED FROM BOREHOLE No. 129)	GROUND WATER CONDITIONS
180.78	GROUND SURFACE	
0.00	FILL, clayey topsoil, with gravel Black	Concrete
0.20	SILTY CLAY, some sand Firm to stiff Mottled brown and grey	
178.80 / 1.98	SILTY CLAY, some sand, trace gravel Stiff to very stiff Mottled brown and grey to grey at about elev. 177.98m	Bentonite and cuttings
177.12 / 3.66	SILTY CLAY, some sand, trace gravel Firm to stiff Grey	Piezometer; Sand
171.18 / 9.60	END OF BOREHOLE	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (180, 179, 178, 177, 176, 175, 174, 173, 172); 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

Water level measured in shallow piezometer at elev. 178.95m on July 22, 2008.3

Water level measured in shallow piezometer at elev. 178.93m on August 11, 2008.

Water level measured in shallow piezometer at elev. 178.95m on September 19, 2008.

Water level measured in shallow piezometer at elev. 178.84m on January 28, 2009.

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-324

METRIC

PROJECT 09-1132-0080
W.P.
LOCATION N 4679664.9 ;E 332002.7
ORIGINATED BY TA
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, SOLID STEM
COMPILED BY DMB
DATUM GEODETIC
DATE January 25, 2010
CHECKED BY

ELEV DEPTH	DESCRIPTION	STRAT PLOT	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
180.85	GROUND SURFACE										
0.00	TOPSOIL, clayey Very stiff Black										
			1	SS	18		180				
179.48 1.37	CLAYEY SILT, some sand, trace gravel, with occasional silt partings Very stiff Brown										
			2	SS	18		179		○ (approx. 19)		
			3	SS	27				○ (approx. 20)		
177.95 2.90	END OF BOREHOLE Borehole dry during drilling on January 25, 2010.						178				

Legend: 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

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-11

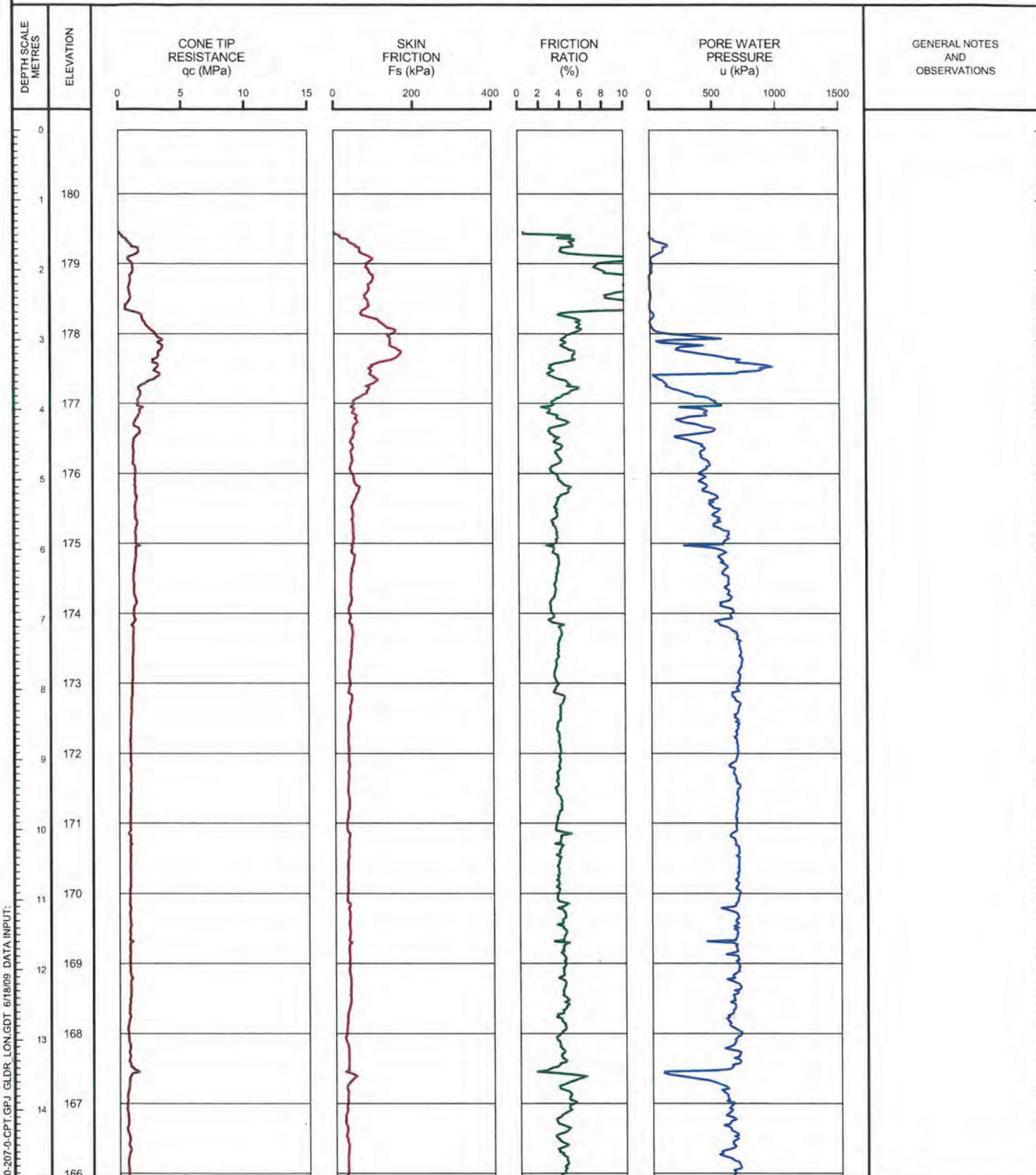
SHEET 1 OF 2

LOCATION: N 4679634.0 ;E 332110.0

TEST DATE: November 10, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 1.46m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



— CONTINUED NEXT PAGE —

DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

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

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-11

LOCATION: N 4679634.0 ;E 332110.0

TEST DATE: November 10, 2006

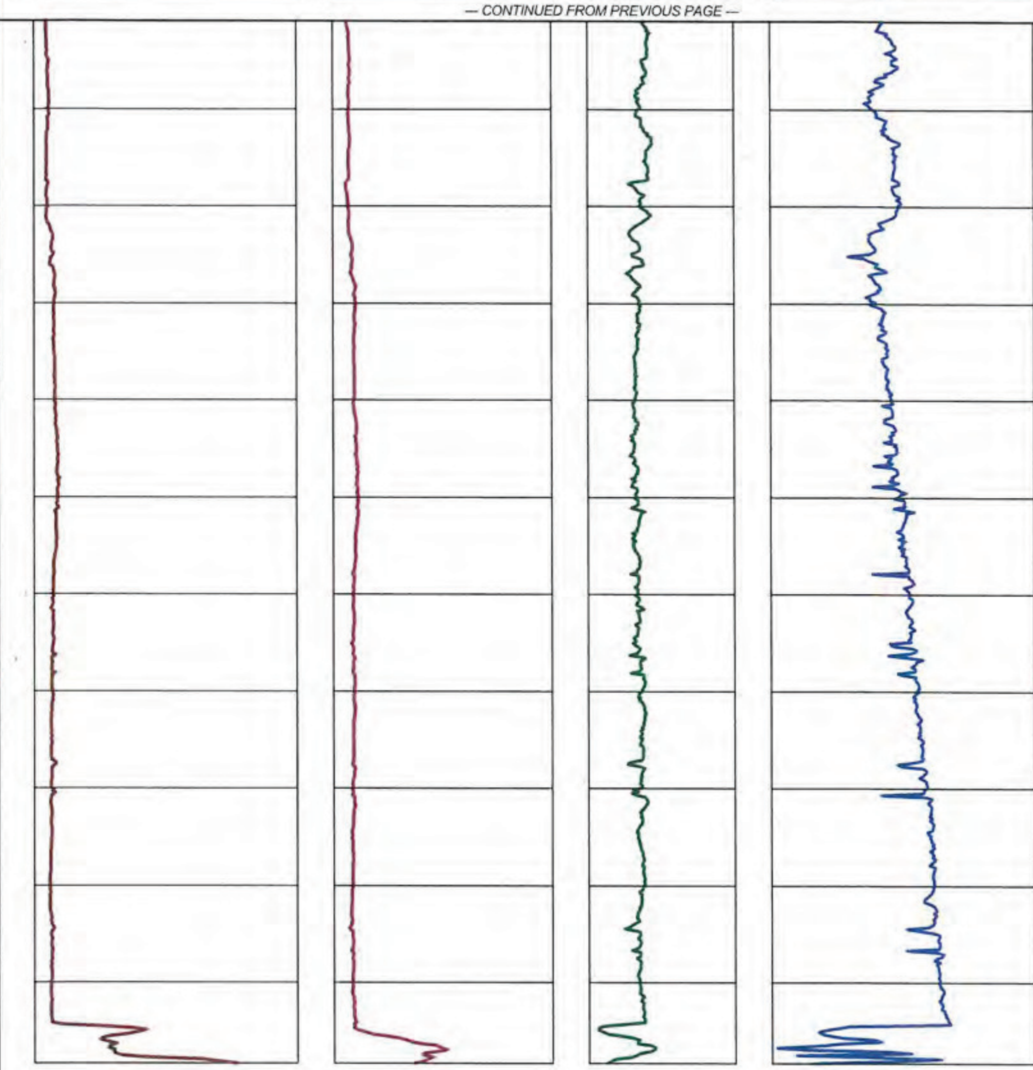
DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 1.46m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013

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

— CONTINUED FROM PREVIOUS PAGE —

Depth (m)	Elevation
15	
16	165
17	164
18	163
19	162
20	161
21	160
22	159
23	158
24	157
25	156
26	
27	
28	
29	
30	



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

DEPTH SCALE
1 : 75

Golder Associates

OPERATOR: CC
CHECKED: SSB

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-324

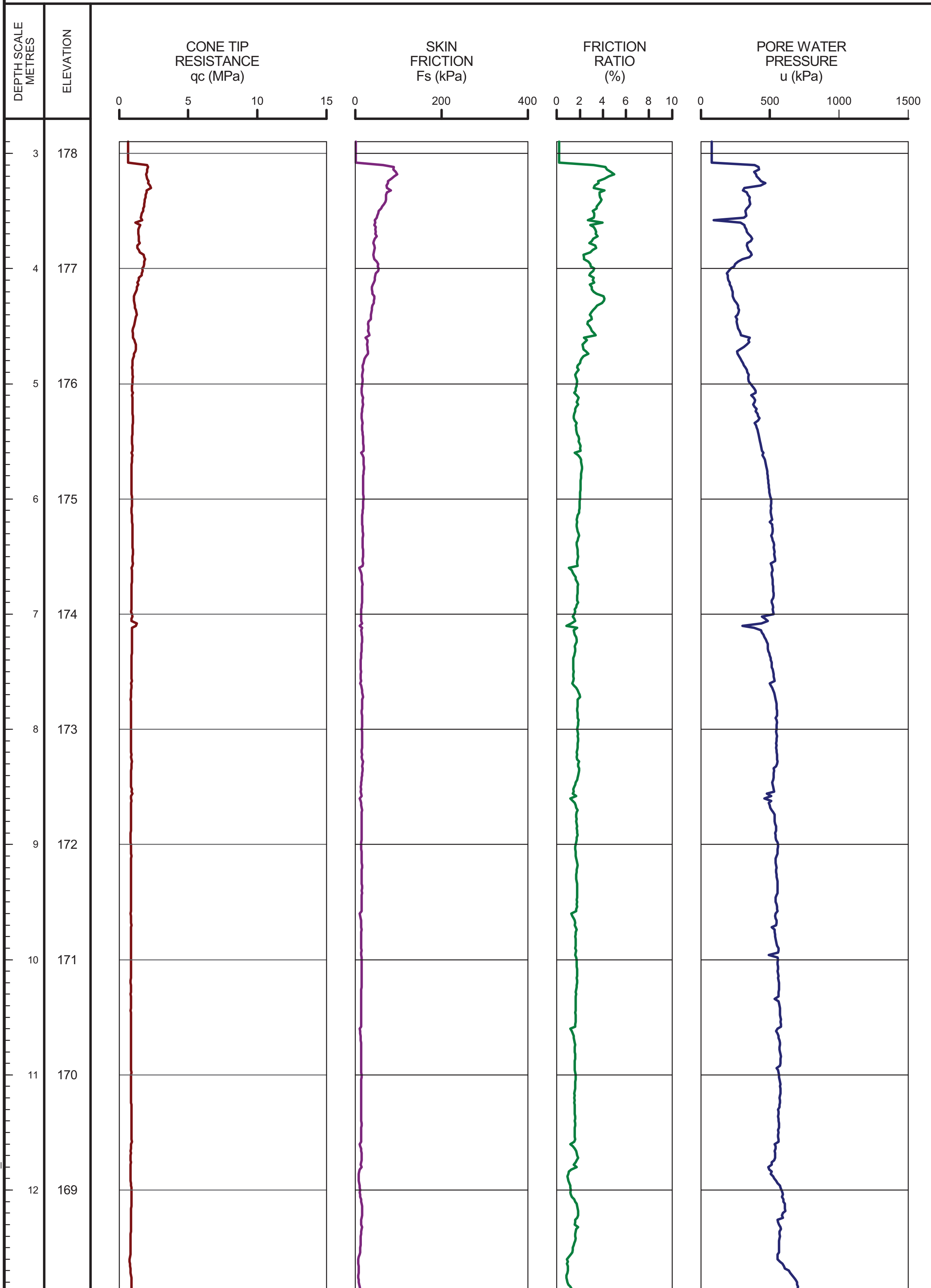
SHEET 1 OF 3

LOCATION: N 4679664.9 ;E 332002.7

TEST DATE: January 25, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 180.85m PREDRILL DEPTH: 2.90m 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-324.

--- 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

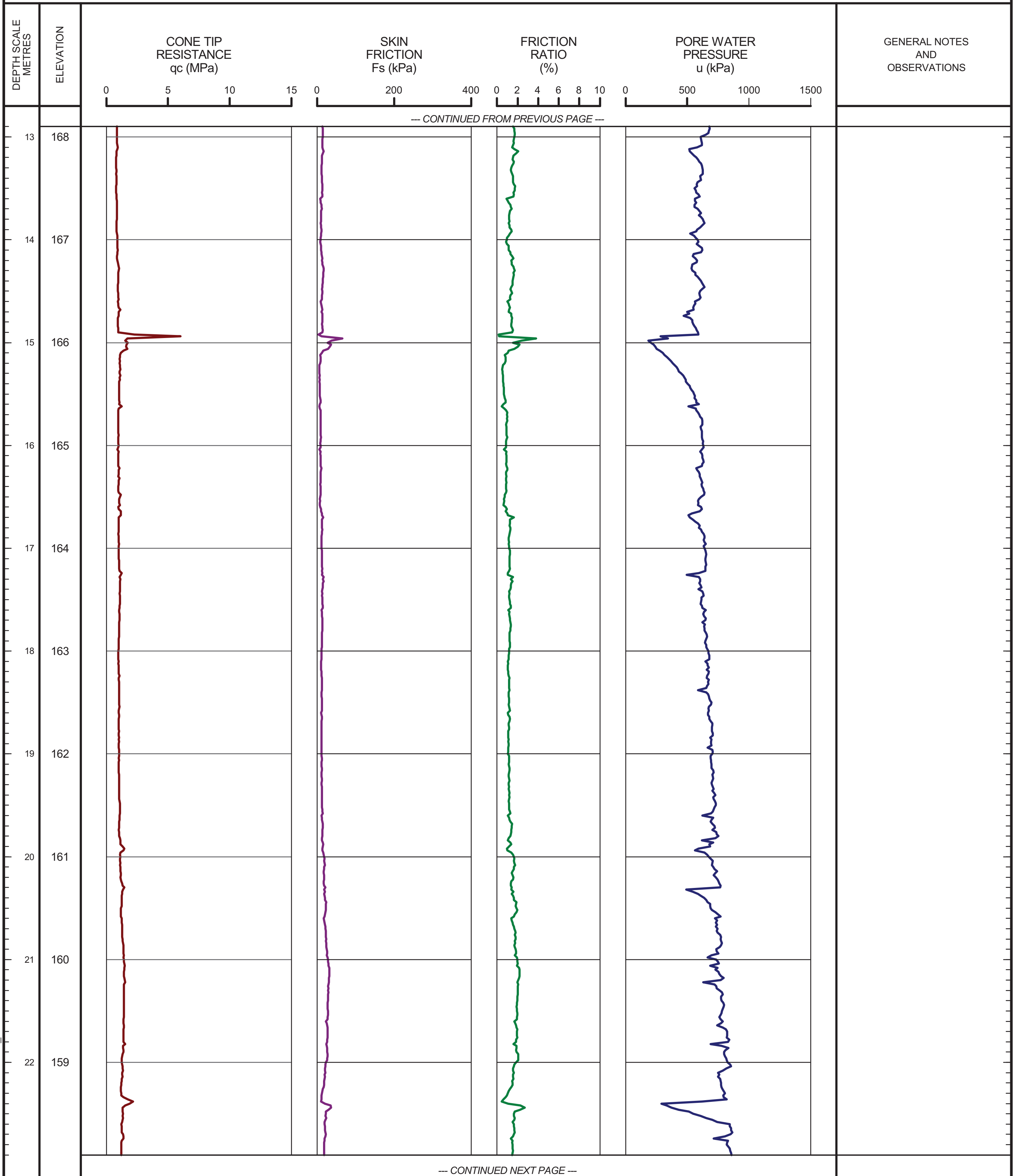
RECORD OF CONE PENETRATION TEST CPT-324

LOCATION: N 4679664.9 ;E 332002.7

TEST DATE: January 25, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 180.85m PREDRILL DEPTH: 2.90m 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-324

LOCATION: N 4679664.9 ;E 332002.7

TEST DATE: January 25, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 180.85m PREDRILL DEPTH: 2.90m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012

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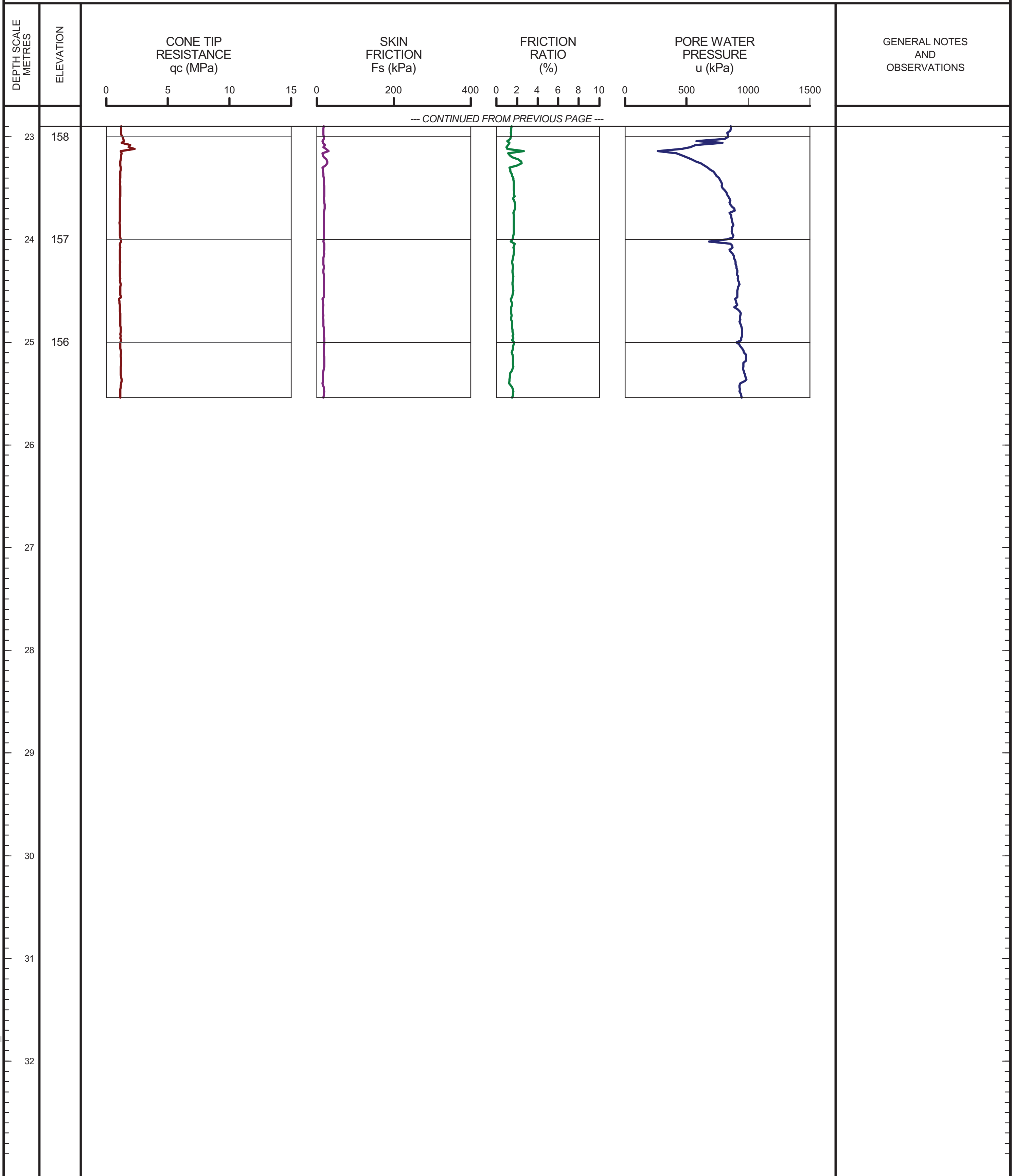


DEPTH SCALE

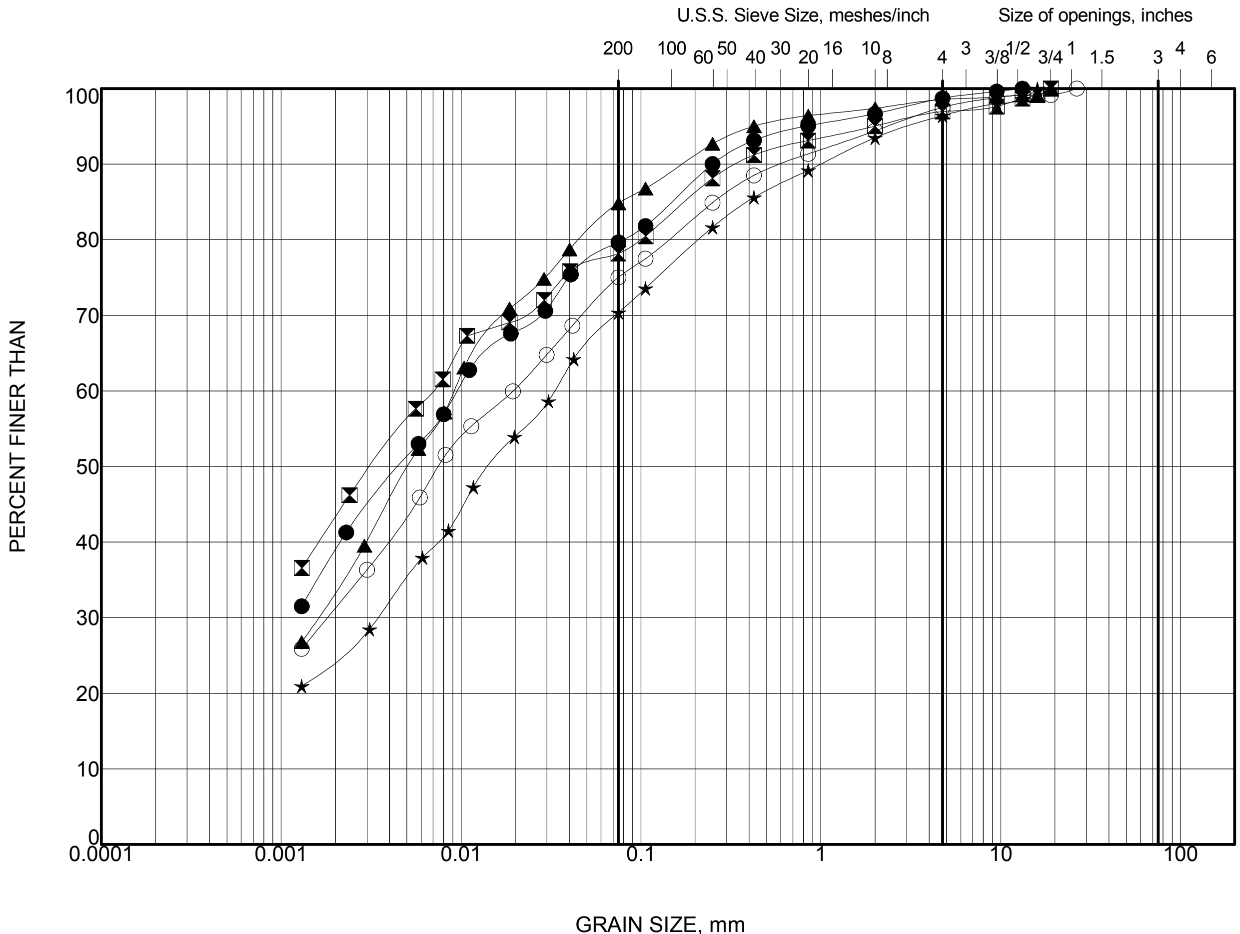
1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

Appendix C Geotechnical Laboratory Test Results



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T6-1/HG-MW-07	8	6.1
⧗	T6-1/HG-MW-07	10	9.1
▲	T6-1/HG-MW-07	13	13.7
★	T6-1/HG-MW-07	16	18.3
○	T6-1/HG-MW-07	19	22.9

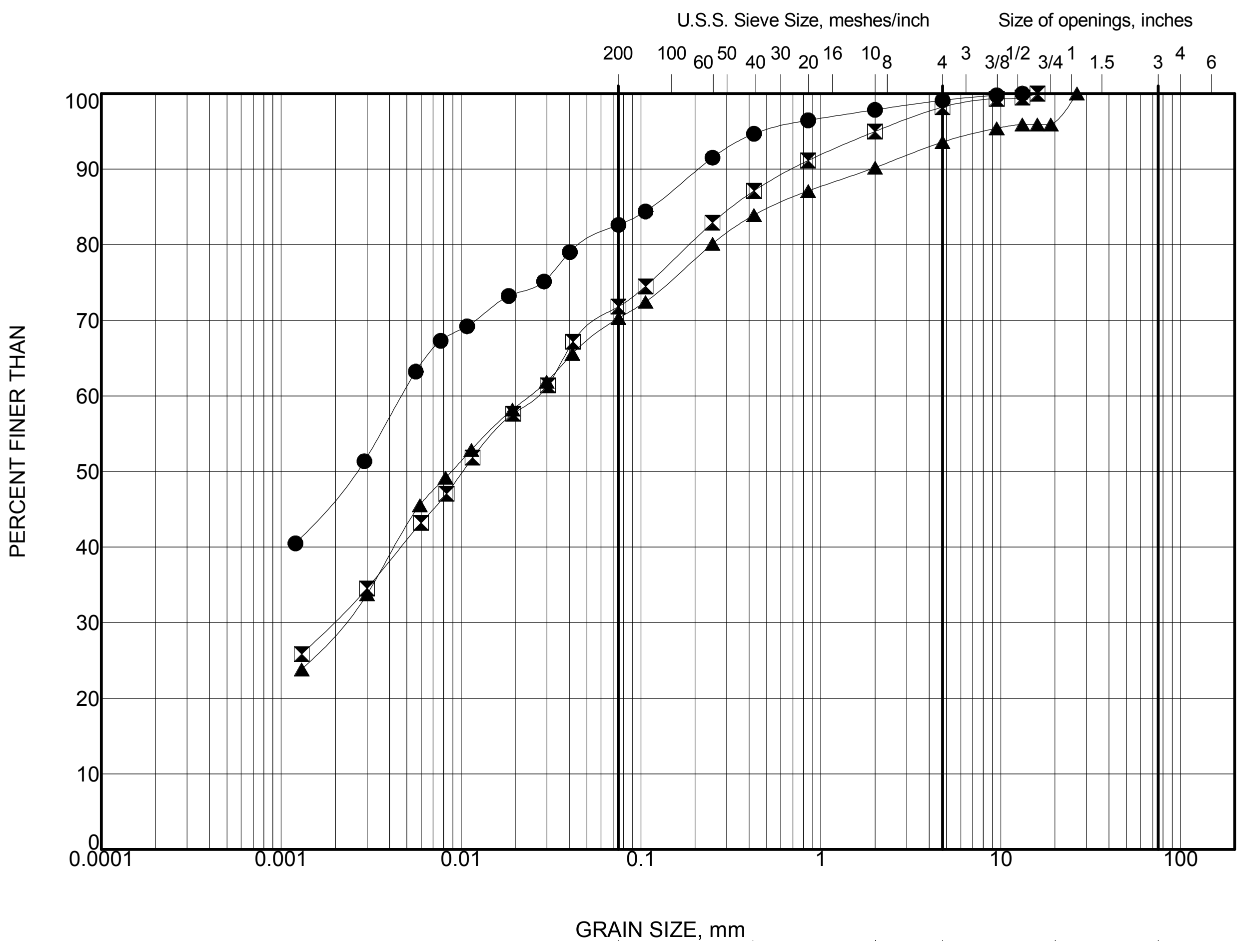
WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 07/08/12

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

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

FIGURE **C.1**



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T6-2	8	6.1
⧗	T6-2	14	15.2
▲	T6-2	19	22.9

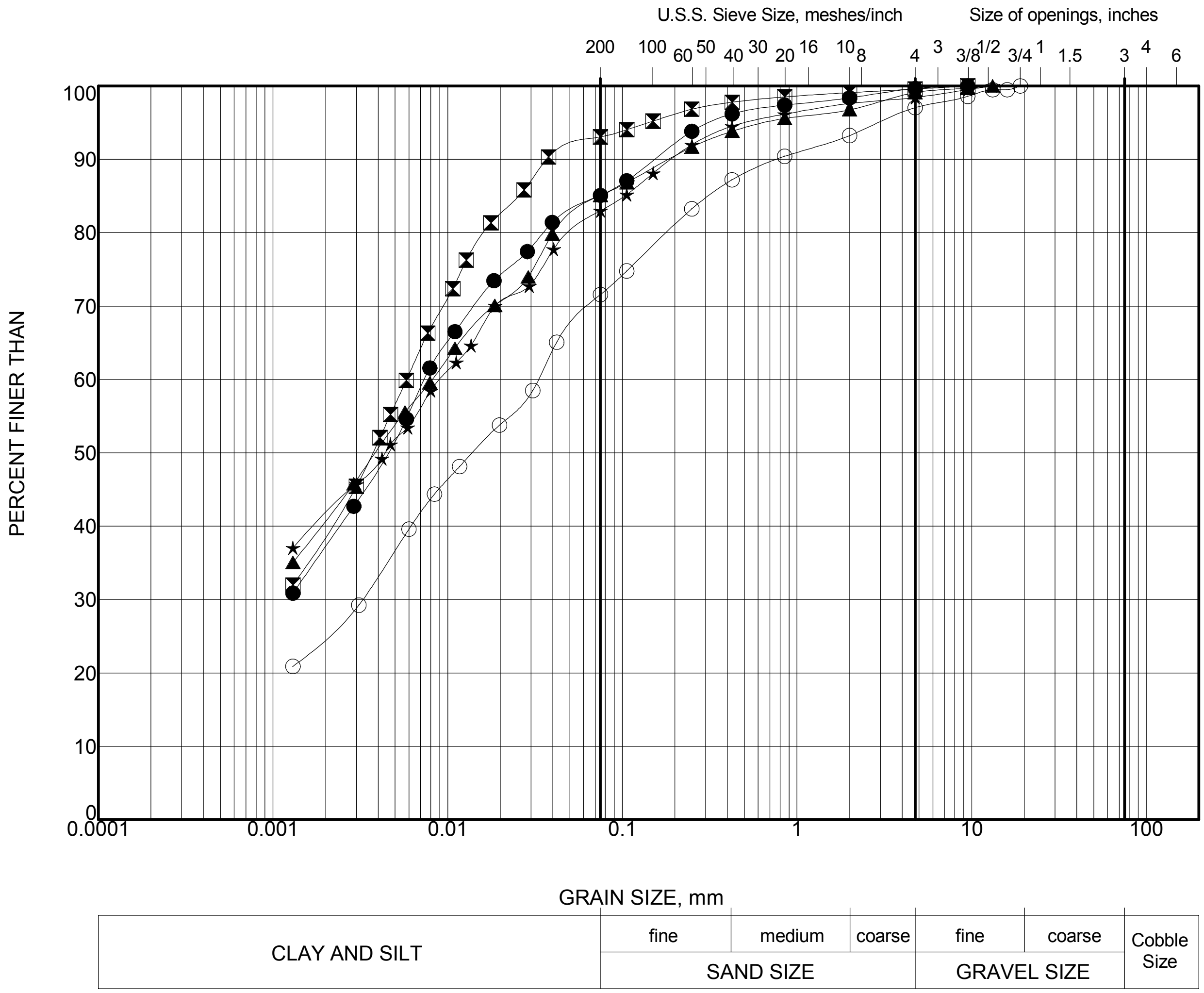
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION
Clayey Silt to Silty Clay

