The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report – Tunnel T-10B (40+840 to 41+000 – SR4)

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PARKWAY INFRASTRUCTURE CONSTRUCTORS
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PROJECT DOCUMENT AND DATA MANAGEMENT

List of Contents and Appendices

Page

List of Tables

List of Drawings

List of Figures

List of Appendices

1 Introduction

1.1 Preface

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

1.2 Report Introduction

This report presents the geotechnical design of Tunnel T-10B, (Highway 3 Hearthwood Tunnel) located in LaSalle sector along Highway 3 (Service Road 4) between Sta. 40+840 to Sta. 41+000 of the proposed Windsor-Essex Parkway (WEP) project. The report includes the results of the additional geotechnical investigation carried out to support the design (i.e., the layout and configuration) available at the time of preparation of this report and addresses review comments from peer reviews and MTO. This is the final report and is issued for construction (IFC).

The 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.

The proposed 160 m long, single-span Tunnel T-10B structure will carry parkland landscape and trails over Highway 3 between Sta. 40+840 and Sta.41+00. Tunnel T-10A is located north of Tunnel T-10B, and carries parkland landscaping and trails over Highway 401. The proposed structural solution incorporates structural deck on concrete girders supported on piles, Reinforced Soil Structure (RSS) wall abutments on piles. RSS wing-walls and return walls will be utilized. The design presented in this report was generally advanced from the preliminary geotechnical design developed for the WEMG proposal in June 2010 (ref. R-43)[1]. The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines as well as the Parkway Infrastructure Constructors (PIC).

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

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

[1] References are listed in Section 9.

2 Background Information

2.1 Geological Setting

The WEP project site is located within the Essex Clay Plain (a part of the St. Clair Clay Plain physiographic region) (ref. R-16, R-18 and R-25). 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. P.P. Hudec (ref. R-25) summarized the overburden geology in Windsor as consisting of the following successive strata: desiccated lacustrine clay, normally consolidated lacustrine clay, silty Tavistock till, glaciolacustrine clay and coarse Catfish Creek till. A distinct change in overburden deposits occurs in the east-west direction along a boundary located generally along the Huron-Church Road. Whereas, the eastern part of Windsor is underlain by firm to stiff glaciolacustrine silts and clays with upper deposits of stiff sandy to silty weathered clay and hard to stiff lacustrine clay-silt crust, the western part of Windsor is characterized by a thin surficial granular deposit underlain by thin crust layer 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). The bedrock geology within the Essex Domain was formed as part of the midcontinent rift south-eastern extension. The midcontinent rift south-eastern extension is composed of Paleozoic cover rocks which form the bedrock foundation of the Essex Domain. The bedrock was deposited in the Paleozoic Era during the Middle Devonian period. Within the Essex Domain the following strata were deposited the Hamilton Group, Dundee Formation, and Detroit River Group Onondaga Formation all consisting of Limestone, Dolostone, and Shale.

2.2 Site Seismic Background

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

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

2.3 Existing Site Conditions and Proposed Tunnel Layout

Tunnel T-10B site is situated near the middle of the LaSalle segment (Highway 3 – SR4) of the Parkway. The tunnel structure will be constructed under WEP Phase I development and will be used to carry trail traffic and parkland over Highway 3 and Highway 401. Highway 3 in the vicinity of Tunnel T-10B will be relocated on the south side of the proposed depressed Highway 401 and south of the proposed tunnel T-10A. Highway 401 and Highway 3 at this location will be constructed within permanent cut. Longitudinal wing walls and return walls flared at 90^0 to the tunnel abutment are indicated at each end of the portals. Based on the highway design profile, the proposed subgrade level below the pavement on Highway 3 varies approximately between elevation 178.5 at the west end and 177.5 at the east end of the tunnel.

The topography of the lands immediately adjacent Tunnel T-10B at Highway 3 is generally flat with elevations ranging from approximately 184.4[2] in the area of northwest abutment to 185.2 at the southwest abutment. Adjacent land use is typically residential.

Tunnel T-10A and Tunnel T-10B are two separate structures that are approximately 15 to 25 m apart (285380-03-060-WIP1-3051). It is understood that Tunnel T-10B will be constructed first. The later construction of T-10A is anticipated to interact with the north abutment at T-10B which will require adequate protection. In addition it may be necessary to consider delaying the completion of the backfill above the north (left) RSS abutment Tunnel T-10B until the construction of the south (right) abutment for Tunnel T-10A.

It is understood that electrical pads (PDA), ducts and maintenance holes for power supply to the tunnels are planned between Tunnel T-10A and Tunnel T-10B. The reported weights of these utilities are negligible in terms of loading on geotechnical structures. However, possible interferences of the buried components of the power supply with the RSS components and the anticipated EPS fill will have to be examined with the suppliers of these proprietary products.

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

2.4 Frost Depth

In accordance with MTO–SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-37) and OPSD 3090.101, the frost depth below the ground surface in Windsor area is estimated to 1.0 m[3]. This estimate is considered applicable for natural soils and/or conventional pavement materials where the ground surface is free from the snow cover.

The insulation effects of riprap and other coarse rockfill cover 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.

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

3 Geotechnical Investigations

3.1 Scope and Procedures of Geotechnical Investigations

Geotechnical investigations involving a number of boreholes, cone penetration tests (CPT) and Nilcon vane tests had been carried out in 2007-09 by Golder Associates (ref. R-16 to R-23) as background information for development of the WEP proposal designs. Additional geotechnical investigation was carried out 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-10B comprised a total of 3 boreholes, 2 CPT and 1 Flat Blade Dilatometer (DMT). Table 3-1 lists the test holes put down at or in close proximity to the tunnel site during both the previous (2007-09) and the current (2011) geotechnical investigations.

Table 3-1: Test Holes at and around Tunnel T-10B Site

Reference	Boreholes	Nilcon Vane Tests	CPT	DMT
This Investigation (2011)	BH - T10-1		BH/CPT - T10-1	BH/DMT T10-1
	BH - T10-2		BH/CPT - T10-2	
	BH TB7A-1		BH/CPT47-RW	
Previous Studies (2007 – 2009)	BH-112/112A	BH-112	CPT-5	
	BH-113/113A		CPT-111	
			BH/CPT-114	
			BH/CPT-312	

Drawing 285380-04-090-WIP1-3053 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-10B (i.e., from Highway 401 Sta. 12+100L to Sta. 12+800L parallel to Highway 3, Service Road 4 between Sta. 40+840 to Sta. 41+000). The test hole locations and stratigraphic sections at the tunnel location are illustrated on Drawing 285380-04-090-WIP1-3054.

3.2 Additional Investigation Fieldwork

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-size 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. At select depths, samples were also taken using 70 mm diameter and 600 mm long thin-walled Shelby tubes. Soil sampling was carried out generally at 0.75 m depth interval in the top 7 to 8 m and at 1.5 m depth intervals thereafter. All samples were identified and placed in airtight containers and were taken to

AMEC's Tecumseh (Windsor) laboratories for further examination and testing[4]. Rock coring of the bedrock was carried out using NQ or HQ sized core barrels with a length of 1.5 m.

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. Table 3-2 summarizes the depths of overburden penetration and rock coring as well as the list of instruments and the accompanying Nilcon vane test.

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 (m)			
			Rock Coring	Nilcon Vane	S-Piez.	VWP
BH-T10-1 (2011)	N4678496, E334122	32.5	152.4 to 150.1		181.8	175.8, 163.3
BH-T10-2 (2011)	N4678357, E334192	32.3	152.5 to 149.0		181.7	178.3, 166.1, 153.8
BH TB7A-1 (2011)	N4678507, E334190	>10.1 (BTEO)	No			
BH – 112 (Pre-Bid)	N4678413, E334221	32.5	152.1 to 146.39	177.8 To 162.8	146.4	
BH-112A (Pre-Bid)	N4678413, E334221	>9.1 (BTEO)	No		175.4	
BH–113 (Pre-Bid)	N4678454, E334070	31.4	153 to 148.4		153	
BH-113B (Pre-Bid)	N4678454, E334070	>9.6 (BTEO)	No		174.8	
BH-312 (Pre-Bid)	N4678320, E334283	>4.5 (BTEO)	No			

Legend: S-Piez. Standpipe piezometer
VWP Vibrating wire piezometer
MSG Spider magnet heave/settlement gauge
INCL Inclinometer casing
BTEO Borehole Terminated Early in Overburden
N/A Not Available

Rock cores were examined and photographed in the field. For each core run, rock core recovery and rock quality designation (RQD) were determined. The recovery and RQD values are given on the borehole logs.

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

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

[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). Pore pressure dissipation tests were carried out at CPT at BH T10-2 and BH 47-RW at 21.5 and 16.4 m below ground surface, respectively.

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. All CPT and DMT were advanced to refusal.

The locations of boreholes, Nilcon tests, and CPTs 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-WIP1-3053. Borehole and CPT logs from the additional investigation are included in Appendix A. Relevant borehole logs from the previous investigations are included in Appendix B.

3.3 Instrumentation

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

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

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

3.4 Geotechnical and Analytical Laboratory Testing

All recovered soil samples and rock cores were examined in the field and the laboratory. Natural moisture content tests were carried out on most of the recovered samples; grain size distribution and Atterberg limit tests were carried out on selected representative samples. Following these soil classification tests, one representative soil sample was selected for advanced tests, namely one direct shear test and two one-dimensional consolidation tests.

Selected samples of the silty clay and silt samples obtained from Boreholes T10-1 and T10-2 were sent to the ALS Environmental Analytical Laboratory in London, Ontario to determine the pH, redox potential, resistivity, sulphide and sulphate content of the soil to assess corrosion potential.

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

3.5 Data Interpretation

Field Vane Test Data Correction: The chart (Figure 3.1[6]) developed initially by Bjerrum (1972) and updated subsequently by Ladd et al (1977) based on circular 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-6 and R-32), the typical value for the silty clay to clayey silt deposit at the WEP. However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), the Canadian Foundations Manual suggests that the vane test data for clays with PI<20 should not be corrected (ref. R-1 and R-8, Figure 3-2). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI. Interestingly, the undrained shear strength (S_u) profiles inferred from the DMT and the S_u values obtained from the conventional field vane tests in boreholes were consistently higher than the Nilcon vane test values.

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

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

Where:

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

Q_t is the corrected total cone tip resistance;

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

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

The CPT based S_u profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles. In this regard, the N_{kt} factor values used to calibrate the CPT strength profiles varied slight for different segments of the WEP and the soil strata. Thus, N_{kt} factor of 14 was used to estimate the undrained shear strength of the clay crust and transition layers. The N_{kt} factors used for the underlying grey silty clay to clayey silt stratum and the lower clayey silt stratum were 15 to 16[7], and 12 to 13[8],

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6 All figures are included at the end of the report text.
7 N_{kt} values for upper silty clay 15 (for 9+700W to 13+500W), 16 (for 13+500W to 13+400L) and 15 (for 13+400L to 10+700T)
8 N_{kt} values for lower clayey silt 13 (for 9+700W to 13+500W), 12 (for 13+500W to 13+400L) and 13 (for 13+400L to 10+700T)

respectively. Figure 3.3 presents the undrained shear strength profiles for WEP segment between Sta. 12+400L and Sta. 12+750L along Highway 401 (which is parallel to the SR-4, Hwy 3 T-10B segment from Sta. 40+840 to Sta. 41+000), and shows that the estimated undrained shear strength profile using the CPT data and measured shear strength profile from Nilcon vane tests show good agreement.

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

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

Where:

S_u is the undrained shear strength,

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

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

S is the normalized strength ratio, Su/σ'_v, of normally consolidated soil,

OCR is the over-consolidation ratio, and

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

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

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

Flat Blade Dilatometer (DMT) Test Data: DMT tests were conducted following the ASTM D6635-01 (2007) method. The soil properties from the results of these tests were developed in general using the guidelines layout in ISSMGE, 2001 (ref. R-26), 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. K_d is the horizontal stress index obtained from DMT reading and is defined 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 (C_c) profiles based on field and laboratory testing from boreholes, CPT and DMT carried out in the vicinity of Tunnel T-10B are presented in Figure 3-3. Also included on the figure are the $0.18 \times \sigma_{vo}'$ curve (representing undrained strength profile for OCR=1 condition) and the 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-11).

4 Subsurface Conditions

The general soil stratigraphy at the borehole locations (ground surface at about elevation 184.5 to 185) 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 184.5 to 183.5, and a lower granular deposit below about elevation 154, overlying limestone and dolostone bedrock below about elevation 152. The thickness of the clayey silt to silty clay deposit varies between about 24.2 m and 30.4 m. The lower granular deposit (sandy silt / silty sand / sand and gravel) varied in thickness between 1.5 to 6.1 m. The bedrock was encountered at depths ranging from about 31.4 m to 32.5 m below the existing ground surface.

4.1 Surficial Fills, Topsoil and Upper Granular Deposit

All boreholes, except for Borehole BH CPT10-2 encountered an up to 0.6 m thick layer of brown to black topsoil. Borehole CPT T10-2 encountered a fill layer consisting of gravel with clay which extended to 0.4 m below existing grade.

Underlying the topsoil in Boreholes BH T10-1 and CPT-312 was a 0.53 to 1.2 m thick fill consisting of primarily clayey silt or silty clay soils. Non-cohesive fine silty sand was encountered in Borehole CPT T10-2 below the topsoil. The thickness of this unit was 0.3 m.

The thickness of the topsoil and fill is expected to vary in quality and thickness through the project area.

4.2 Silty Clay to Clayey Silt Stratum

The cohesive silty clay stratum was encountered directly underlying the surficial topsoil or fill/granular deposit. The encountered depth below existing ground surface was from 0.2 to 0.8 m. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into 4 layers as follows: brown desiccated stiff to very stiff clay crust, transition zone, upper grey silty clay to clayey silt deposit (referred to hereafter as upper silty clay), and then a lower grey clayey silt deposit (referred to as lower clayey silt). The natural water content, Atterberg limits and bulk unit weights determined on the samples recovered during the additional geotechnical investigation of the clay sub-strata are summarized in Table 4-1.

Table 4-1: Summary of Index Properties and Undrained Shear Strength from Nilcon Vane Test

Property[1]	Clay Crust	Transition	Grey Silty Clay	Clayey Silt
Elevation Range (m)	184.4 to 178.3	177.2 to 175.3	174.2 to 163.1	162.0 to 154.1
Natural Water Content, w_N, %	12 to 21	13 to 16	12 to 44	10 to 27
Liquid Limit, w_L, %	19 to 26	20 to 24	28 to 41	22 to 36
Plastic Limit, w_P, %	12 to 15	12 to 13	15 to 20	14 to 17
Plasticity Index, PI	7 to 11	7 to 12	13 to 21	8 to 21
Liquidity Index, LI	<0.14	0.06 to 0.31	0.31 to 1.89	-0.43 to 0.44
Unit Weight, γ, kN/m^3	19.9 to 22.3	21.7	18.5 to 21.2	20.4 to 23.2
Undrained Shear Strength, Su, kPa	81	79 to 45	60 to 43	N/A

1 – Index Properties are bases on laboratory results from BH112, BH113 and BHT10-2.
2 - Varies

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

- Crust layer: > 100±20 kPa
- Transition layer: 100±20 kPa to 65±10 kPa
- Upper silty clay: 65±10 kPa to 55±10 kPa
- Lower clayey silt: >75 kPa (Change values appropriately)

The stress-strain properties and the effective shear strength properties of the silty clay to clayey silt soils were based on published correlations (Kulhawy and Mayne, 1990, ref. R-29, Leroueil et al, ref. R-33 and Terzaghi et al. Ref. R-42) and confirmed by tests reported in Golder's Subsurface Condition Interpretation Report (ref. R-19) and the tests performed during the additional geotechnical investigation carried out as part of the detailed design development for the entire WEP length.

The stress-strain relationships are correlated to natural water content (w_N, expressed as percent) as illustrated in Figure 4.1 and 4.2 and 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-10B site are summarized as follows:

Table 4-2: Summary of Interpreted Compressibility Properties

Property	Clay Crust	Transition	Grey Silty Clay	Clayey Silt
Natural Water Content, w_N, %	14	15	23	19
Virgin Compression Index, Cc	0.111	0.120	0.189	0.155
Recompression Index, C_r	0.0123	0.0132	0.0284	0.0170
Swelling Index, C_s	0.0280	0.0301	0.0473	0.0387
Secondary Compression Index, C_α	0.00313	0.00337	0.0053	0.00433

Oedometer testing carried out on samples in the upper grey silty clay from Borehole BH T10-1 (Sample TW16, 18.9 m depth) and Borehole BH T10-2 (Sample TW20A, 20.1 m depth) indicated the following compressibility indices: C_c = 0.130 and 0.097, C_r = 0.018 and 0.011, C_s = 0.030 and 0.017, which are lower than the values in Table 4-2.

The effective shear strength properties applicable to the silty clay to clayey silt stratum were determined from triaxial compression tests performed during the pre-bid and additional geotechnical investigations (Figure 4-3) and supported also by published PI versus ø' relationships (ref. R-28, R-34 and R-42, Figure 4-4), and are summarized as follows:

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

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

One direct shear testing as conducted on sample BH T10-1 sample TW16 (depth of 17 m) indicate an effective friction angle of 27 degrees.

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

$E_u = 300\ S_u$

$E' = 0.9E_u$

Table 4-3: Summary of Interpreted Elastic Moduli Properties

Soils Stratigraphy	Elastic Modulus-Undrained, MPa	Poisson's Ratio-Undrained (*)	Elastic Modulus-Drained, MPa	Poisson's Ratio-Drained (*)
Clay Crust	35	0.49	31.5	0.35
Transition	19.5		17.5	
Grey Silty Clay	14.4		12.9	
Clayey Silt	29		26	

Note: (*) Assumed value

The hydraulic conductivity of the silty clay to clayey silt stratum was interpreted from pore pressure dissipation tests carried out in the CPT probes as well as the laboratory oedometer tests. The hydraulic conductivity values obtained from previous (2007-09) and additional (2011) investigations are plotted on Figure 4.5.

4.3 Lower Granular Deposit

Underlying the silty clay to clayey silt stratum and overlying the bedrock, a heterogeneous non-cohesive material deposit (varying from silty sand, to sand and gravel, and clayey silts with sand) was encountered. Based on SPT N-values ranging generally from 12 to 79, this material is considered to be in a compact to very dense state. This layer was approximately 1.5 to 6.1 m thick within the site 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 elevation ranging from 152.1 to 153 in the vicinity of Tunnel T-10B. The Rock Quality Designation (RQD) of the recovered rock varied between 0 to 100 per cent. The RQD values generally ranged from 38% to 100% with most of the values greater than 25% indicating poor to excellent quality. The RQD values generally increased with depth. Photographs of rock cores recovered from the additional investigation are provided in Appendix H.

Based on this core logging the rock mass classification was estimated to range from 2.8 to 5 for the Q-System (Barton *et. al.*, 1974, ref. R-3) and 53 to 58 for the Rock Mass Rating (RMR) based on Bieniawski (1976) (ref.. R-5) and indicates that the rock mass can be considered as a Fair quality rock mass based on the latter system. With the exception of Borehole BH-314, rock quality generally improved with depth.

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

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). The piezometric levels within the silty clay overburden and the underlying granular stratum/bedrock varied generally from 181.2 to 183.4 and 176.9 to 177.9, respectively (Table 4-5). The highest piezometric levels within the overburden and the bedrock were recorded at elevations 183.7 and 177.9, respectively. These observations suggest a downward gradient between the overburden and the bedrock. However, given the experience at locations along the Parkway project occurrence of localised artesian conditions in bedrock cannot be ruled out.

Table 4-5: Summary of Measured Water Levels at Installed Piezometers

Borehole	Surface El., m	Piezo. Type	Screen / Sensor El., m	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	El., m
BH T10-1	184.9	S-Piez	181.9 to 183.4	Silty Clay	2011-07-29	183.7
		VWP	175.8	Silty Clay	2011-07-29	183.4
		VWP	163.3	Silty Clay	2011-07-29	182.4
BH T10-2	184.8	S-Piez	181.9 to 183.4	Silty Clay	2011-10-19	183.2
		VWP	178.3	Silty Clay	2011-08-29	182.2
		VWP	166.2	Silty Clay	2011-08-29	181.2
		VWP	153.8	Lower Granular	2011-08-29	177.4
BH-112	184.6	S-Piez	175.7	Silty Clay	2011-07-24	182.5
		S-Piez	175.5 to 175.8	Limestone	2011-07-10	177.9
BH-113	184.4	S-Piez	174.7 to 175	Silty Clay	2011-07-24	182.6
		S-Piez	153 to 154.5	Lower Granular	2011-07-24	176.9

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 ppm and can be corrosive at concentrations of about 2 ppm to 3 ppm in the groundwater. The gas odour was not detected during the drilling at the Tunnel T-10B site.