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.
		SCALE / REV.
DRAWN	SS	
CHECK	GN	FIGURE C.2

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 07/08/12



LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T6-3	11	10.7
⧗	T6-3	12	12.2
▲	T6-3	14	15.2
★	T6-3	15	16.8
○	T6-3	17	19.8

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE

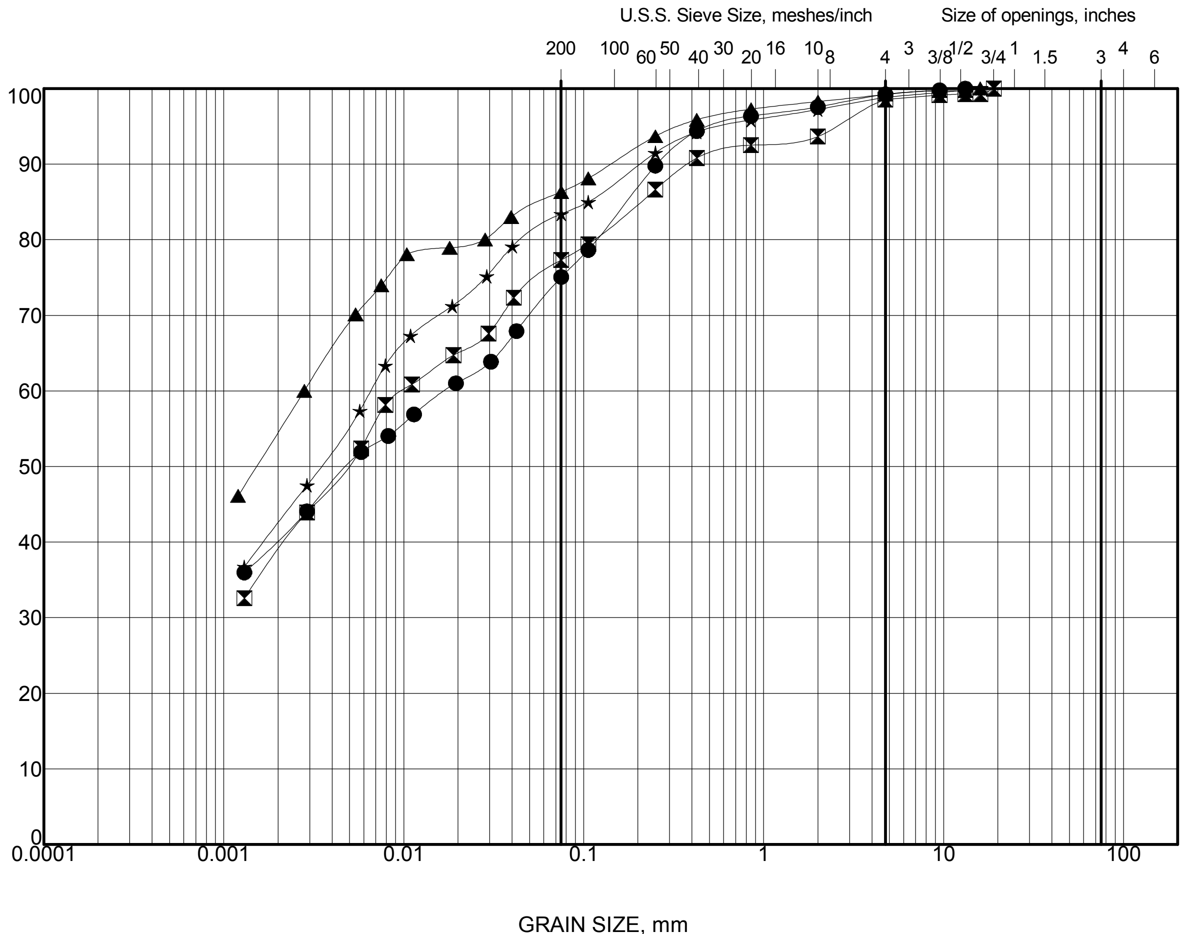
GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.	
		SCALE	REV.
DRAWN	SS	FIGURE C.3	
CHECK	GN		

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 07/08/12



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	TP7	1	1.5
⧗	TP7	2	2.5
▲	TP7	3	4.2
★	TP7	4	10.5

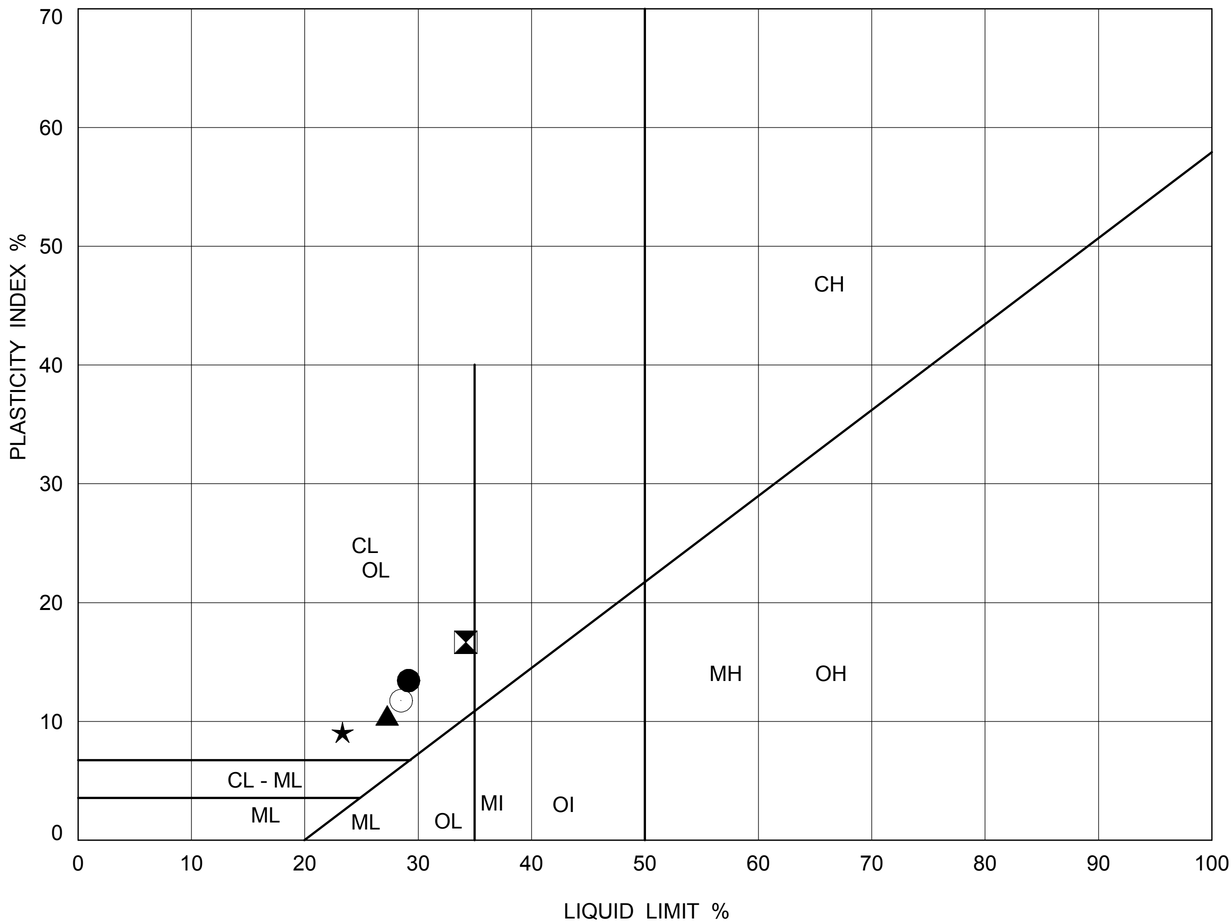
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION
Clayey Silt to Silty Clay

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.	
		SCALE	REV.
DRAWN	SS	FIGURE C.4	
CHECK	GN		

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 07/08/12



SOIL TYPE

C = Clay
M = Silt
O = Organic

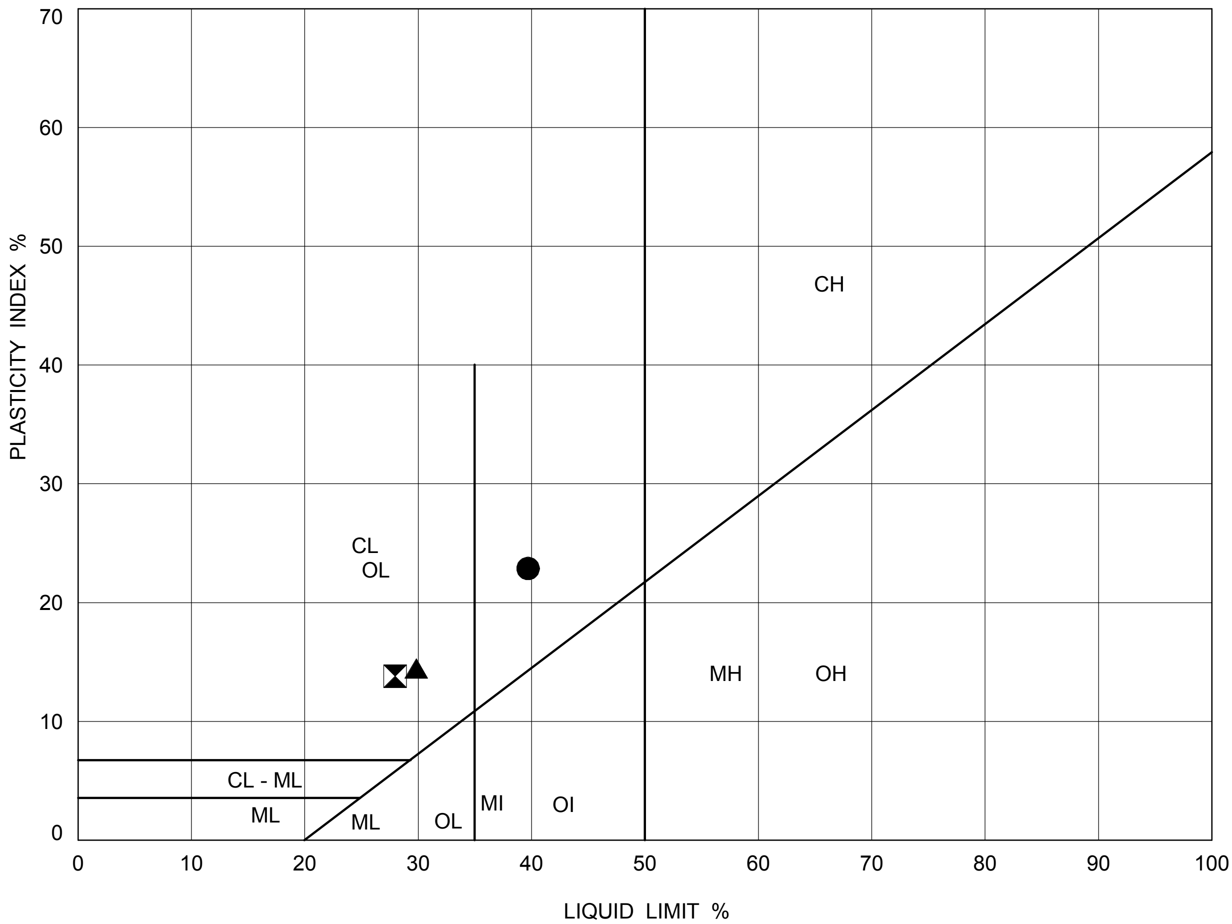
PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T6-1/HG-MW-07	8	6.1	**29**	**16**	**13**
⧗	T6-1/HG-MW-07	10	9.1	**34**	**18**	**16**
▲	T6-1/HG-MW-07	13	13.7	**27**	**17**	**10**
★	T6-1/HG-MW-07	16	18.3	**23**	**14**	**9**
○	T6-1/HG-MW-07	19	22.9	**28**	**17**	**11**

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE	PLASTICITY CHART **Clayey Silt to Silty Clay**		
Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PROJECT No. SW8801.1004.101	FILE No.	
		SCALE	REV.
	DRAWN SS	FIGURE **C.5**	
	CHECK GN		



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T6-2	8	6.1	**40**	**17**	**23**
⧗	T6-2	14	15.2	**28**	**14**	**14**
▲	T6-2	19	22.9	**30**	**15**	**15**

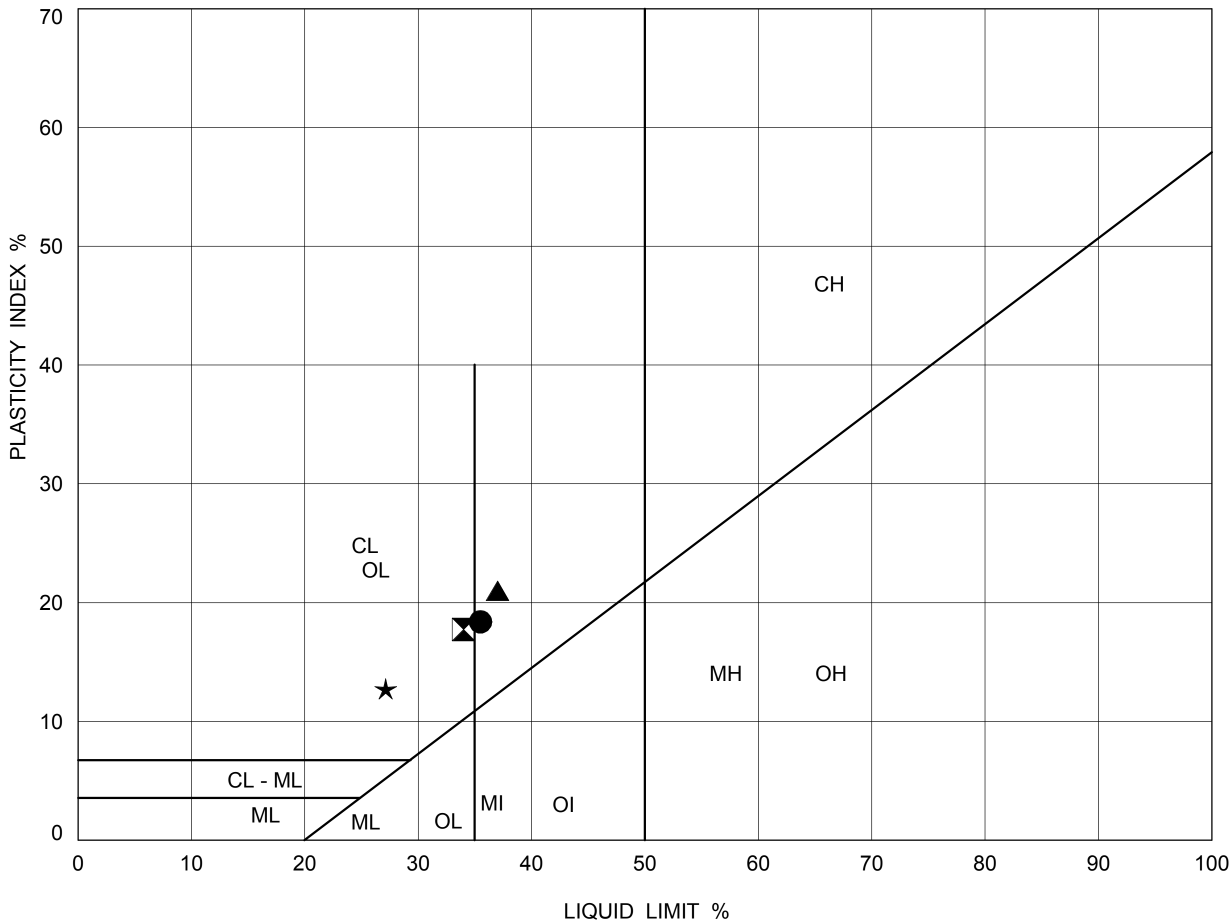
WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 07/08/12

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
PLASTICITY CHART
Clayey Silt to Silty Clay

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.
		SCALE	REV.
DRAWN	SS	
CHECK	GN	FIGURE **C.6**



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T6-3	11	10.7	**35**	**17**	**18**
⧗	T6-3	14	15.2	**34**	**16**	**18**
▲	T6-3	15	16.8	**37**	**16**	**21**
★	T6-3	17	19.8	**27**	**14**	**13**

PROJECT

Windsor Essex Parkway (WEP)
Windsor, Ontario

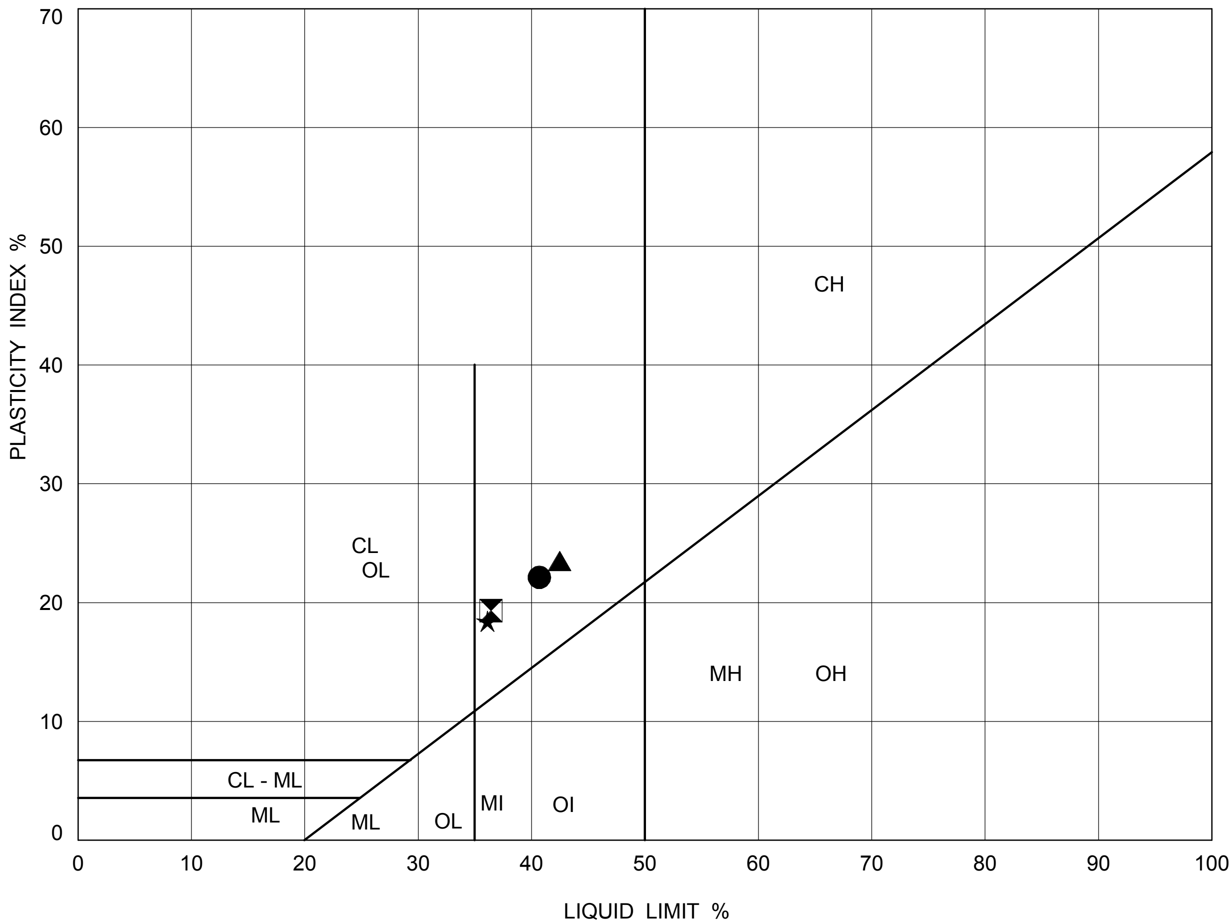
TITLE

PLASTICITY CHART

Clayey Silt to Silty Clay

Parkway Infrastructure Engineers amec Hatch Mott MacDonald	PROJECT No. SW8801.1004.101		FILE No.	
			SCALE	REV.
	DRAWN	SS	FIGURE **C.7**	
	CHECK	GN		

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 07/08/12



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	TP7	1	1.5	**41**	**19**	**22**
⧗	TP7	2	2.5	**36**	**17**	**19**
▲	TP7	3	4.2	**42**	**19**	**23**
★	TP7	4	10.5	**36**	**18**	**18**

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario
TITLE	PLASTICITY CHART **Clayey Silt to Silty Clay**
Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PROJECT No. SW8801.1004.101; FILE No.; SCALE; REV.; DRAWN SS; CHECK GN; FIGURE **C.8**

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 07/08/12 amec

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 14-Nov-11 **Sample ID:** T6-1_TW12 **Depth(m):** 12.2

Test Data

Ring # : A	Ring Height (in) =	0.758	Wt of dry filter paper (g)	0.8
Wet soil + Ring Wt (g)		205.34	Wt of ring (g)	76.58
Wet soil + Wet Paper + Ring (g)		204.00	Wet Paper (g)	2.28
Dry Soil + Dry Paper + Ring (g)		184.44	Ring Dia (in)	2.498
Initial moisture Content (%)		20.27	Final moisture Content (%)	16.89
Area of Ring (in^2)		4.90	Initial Volume (in^3)	3.7149
Initial Bulk Density (kg/m^3)		2115	Initial Dry Density (kg/m^3)	1759
Specific Gravity of Soil		2.73	Eqiv. Thick. of solids (mm)	12.389
Final Bulk Density (kg/m^3)		2186	Final Dry Density (kg/m^3)	1870
Initiall gauge reading for Load 1		0.2558	Gauge reading for last Loading	0.2106
Initial Voids Ratio		0.554	Final Void Ratio	0.461
Initial Degree of Saturation (%)		100	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	4.75	7.0	10.5	15.75	23.75	35.5	53.5
Load (tsf)	0.0494	0.0728	0.109	0.164	0.247	0.369	0.556
Gauge Reading (in)	0.2558	0.2554	0.2548	0.2517	0.2496	0.24684	0.2434
(H-Hs) mm	6.864	6.854	6.838	6.760	6.706	6.636	6.549
Voids ratio	0.554	0.553	0.552	0.546	0.541	0.536	0.529
t90 (min)			5.71	6.76	11.56	9.00	7.56
Cv (m^2/day)			0.020	0.017	0.010	0.012	0.015
k' (MPa)			4.209	1.299	2.872	3.201	3.919
Mv (mm^2 / N)			0.2376	0.7700	0.3482	0.3124	0.2551

Trial #	8	9	10	11	12	13	14
Load (kPa)	80.0	120.0	80.0	53.5	80.0	120.0	180.0
Load (tsf)	0.832	1.248	0.832	0.556	0.832	1.248	1.872
Gauge Reading (in)	0.2393	0.2347	0.2352	0.2357	0.2352	0.2340	0.2282
(H-Hs) mm	6.445	6.328	6.341	6.353	6.341	6.310	6.163
Voids ratio	0.520	0.511	0.512	0.513	0.512	0.509	0.497
t90 (min)	7.02	6.76					6.25
Cv (m^2/day)	0.016	0.016					0.017
k' (MPa)	4.819	6.448					7.616
Mv (mm^2 / N)	0.2075	0.1551					0.1313

Trial #	15	16	17	18	19	20	21
Load (kPa)	270.0	405.0	607.5	910.0	1375.0	685.0	340.0
Load (tsf)	2.808	4.212	6.318	9.464	14.300	7.124	3.536
Gauge Reading (in)	0.21968	0.2094	0.1986	0.1863	0.1733	0.1751	0.1775
(H-Hs) mm	5.946	5.686	5.410	5.099	4.769	4.814	4.874
Voids ratio	0.480	0.459	0.437	0.412	0.385	0.389	0.393
t90 (min)	7.02	6.25	6.25	6.25	6.25		
Cv (m^2/day)	0.015	0.016	0.016	0.015	0.015		
k' (MPa)	7.715	9.508	13.245	17.304	24.665		
Mv (mm^2 / N)	0.1296	0.1052	0.0755	0.0578	0.0405		

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

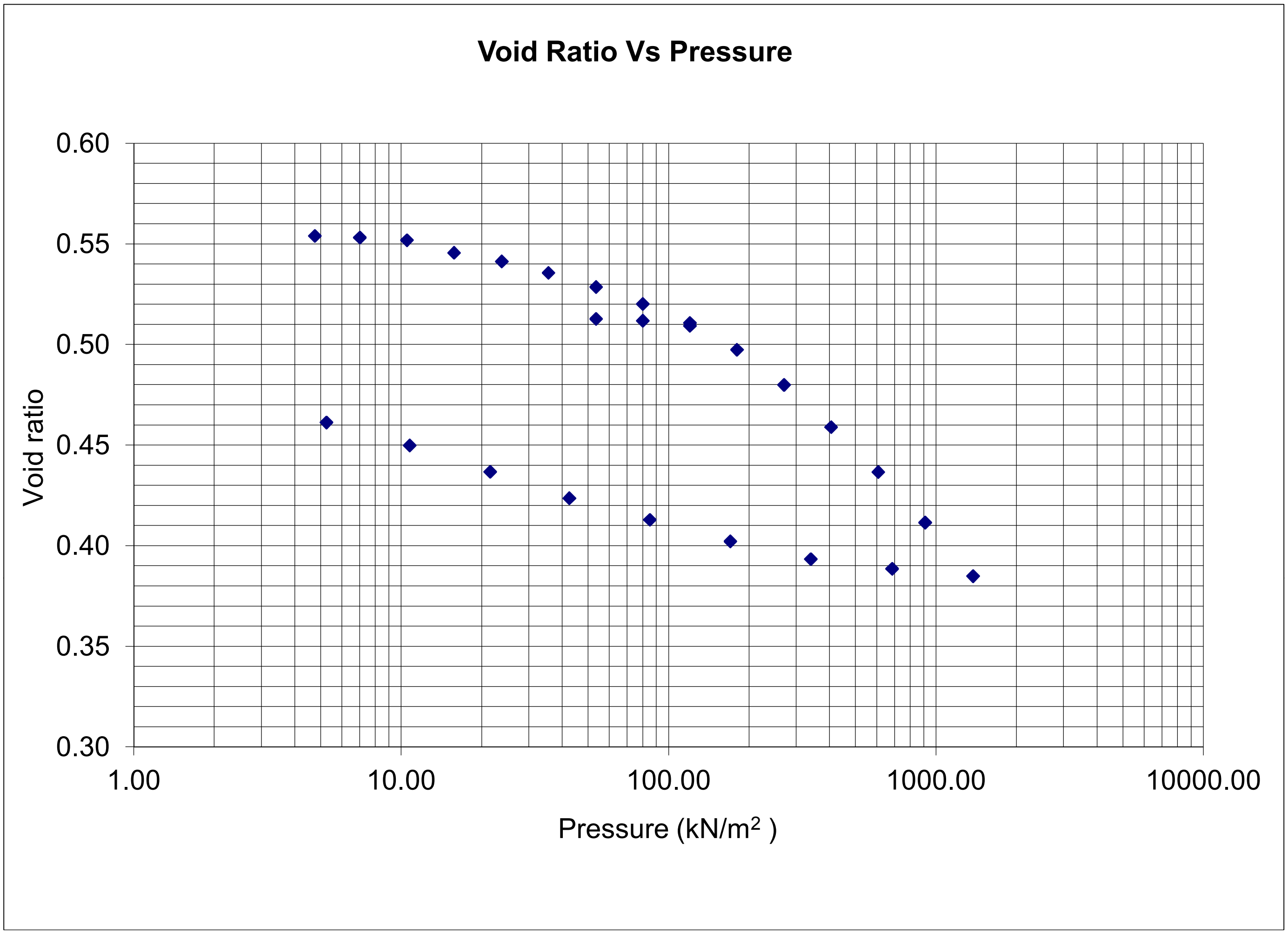
Project: WEP
Job No.: SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 14-Nov-11
Sample ID: T6-1_TW12
Depth(m): 12.2

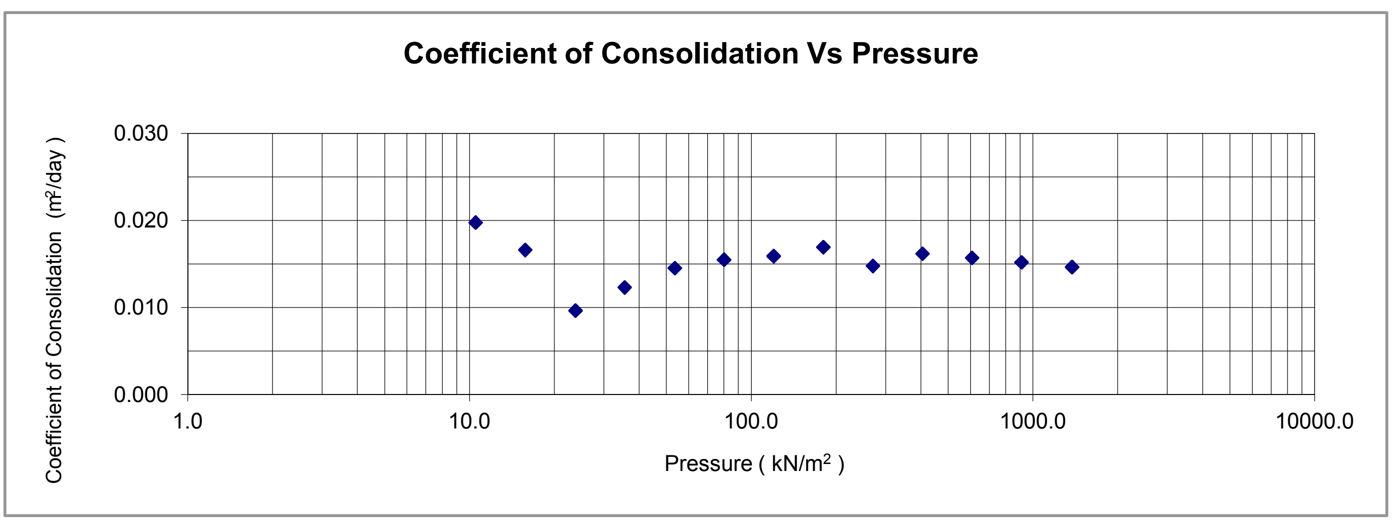
Trial #	22	23	24	25	26	27	
Load (kPa)	170.0	85.0	42.5	21.5	10.75	5.25	
Load (tsf)	1.768	0.884	0.442	0.224	0.112	0.055	
Gauge Reading (in)	0.1818	0.1870	0.1922	0.1986	0.2050	0.2106	
(H-Hs) mm	4.983	5.116	5.248	5.411	5.574	5.715	
Voids ratio	0.402	0.413	0.424	0.437	0.450	0.461	
t90 (min)							
Cv (m^2/day)							
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: 14-Nov-11 **Sample ID:** T6-1_TW12 **Depth(m):** 12.2

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: 14-Nov-11 **Sample ID:** T6-1_TW12 **Depth(m):** 12.2