However, although the H_2S and CH_4 gases were not detected during the 2011 geotechnical investigation at T-10B site, their presence cannot be entirely ruled out. Pumping tests were conducted at three locations across the proposed parkway to determine concentration levels of hydrogen sulphide gas in the groundwater of the area. A summary of the results of these tests is provided in Table 4-6, which suggest very low concentration in Tunnel T-10A/T-10B area.

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

5.1 Tunnel Configuration

As indicated previously in Sections 1.2 and 2.3, Tunnel T-10B will be constructed along the below-grade section of the WEP along Highway 3, Service Road 4 between Sta. 40+840 to Sta. 41+000 (Drawing 285380-03-060-WIP1-3051). The proposed Tunnel T-10B is 160 m long and its width is 26.3 m. The tunnel T-10B is a single-span deck-on-girder structure incorporating semi-integral abutments and RSS false abutments. Deck elevations were estimated using the elevation of WP #2 and calculated for each section location using the grades shown on Drawing 285380-03-060-WIP1-3051. The abutments consist of 1.7 m wide × 1.5 high pile caps founded on deep end-bearing HP 310×110 steel piles (Drawing 285380-03-061-WIP1-3058) to bedrock.

Table 5-1 summarizes the 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 (WP #1) – Sta. 40+920	185.0	188.01	186.01 (WP#1)	184.6	178.9
South Wall – Centerline Tunnel (WP#2) Sta. 40+920	185	187.5	186.55 (WP#2)	184.1	178.9

(*) Elevations as interpreted from highways drawing sections 285380-20-024-MST1-0001 (Hwy 3, SR4 STA 40+840 to STA 40+100)
(**) Top of finished grade assumed to be 1 m above deck elevation except where trails are present.

Notes: 1. Top of deck elevations at the North Abutment and South Abutment at Sta. 40+920 were calculated based on the top of deck elevation at WP #1 and WP#2 respectively. Grades at other locations can be calculated based on the following deck slops of 0.5% parallel and 2.0% perpendicular to the tunnel alignment as shown in Drawing 285380-03-060-WIP1-3051.

Geotechnical designs incorporating false abutments with various sections 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 a reinforced granular matt (RGM), which in turn will be founded over undisturbed native silty clay subgrade.

5.2 Geotechnical Design Criteria and Considerations

The geotechnical design has been completed in compliance with the requirements of the execution version of the Project Agreement Schedule 15-2 Part 2, Article 5 (PA) for the Windsor-Essex Parkway Project, 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-8 and CHBDC, R-9).

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 not less than 1.3.

Tunnel T-10B construction is expected to involve the following sequence of earthwork, design elements and loading stages:

- Temporary excavations to about 9.25 m depth below grade;
- Installation of a 1.5 m thick Reinforced Granular Mats (RGM) foundation at the north and south abutments (Void forms are anticipated to be incorporated within the RGM to accommodate later pile installation through the RGM), including base drain;
- Installation of piles (HP310x110) for all tunnel supports;
- 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
- Filling of the CSP casing with concrete followed by construction of the structural abutment (pile cap) and tunnel deck
- Placement of EPS fill on top of the RSS structure
- Completion of final stage of backfill behind the semi-integral abutments
- Completion of the final topsoil placement and trail materials
- The approximate excavation profile for this structure is shown in Drawing 285380-04-091-WIP1-3095 which was developed on the basis of the roadway cross section at Highway 3 SR4 Sta. 40+975.It should be noted that an existing dwelling is located within 25 m of the proposed south abutment at Highway 3 SR4 Sta. 40+950.

5.3 Design Soil Properties

As described in Section 3, the design soil properties for the silty clay to clayey silt deposit were interpreted from the available CPT and Nilcon vane test profiles and the laboratory test results. The undrained shear strength, S_u profiles were estimated from the CPT based on the calibration described in Section 4.2. The interpreted S_u and pre-consolidation pressure profiles are shown in Figure 3.3. Selected typical 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	Undrained Shear Strength (Su), kPa	Effective Stress Parameters	Pre-consolidation Pressure (σp'), kPa	Over Consolidation Ratio
Clay Crust	75 (*)	Cohesion, c' = 0 Friction Angle, ǿ = 30°	450 to 650	>4
Transition	55 to 75		325 to 450	2
Grey Silty Clay	52 to 55		325 to 240	1.5
Clayey Silt	55to 100		400 to 500	1.2

(*) 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 horizontal permeability (k_h) and the hydraulic conductivity anisotropy ratio (A) used for the analysis of stress and deformation response of the soils are provided in Table 5-3. These values are typically 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: Design Hydraulic Conductivity Parameters for Silty Clay Stratum

Clay Substratum	Horizontal Permeability, cm/s	Anisotropy ratio, k_h/k_v (*)	Initial Void Ratio, e_0
Clay Crust	6.8×10^{-7}	1	0.37
Transition	4.1×10^{-7}	2	0.40
Silty Clay	1.1×10^{-7}	1	0.61
Clayey Silt	1.1×10^{-7}	1	0.51
Lower Granular	1.2×10^{-5}	1	

(*) Assumed

The initial groundwater conditions were assumed to be hydrostatic at elevation 183.5.

5.4 Excavation and Temporary Cut Slopes

The discussion of the temporary slopes in this report relates only to the anticipated subsurface conditions to assist the designer of temporary works. The shapes and slopes of the temporary excavations shown do not constitute the actual design of the temporary slopes. The Contractors are fully responsible for the design, construction methods, performance (stability, deformability and deterioration) and maintenance 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 shallow granular soils and underlain by stiff to firm silty clay to a depth of up to 9.2 m below grade (up to elevation 176 m). The anticipated approximate excavation profile for this structure is shown in Figures I.1 and I.2, at various locations along the Tunnel.

Basal hydrostatic uplift was calculated based on the highest measured water level in the bedrock (178.7), anticipated deepest excavation depth (RGM base at elevation 176), and the weight of the silty clay cap of 17.3 m below the deepest excavation. The factor of safety against hydrostatic uplift was 1.7.

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 WEP alignment 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 and accordingly they would mobilize an Ultimate Limit States (ULS) axial geotechnical resistance in excess of 4000 kN. Hence, a factored ULS resistance of at least 2000 kN is anticipated.

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

Based on the available borehole data at this structure, the bedrock surface elevation varies between 152.1 and 153, where the tips of piles are anticipated to be set.

In cases where some of the piles cannot be driven to bedrock due to refusal within 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 than assumed 4000 kN 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 the Tunnel T-10B 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 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.

For preliminary estimates a conventional serviceability limit states (SLS) resistance of 70 to 80 kN along the strong axis of the HP310x110 and 40 kN along the weak axis of the HP310x110 can be considered. This conventional SLS resistance is the lateral shear force applied on a free-head pile section at the level of the ground surface causing a lateral deflection of 10 mm.

Both the USL and SLS to lateral loads resistances are also strongly dependent on the structural and load configuration and on the acceptable deformations. The preliminary design of the piles to lateral loads may be carried out using the horizontal subgrade reaction method. The coefficient of horizontal subgrade reaction, k_h, is based on the following equations:

k_h	$= n_h$ (z/d)	for cohesionless soils, and
	= 67 (S_u/d)	for cohesive soils.

Where:

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

The recommended ranges of soil parameters are tabulated as follows:

Table 5-4: Soil Parameters for Lateral Load Resistance Calculations

Soils Around the Piles	Elevation Range	n_h, MPa/m	Undrained Shear Strength (S_u)
Compact / Dense Sand (within RSS structures(*))	-	10 to 15	
Native Silty Clay Crust	182 to 177	-	0.075 MPa
Native Stiff Silty Clay	El.177 to 175	-	Decreases linearly with depth from 0.075 MPa to 0.05 MPa
Native Firm Silty Clay	Below El. 175	-	0.05 MPa

(*)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.

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

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

Table 5-5: Lateral Load Capacity Reduction Factor For Pile Groups using the Horizontal Subgrade Reaction method (*)

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

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

Alternative Nonlinear 'p-y' Curve Method:

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 (ref. R-15). For a given foundation configuration, pile size, and soil stratification, the soil properties required for the generation of the p-y curves are provided in the table below. "Stiff clay" p-y curves, as given in the LPILE manual, should be developed appropriate for either static or cyclic loading conditions in absence of free water. For p-y curves below the water table, effective unit weights in the soil mass shall be used.

Table 5-6: 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	21.5	Decreases linearly with depth from 75 to 55	0.007
Upper Silty Clay - 1	175 to 166	21	Decreases linearly with depth from 55 to 52	0.009
Upper Silty Clay – 2	166 to 163	20.5	Increase linearly with depth from 52 to 60	0.009
Native Lower Clayey Silt - 1	163 to 162	22	Increases linearly with depth from 60 to 100	0.007
Lower Clayey Silt - 2	162 to 155	22	100	0.005

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

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

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:

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

where:

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

Table 5-7: Lateral Load Capacity Reduction Factor For Pile Groups for p-y Method

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

The modifier factor applies to the "p" values.

The space between the piles under the abutments is approximately 1.74 m (Drawing 285380-03-061-WIP1-3055). Group reduction factors will apply for lateral pile loadings at the abutments.

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

5.5.3 Soil Pile Interaction Assessment

Downdrag Loads (Negative Skin Friction – NSF):

Potential for downdrag loads on piles was considered in conjunction with the anticipated ground movements (rebound and settlements) that are assumed to occur during and following excavation of the overburden of up to 9.5 m to accommodate the future depressed highways, followed by partial re-placement of fills to construct the tunnel abutments.

Soil stress-deformation analyses described later in Section 5.6.2 were conducted using the SIGMA/W software. The net estimated vertical ground movement (settlement/heave) after excavation in the vicinity of the pile shaft at representative stages, namely, after RSS completion (Short-term – ST), after completion of the top backfill against the tunnel diaphragm (End of Construction – EC) and in long-term (LT), and associated is presented in Figures F.7. The analyses indicated the following:

- Ground settlement is expected to occur along the pile shaft during construction of the RSS wall, completion of the associated backfill, and tunnel structure, and
- Ground rebound is expected to occur after substantial completion of the ground surface loading.

Considering the construction staging, the anticipated settlement-rebound of the soils and the transient nature of the downdrag at the site, the estimated maximum unfactored downdrag loads on the piles was estimated to be less then 500 kN.