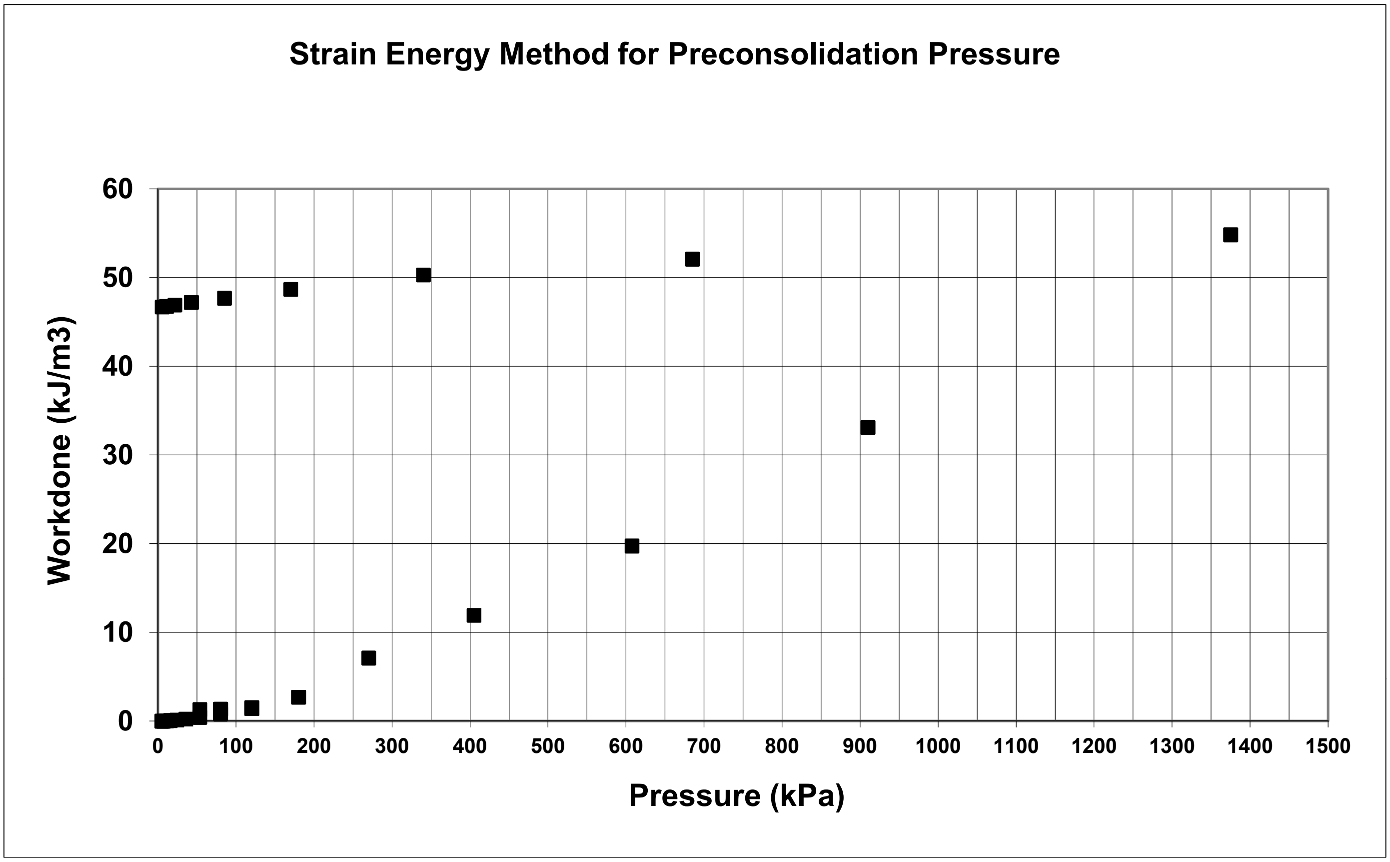
Strain Energy Data

Presssure (kN/m^2)	c_v (m^2/day)	Void ratio
4.75		0.554
7.0		0.553
10.5	0.020	0.552
15.8	0.017	0.546
23.75	0.010	0.541
35.5	0.012	0.536
53.5	0.015	0.529
80.0	0.016	0.520
120.0	0.016	0.511
80.0		0.512
53.5		0.513
80.0		0.512
120.0		0.509
180.0	0.0169	0.497
270.0	0.0148	0.480
405.0	0.0162	0.459
607.5	0.0157	0.437
910.0	0.0152	0.412
1375.0	0.0147	0.385
685.0		0.389
340.0		0.393
170.0		0.402
85.0		0.413
42.5		0.424
21.5		0.437
10.75		0.450
5.25		0.461

Presssure (kN/m^2)	Height mm	Total Work (kJ/m^3)
4.75	19.253	0.000
7.0	19.243	0.003
10.5	19.227	0.010
15.75	19.149	0.064
23.75	19.096	0.119
35.5	19.026	0.227
53.5	18.938	0.432
80.0	18.834	0.799
120.0	18.717	1.419
80.0	18.731	1.347
53.5	18.742	1.306
80.0	18.730	1.349
120.0	18.699	1.513
180.0	18.552	2.695
270.0	18.188	7.106
405.0	17.928	11.937
607.5	17.652	19.741
910.0	17.341	33.115
1375.0	17.011	54.837
685.0	17.056	52.100
340.0	17.116	50.306
170.0	17.225	48.679
85.0	17.358	47.696
42.5	17.490	47.209
21.5	17.653	46.911
10.75	17.816	46.763
5.25	17.957	46.699

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project:	**WEP**		**Job No.:**	**SW8801.1004.101**
Client:	**Hatch Mott MacDonald Limited**			
Date:	**14-Nov-11**	**Sample ID: T6-1_TW12**	**Depth(m):**	**12.2**



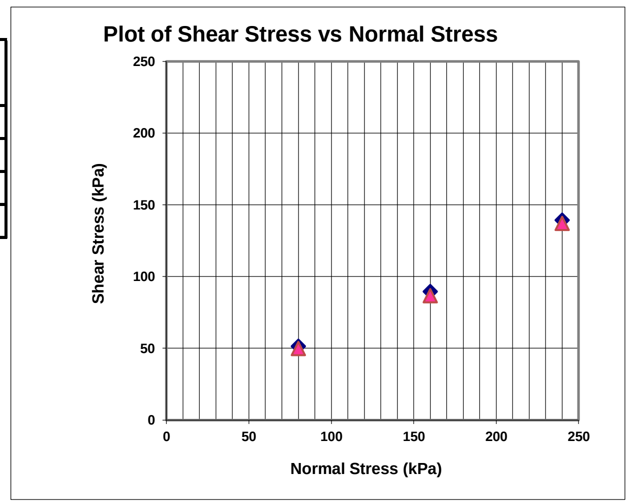
DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**14 November 2011**
Sample ID.:	**T6-3_TW15**	**Tested By:**	**CZ/SB**
Lab No.:	**AdS090_2011**	**Checked By:**	**SB**

Specimen ID	**1**	**2**	**3**
Date of Test	**15-Nov-11**	**16-Nov-11**	**17-Nov-11**
Normal Stress (kPa)	**80**	**160**	**240**
Rate of displacement (mm/min)	0.05	0.06	0.06
Initial thickness of specimen (mm)	24.10	24.10	24.10
Initial diameter of specimen (mm)	63.30	63.30	63.30
Initial moisture content (%)	16.0	15.2	15.2
Density (kN/m^3)	8.4	7.9	8.1
Final moisture (%)	15.3	15.3	13.1

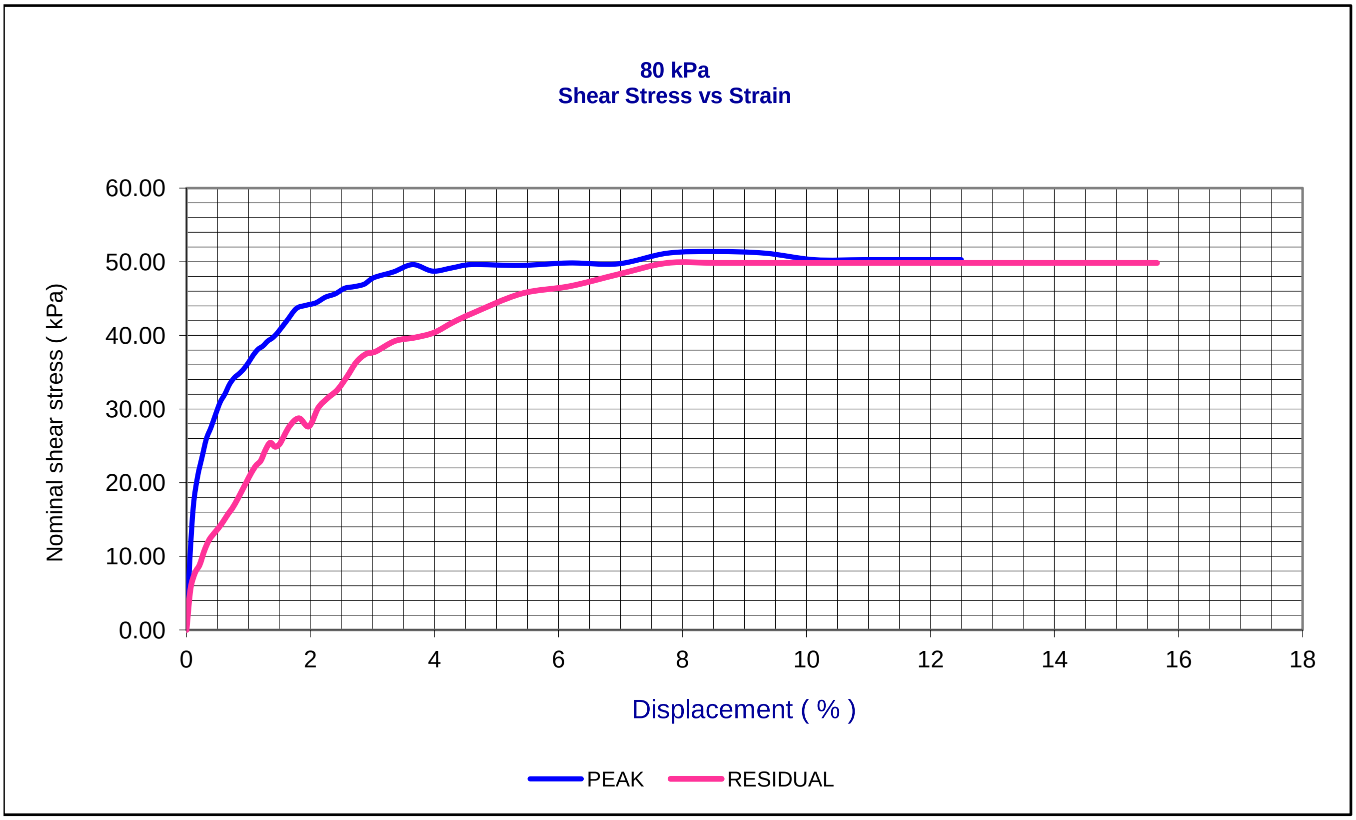
Specimen ID	**Normal Stress**	**Peak Shear Stress**	**Residual Shear Stress**
	kPa	**kPa**	**kPa**
1	80.0	51.4	49.8
2	160.0	89.5	86.6
3	240.0	139.3	137.0

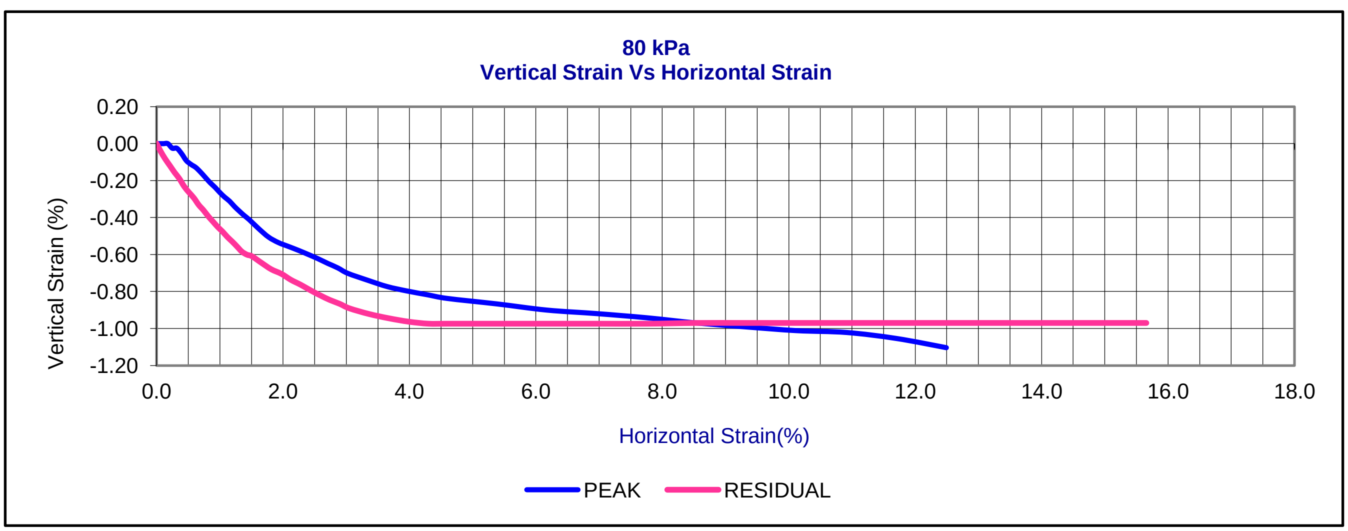
Note: Test specimens were inundated with water.



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

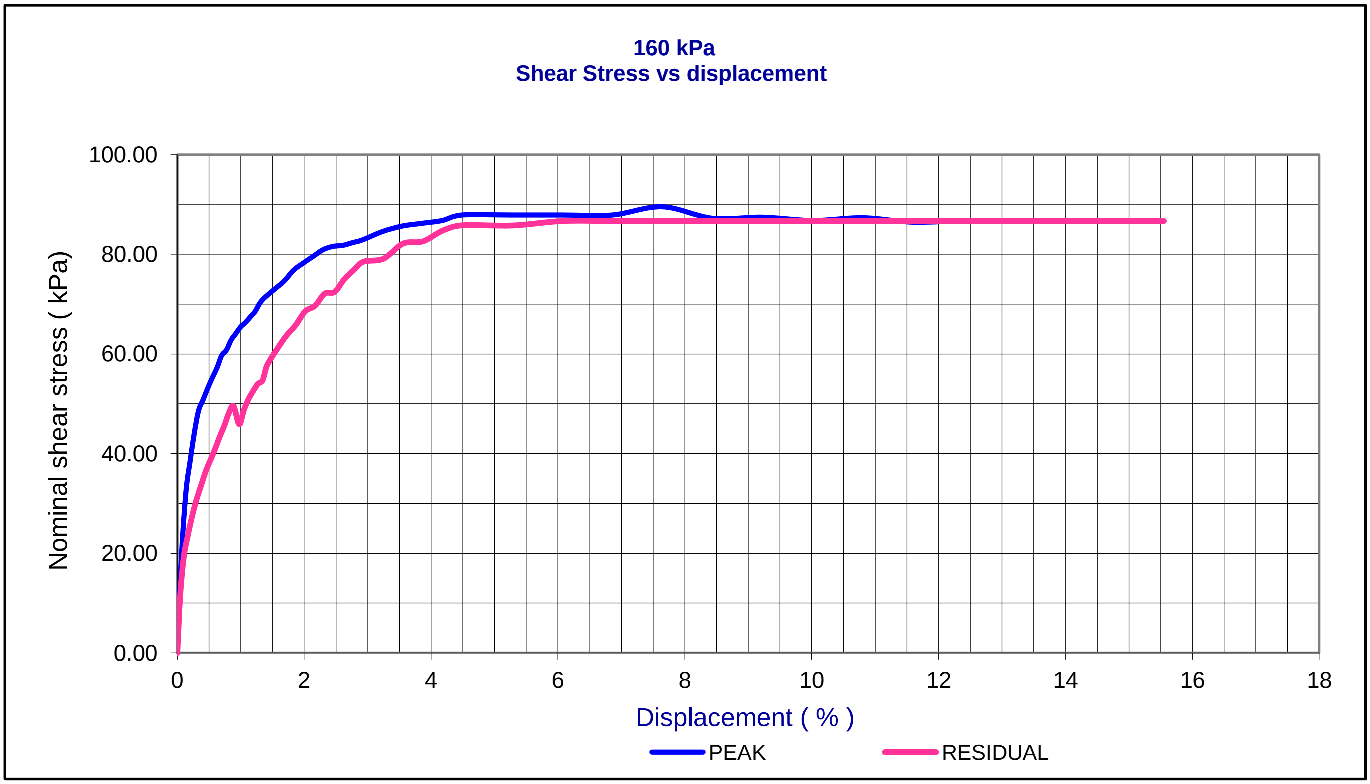
Project:-	WEP	**Job#:**	SW8801.1004.101
Client:-	Hatch Mott MacDonald Limited	**Date:**	14-November-2011
Sample ID.:	T6-3_TW15	**Tested By:**	CZ/SB
Lab No.:	AdS090_2011	**Checked By:**	SB

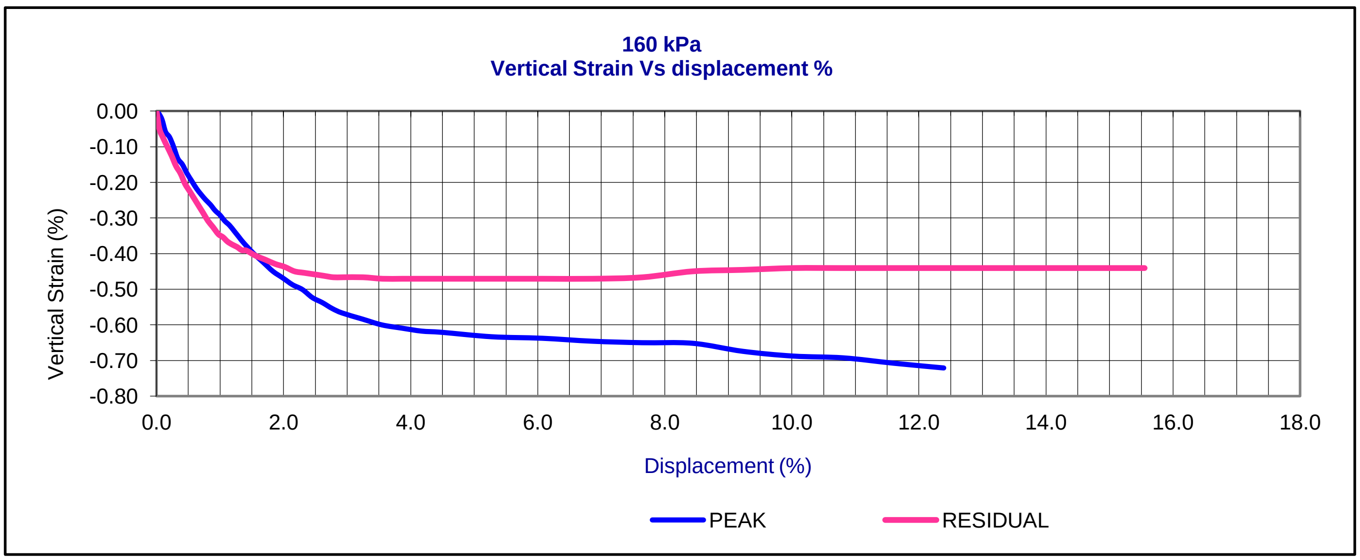




DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

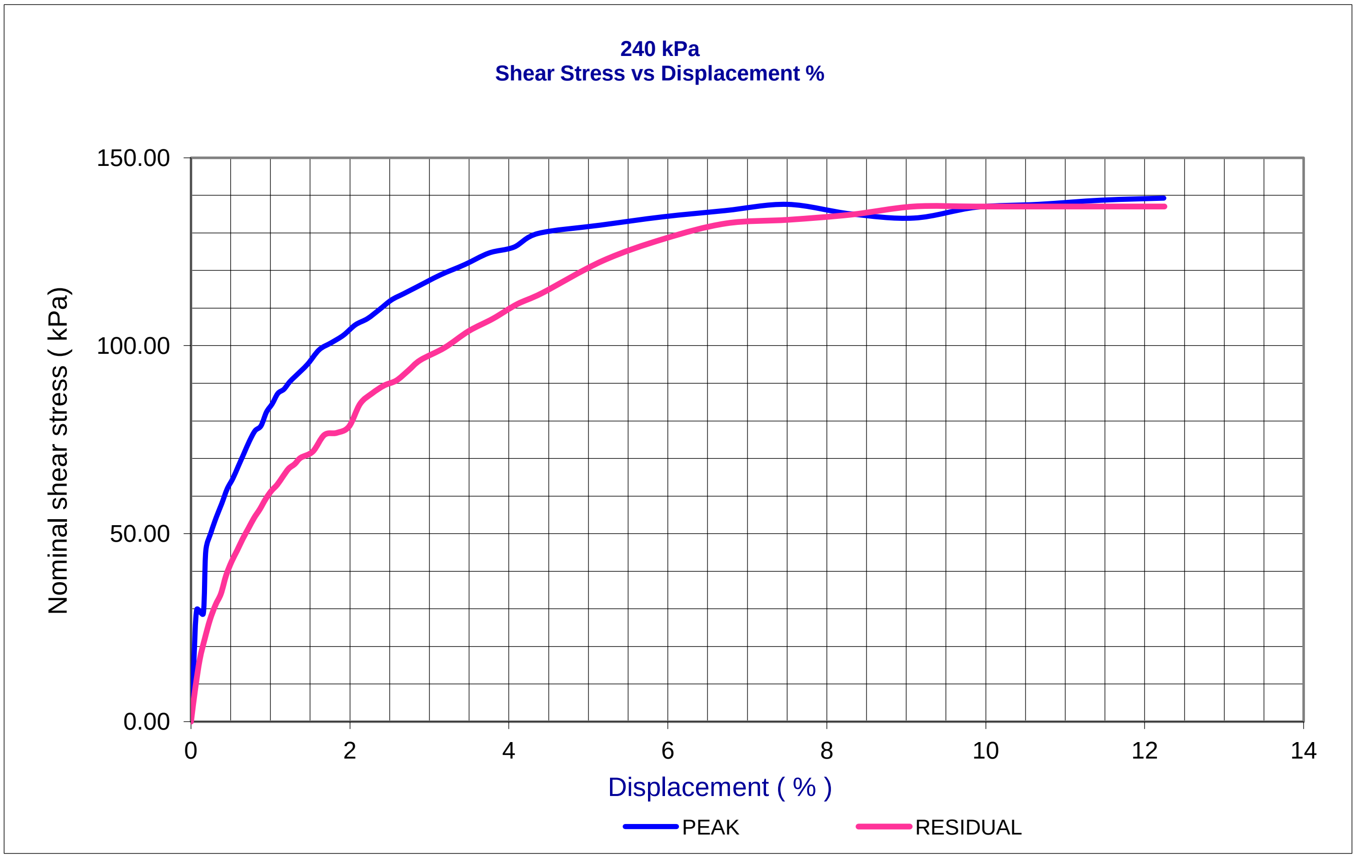
Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**14-November-2011**
Sample ID.:	**T6-3_TW15**	**Tested By:**	**CZ/SB**
Lab No.:	**AdS090_2011**	**Checked By:**	**SB**

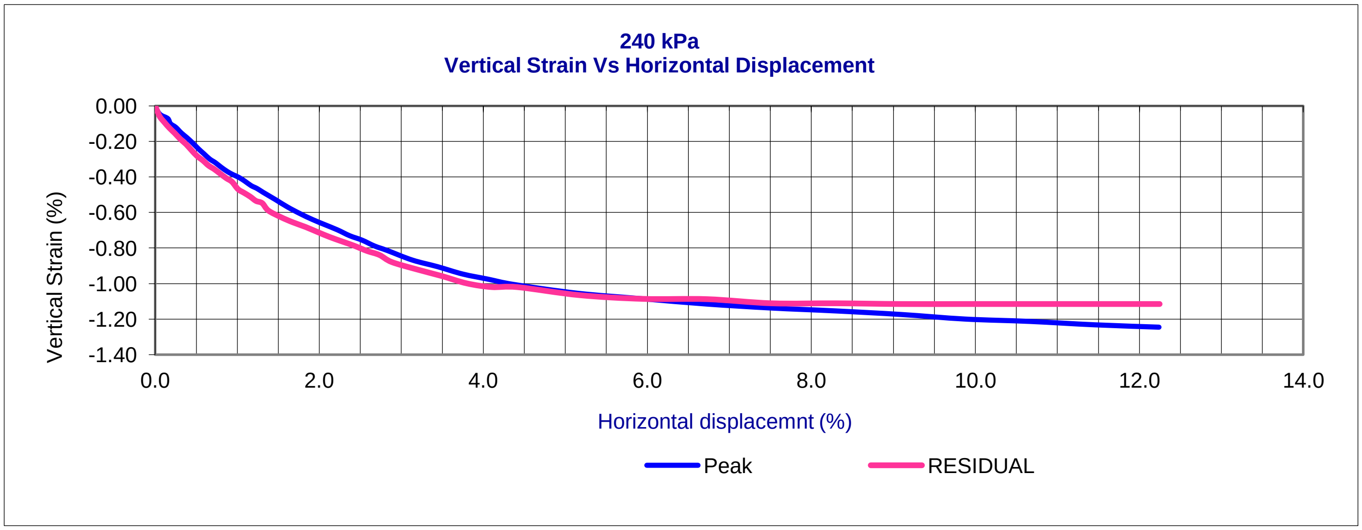




DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**14 November 2011**
Sample ID.:	**T6-3_TW15**	**Tested By:**	**CZ/SB**
Lab No.:	**AdS090_2011**	**Checked By:**	**SB**





Appendix D Analytical Laboratory Test Results

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

Date Received: 25-JUL-11
Report Date: 29-JUL-11 20:50 (MT)
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1035552

Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 112827

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1035552-1	L1035552-2			
	Description	SOIL	SOIL			
	Sampled Date	22-JUL-11	22-JUL-11			
	Sampled Time					
	Client ID	T6-1,TW70,30',SILTY CLAY, GREY	T6-2,TW11,35',SILTY CLAY, GREY			
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	18.9	18.4			
	pH (pH units)	8.11	8.00			
	Redox Potential (mV)	111	105			
	Resistivity (ohm cm)	2600	2820			
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20	<0.20			
Anions and Nutrients	Sulphate (mg/kg)	436	451			

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:

112827

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.

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

Date Received: 18-JUL-11
Report Date: 25-JUL-11 15:04 (MT)
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1032530

Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 092959-E

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L1032530-1 T6-3,SS10@30' GREY SILTY CLAY Sampled By: CLIENT on 15-JUL-11 Matrix: SOIL							
Physical Tests							
% Moisture	19.6		0.10	%	18-JUL-11	18-JUL-11	R2220531
pH	8.14		0.10	pH units	22-JUL-11	22-JUL-11	R2223567
Redox Potential	111		-1000	mV	22-JUL-11	22-JUL-11	R2223536
Resistivity	2310		100	ohm cm	22-JUL-11	22-JUL-11	R2223537
Leachable Anions & Nutrients							
Sulphide	<0.20		0.20	mg/kg	21-JUL-11	21-JUL-11	R2222299
Anions and Nutrients							
Sulphate	374		20	mg/kg	20-JUL-11	20-JUL-11	R2222247

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

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:

092959-E

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
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 weight of sample
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
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.

Appendix E Core Photographs

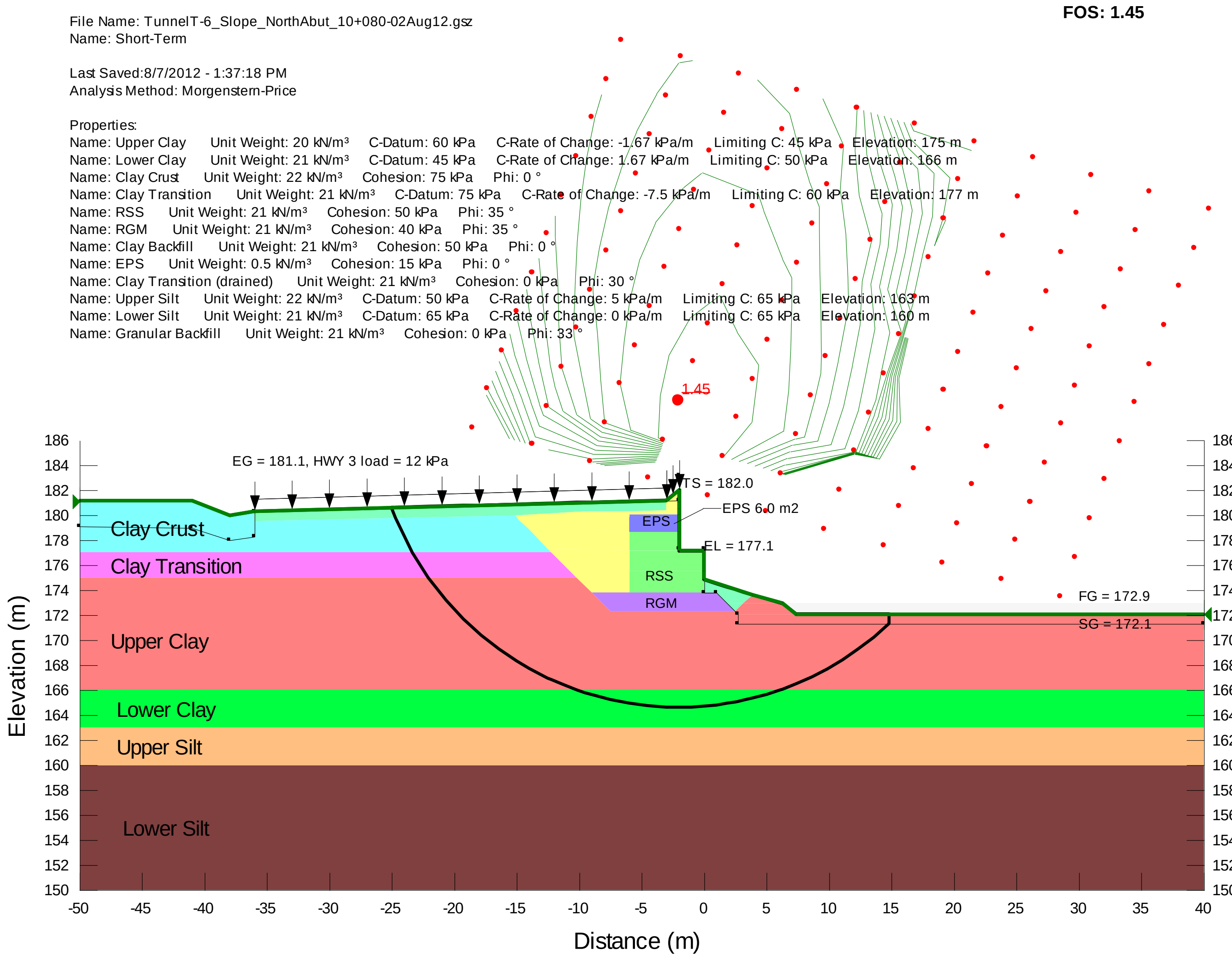
Photograph E-1: Borehole T6-1 – Rock Core Elevation 147.2 to 145.4m

Photograph E-2: Borehole T6-2 Rock Core Elevation 148.3 to 146.1m

Photograph E-3: Borehole T6-3 – Rock Core Elevation 146.9 to 145.3 m

Appendix F Slope Stability Analysis Results

Figure F-1: Global Stability Result – North Abutment (10+080)– Short Term (Undrained) Loading



Project: Windsor-Essex Parkway

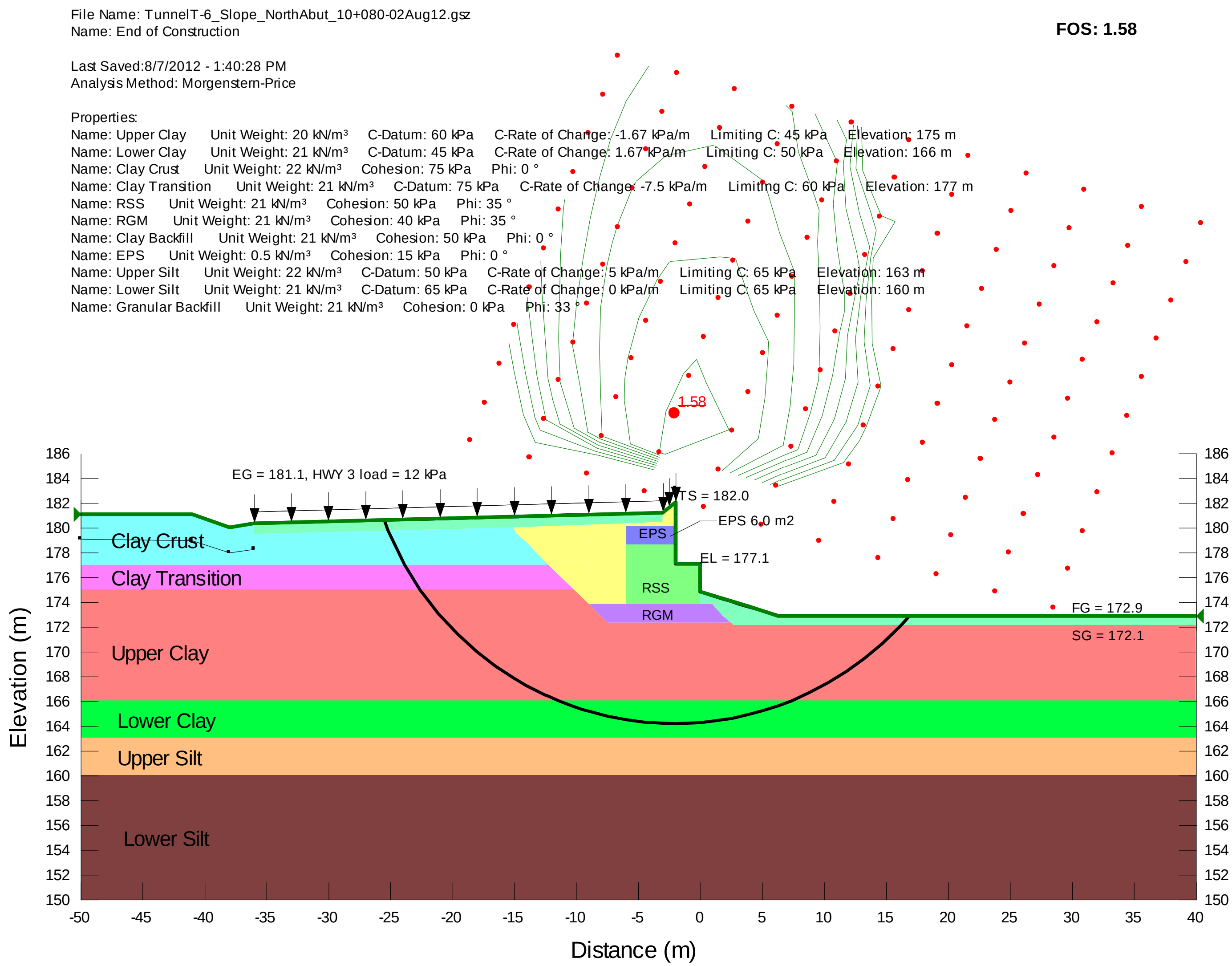
Document: Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)

Doc No.: 285380-04-119-0084 (Geocres No. 40J6-44)

Date: September / 2012

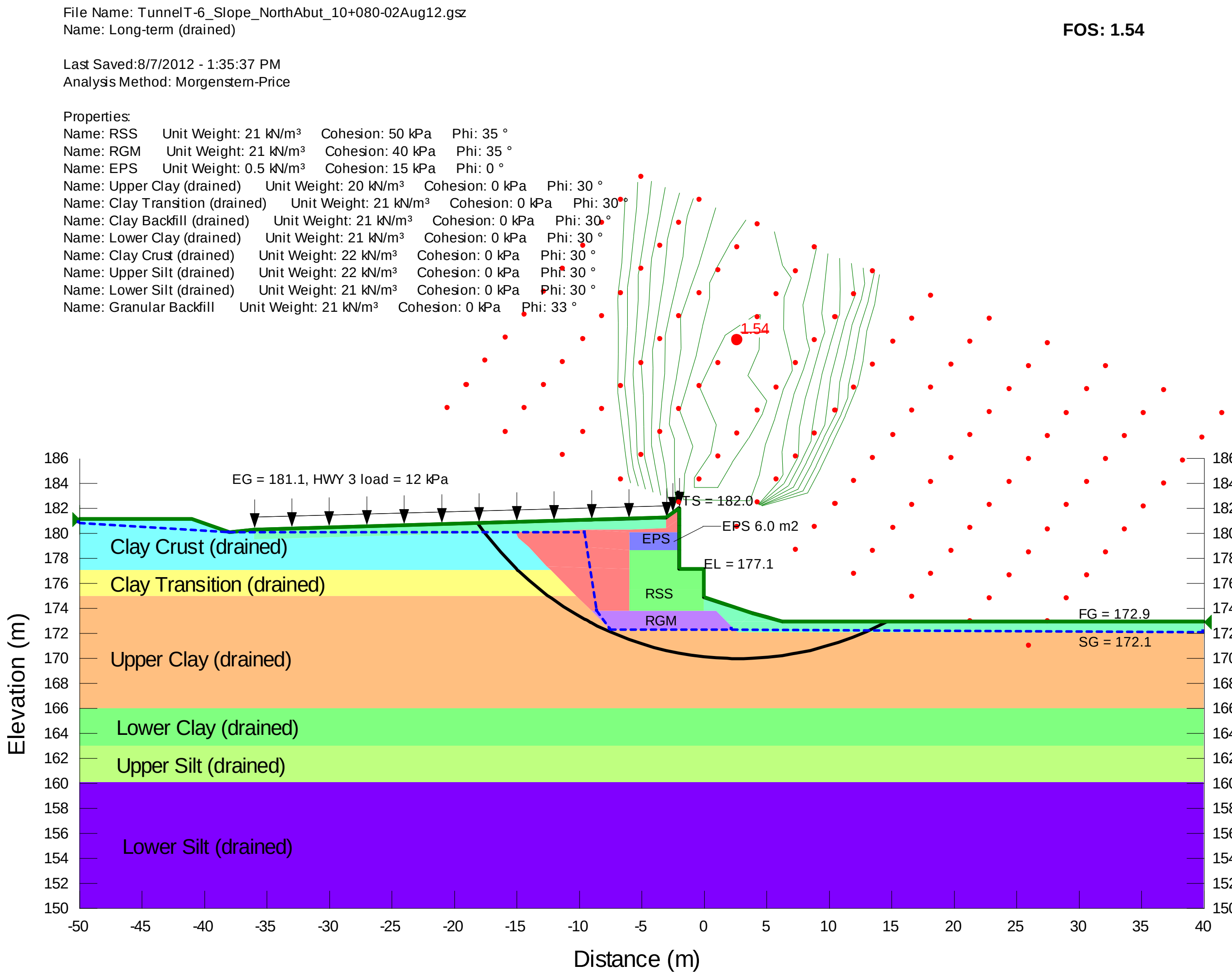
Rev: 0

Figure F-2: Global Stability Result – North Abutment (10+080)– End of Construction (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 2 of 27

Figure F-3: Global Stability Result – North Abutment (10+080)– Long-term (Drained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 3 of 27

Figure F-4: Global Stability Result – North Abutment (10+125) – Short Term (Undrained) Loading

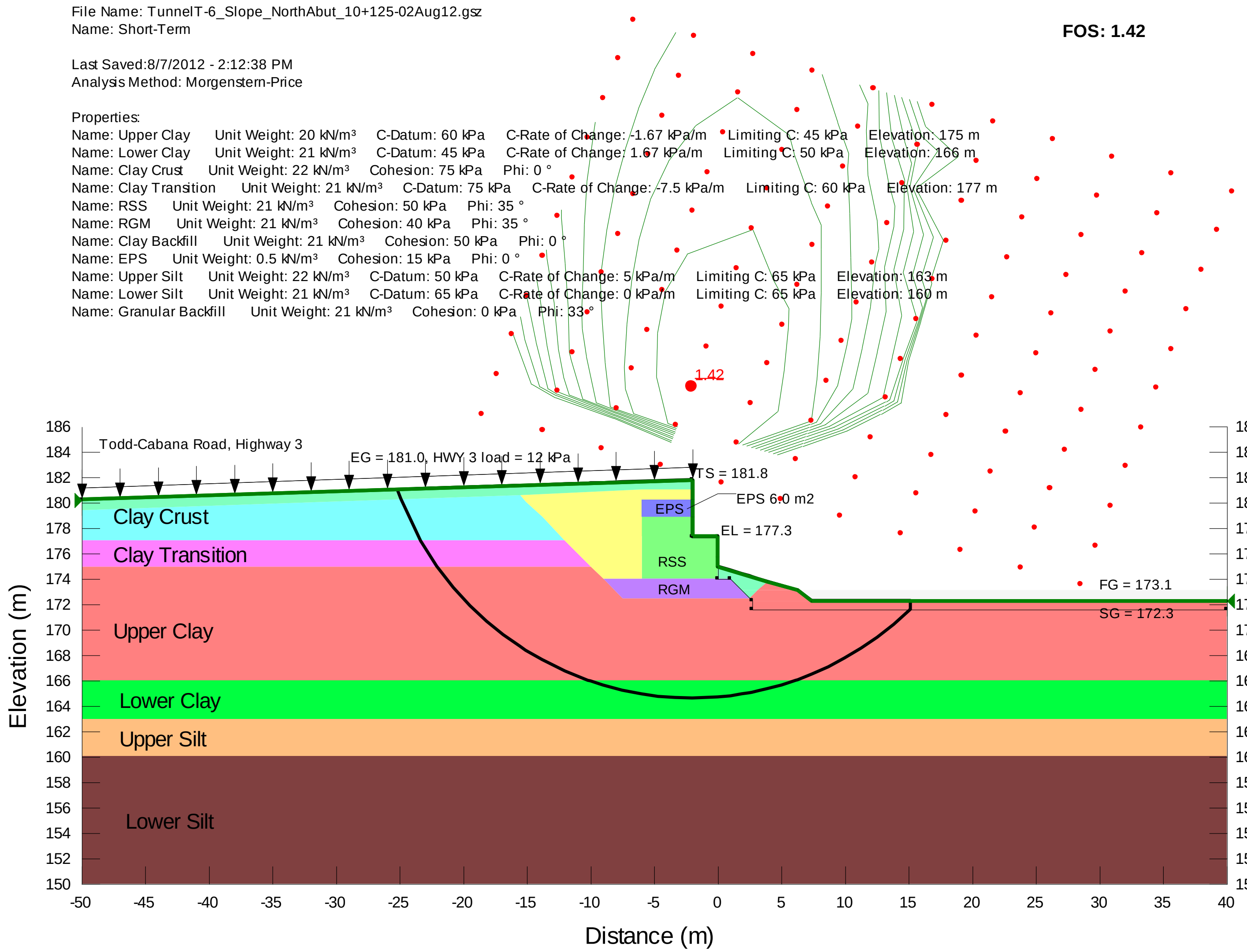
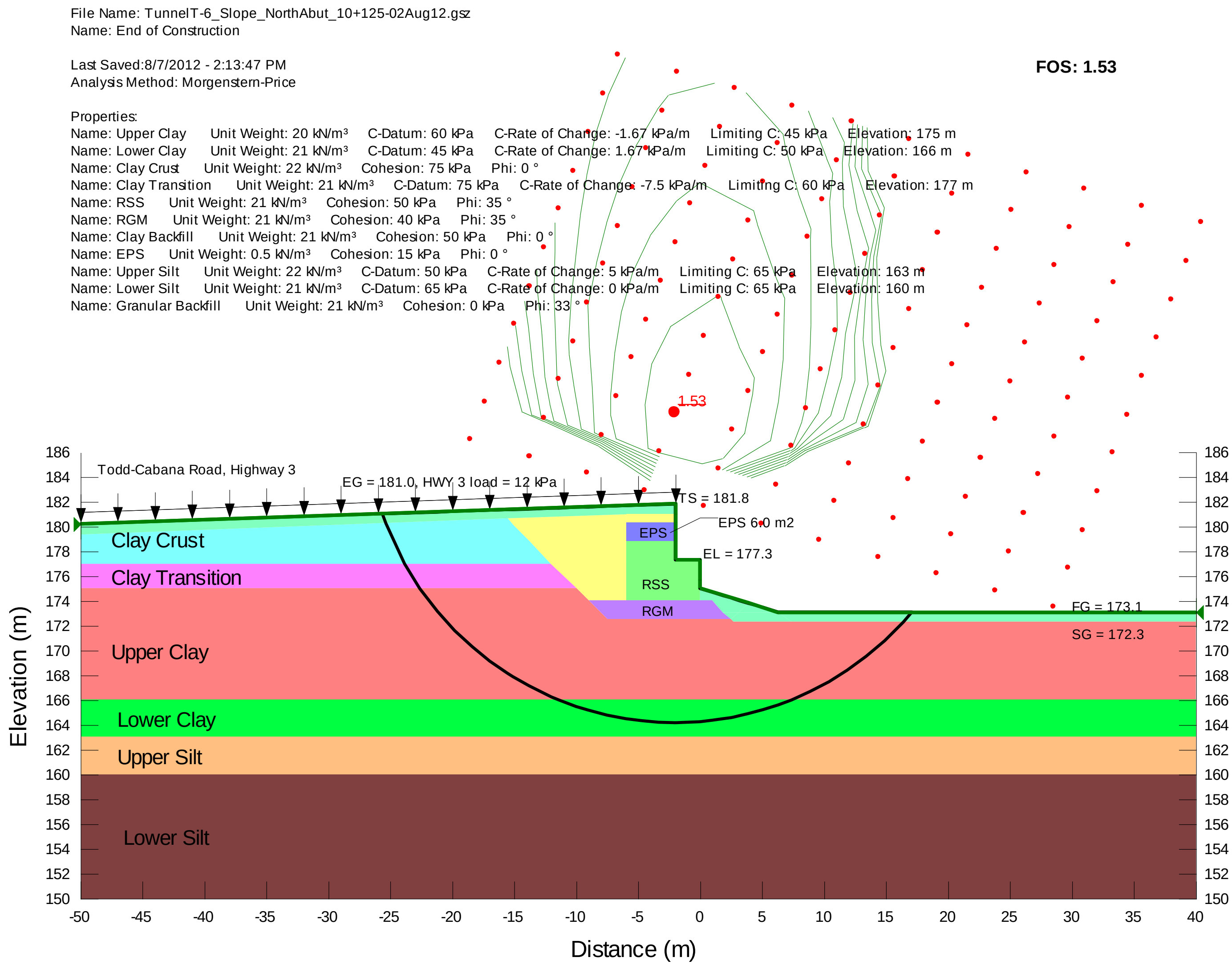
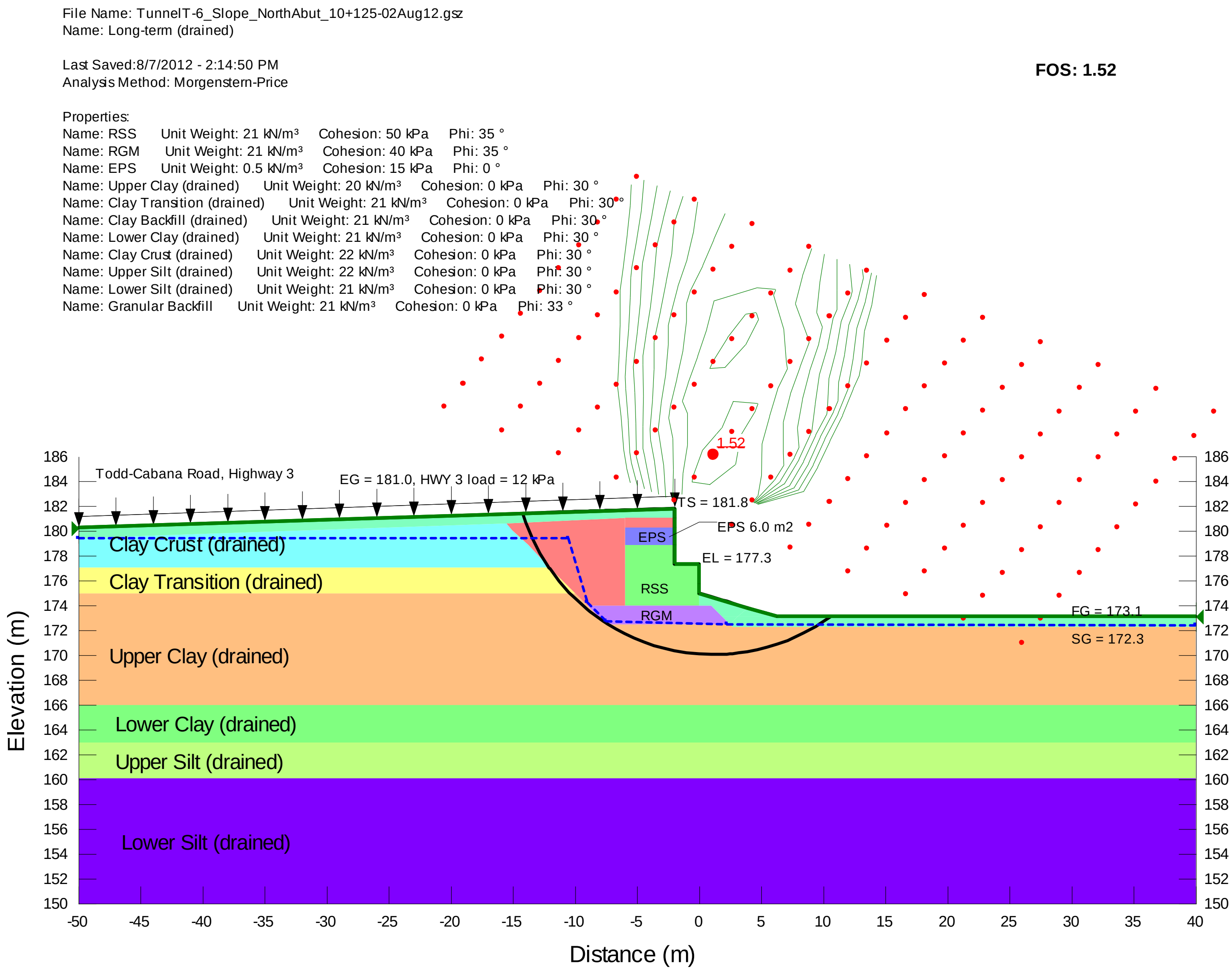


Figure F-5: Global Stability Result – North Abutment (10+125) – End of Construction (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 5 of 27

Figure F-6: Global Stability Result – North Abutment (10+125) – Long Term (Drained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 6 of 27

Figure F-7: Global Stability Result – North Abutment (10+170) – Short Term (Undrained) Loading

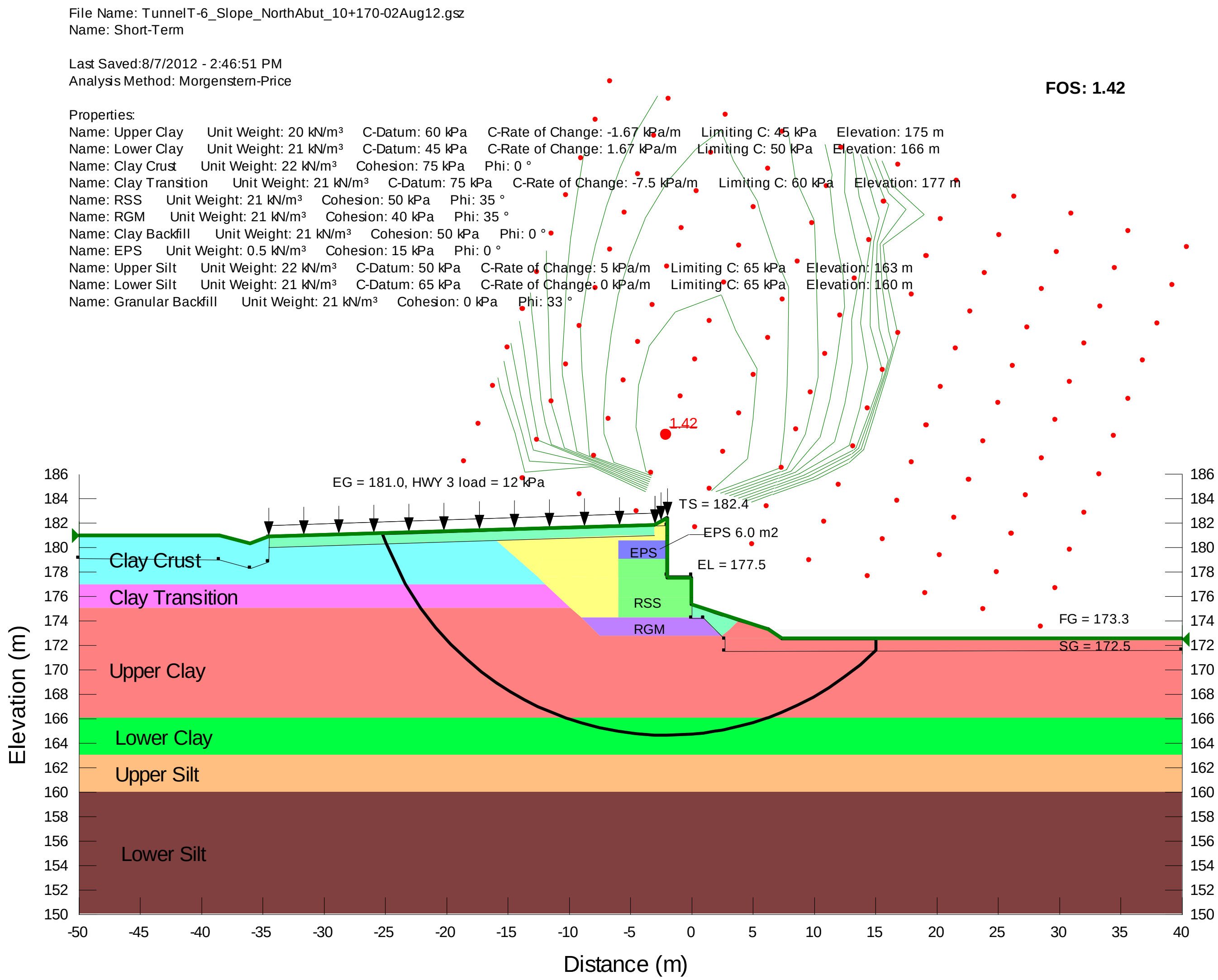
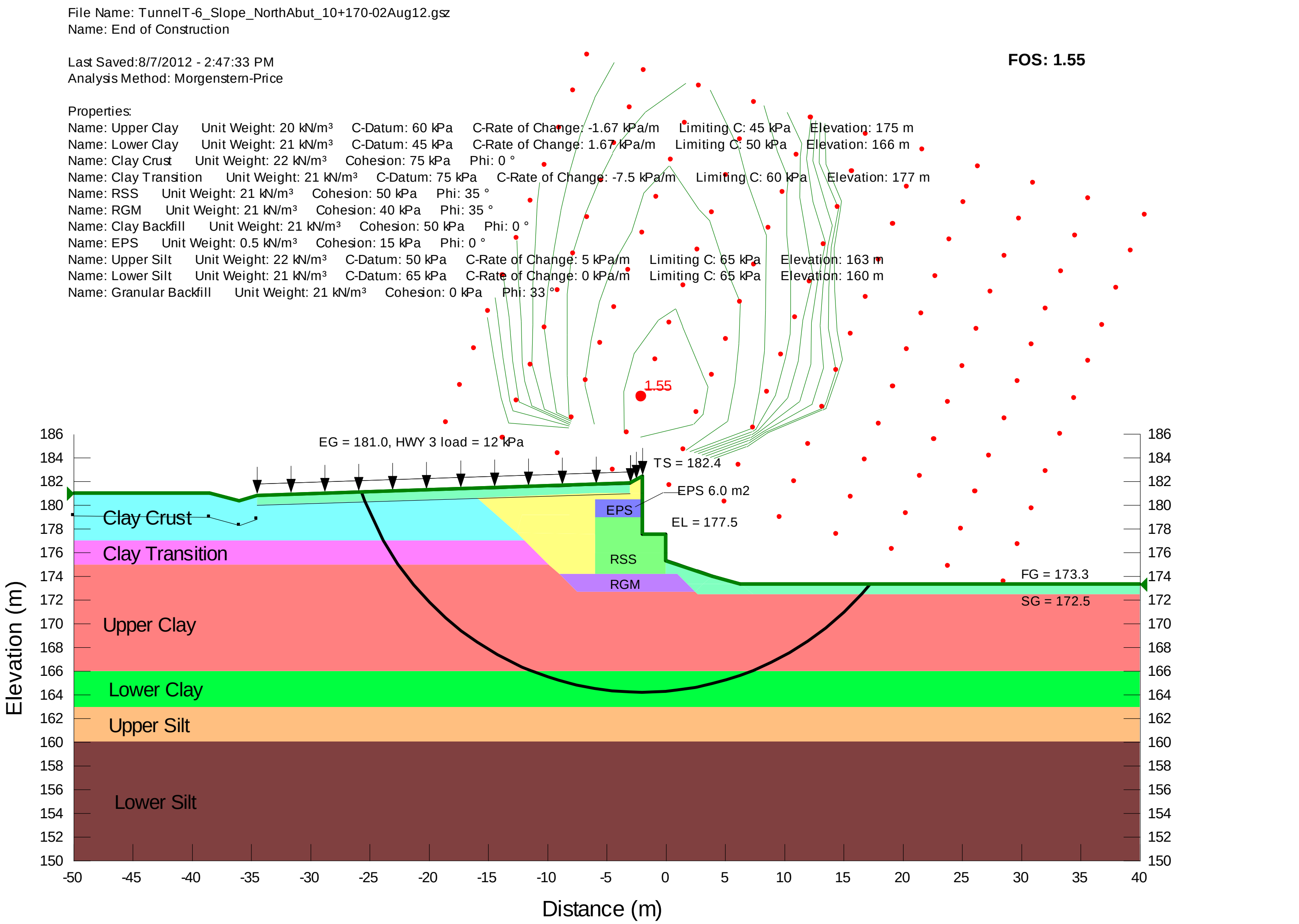


Figure F-8: Global Stability Result – North Abutment (10+170) – End of Construction (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 8 of 27

Figure F-9: Global Stability Result – North Abutment (10+170) – Long Term (Drained) Loading

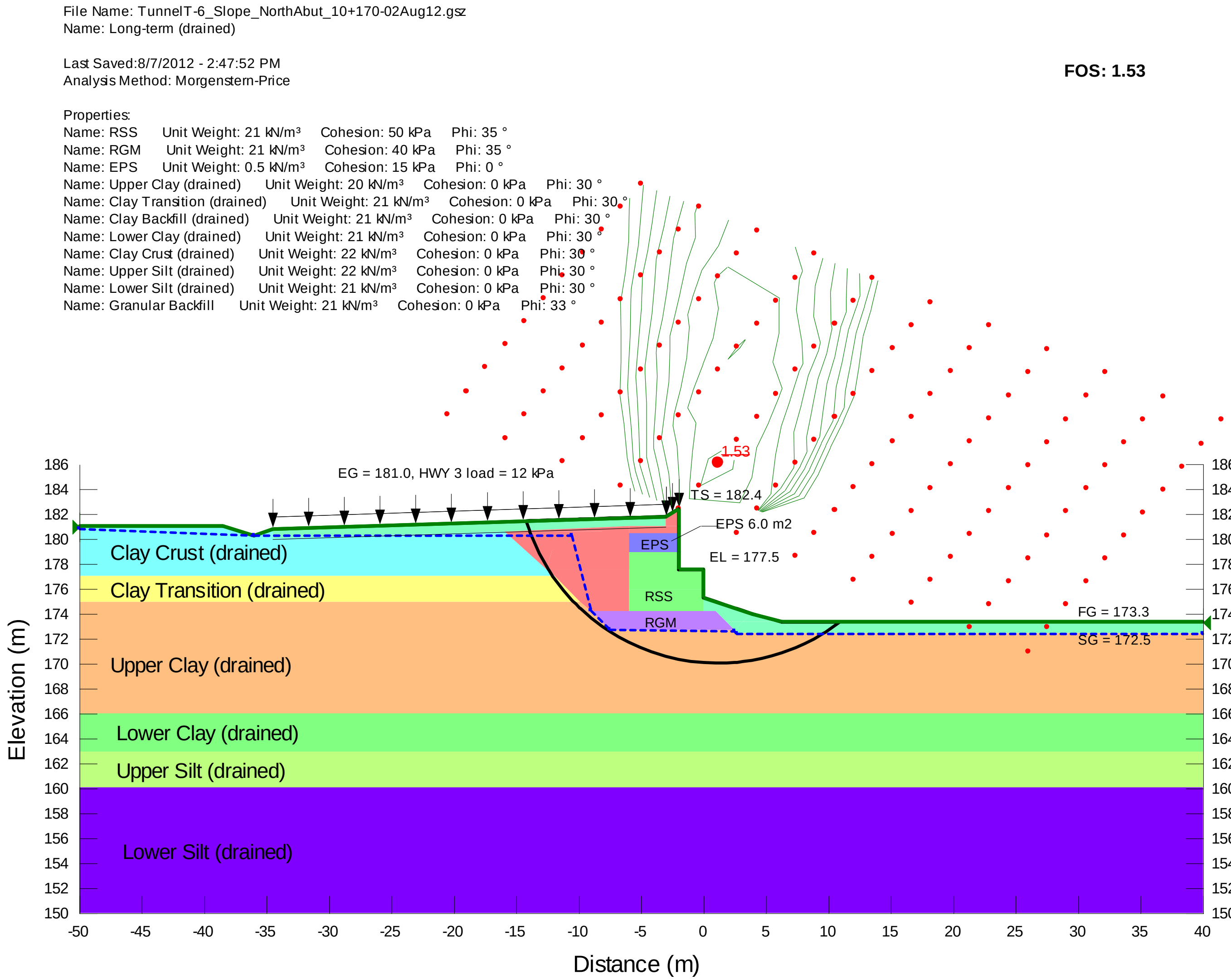
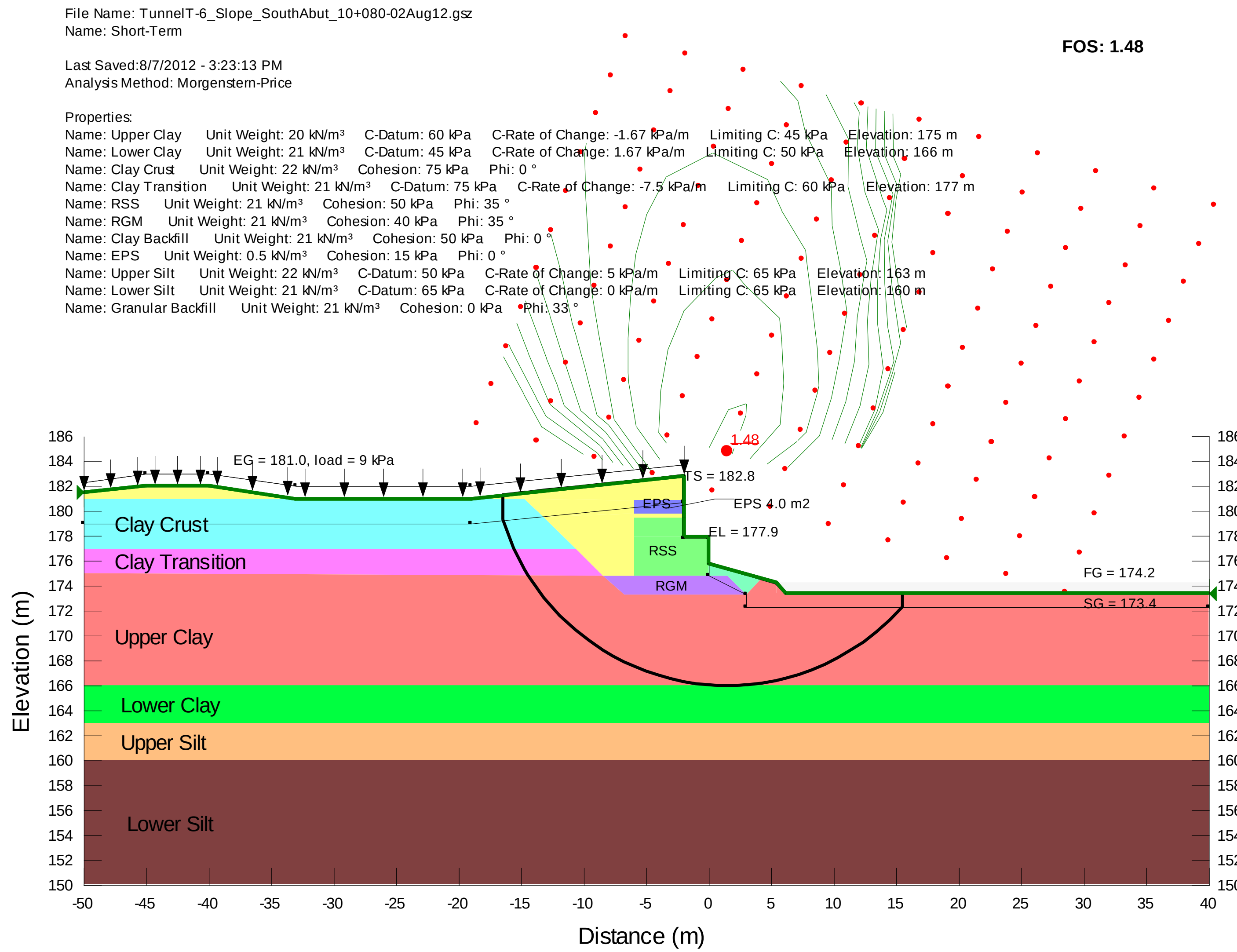
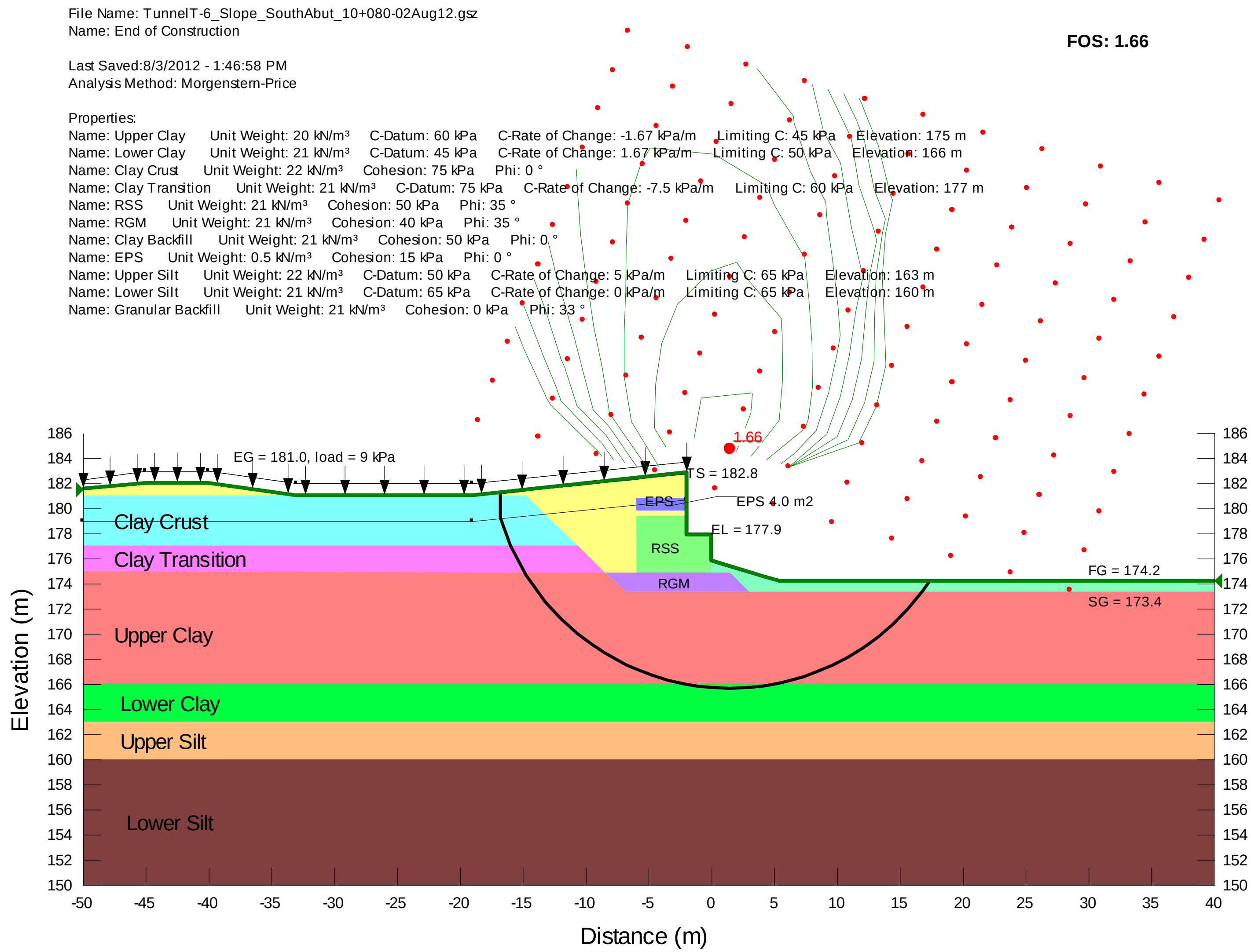