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

- Permanent load plus downdrag load, but no transient live load; and
- Permanent load and transient live load, but no downdrag load

Shaft Bending:

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

- The pile was modelled with a 500 mm diameter collar section (CSP pipe filled with concrete around the pile shaft) within the RSS wall. Below the RSS wall, the pile section was HP section.
- The ground lateral movement (Appendix G Figure G-1) along the pile shaft anticipated to occur after the installation of the piles was estimated using the stress-deformation analysis described below in Section 5.6.2. The earth pressures from backfill and surcharge loads against the pile cap were not considered in the analyses.
- The above soil deformation field was imposed as “loads” along the pile shaft in conjunction with the loads (shear force and bending moment) estimated at the pile cap. The calculation was conducted using the “p-y” model (L-Pile-5 model Ensoft 2010). The “p-y” curves were generated using the Reese method described in the Technical manual for L-PILE, using the soil parameters provided in Section 5.2.5.
- The earth pressures from backfill and surcharge loads against the pile cap were not considered in the analyses.

Based on the above approach the estimated unfactored bending moments, shear force and deflection in the pile shaft due to the deformations of the soil mass around the piles are listed in Table 5-9.

Table 5-8: Estimated Unfactored Loads on Pile Shaft

Maximum Induced Bending Moment, kN-m	Lateral Load transferred by Pile Shaft to RSS Wall, kN	Deflection of the Pile at underside of RSS wall, mm
60	95	<5

These results are subject to verification by the structural and RSS wall designers. These bending moment, shear force and deflection are in addition to structural response assessed in the pile due to imposed loads by the bridge structure.

These results should be considered in the structural design of the piles and in the design of RSS structural components.

Pile Cap/Abutment Stem Anchoring:

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

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

Unit weight:	21 kN/m^3
Friction Angle (Φ):	35^0
Active Earth Pressure Coefficient (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-38)

where:

σ_v is the vertical stress at the point of calculation including the effects of the dead loads and applicable live loads

$\Delta\sigma_H$ is the 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 is the active earth pressure coefficient

K_r is the correction factor varying from 1.2 to 2.5 depending on the type of reinforcement (extensible like geosynthetics, or inextensible like metal strips or metal bar mats & welded wire grids), and depth of calculation section

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

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

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

All 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 for the north and south abutments:

ELL = 5 kN/m	Earth pressure from Live Loads (LL=9 kPa) from Trail (same for both abutments)
EDL = 45 kN/m (North)	Earth pressure from Dead Surcharge load above the pile cap and earth pressure due to backfill – North Abutment
EDL = 40 kN/m (South Abutment Section 1 Figure J.1) EDL = 35 kN/m (South Section 2 Figure J.1)	Earth pressure from Dead Surcharge load above the pile cap and earth pressure due to backfill

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

The general configurations developed for the typical abutments at Tunnel T-10B South and North abutments are shown in Figures J.1. The abutments comprise retained soil structure (RSS) founded on the reinforced granular mat (RGM), light weight fill (EPS), and engineered backfill. These 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 and (b) the foundation soil bearing resistances. The design assessments were based on (a) assumed strength and deformation properties of the proprietary components (RSS, RGM and EPS), which will have to be confirmed by proprietary suppliers, and (b) the assumed external loads and backfill properties. The final design of the abutment may require adjustments based on the proprietary components and structural design.

The properties of the proprietary products used in the geotechnical analyses are described in Table 5-9.

Table 5-9: Assumed Proprietary Product Properties

Backfill Material	Unit weight, kN/m^3	Limit Equilibrium Analyses (Drained)		Stress Deformation Analyses	
		Friction Angle, °	Apparent Cohesion, kPa	Modulus of Elasticity, E, MPa	Poisson's Ratio, μ
RSS with Approved Granular Fill	21	35	50	40	0.35
RGM	21	35	50	60	0.35
EPS	1.5	0	15	10	0.20

The properties assumed for the backfill materials are also given in Table 5-10.

Table 5-10: Assumed Backfill Material Properties

		Limit Equilibrium Analyses		Stress Deformation 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
Roadway Granular Backfill	22	N/A	35	22.5	0.35
Light Weight Fill (LWF)	12	N/A	35	30	0.35

Preliminary dimensions of the abutments are listed in Table 5-14.
* 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 E-1 to E-8 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 for two cases: a) short-term during construction without the pavement structure (ST-Construction), b) end of construction case assuming the pavement structure over the subgrade at the toe of the slope in place (ST-EOC). The drained analyses assumed that all the components of the system are present and steady state condition had been achieved. The presence of the piles was not considered in the stability models (somewhat conservative approach). Surcharge of 9 kPa for short-term and long-term model was applied at the top of ground surface, while tension crack was assumed for the undrained condition only.

The calculated factors of safety (FS) exceed 1.3 against global instability of the abutments, as shown in Figures E-1 to E-6 and summarized in Table 5-11. Stability analysis results for the wing-wall are presented in Table 5-17.

Table 5-11: Summarized Global Stability Results

Abutment	Factor of Safety for Loading Condition (*)			Figure
	Short-Term (**) (Undrained) (During Construction)	End of Construction (Undrained)	Long-term (Drained)	
South Abutment – Sta. 40+990	1.63 (1.40)	1.78 (1.59)	1.81 (1.58)	E-1 to E-3
South Wall – Section 2	1.72 (1.44)	1.95 (1.62)	1.75 (1.48)	E-4 to E-6

(*) Values outside and within the brackets refer to circular and non-circular failure surfaces, respectively.
(**) Short-Term Loading assumes no pavement structure final landscape grading in place,

Surcharge of 12 kPa for short term and long term end of construction model was applied at the top of ground surface.

The calculated factors of safety are in excess of 1.3 against global instability of the abutments.

The design ground water level for the analysis was taken as 183.75.

As indicated earlier, the abutment configurations were developed in consideration of both the global stability and the geotechnical bearing of the false abutments using the applicable soil characteristics and the design undrained strength profiles. The tunnel abutment designs have been governed primarily by the geotechnical bearing requirement, and that is the reason that global stability FS values are significantly higher than the required minimum value of 1.30.

5.6.2 Stress Deformation Analyses

Stress-deformation analyses (SDA) were carried out by finite element modeling using SIGMA/W software. 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, piles) were not included in the model, albeit their presence was simulated with boundary restraints.

The configuration of the calculation model is presented in Figure F.1. 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 a K_0 factor of 0.75 for the soil deposit

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

c) Construction of the RSS structure and associated backfill

d) Completion of the remaining fill above the RSS structure

e) Completion of the pavement structure for Highway 3.

The stratigraphy and selection of the soil properties (except for the RSS structure and pavement box) was based on the design soil properties discussed at Section 5.2.

For the SDA purposes, the RSS structure, RGM and pavement were assumed as homogeneous elastic materials described by E=60 MPa, μ= 0.35 and γ = 21 kN/m^3 (Table 5-3 above).

The SDA were carried out for drained (effective stress) (Figure F.1) soil behaviour. The phreatic surface was assumed to correspond to the initial groundwater level at elevation 183.5 and follow the excavation and subgrade surfaces. Elastic-plastic Mohr-Coulomb constitutive models were associated to all soils and backfill except for the unweathered soft and firm silty clay which was described by the Modified Cam-Clay model.

5.6.3 Serviceability Limit States (SLS) Performance

The SLS performance was assessed on the basis of the SDAs described above in Section 5.5.2. The cumulative deformations are summarized as follows:

Table 5-12: Summary of Calculated Cumulative Deformations

Parameter	End of RSS Construction	End of Construction (Undrained)	Long-term (Drained)	Remarks
Settlements on Top of Approachway (mm) at Distances (m) from the Edge of Deck of:				Figure F.3a F.3b
0 m	N/A	25 mm (*)	20 mm	
5 m	N/A	30 mm (*)	20 mm	
10 m	N/A	35 mm (*)	25 mm	
At 20 m	-30	40 mm (*)	25 mm	
Settlement at the top of RSS facing (mm)	40 mm (*)	60 mm	45 mm	Figure F.4
Lateral displacement at the base of RSS facing (mm)	<5 mm	<5 mm	10 mm	Figure F.5
Rotation of the RSS facing	< 0.002	<0.002	<0.002	Figure F.5
Maximum Heave (rebound) at Highway 3 C/L	45 mm	45 mm	80 mm	Figure F.6

N/A Not Applicable – Area located within the temporary excavation.
Note: The above values do not include deformations induced by excavation for Highway 401. Distances are measured perpendicular to the bridge abutment.
(*) Indicates calculated settlement at top of wall / abutment backfill to be compensated during constructions.

- Estimated RSS structure rotation during backfilling was less than 0.002. A chart of calculated horizontal deflection is provided on Figure F.5.
- The ground movements generated by the construction loads are anticipated to stabilize within approximately 10 to 15 years following completion of construction.
- Due to the relatively smooth changes in the geometry of the tunnel, the above settlement changes along Highway 3 are anticipated to be gradual in longitudinal profile.
- All the 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 approximate indication of the magnitude of the soil response. These estimates will be verified and refined with respect to the actual performance monitoring in the field.
- The ground movement and deformation discussed above do not account for the future excavation of Highway 401. A preliminary estimate of the potential effects of the future Tunnel T-10A on Tunnel T-10B is discussed later in Section 5.10.

5.6.4 RSS 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): 300 kPa (based on average shear strength of 60 kPa).
- Long-term (drained): 525 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.

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

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

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	50		

5.6.5 Abutment Configurations

Based on geotechnical analyses discussed in Sections 5.1 to 5.7, abutment configurations and dimensions were determined (Table 5-14). 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 Tentative Dimensions

Abutment Location	Assumed Total Height(1), m	RGM Size (Thickness x Min. Length at Base)	Light Weight Fill (EPS) ,m^3/m	RSS Structure Size (Width x Height)(3), m
South- at Trail crossings	9.5	1.5 x 9.5	6.75	6.5 x 4.7
South – between trails	9.2	1.5 x 9	4	6 x 4.7
North-at Trail crossing	9.9	1.5 x 19.5	6.75	6.5 x 4.
North – No Trails	9.0	1.5 x 9	4	6 x 4.1

1) Measured from top of finished grade at tunnel edge to the base of the RSS structure
2) 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.
3) The use of RGM and EPS required to meet the ULS design for undrained short-term condition

5.7 RGM Foundation Loads

1.5 m thick RGM foundations were considered under the RSS false abutment walls to improve the bearing soils and satisfy the WS bearing capacity requirements for undrained conditions at the North and South abutments. For preliminary estimates, 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 within a 45° angle. The following loads in (Table 5-15) were estimated to act on top of the RGM on the basis of conventional calculation of the bearing pressures under gravity retaining walls:

Table 5-15: Estimated load on RGM at the underside of RSS

Abutment Location	Maximum Edge Bearing Pressure, kPa	Average Unfactored Bearing Pressure, kPa
South Wall – Sta. 40+950	170	160
South Wall – Sta. 40+990	178	162
North Wall – Sta. 40+915	160	160
North Wall – Sta. 40+990	150	150

Based on the above load on RGM, an estimated unfactored horizontal tensile load of 65 kN per meter of RGM for both walls was estimated across the entire height of 1.5 m. For cost estimates, it is considered that this tensile load can be accommodated by 3 layers of UX1000HS, or equivalent.