Figure F-10: Global Stability Result - South Abutment (10+080) - Short Term (Undrained) Loading



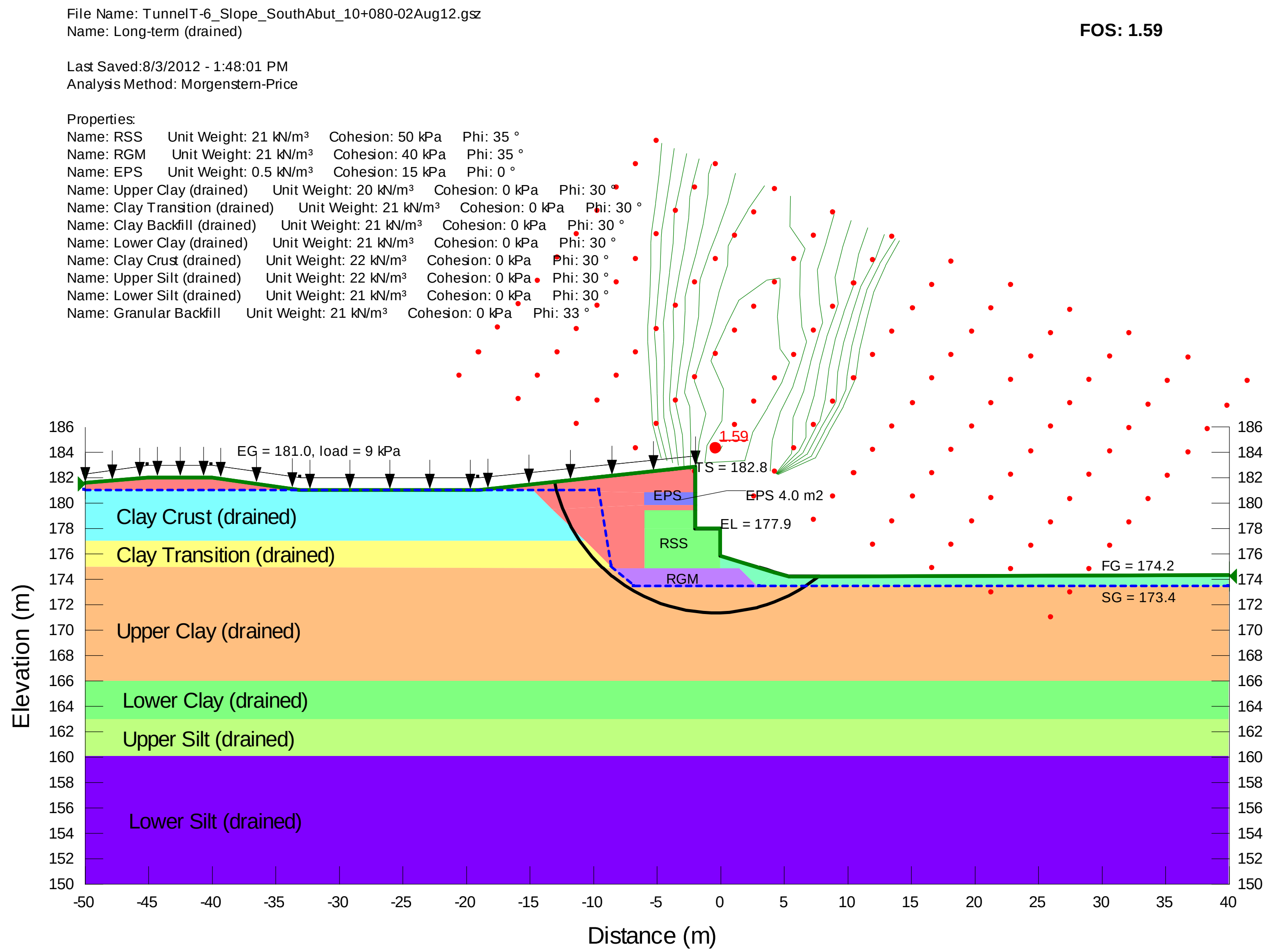
Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 10 of 27

Figure F-11: Global Stability Result – South Abutment (10+080) – End of Construction (Undrained) Loading



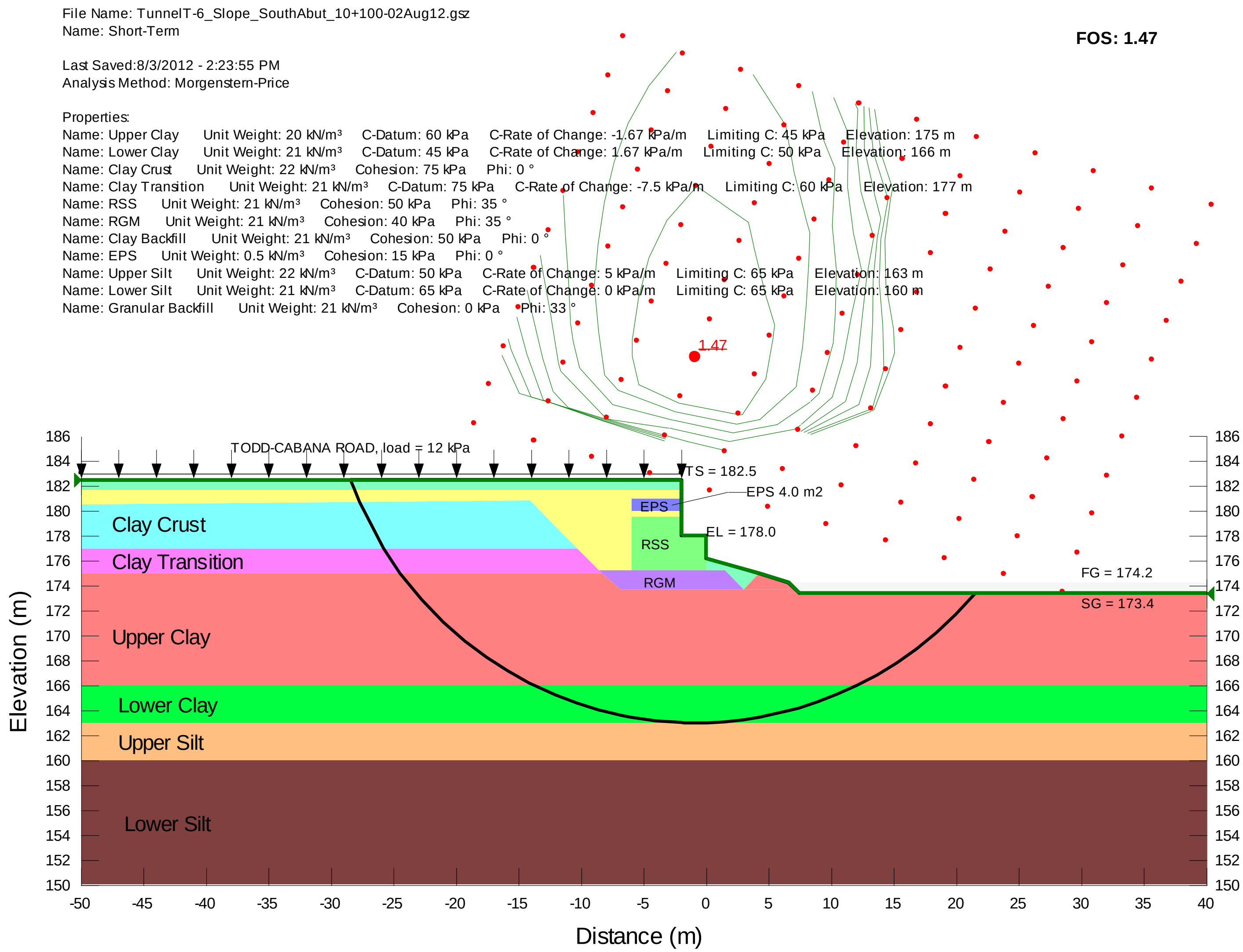
Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 11 of 27

Figure F-12: Global Stability Result – South Abutment (10+080) – Long Term (Drained) Loading



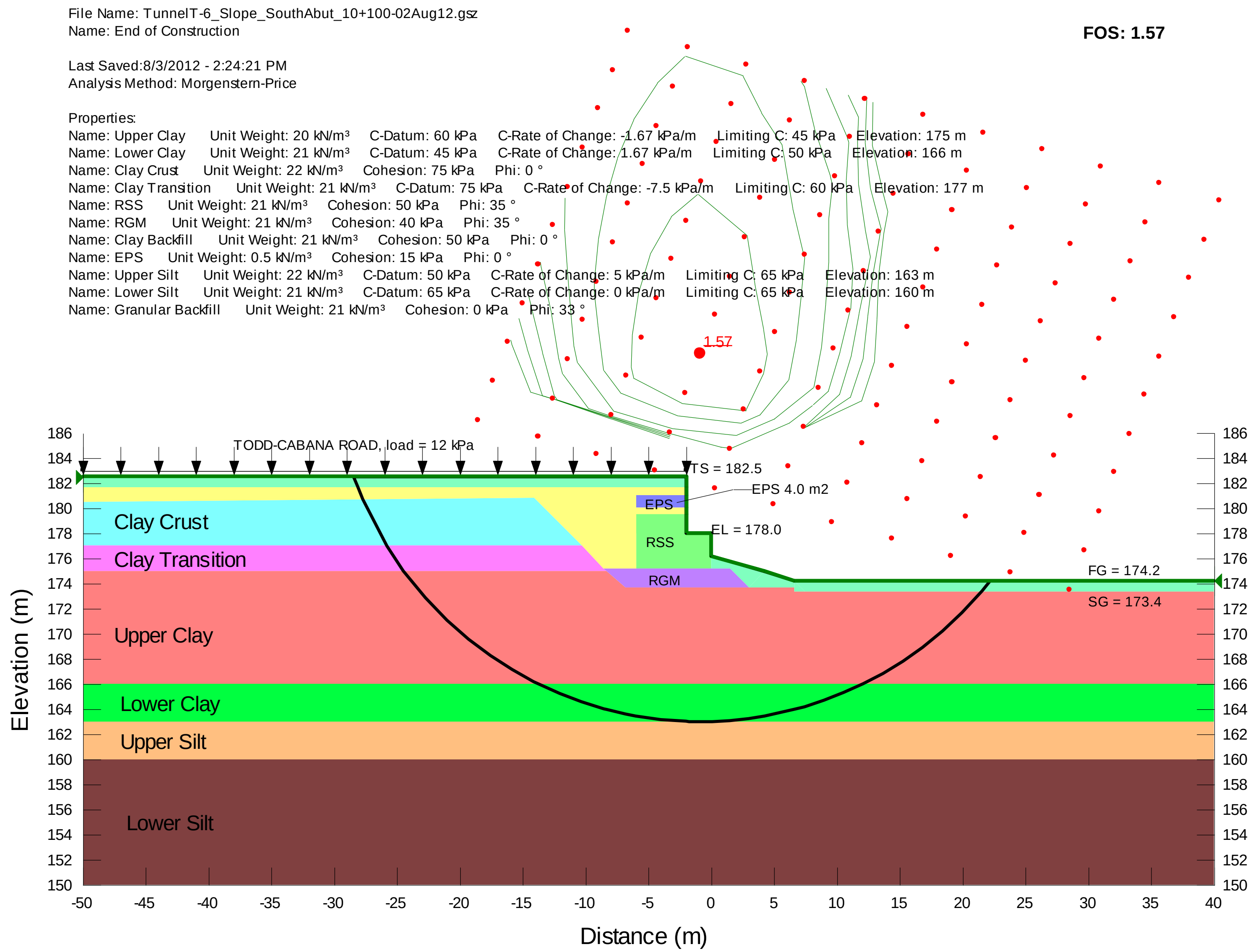
Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 12 of 27

Figure F-13: Global Stability Result - South Abutment (10+100) - Short Term (Undrained) Loading



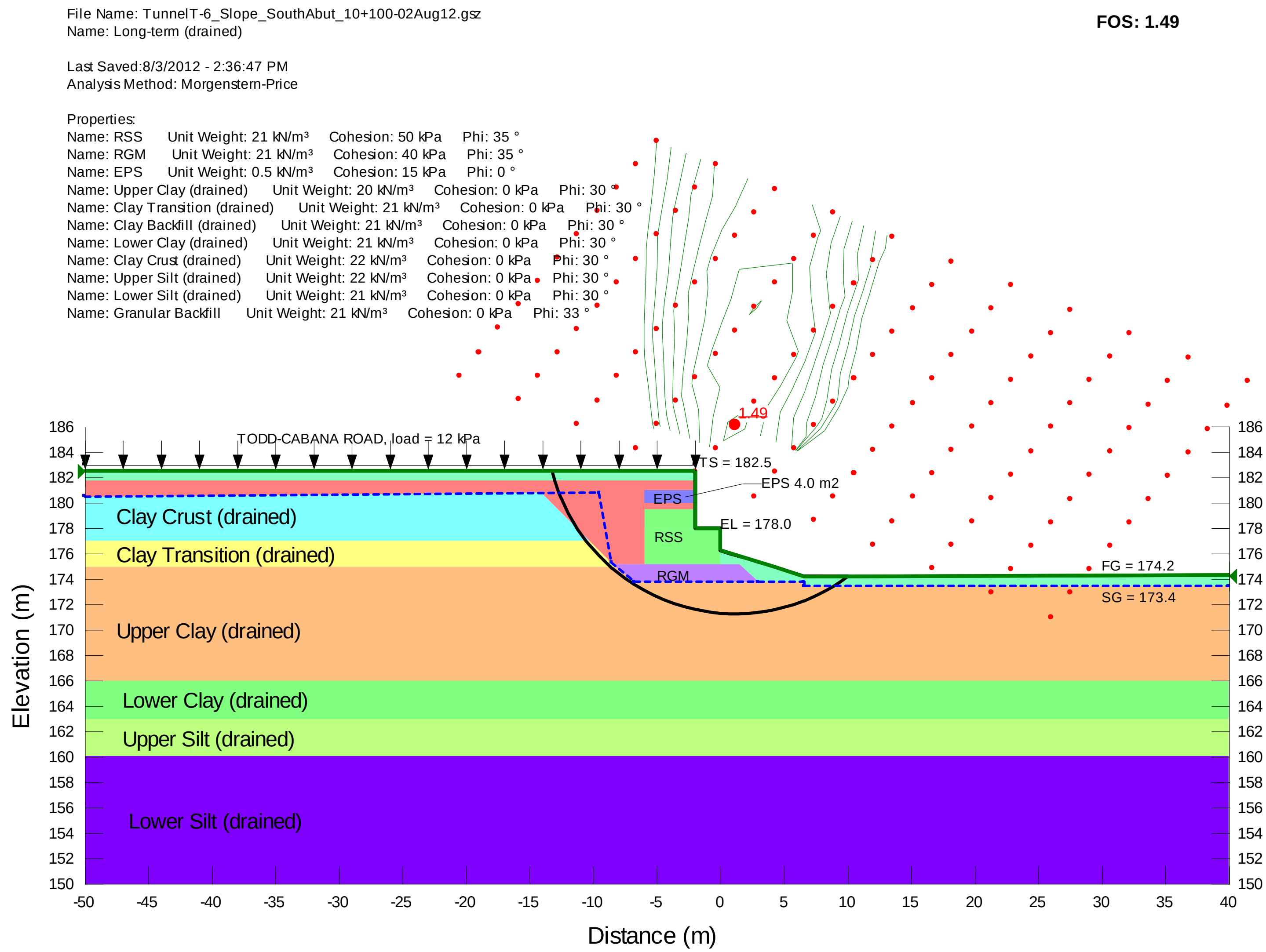
Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 13 of 27

Figure F-14: Global Stability Result – South Abutment (10+100) – End of Construction (Undrained) Loading



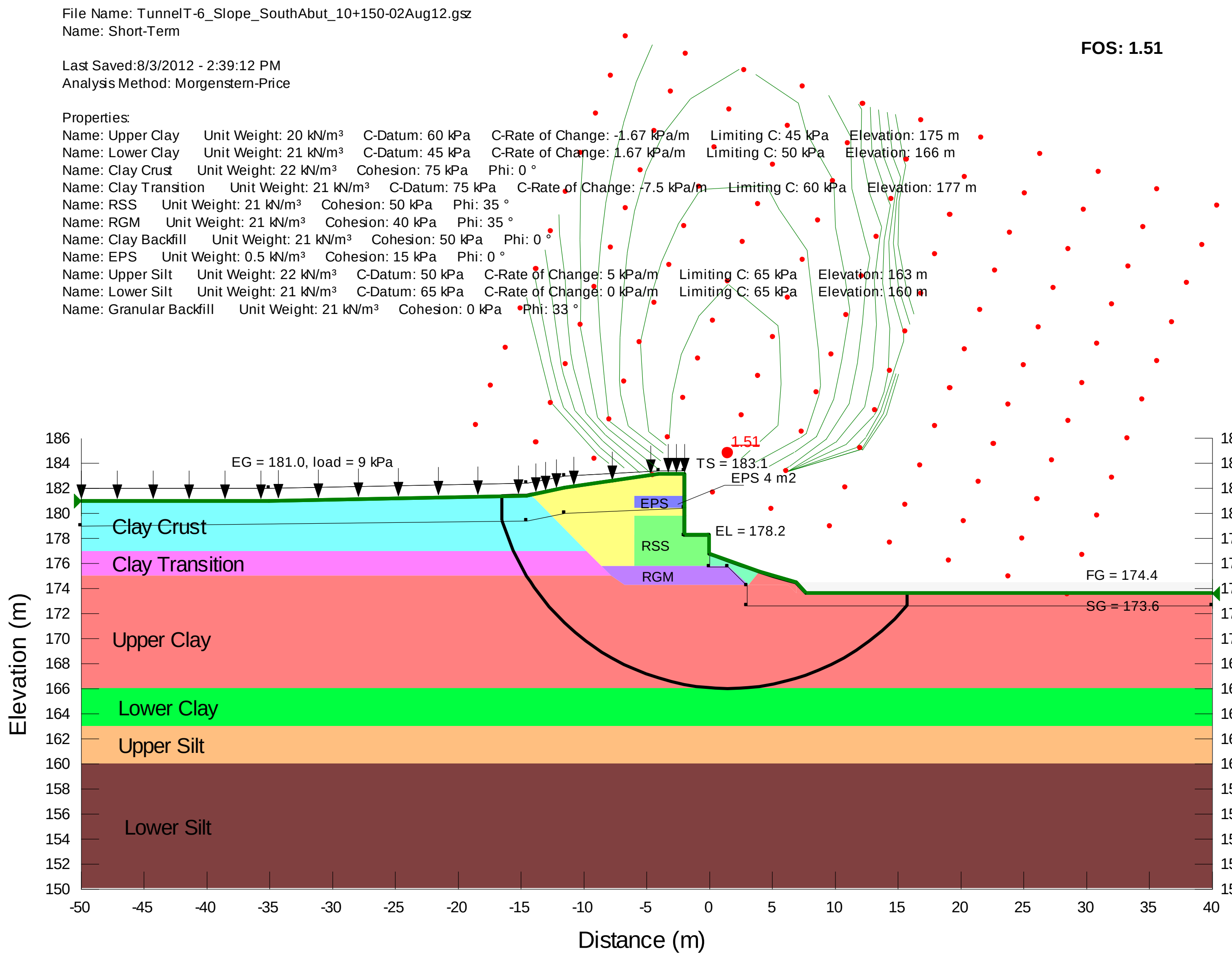
Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 14 of 27

Figure F-15: Global Stability Result – South Abutment (10+100) – Long Term (Drained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 15 of 27

Figure F-16: Global Stability Result – South Abutment (10+150) – Short Term (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 16 of 27

Figure F-17: Global Stability Result - South Abutment (10+150) - End of Construction (Undrained) Loading

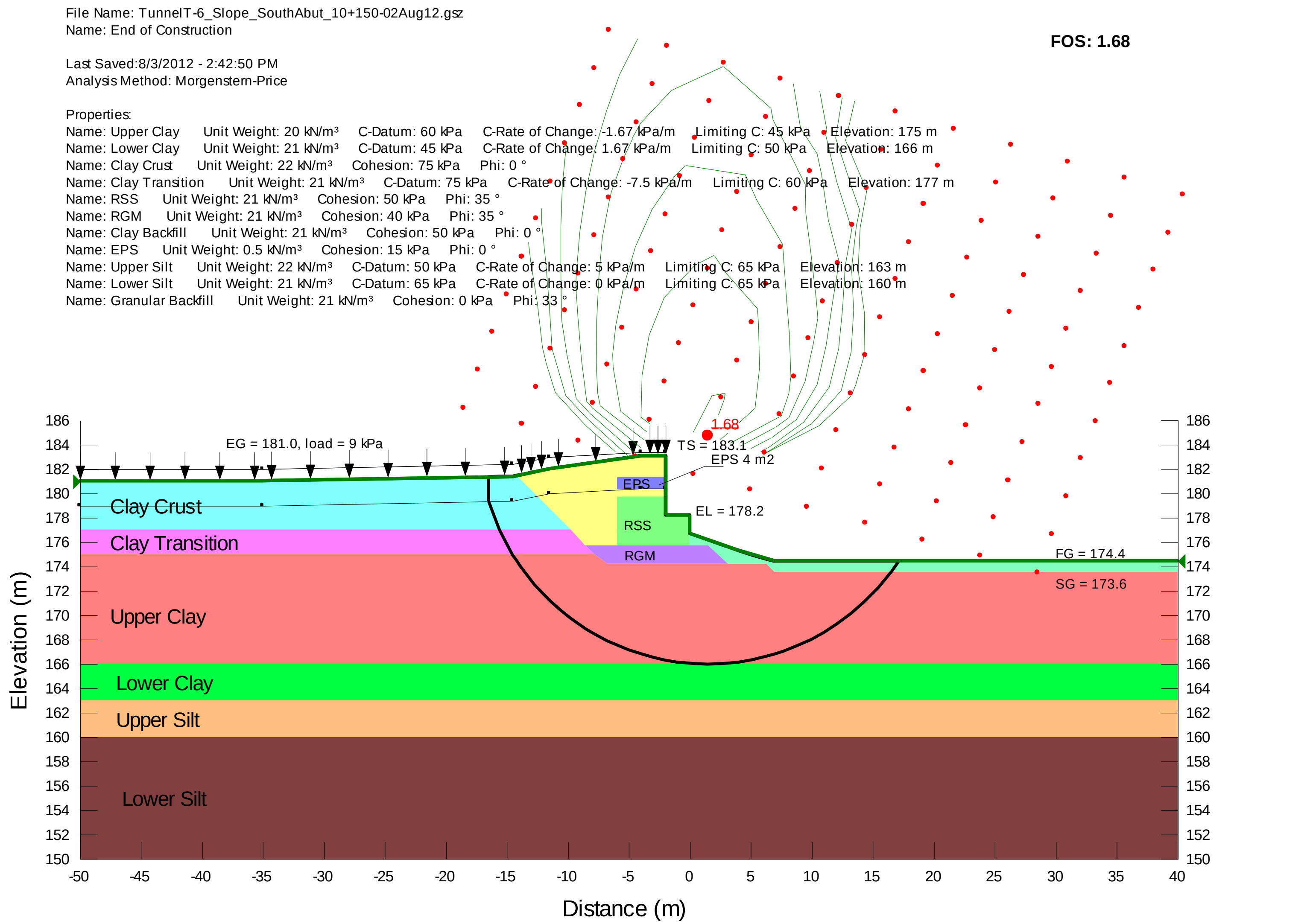
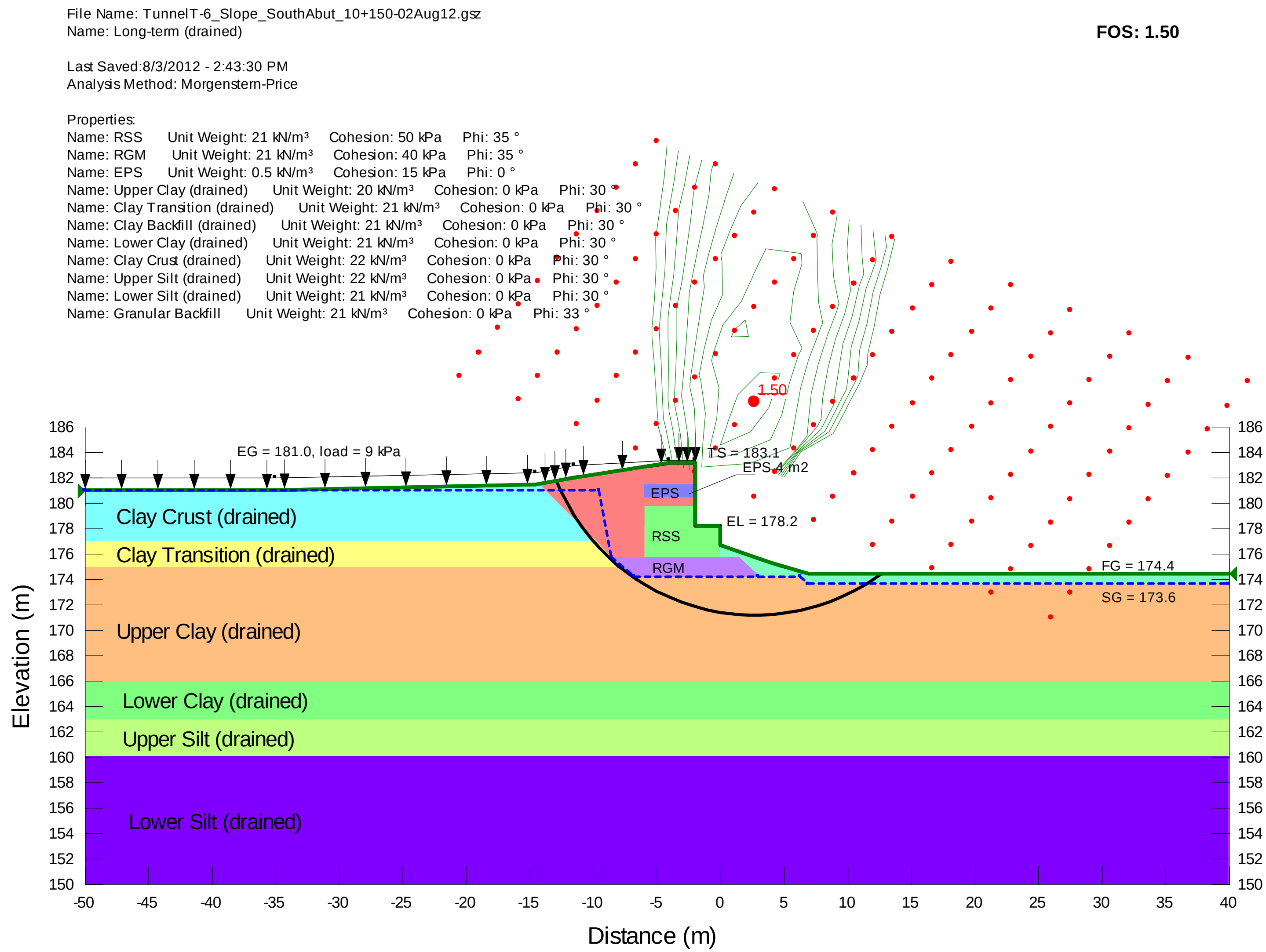


Figure F-18: Global Stability Result – South Abutment (10+150) – Long Term (Drained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 18 of 27

Figure F-19: Global Stability Result – Extension Wing Wall North (10+200) – Short Term (Undrained) Loading

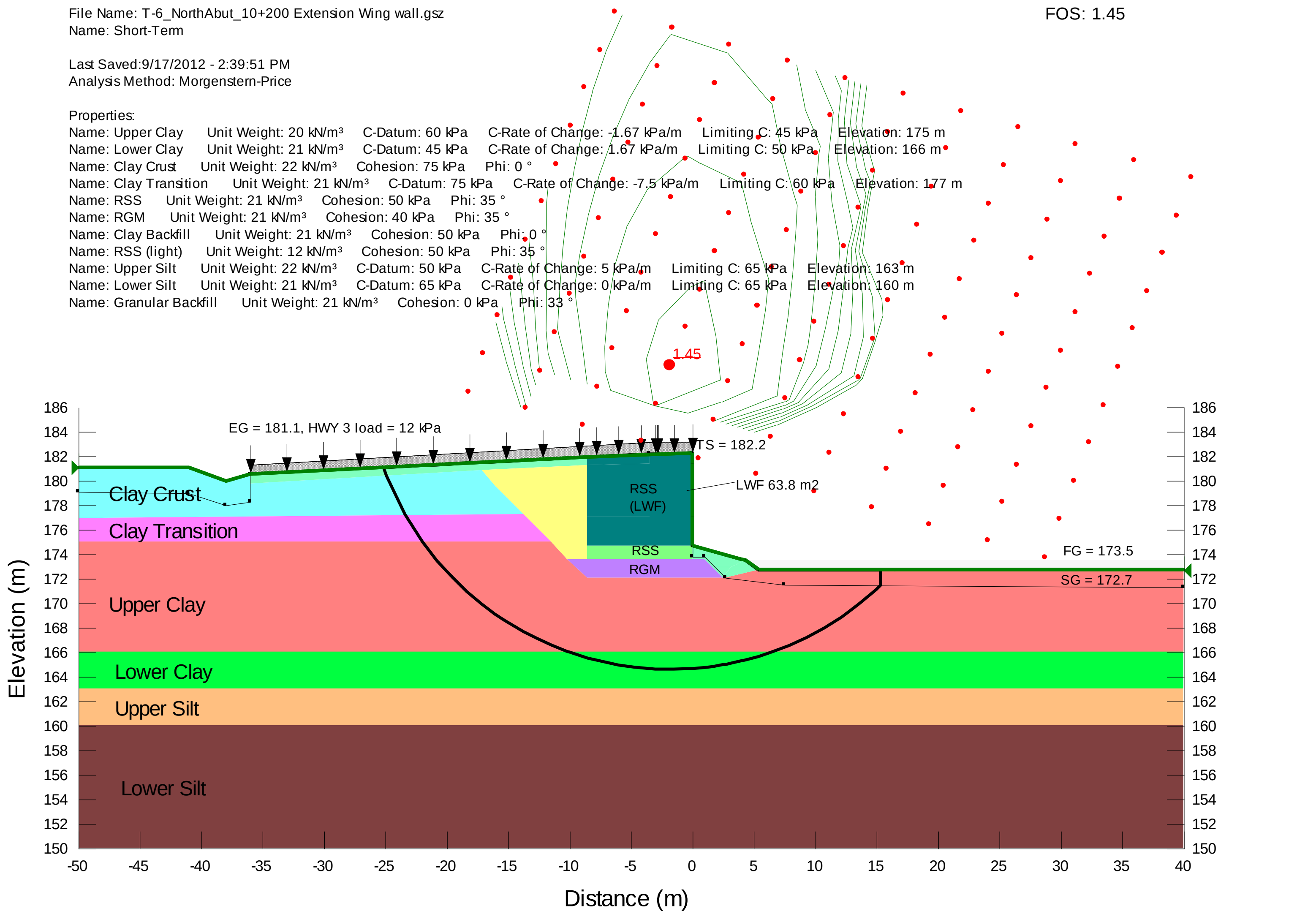
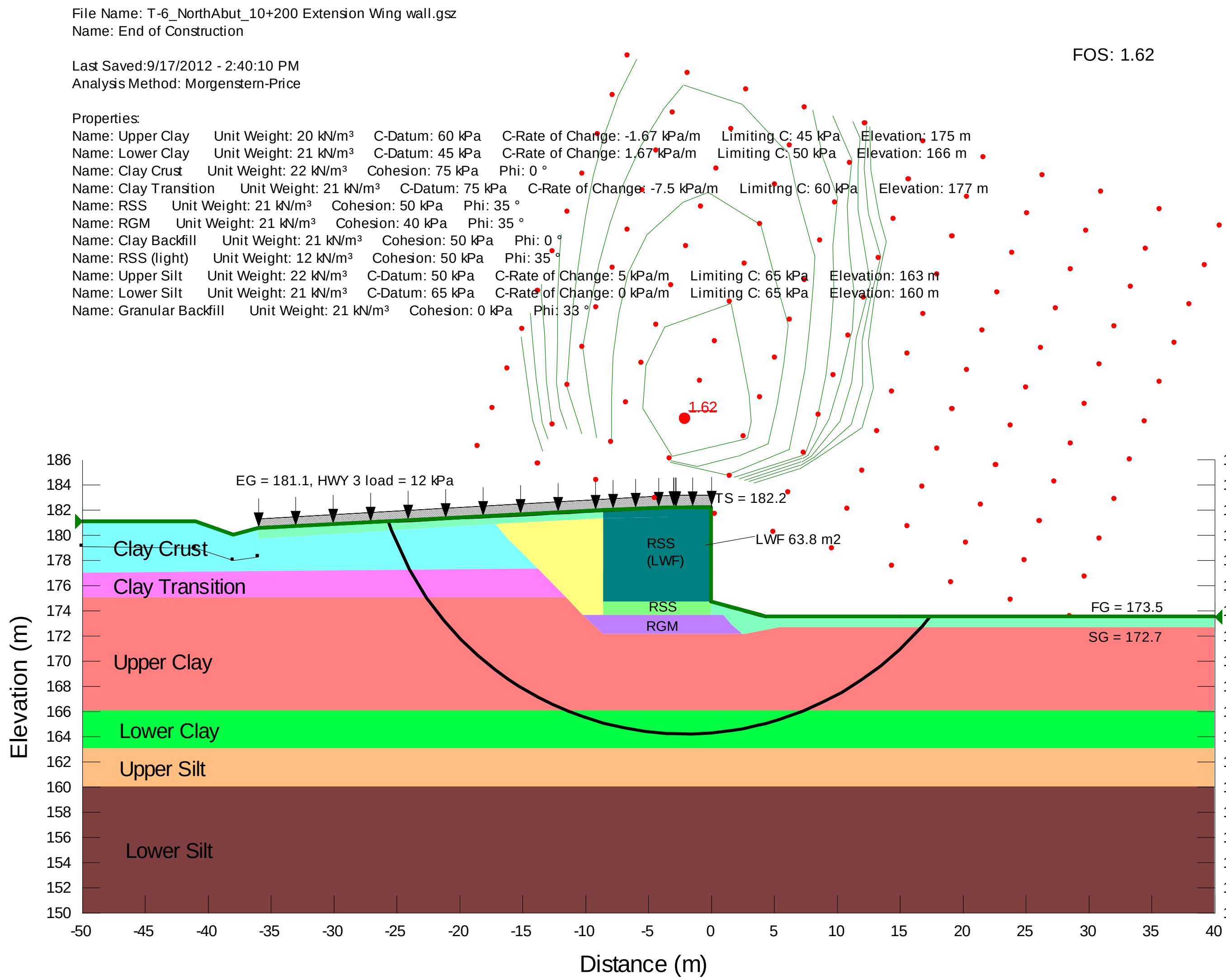