5.8 Approach Embankment

The tunnel will be crossed by a number of recreational trails. Settlements of up to 35 mm were estimated to occur at the ground surface.

5.9 Wing-walls and Return Walls

As mentioned earlier, RSS return walls flared at 90° to the tunnel diaphragm are indicated at each corner of the structure. The tapered RSS wing walls are extended beyond the tunnel portals at north and south sides. The RSS walls have also been checked to meet the external stability requirements similar to Section 5.6.4 and the analysis results utilized to develop RSS wall configurations.

Table 5-16 summarizes the results of slope stability analyses carried out for the RSS wing wall. Due to the trail loading, only the wing-wall at the southeast section was checked as this wall height is much higher than other walls.

Table 5-16: Summary of the Results of Southeast Wingwall Global 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 Wall – Southeast	1.66 (1.34)	1.85 (1.56)	1.79 (1.61)	E-7 to E-9

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

Similar to the abutment walls, the tapered and return RSS walls have been checked for external stability.

Based on geotechnical analyses discussed in Section 5.2 to 5.6, tentative wing wall configurations and dimensions summarized in Table 5-17 were determined. The wing wall configurations and dimensions indicated in these analyses are preliminary (e.g., the indicated width of the RSS is the minimum width) and are to be finalized by proprietary suppliers. The design of the abutments is to be developed in consultation with the proprietary component suppliers.

Table 5-17: Summary of the Tentative Wing-wall Structures

Wingwall Location	Wing Wall Type	RSS Structure (Width × Height)(1), m	RGM (Width x Thickness), m	Quantity of Lightweight Fill, m^3/m
Southeast Wall – 41+000 "Long Panel"(3)	Extension	6.5 × 7.9	9.5 x 1.5	29.25
Northeast, Northwest and Southwest "Long-Panels" and Southeast "Short Panel"	Tapered	5.0 × 6.4*	8 x 1.5	0
Northeast, Northwest and Southwest "Short-Panels"	Tapered	4.0 x 3.1*	5 x 0.5	0
All return wall locations	Return	3.8 x 4.9	-	13.5

(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, except for southwest wall "long – panel" which includes requires additional fill for trail.
(2) Measured into the abutment wall (parallel to the Highway 401 alignment)
(3) "Long Panel" refers to the higher wing wall sections. "Short panel" refers to stepped wing-wall section
(*) Dimensions vary, tallest height shown

5.10 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 5-18.

Table 5-18: Soil Parameters for Earth Pressure Calculations

Soil Parameter	Group I Soils	Group II Soils	Group III Soils
Fill Unit Weight, kN/m^3	22	21	20.5
Friction angle, ∅ (degrees)	33 -35	29-32	22-30
Coefficients of Static Lateral Earth Pressure:			
'Active' or Unrestrained, $K_a^{(*)}$	0.27 to 0.30	0.310 to 0.35	0.33 to 0.45
'At Rest' or Restrained, $K_o^{(*)}$	0.43 to 0.46	0.47 to 0.52	0.50 to 0.62
'Passive', $K_p^{(*)}$	3.3 to 3.7	2.9 to 3.2	2.2 to 3.0

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

Compactable Group III soils may be used as general backfill within approved areas

5.11 Permanent Subdrainage System

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

Use of free-draining granular soils for the retained soil mass within the RSS structures and the RGM, as recommended, will ensure that these structures will act as a "natural" drain convening the seepage from the groundwater and infiltrations from surface precipitations toward the toe of the wall facing and base of the RGM. In order to prevent accumulation and stagnation of groundwater within the RGM, the subgrade should be graded to direct the collected groundwater to manholes or sumps.

Depending on the grain size of the backfill, RSS and RGM materials, a granular filter layer may be required at the interface between the native soil (e.g., at the excavation slope) and the backfill.

Simplified steady-state models (Appendix H) were used to estimate seepage rates associated with the long-term drawdown of the groundwater along a typical cross-section of Tunnels T-10A and T-10B system. SEEP/W 2007 software was used for these analyses. The initial groundwater table was assumed at elevation 178 for both abutment models and a high water level of elevation 184 m was used for the Wolfe Drain in the north abutment model. Groundwater recharge from infiltrations from ground surface sources was also considered. The rates of recharge were estimated on the basis of saturated hydraulic conductivity of the soils. A ground surface infiltration of $1x10^{-4}$ m/day was accommodated by trial-and-error approach to ensure a sustained groundwater level without excessive mounding.

Based on the above, the flow rate from groundwater seepage across the entire tunnel cross section was estimated to be about 2.0 litre/day per meter length at Tunnel T-10B. 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.

5.12 Flood Events

Based on the estimated elevation of 177.5 for the 100year flooding event and 177.9 for the regional storm event from Pump Station 6 in the vicinity of Tunnel T10B, flooding of the roadway in this tunnel will not occur during a 100-year storm event. As such, submergence of the EPS or LWF material is not anticipated to occur in the area of Tunnel T-10B.

5.13 Interactions with Tunnel T-10A

As mentioned earlier, it is anticipated that the construction for Tunnel T-10A may cause some interactions with the structures of Tunnel T-10B.

It is likely that the zones of temporary excavation for Tunnel T-10A will encroach on the RSS and Lightweight fill at Tunnel T-10B. It is recommended that the RSS manufacturer be consulted to ensure that the integrity of the RSS is not adversely affected and it maintains its load carrying capabilities. A "sacrificial" reinforced earth type wall facing may need to be considered at the north side of the RSS wall for Tunnel T-10B. Delaying of the placement of the lightweight fill or EPS and final grading for Tunnel T-10B north abutment in areas of interference may be necessary until Tunnel T-10A south abutment is constructed. A simplified SIGMA model (Appendix F.8) was used for a preliminary evaluation of the deformations imposed on Tunnel T-10B components by the excavations for Tunnel 10A. Select charts showing settlements and lateral displacement of the Tunnel T-10B walls and roadway are also provided in Appendix F.9 and F.10. A summary of representative deformations as obtained from the SIGMA model is provided in Table 5-19.

Table 5-19: Estimated Effect of Excavation for Tunnel T-10A on Tunnel T-10B and Highway 3 Subgrade

Deformation Item	North Abutment	South Abutment	Highway 3 Subgrade
Lateral Displacement, mm (*)	65 to 75	30 to 40	75 to 40
Wall Face Rotation	<0.003	<0.003	N/A
Settlement, mm	40	20	40 to 20

(*) lateral movement toward Highway 401

Due to the relatively smooth changes in the geometry of the tunnel and assuming that the excavations for Highway 401 will progress uniformly, the ground movements given above are expected to be gradual.

As mentioned earlier the ground movement and deformations discussed above are estimates based on soil deformation / compressibility properties interpreted from laboratory tests and empirical correlations. Therefore, the reported values are approximate and should be considered only as an approximate indication of the magnitude of the soil response. Monitoring of Tunnel T-10A and its impact should be monitored at strategic locations during and after excavations. The above estimates will be verified and refined with respect to the actual performance monitoring in the field.

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 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 decisions. References to construction methods are not intended to be the suggestions or directions on the construction methodologies. Contractors should be aware that the data presented in this report and their interpretations may not be sufficient to assess all factors that may affect the construction.

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

- All excavation works should be carried out in accordance with the guidelines outlined in Occupational Health and Safety Act (OHSA) and Ontario Provincial Standard Specification (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 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, geo-fabrics, 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.
- Riprap and other coarse rockfill covers are considered to have half the insulation effect as offered by the fine grained soil deposits/cover, and therefore, the depth of frost penetration will have to be increased proportionally.
- During the excavation of the south abutment of Tunnel T-10A, adequate protection of the RSS walls of the North abutment of Tunnel T-10B, as determined by the RSS supplier, will be required. This may include limiting the exposed slope against the rear of the RSS wall especially at the west end of the tunnels were the two tunnels are closest.
- The contractor shall monitor for the potential emissions of natural gas (primarily hydrogen sulphide (H_2S) and methane (CH_4) gases) during construction.

6.3 Instrumentation and Monitoring during Construction

As mentioned earlier in Section 5.3, a program of site instrumentation and monitoring of the temporary works during construction should be implemented by the Contractor in addition to the limited instrumentation already installed during the geotechnical investigation.

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

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

Instrumentation: A limited number of geotechnical instruments[9] were installed during the recent geotechnical investigation at the locations of boreholes as follows:

Table 6-1: Instrumentations in Boreholes during Additional Investigation

Locations	VWP at Elevation
Borehole T10-1	175.8 163.3
Borehole T10-2	178.3 166.1 153.8

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

- Heave/ settlement gauges (MHSG) placed typically at about 1.5 to 2 m below the subgrade level;
- Low displacement type of piezometers (vibratory wire piezometers, VWP) placed typically below the subgrade level at depths of 0.5 to 1.5 times the depth of excavation;
- Shallow survey pins (stakes) typically driven >600 mm into the ground; and
- Inclinometers.

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

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

The instrument monitoring should be completed on a regular basis. As a general guideline, the following schedule should be considered after the completion of the baseline survey:

Table 6-2: Monitoring Schedule of the Instruments

Instruments	Active Excavation	Active Construction inside the Excavation	Backfilling	Post-Construction
Piezometers	EOD	D	W	M
Heave Gauge	EOD	EOD	W	M
Inclinometer	TPW	EOD	BW	M
Survey Pins	TPW	EOD	BW	M

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

[9] Vibrating wire piezometers (VWP) to measure pore water pressure,

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

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

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

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

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

Two inclinometers are proposed between Tunnels T-10A and T-10B to ensure that the impact of the construction of T-10A is within allowable limits.

6.4 Corrosion Potential

Analytical testing was carried out on a sample of the clay obtained in borehole T10-2 (sample 7). The following Table 6-3 provides the results of various analyses carried out on the soil samples to assess the potential for corrosion on concrete:

Table 6-3: 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 T10-2 (Sample 7)	179.6	7.86	228	3620	<0.2	180

- The reported results of laboratory testing indicate that based on CSA A23.1, concrete in contact with the tested soil material would have a negligible degree of exposure to sulphate attack.
- Based on the measured electrical resistivity, pH, redox potential, sulphide contents etc., the soil would be considered to have a potential for corrosion to buried metallic elements.
- A corrosion specialist should review the test result and satisfy with their adequacy.

6.5 Construction Quality Control

To ensure that construction is carried out in a manner consistent with the intent of the PA, the design and the recommendations set forth in this report, a construction quality control program, including geotechnical inspection, 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 design for Tunnel T-10B was developed by Mr. Tommi Leinala, P.Eng. under design direction of Dr. Dan Dimitriu, P.Eng. (Lead Designer). The project was executed under the technical direction of Dr. Narendra S. Verma, P.Eng. (Technical Director) who also provided the senior review of the report. Mr. Matt Oldewening, P.Eng. managed the geotechnical investigation and Mr. Brian Lapos, P.Eng. is the project manager.

Mr. Zuhtu Ozden, P.Eng. and Dr. Andrew Smith of Coffey Geotechnics provided the peer review.

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

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

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Associate Geotechnical Engineer

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Principal Geotechnical Engineer
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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 3
HEARTHWOOD TUNNEL T–10B
GENERAL ARRANGEMENT

SHEET S3051

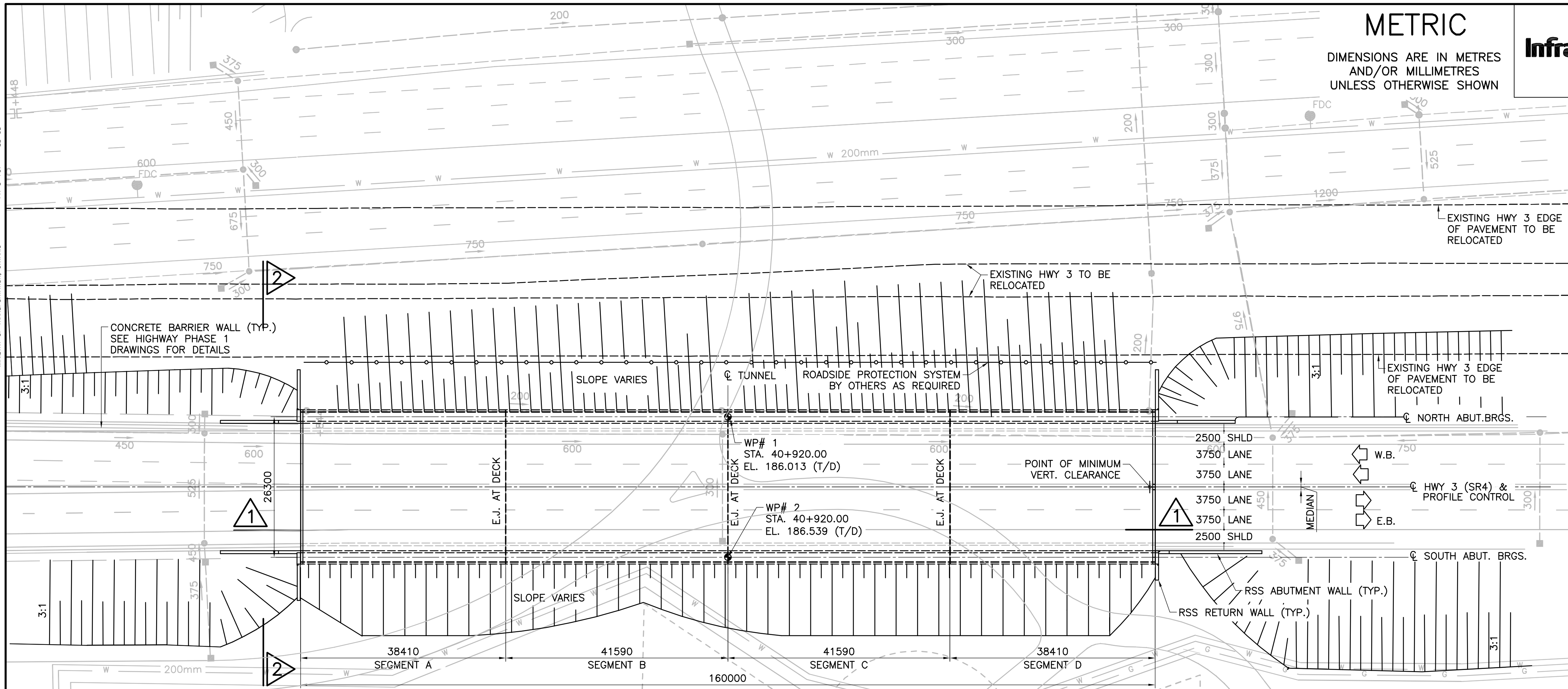
Phase 1

IFC

NORTH FOR CONSTRUCTION

NOTE:

1. SEE SHEET S3052 FOR GENERAL NOTES.



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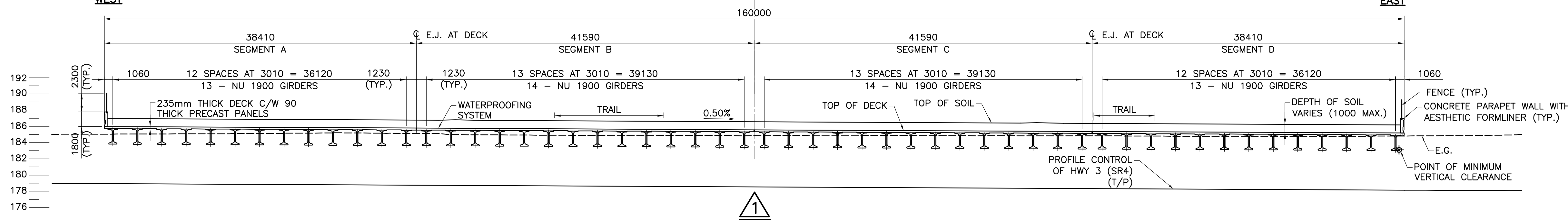

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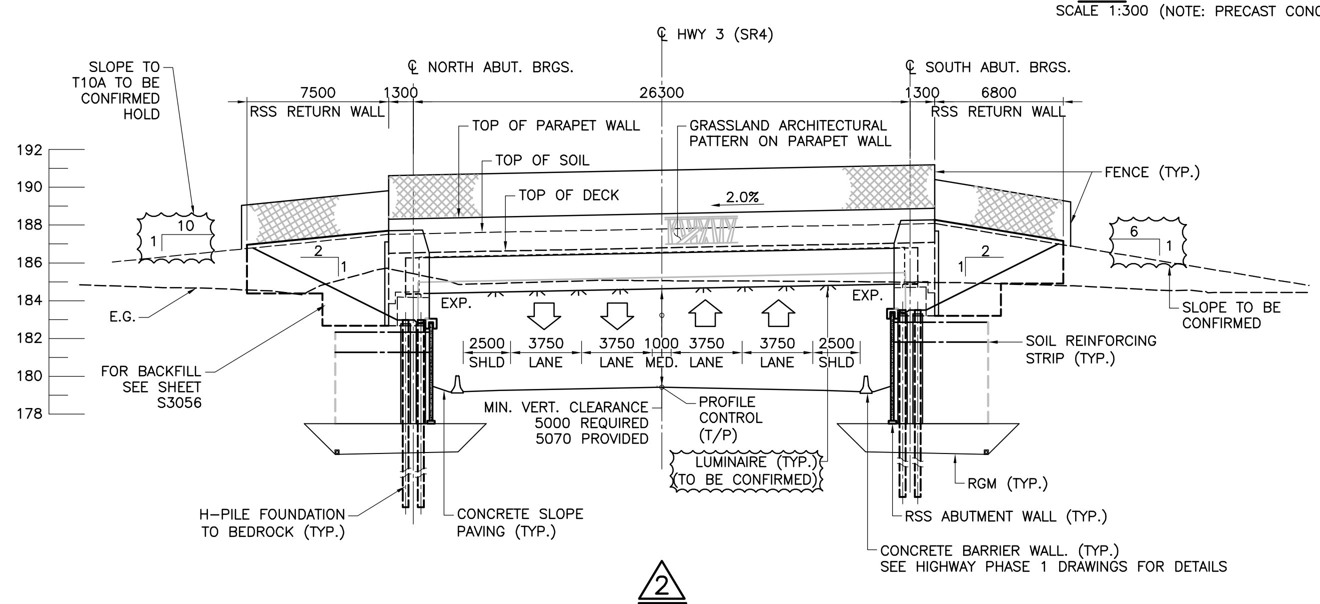

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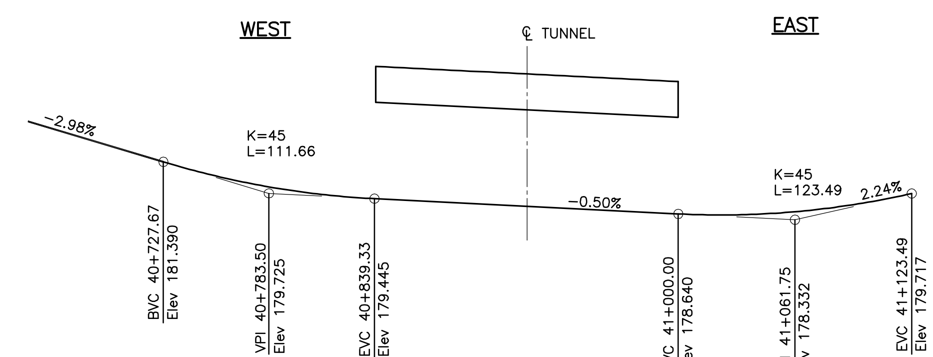

PROFILE OF HWY 3 (SR4)
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REVISIONS