Figure F-20: Global Stability Result – Extended Wing Wall North (10+200) – End of Construction (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 20 of 27

Figure F-21: Global Stability Result – Extended Wing Wall North (10+200) – Long Term (Drained) Loading

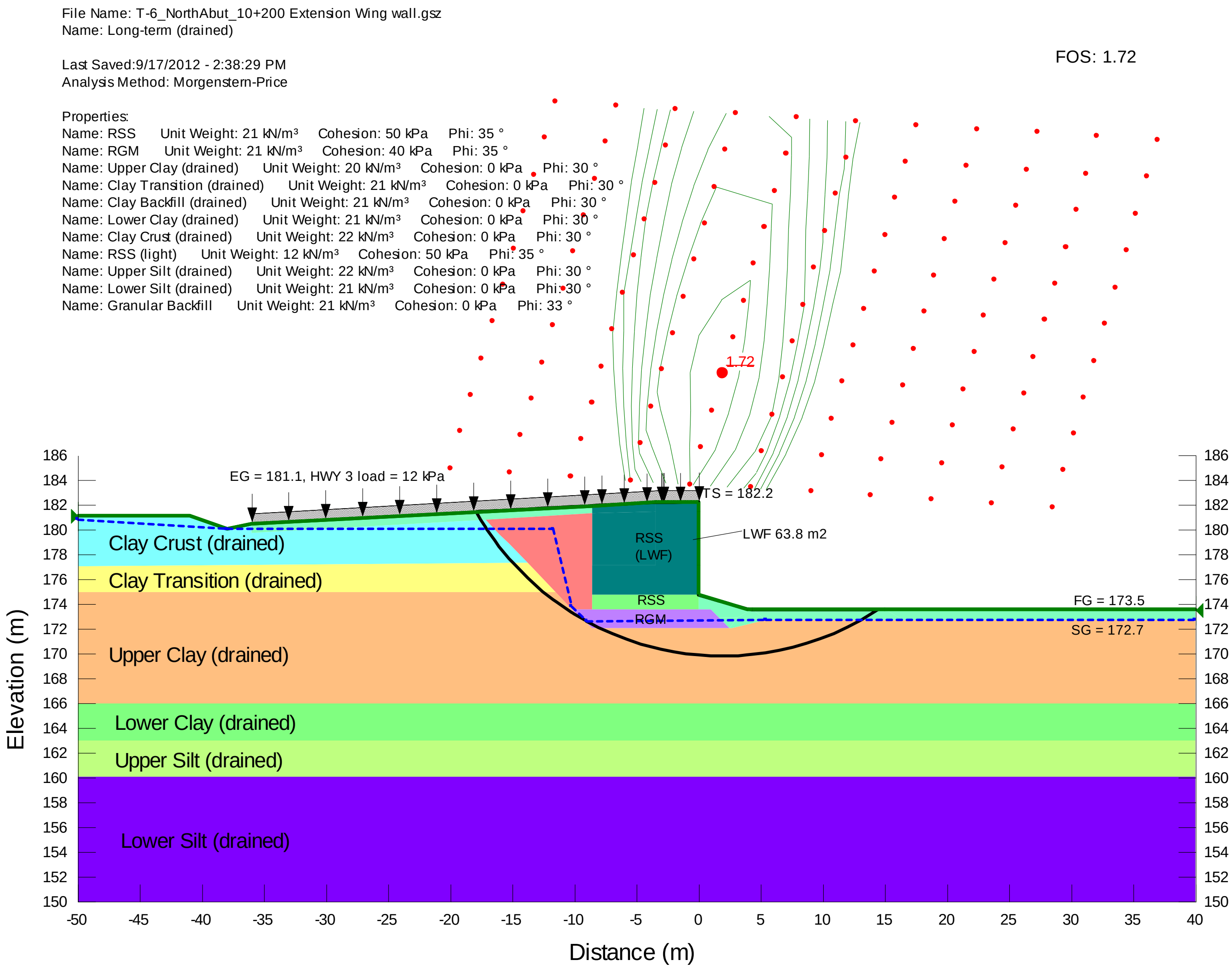


Figure F-22: Global Stability Result – Tapered Wing Wall South (10+080) – Short Term (Undrained) Loading

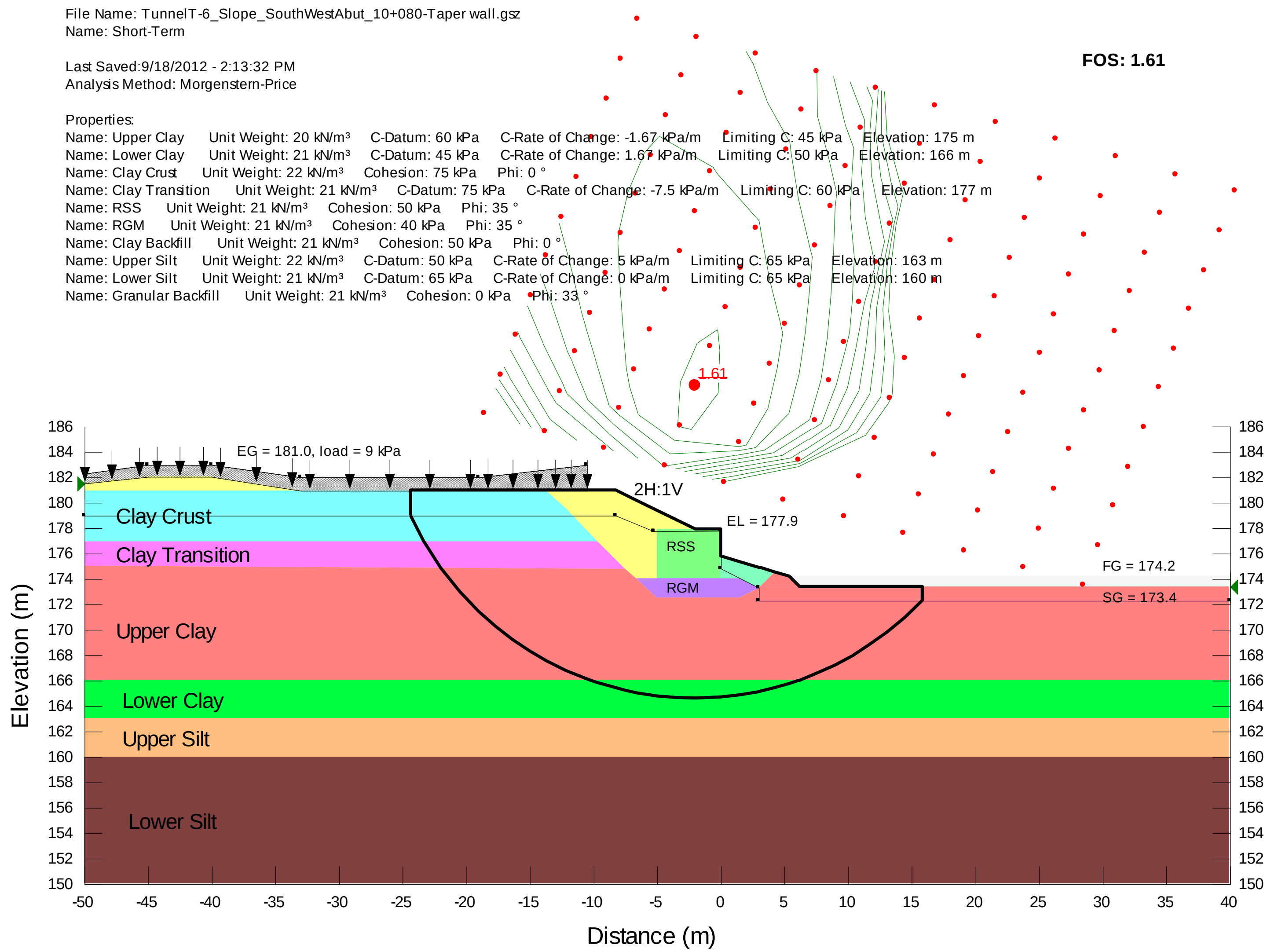
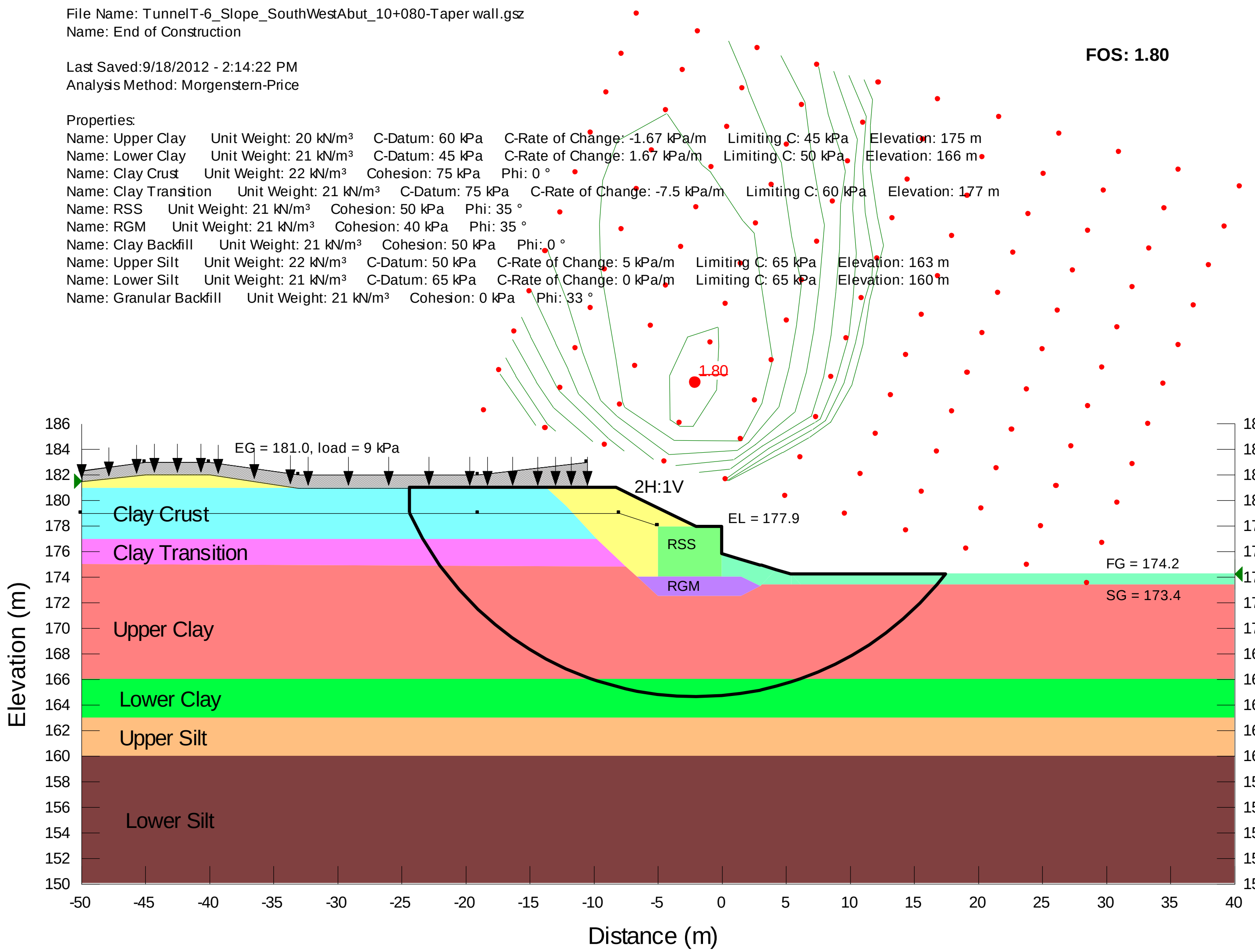


Figure F-23: Global Stability Result – Tapered Wing Wall South (10+080) – End of Construction (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 23 of 27

Figure F-24: Global Stability Result – Tapered Wing Wall South (10+080) – Long Term (Drained) Loading

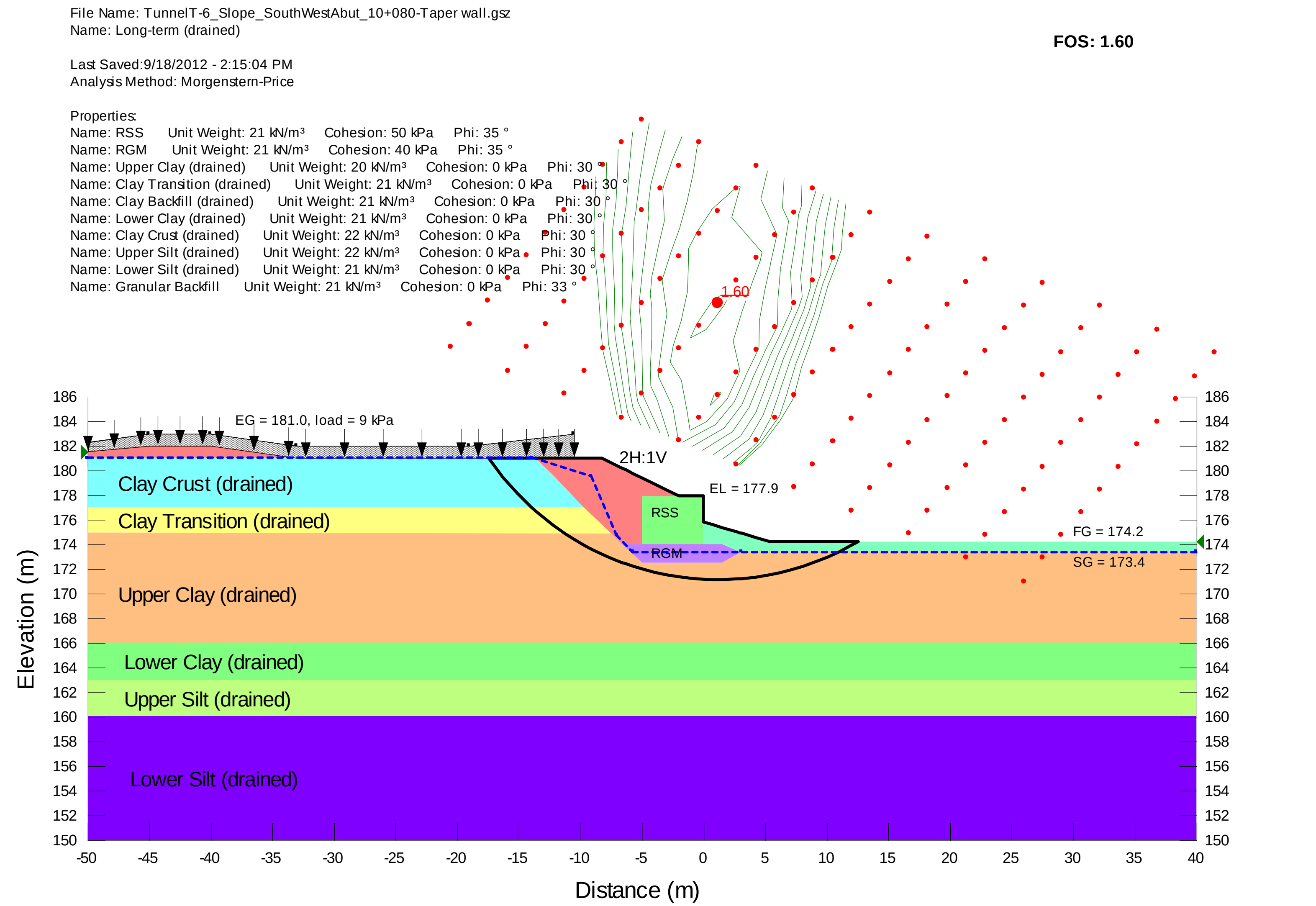
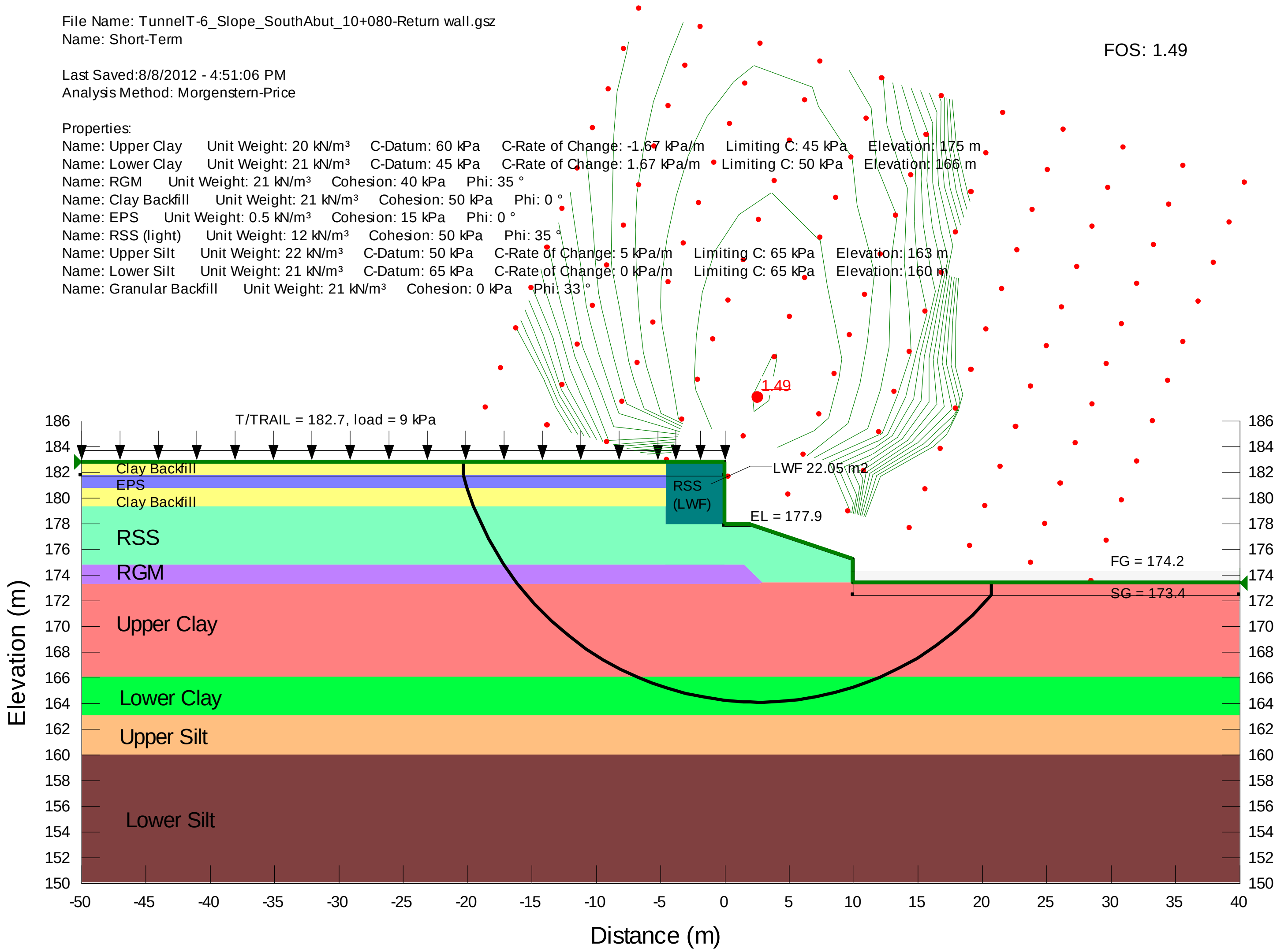
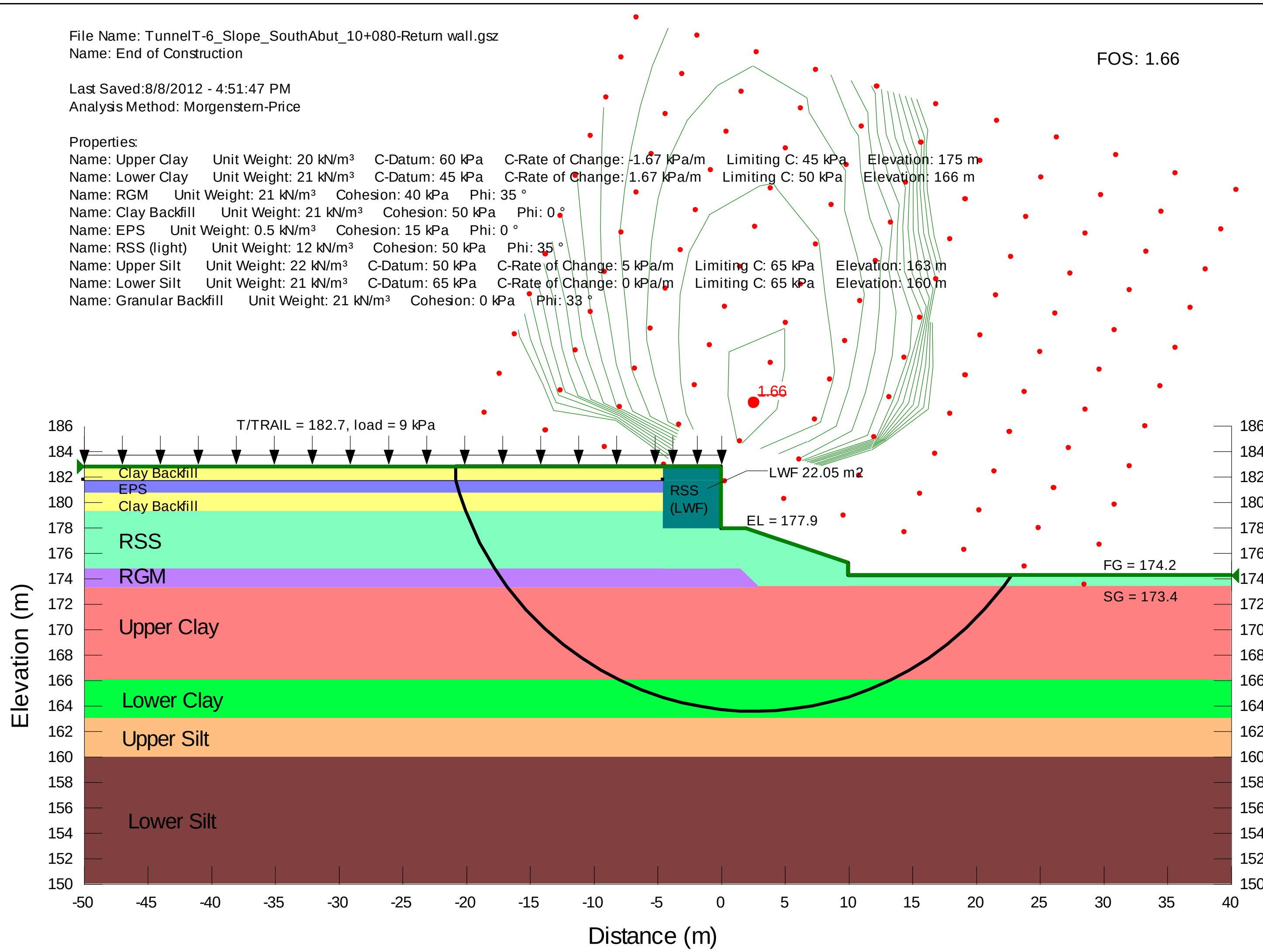


Figure F-25: Global Stability Result – Return Wing Wall South (10+080)– Short Term (Undrained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 25 of 27

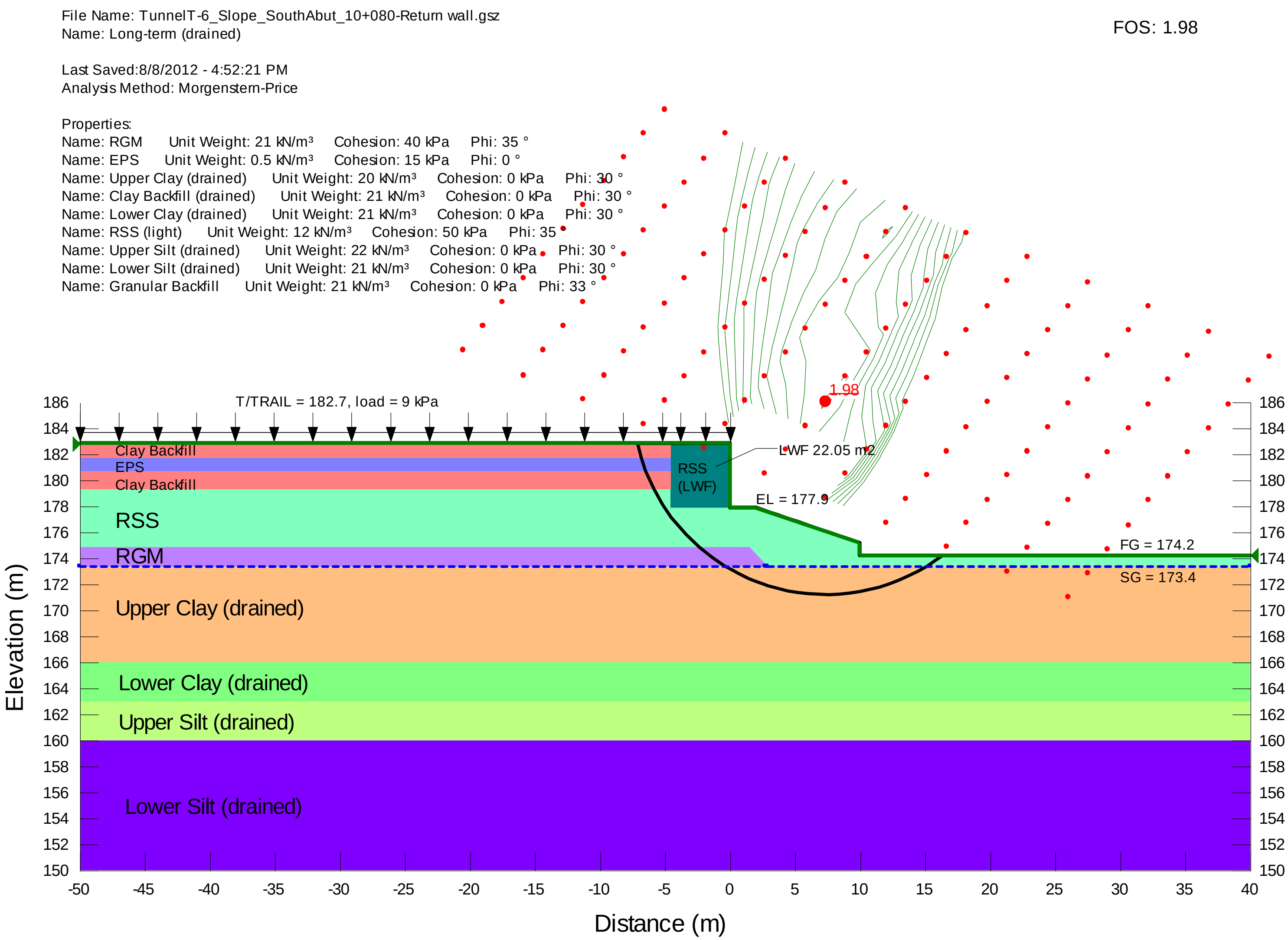
Figure F-26: Global Stability Result – Return Wing Wall South (10+080)– End of Construction (Undrained) Loading



Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

Figure F-27: Global Stability Result – Return Wing Wall South (10+080)– Long Term (Drained) Loading



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix F 27 of 27

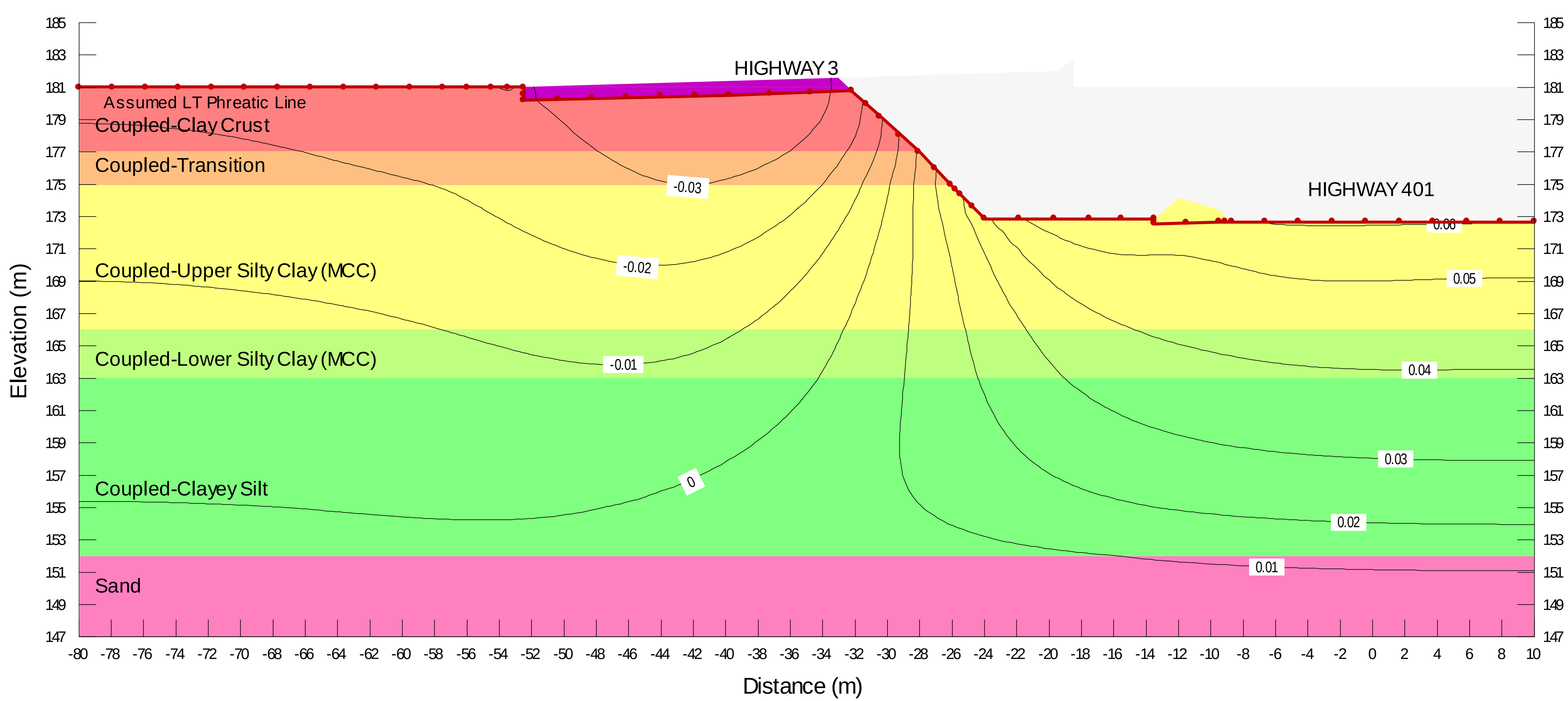
Appendix G Stress-Deformation Analysis Results