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METRIC

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

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

LOCATION PLAN & INTERPRETED STRATIGRAPHIC PROFILE

STA 12+100L TO STA 12+800L

SHEET G3052

Phase 1

IFC

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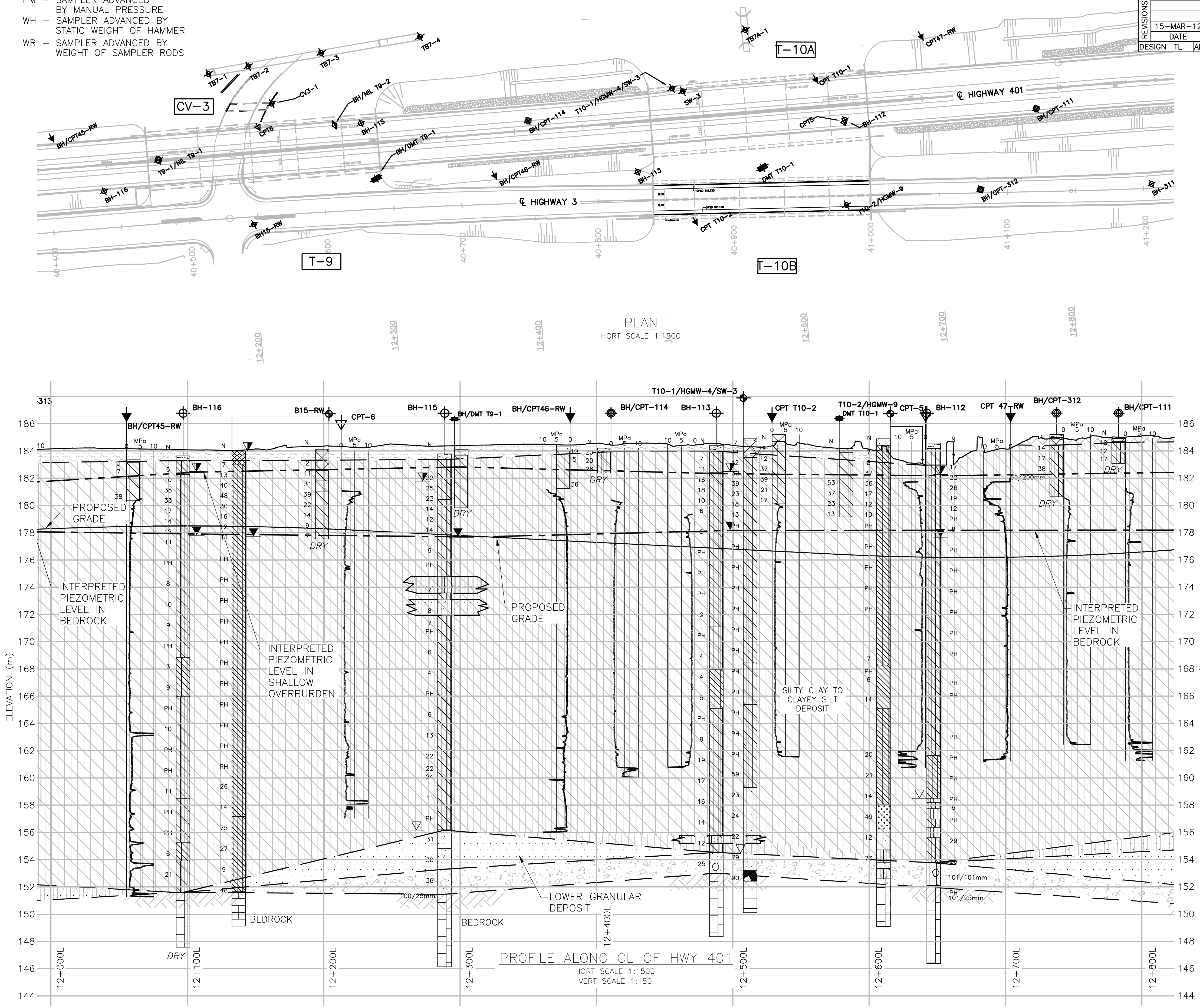

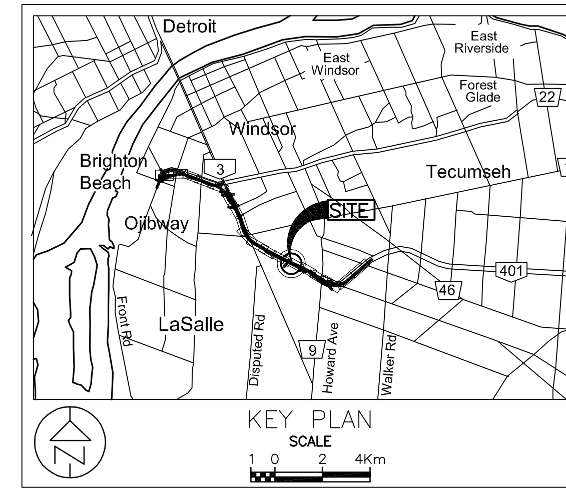

LEGEND

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

NOTES

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

SCALES

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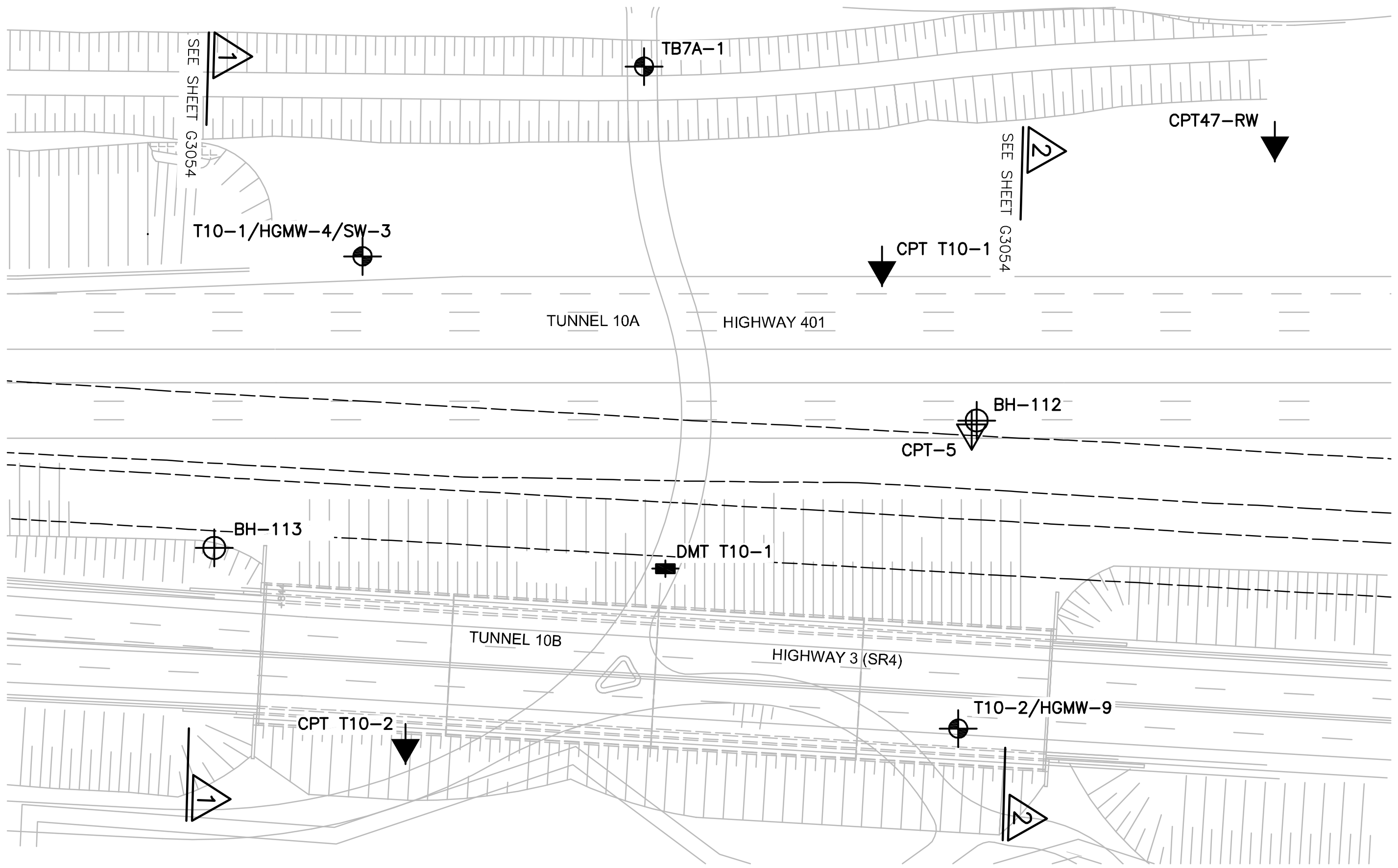
Windsor-Essex Parkway Project RFP No. 09-54-1007

NEW CONSTRUCTION
HWY 3
HEARTHWOOD TUNNEL T-10B
BOREHOLE LOCATIONS & SOIL STRATA

SHEET G3053

Phase 1

IFC



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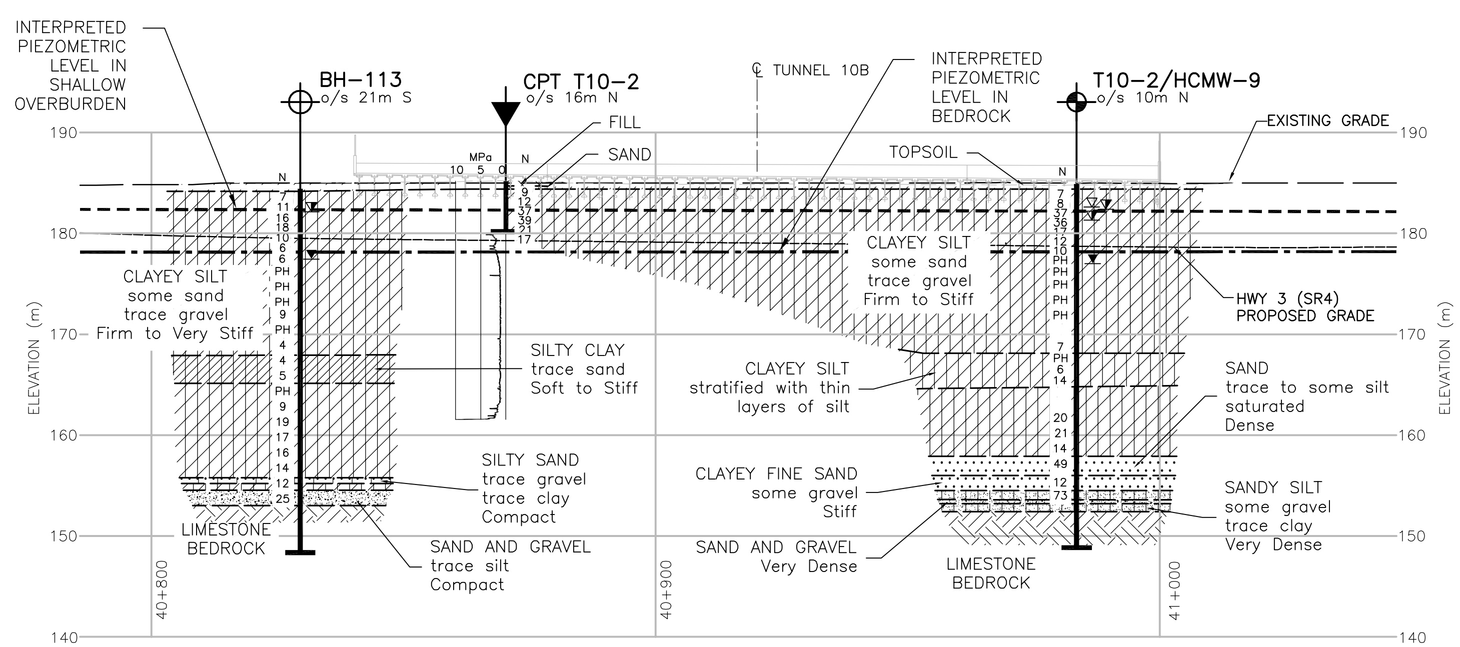