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

Coupled-Excavation
End of Excavation
Last Solved Date: 8/15/2012

(+) Denotes Heave
(-) Denotes Settlement

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 Dilation Angle: 0 °
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 Dilation Angle: 0 °
Name: Coupled-Transition Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: Coupled-Upper Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 2.8 Poisson's Ratio: 0.35 Lambda: 0.0882 Kappa: 0.009699 Initial Void Ratio: 0.69 Unit Weight: 20 kN/m³ Phi': 25 °
Name: Coupled-Lower Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 1.3 Poisson's Ratio: 0.35 Lambda: 0.0713 Kappa: 0.007839 Initial Void Ratio: 0.56 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Coupled-Clayey Silt Model: Elastic-Plastic Effective Young's Modulus (E'): 23000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³ Dilation Angle: 0 °



Project:	Windsor-Essex Parkway	**Date:**	September/2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix G 1 of 12

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

Coupled-Excavation
End of Excavation
Last Solved Date: 8/15/2012

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 Dilation Angle: 0 °
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 Dilation Angle: 0 °
Name: Coupled-Transition Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: Coupled-Upper Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 2.8 Poisson's Ratio: 0.35 Lambda: 0.0882 Kappa: 0.009699 Initial Void Ratio: 0.69 Unit Weight: 20 kN/m³ Phi': 25 °
Name: Coupled-Lower Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 1.3 Poisson's Ratio: 0.35 Lambda: 0.0713 Kappa: 0.007839 Initial Void Ratio: 0.56 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Coupled-Clayey Silt Model: Elastic-Plastic Effective Young's Modulus (E'): 23000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³ Dilation Angle: 0 °

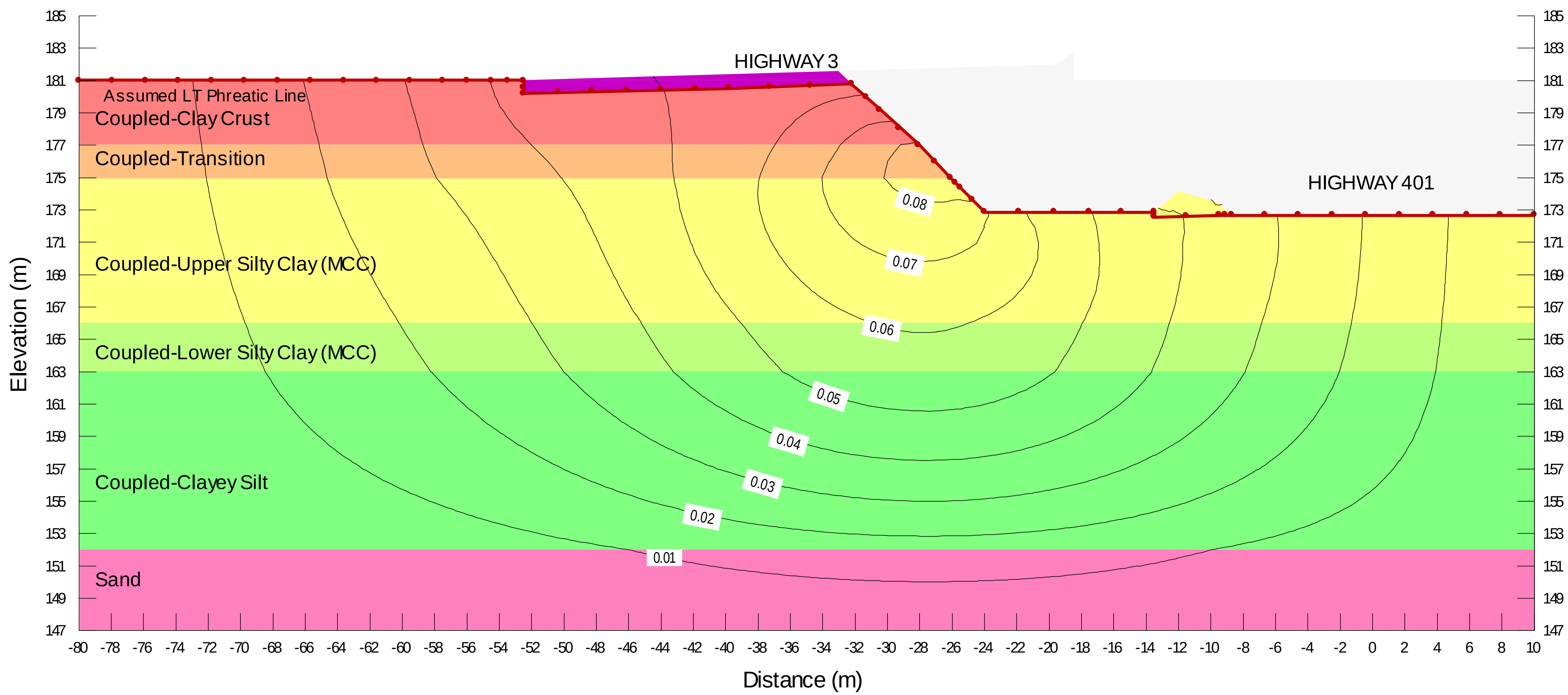


Figure G-3: Cumulative Heave/Settlement – End of Construction

Coupled-Roadway Backfill
End of Construction
Last Solved Date: 8/15/2012

(+) 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 Dilation Angle: 0 °
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 Dilation Angle: 0 °
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 Dilation Angle: 0 °
Name: Coupled-Transition Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: Coupled-Upper Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 2.8 Poisson's Ratio: 0.35 Lambda: 0.0882 Kappa: 0.009699 Initial Void Ratio: 0.69 Unit Weight: 20 kN/m³ Phi': 25 °
Name: Coupled-Lower Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 1.3 Poisson's Ratio: 0.35 Lambda: 0.0713 Kappa: 0.007839 Initial Void Ratio: 0.56 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Coupled-Clayey Silt Model: Elastic-Plastic Effective Young's Modulus (E'): 23000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³ Dilation Angle: 0 °
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 Dilation Angle: 0 °

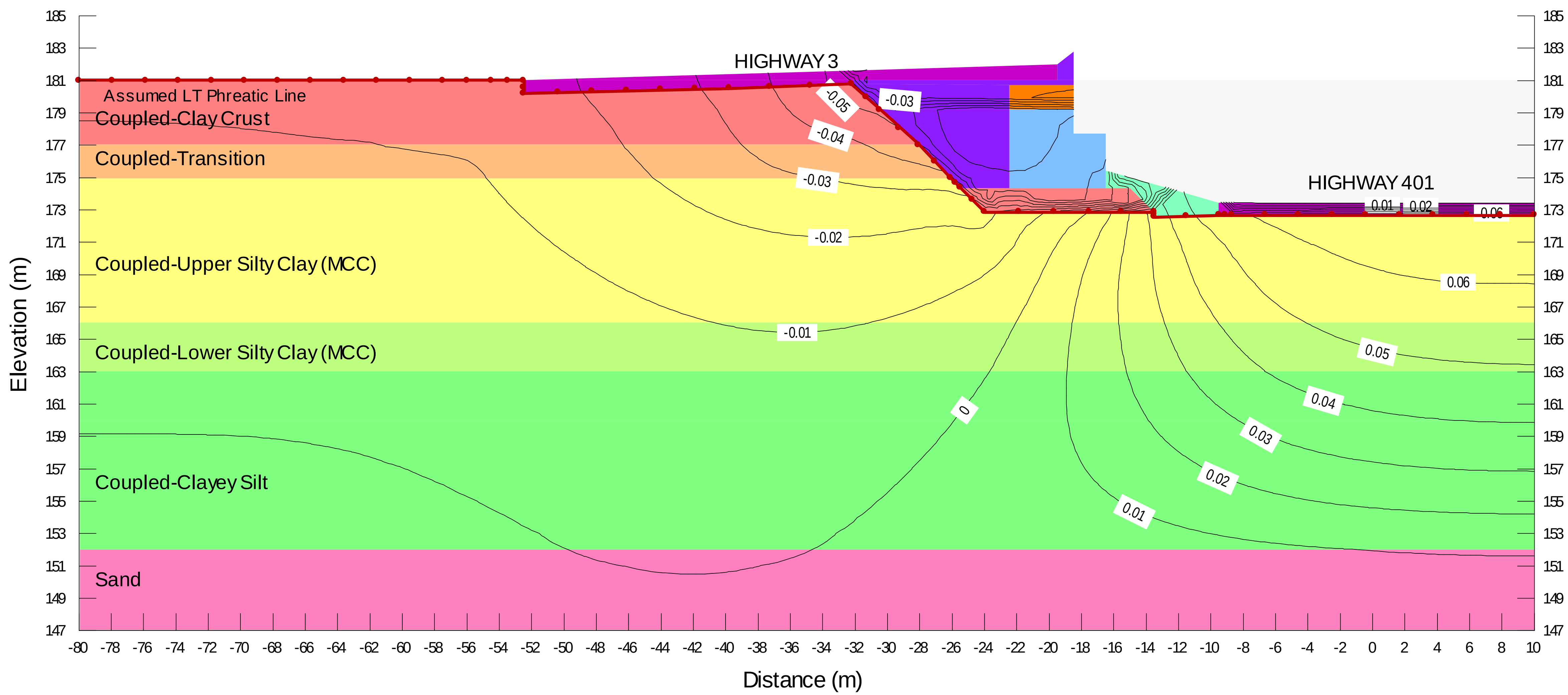


Figure G-4: Cumulative Heave/Settlement - Long-term (Drained)

Coupled-Dissipation
Long Term
Last Solved Date: 8/15/2012

(+) 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 Dilation Angle: 0 °
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 Dilation Angle: 0 °
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 Dilation Angle: 0 °
Name: Coupled-Transition Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ Dilation Angle: 0 °
Name: Coupled-Upper Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 2.8 Poisson's Ratio: 0.35 Lambda: 0.0882 Kappa: 0.009699 Initial Void Ratio: 0.69 Unit Weight: 20 kN/m³ Phi': 25 °
Name: Coupled-Lower Silty Clay (MCC) Model: Soft Clay (MCC) O.C. Ratio: 1.3 Poisson's Ratio: 0.35 Lambda: 0.0713 Kappa: 0.007839 Initial Void Ratio: 0.56 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Coupled-Clayey Silt Model: Elastic-Plastic Effective Young's Modulus (E'): 23000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³ Dilation Angle: 0 °
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 Dilation Angle: 0 °

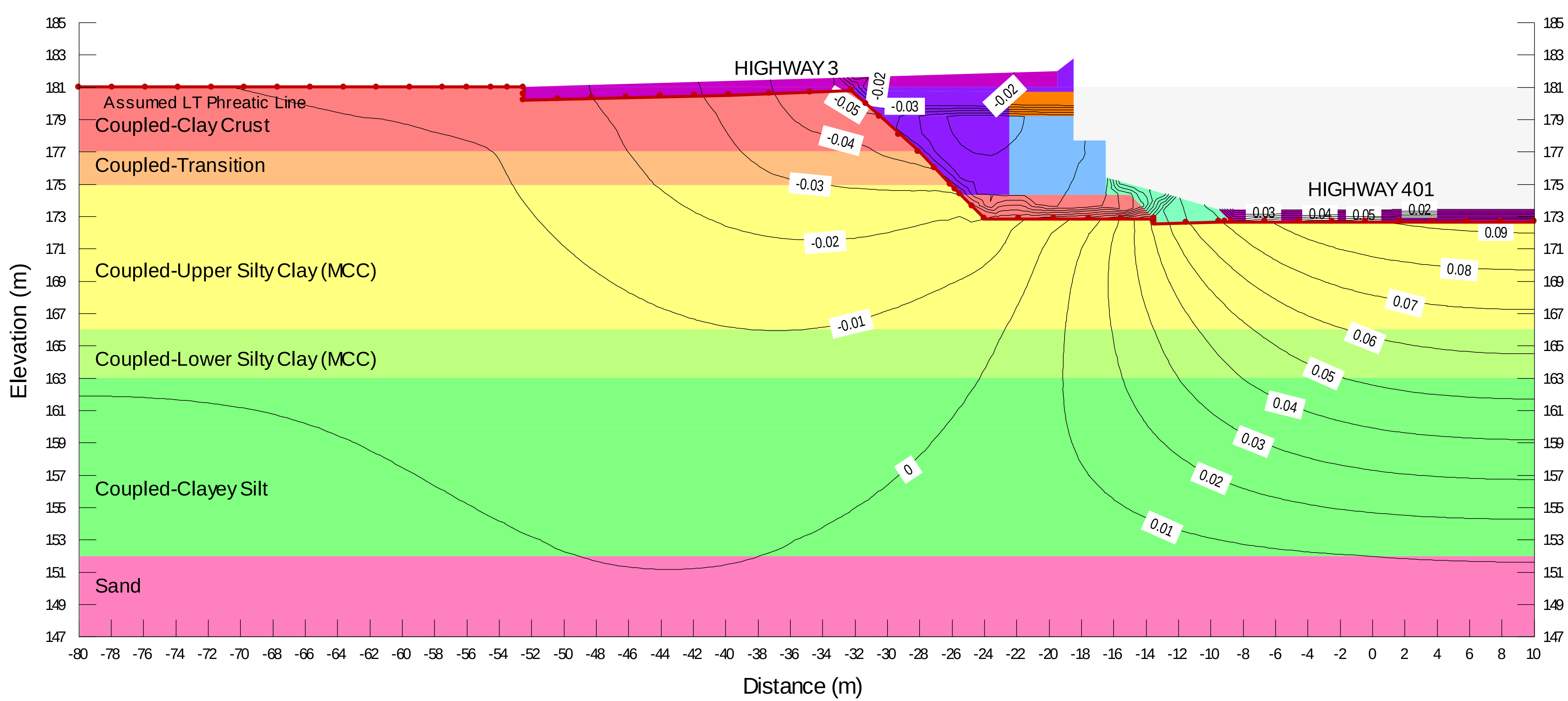


Figure G-5: Stabilized Porewater Pressure Contours – Long-term (Drained)

Coupled-Dissipation
Long Term
Last Solved Date: 8/15/2012

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   Dilation Angle: 0 °
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   Dilation Angle: 0 °
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   Dilation Angle: 0 °
Name: Coupled-Transition   Model: Elastic-Plastic   Effective Young's Modulus (E'): 18000 kPa   Poisson's Ratio: 0.35   Cohesion': 0 kPa   Phi': 30 °   Unit Weight: 21 kN/m³   Dilation Angle: 0 °
Name: Coupled-Upper Silty Clay (MCC)   Model: Soft Clay (MCC)   O.C. Ratio: 2.8   Poisson's Ratio: 0.35   Lambda: 0.0882   Kappa: 0.009699   Initial Void Ratio: 0.69   Unit Weight: 20 kN/m³   Phi': 25 °
Name: Coupled-Lower Silty Clay (MCC)   Model: Soft Clay (MCC)   O.C. Ratio: 1.3   Poisson's Ratio: 0.35   Lambda: 0.0713   Kappa: 0.007839   Initial Void Ratio: 0.56   Unit Weight: 21 kN/m³   Phi': 26 °
Name: Coupled-Clayey Silt   Model: Elastic-Plastic   Effective Young's Modulus (E'): 23000 kPa   Poisson's Ratio: 0.35   Cohesion': 0 kPa   Phi': 30 °   Unit Weight: 21.5 kN/m³   Dilation Angle: 0 °
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   Dilation Angle: 0 °

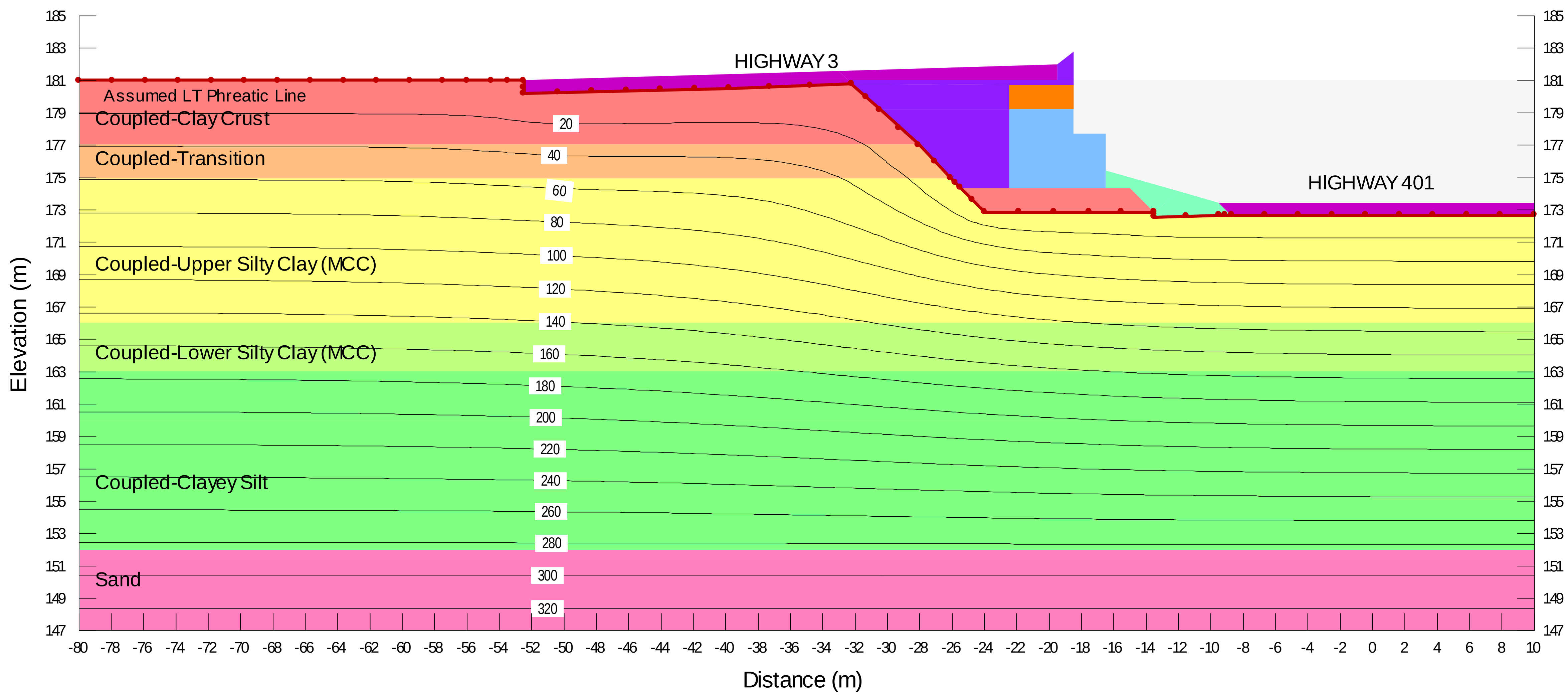
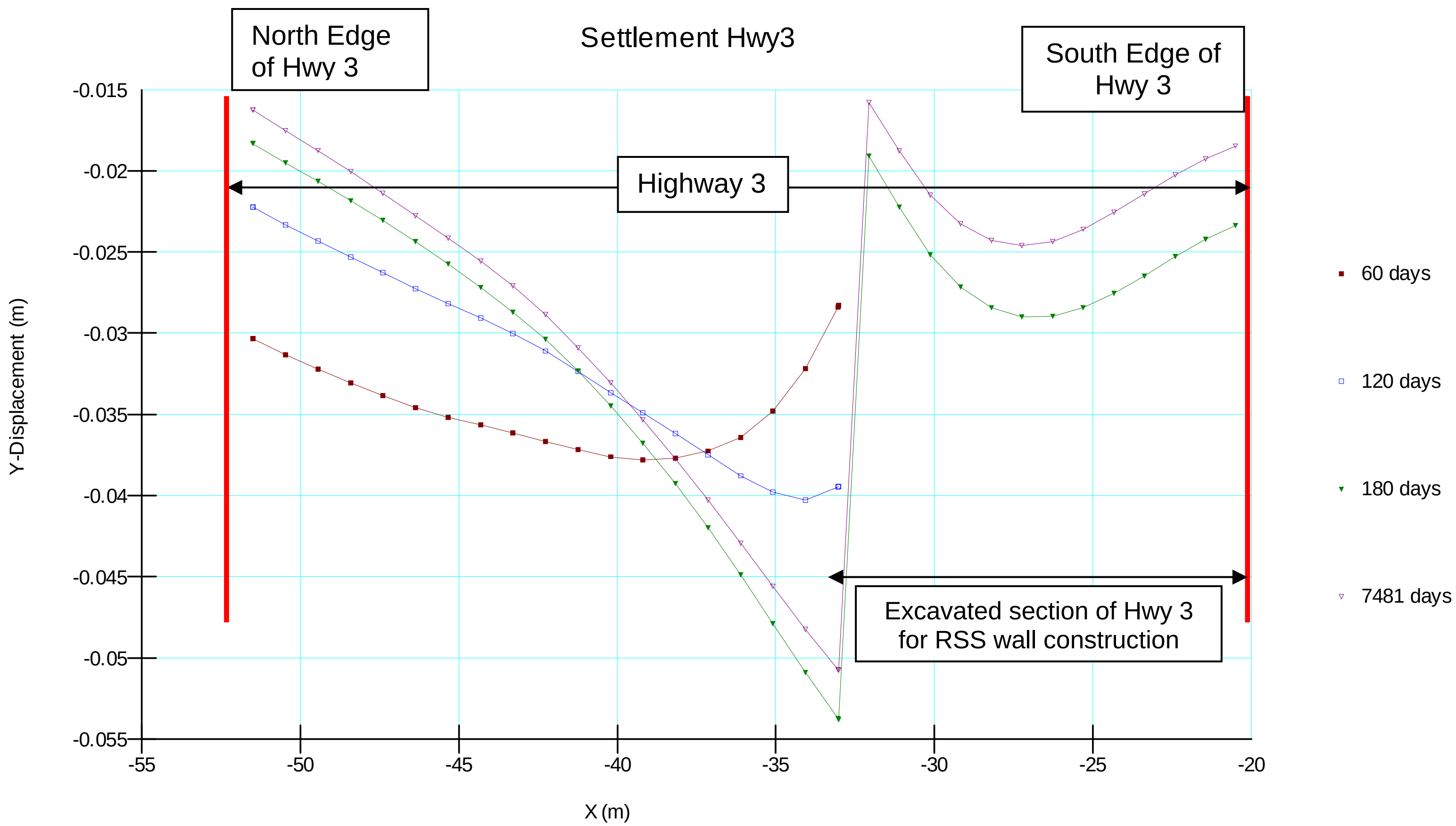


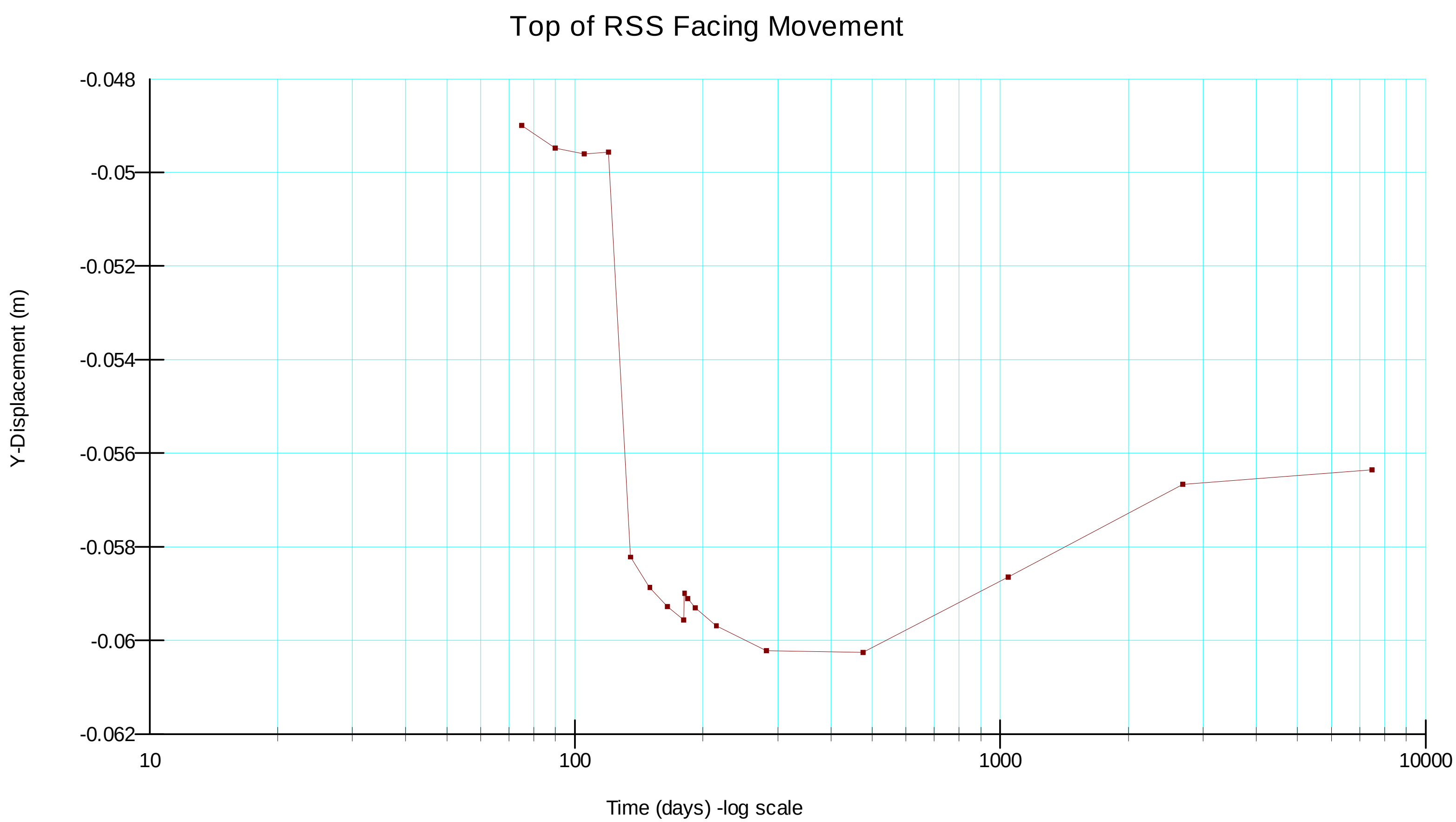
Figure G-6: Cumulative Ground Settlement at Highway 3



60 days = Excavation
120 days = End of RSS Construction
180 days = End of Construction
7481 days = Long Term Condtion

(+) Denotes Heave
(-) Denotes Settlement

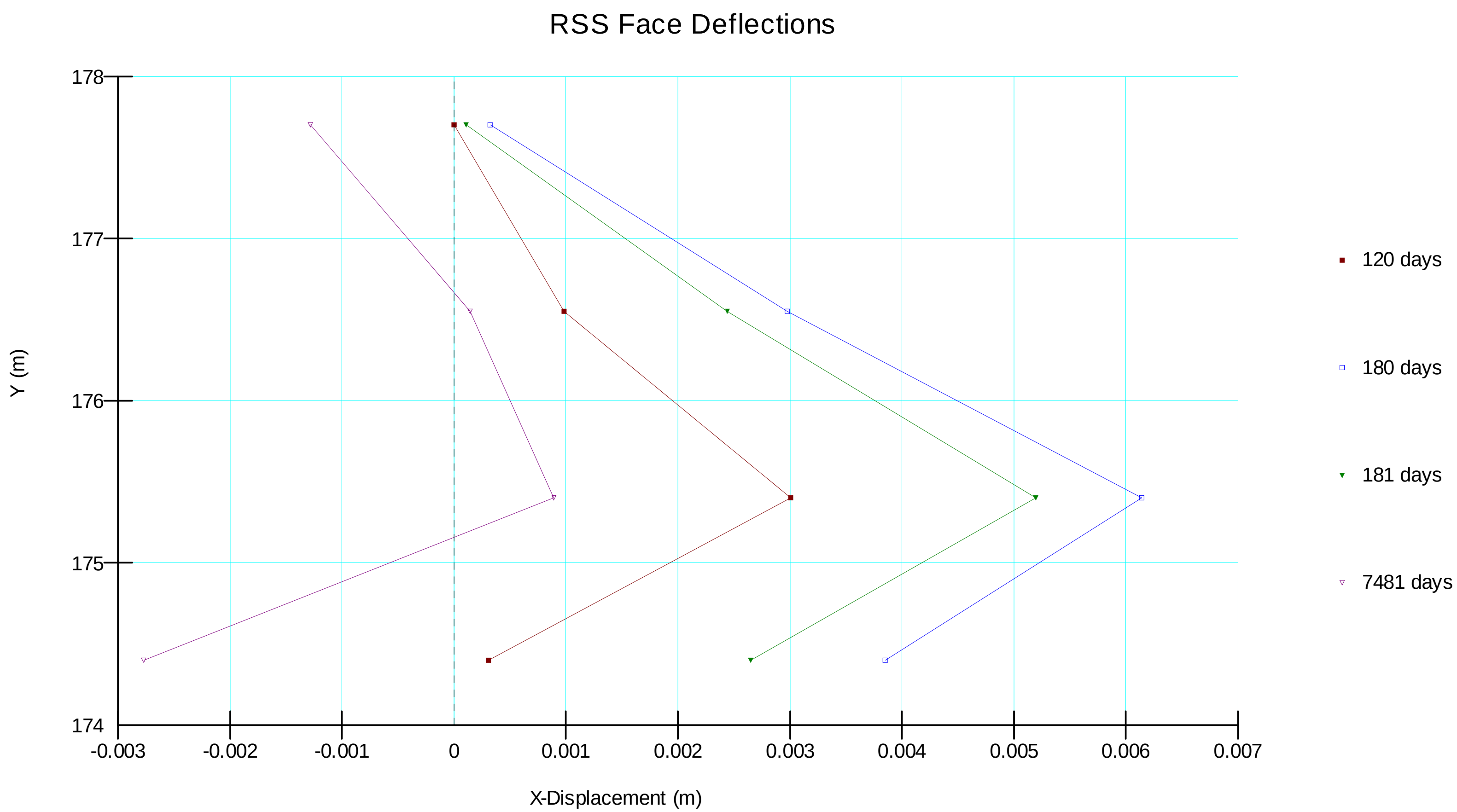
Figure G-7: Cumulative Settlement at Top of RSS Wall Facing



60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
7481 days = Long-term Condition

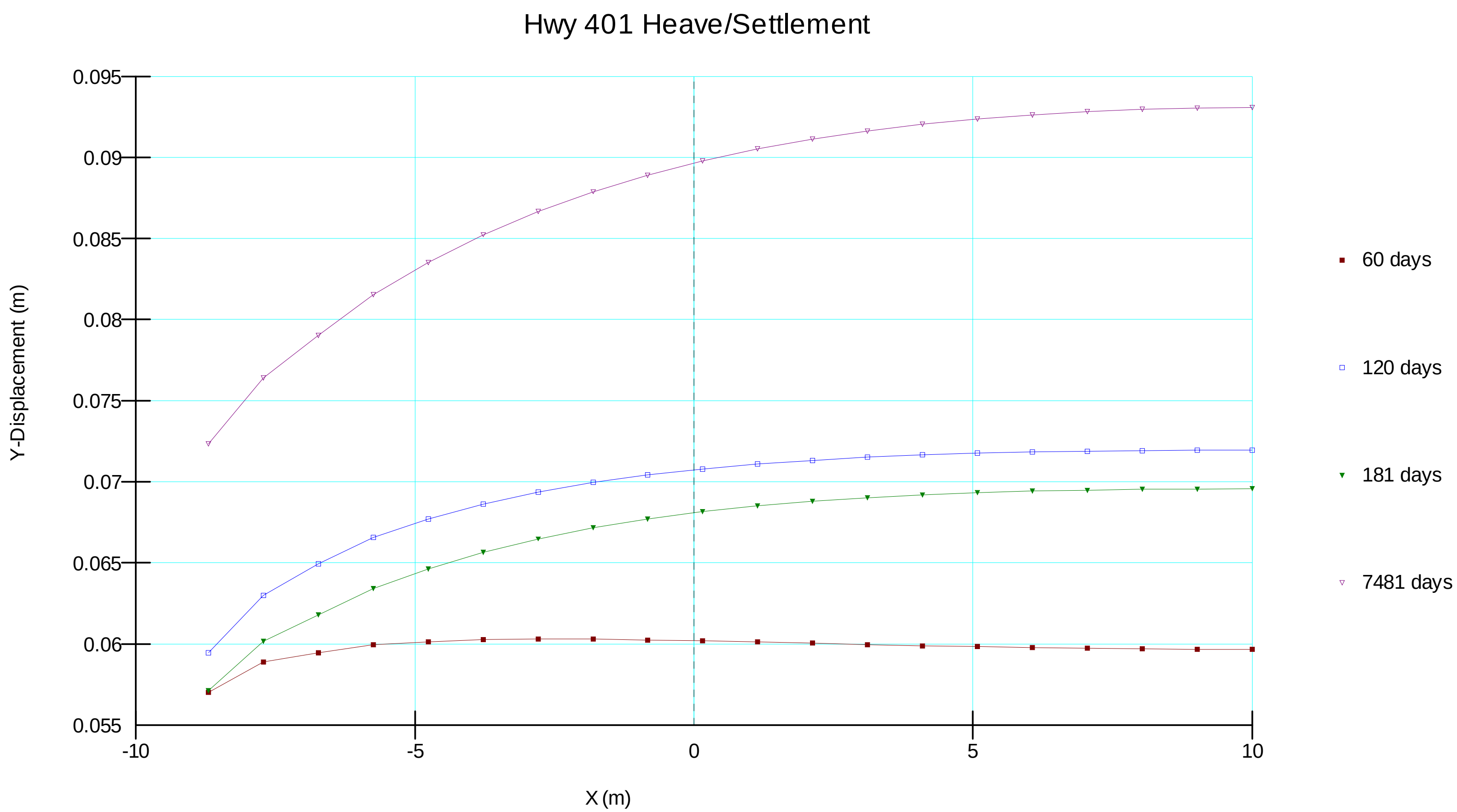
(+) Denotes Heave
(-) Denotes Settlement

Figure G-8: Cumulative Lateral Deflection of RSS Wall



120 days = RSS Completion
180 days = Abutment Completion
181 days = End of Construction
7481 days = Long-term Condition

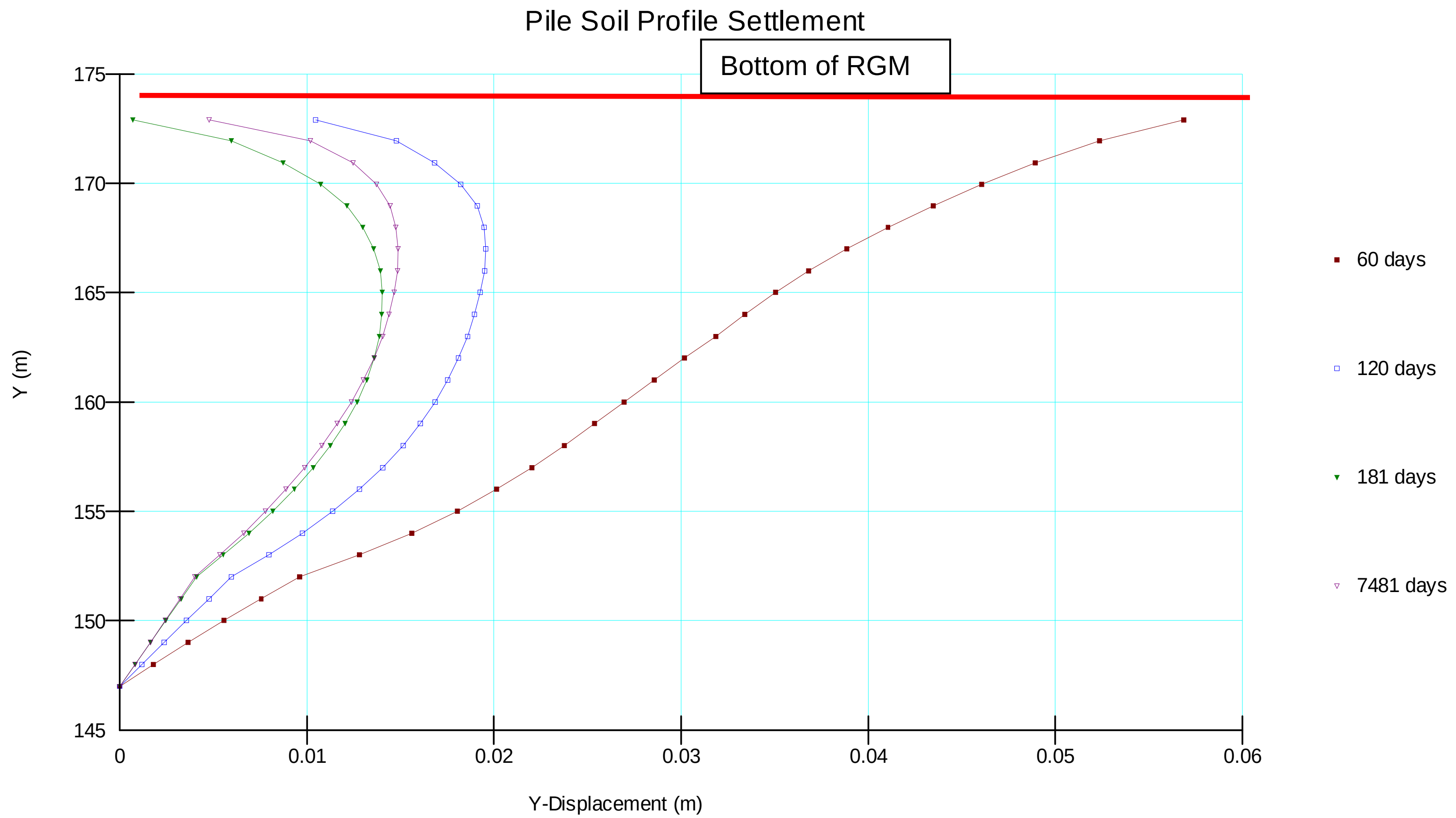
Figure G-9: Cumulative Highway 401 Settlement/Heave (Subgrade Level)



60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
7481 days = Long-term Condition

(+) Denotes Heave
(-) Denotes Settlement

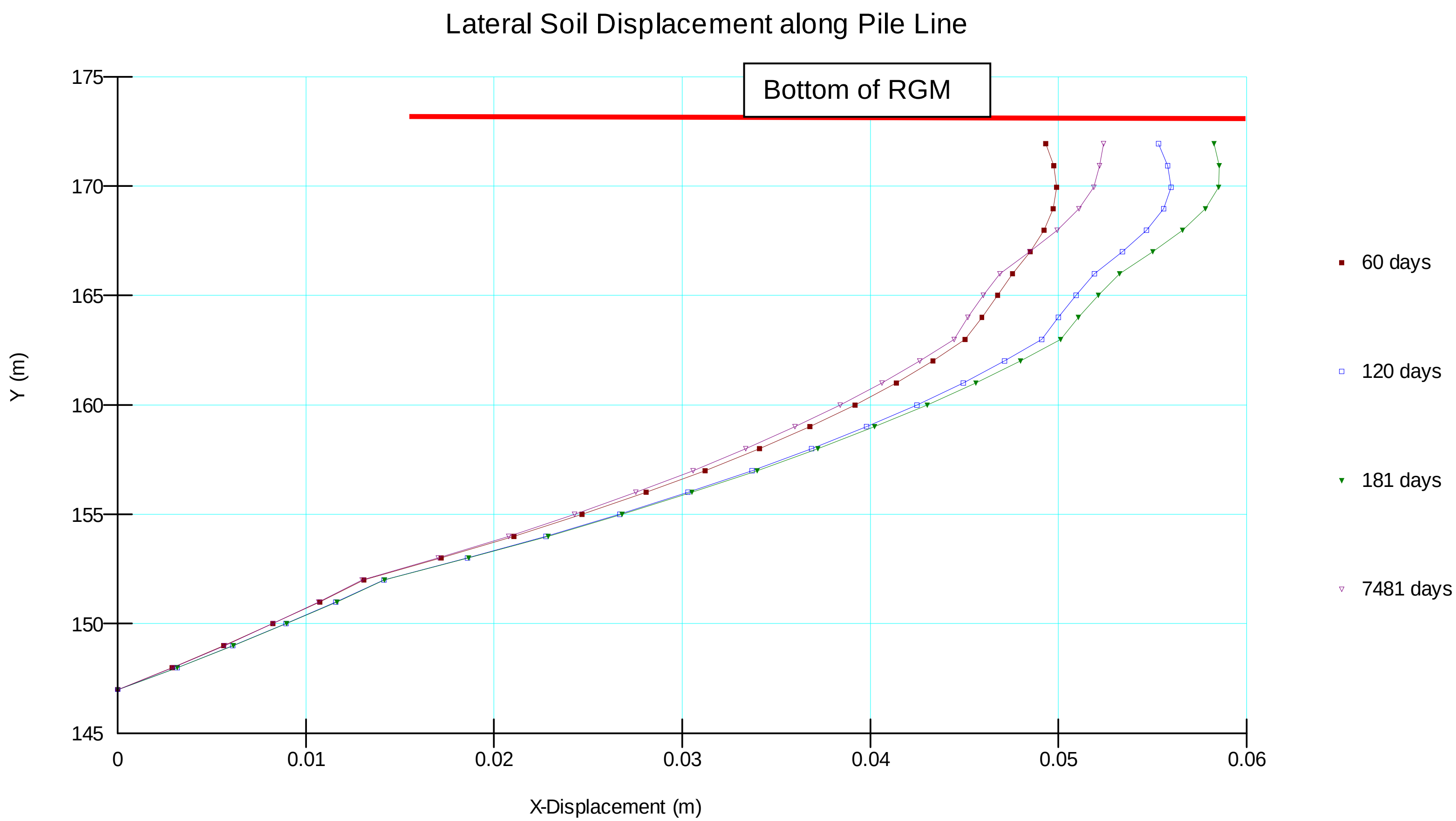
Figure G-10: Cumulative Soil Settlement Profile along Pile Line



60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
7481 days = Long-term Condition

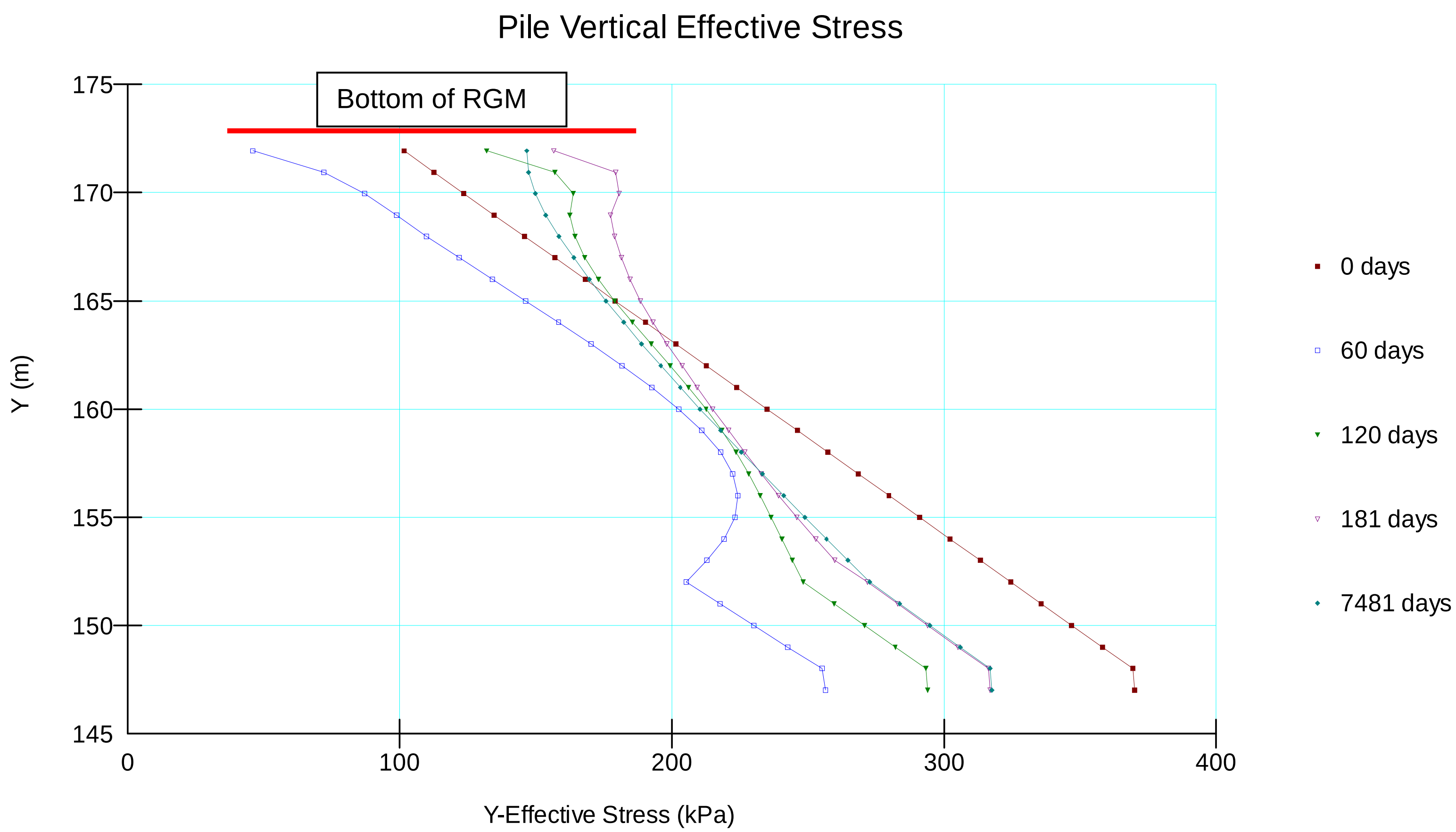
(+) Denotes Heave
(-) Denotes Settlement

Figure G-11: Cumulative Lateral Soil Displacement Profile along Pile Line



60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
7481 days = Long-term Condition

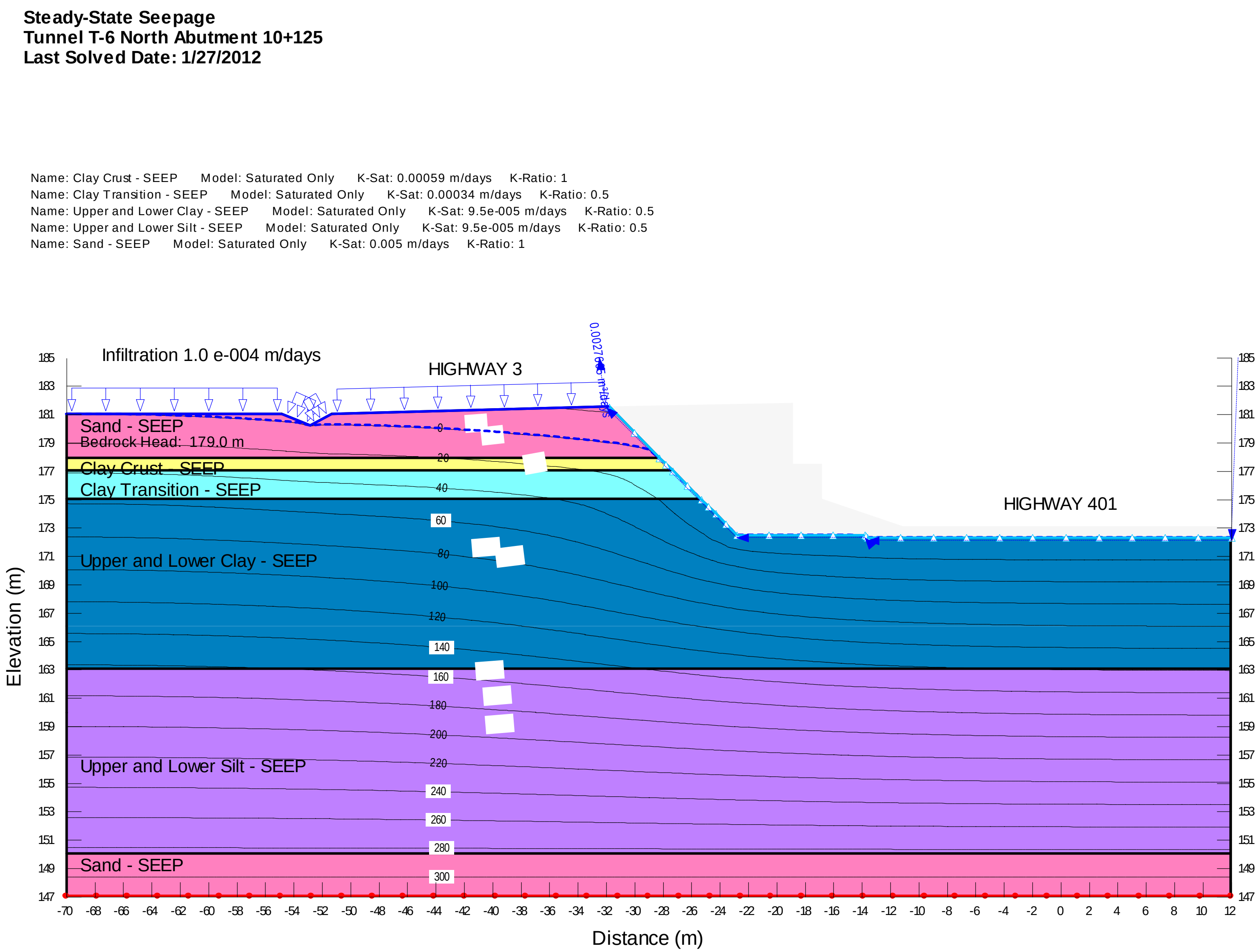
Figure G-12: Vertical Effective Stress Profile along Pile Line



60 days = End of Excavation
120 days = RSS Completion
181 days = End of Construction
7481 days = Long-term Condition

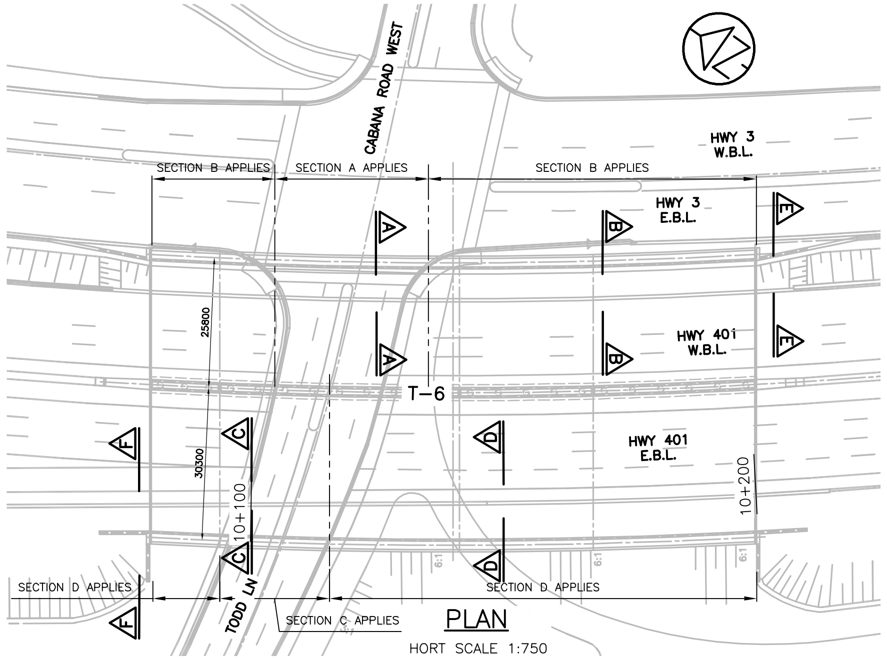
Appendix H Seepage Analysis Results

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



Project:	Windsor-Essex Parkway	**Date:**	September / 2012
Document:	Geotechnical Investigation and Design Report - Tunnel T-6 (Sta. 10+080L to 10+200L)	**Rev:**	0
Doc No.:	285380-04-119-0084 (Geocres No. 40J6-44)	**Page No.:**	Appendix H 1 of 1

Appendix I Conceptual Drawings



PLAN
HORT SCALE 1:750

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 NORTH AND SOUTH ABUTMENTS OF TUNNEL T-6 BASED ON GEOTECHNICAL DESIGN ANALYSES.
3. THE ILLUSTRATED RSS WALL WIDTH AND RGM DIMENSIONS REPRESENT THE MINIMUM DIMENSIONS BASED ON GLOBAL STABILITY REQUIREMENTS. THE INTERNAL DESIGN OF THE RSS WALL AND RGM BY SUPPLIERS.
4. TUNNEL ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGN WERE INTERPRETED FROM LIMITED INFORMATION INDICATED ON STRUCTURAL DRAWINGS AVAILABLE IN JULY 2012. ABUTMENT ELEVATIONS VARY ALONG TUNNEL T-6.
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 DURING CONSTRUCTION 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 SLOPES ARE SUSCEPTIBLE TO DETERIORATION AND MAY EXPERIENCE DEFORMATIONS AND INSTABILITY. THE TEMPORARY SLOPES MUST BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED, MONITORED AND TREATED, AS REQUIRED.
7. RGM DRAIN TO BE AT THE BOTTOM OF RGM.
8. BACKFILL ABOVE THE RSS MAY ONLY BE PLACED AFTER SUBSTANTIAL COMPLETION OF THE BACKFILL AT THE TOE OF THE RSS.
9. SEE ACCOMPANYING DRAWINGS FOR APPLICABLE BACKFILL, LWF, AND EPS SPECIFICATIONS.

LEGEND:

RSS – REINFORCED SOIL STRUCTURE

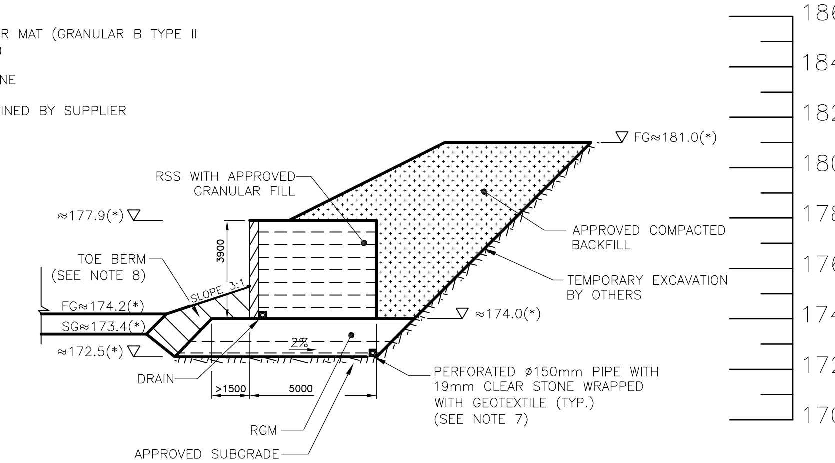
LWF – LIGHT WEIGHT FILL

RGM – REINFORCED GRANULAR MAT (GRANULAR B TYPE II COMPACTED TO 100%)

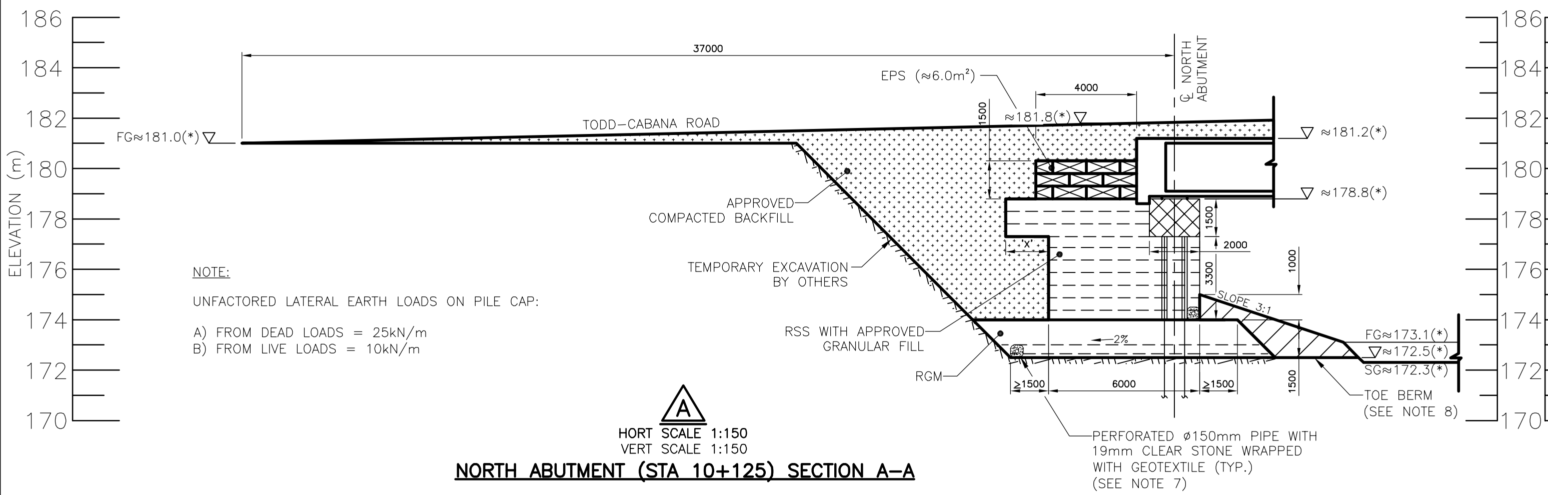
EPS – EXPANDED POLYSTYRENE

'X' – LENGTH TO BE DETERMINED BY SUPPLIER

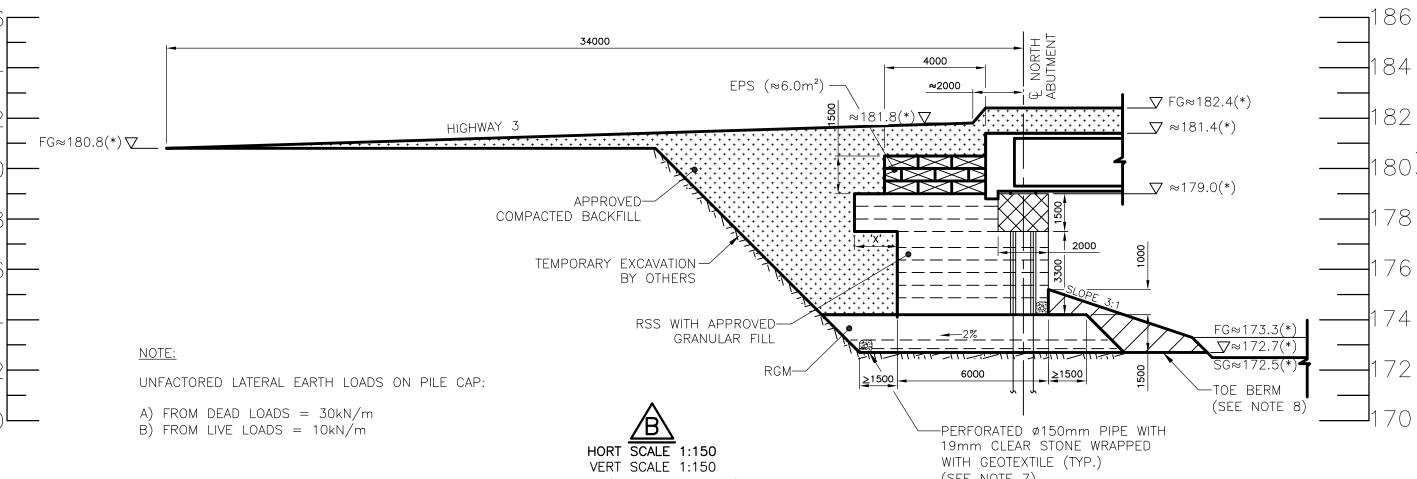
(*) – VARIES



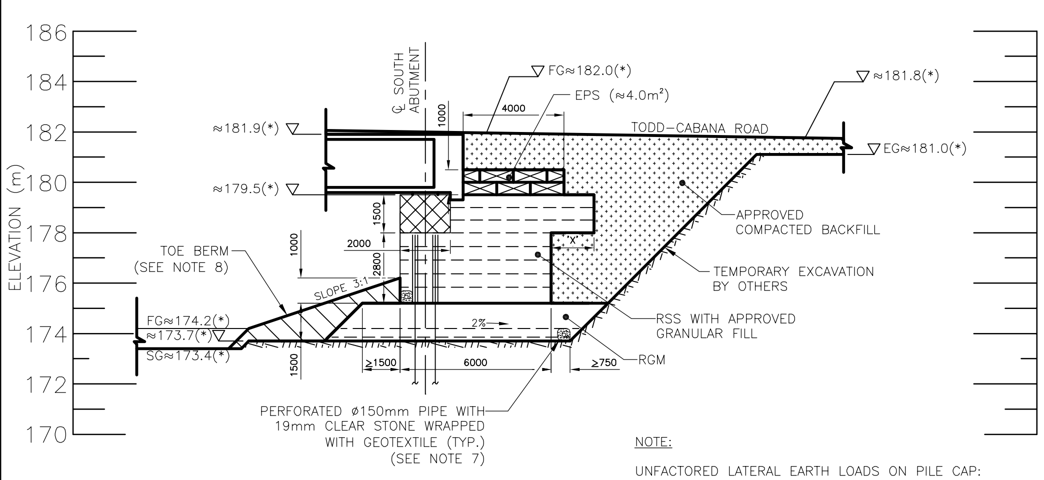
F
HORT SCALE 1:150
VERT SCALE 1:150
SOUTH WEST TAPERED WALL SECTION F–F



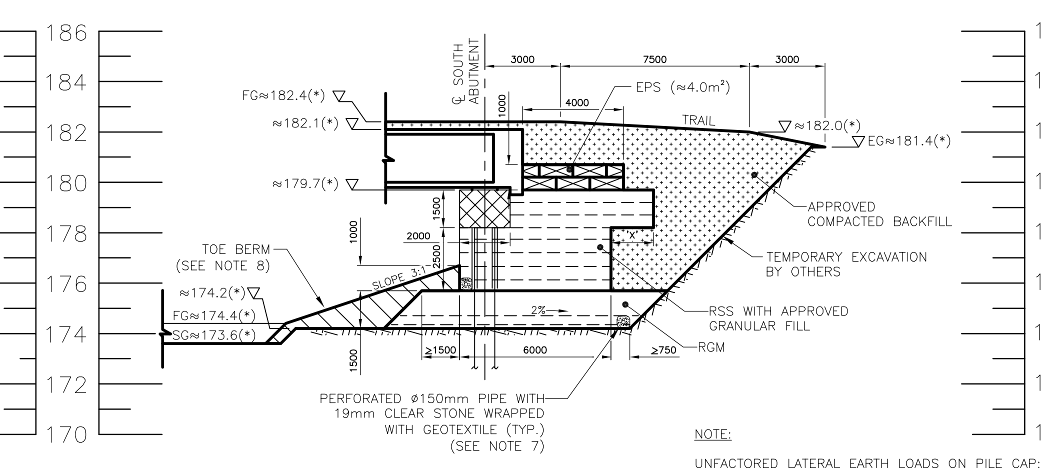
A
HORT SCALE 1:150
VERT SCALE 1:150
NORTH ABUTMENT (STA 10+125) SECTION A–A



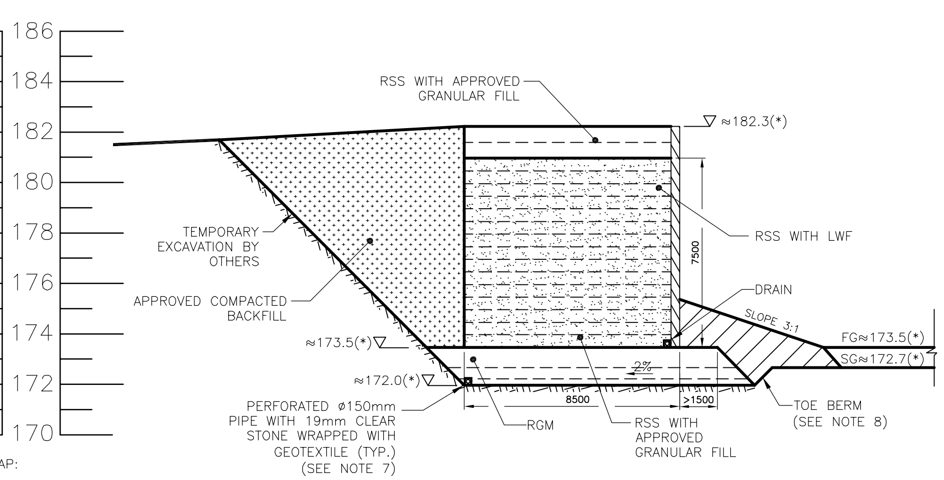
B
HORT SCALE 1:150
VERT SCALE 1:150
NORTH ABUTMENT (STA 10+170) SECTION B–B



C
HORT SCALE 1:150
VERT SCALE 1:150
SOUTH ABUTMENT (STA 10+100) SECTION C–C



D
HORT SCALE 1:150
VERT SCALE 1:150
SOUTH ABUTMENT (STA 10+150) SECTION D–D



E
HORT SCALE 1:150
VERT SCALE 1:150
NORTH EAST EXTENSION WALL SECTION E–E

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

TUNNEL T-6
HWY 401 TODD–CABANA TUNNEL
PLAN AND SECTIONS OF ABUTMENT WALLS

DWG. BY: SJL/KCT	CHK. BY: JF/GN	FIGURE NO.: I.1
DATE: Sept-12	SHEET: 1 OF 1	

DOC: T-6 PLAN W ABMT SECTIONS_FIG I.1