PROFILE ALONG CL OF HWY 3
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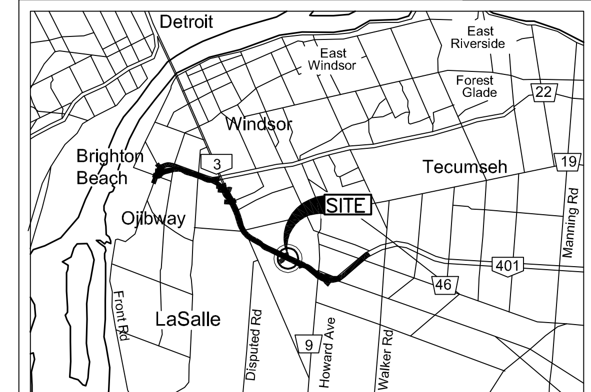

KEY PLAN
SCALE
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LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

LEGEND

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

No.	ELEVATION	CO-ORDINATES (UTM, NAD 83 ZONE 17) NORTHING	EASTING
AMEC BOREHOLES			
CPT T10-1	184.9	4678450.6	334217.4
CPT T10-2	185.2	4678403.2	334089.2
DMT T10-1	184.6	4678412.4	334151.5
T10-1/HGMW-4/SW-3	184.9	4678495.6	334122.3
T10-2/HGMW-9	184.8	4678358.2	334191.8
TB7A-1	184.8	4678506.6	334190.2
CPT47-RW	185.4	4678440.3	334300.2
PREVIOUS BOREHOLES			
BH-112	184.6	4678413.3	334221.3
BH-113	184.4	4678454.5	334070.3
CPT-5	184.7	4678413.0	334220.0

NOTES

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2. THE INTERPRETED STRATIGRAPHY REPRESENTS SIMPLIFIED SUBSURFACE CONDITIONS. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN DEFINED AT BOREHOLE LOCATIONS ONLY. CONDITIONS BETWEEN BOREHOLE LOCATIONS COULD DIFFER FROM ILLUSTRATED CONDITIONS.
3. ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS DATE	REV.	BY	DESCRIPTION
15-MAR-12	0	TL	ISSUED FOR CONSTRUCTION

DESIGN TL	CHK NSV	CODE CAN/CSA S6-06	LOAD CL-625-ON
DRAWN MM	CHK DD	SITE 6-710-1	DATE 18-JUL-11

DOC: 285380-04-090-WIP1-3053

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

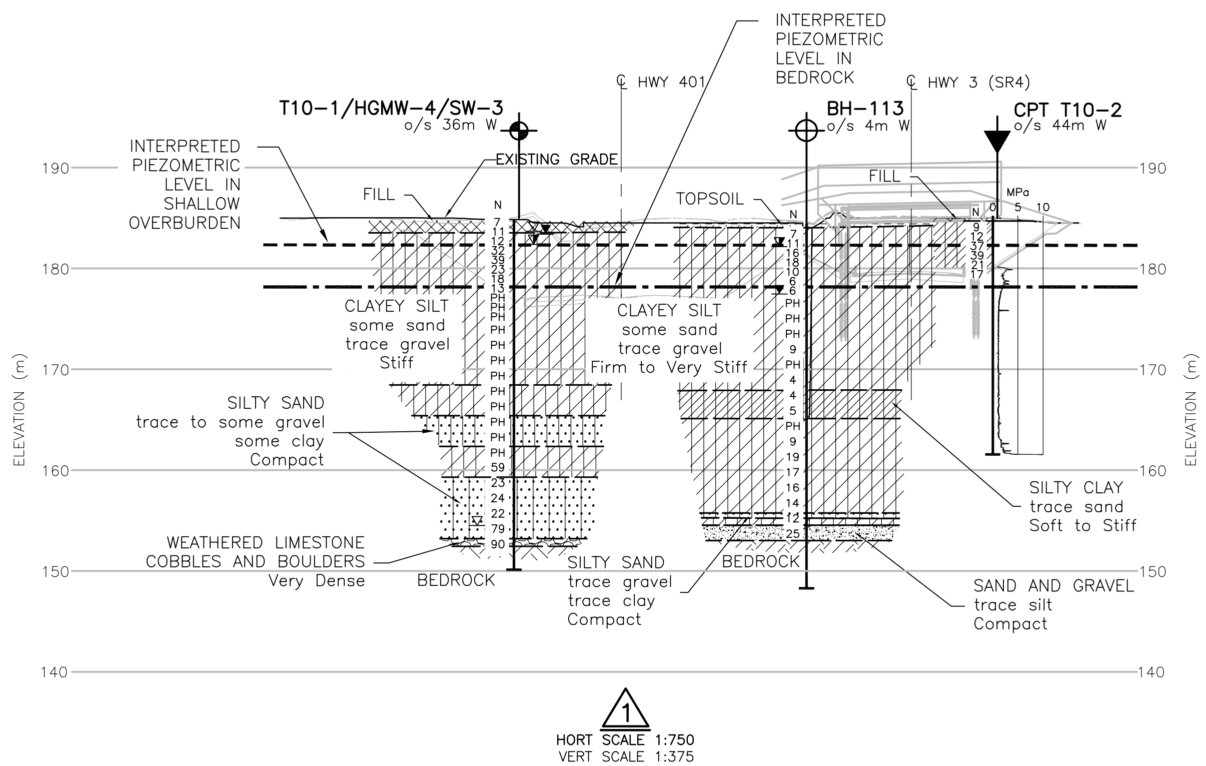
Windsor–Essex Parkway Project
RFP No. 09–54–1007

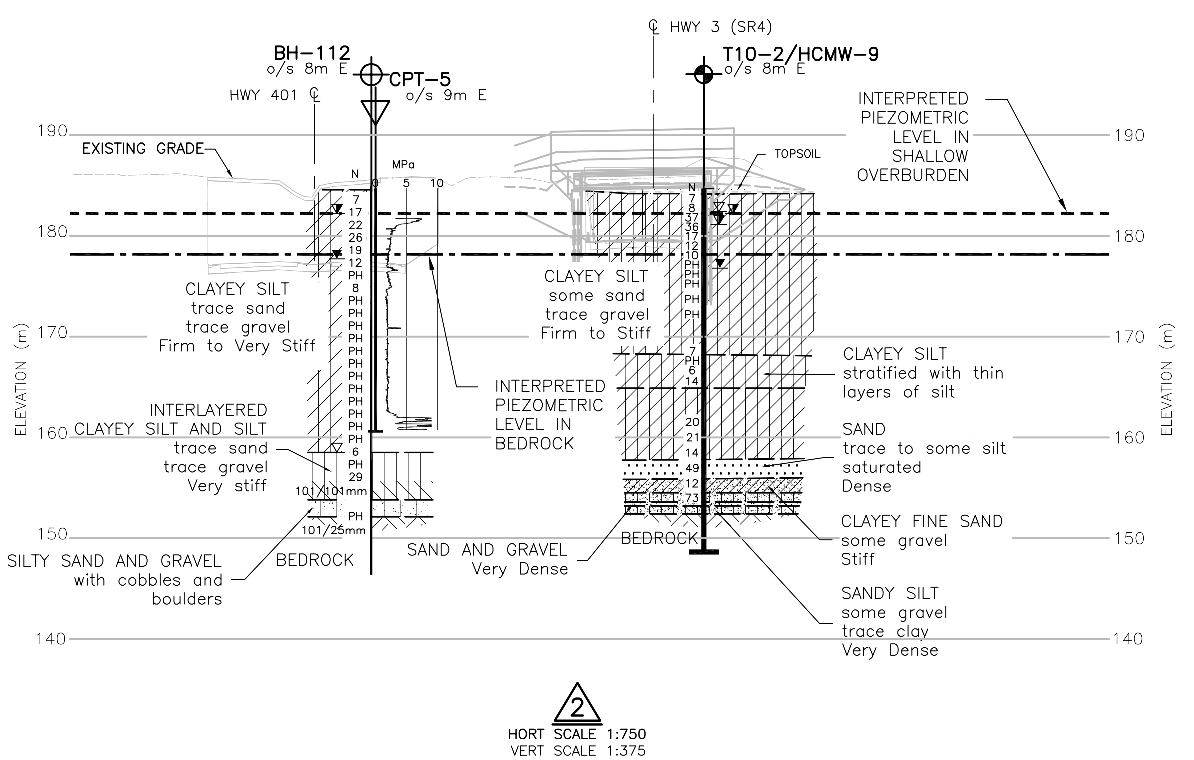
NEW CONSTRUCTION
HWY 3
HEARTHWOOD TUNNEL T–10B
SOIL STRATIGRAPHY

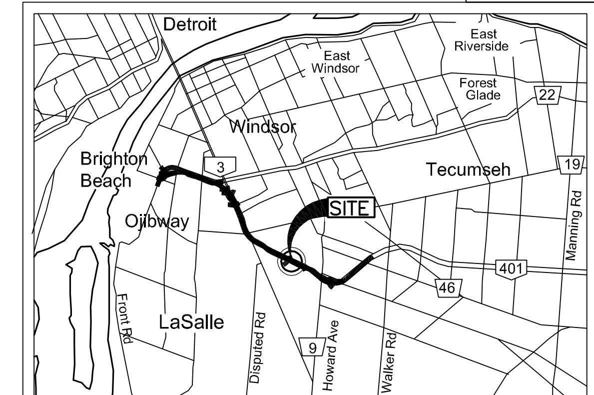
SHEET G3054

Phase 1

IFC







KEY PLAN
SCALE 1 0 2 4Km

LEGEND

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

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

No.	ELEVATION	CO–ORDINATES (UTM, NAD 83 ZONE 17) NORTHING	EASTING
AMEC BOREHOLES			
CPT T10–1	184.9	4678450.6	334217.4
CPT T10–2	185.2	4678403.2	334089.2
DMT T10–1	184.6	4678412.4	334151.5
T10–1/HGMW–4/SW–3	184.9	4678495.6	334122.3
T10–2/HGMW–9	184.8	4678358.2	334191.8
TB7A–1	184.8	4678506.6	334190.2
CPT47–RW	185.4	4678440.3	334300.2
PREVIOUS BOREHOLES			
BH–112	184.6	4678413.3	334221.3
BH–113	184.4	4678454.5	334070.3
CPT–5	184.7	4678413.0	334220.0

NOTES

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

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS DATE	REV.	BY	DESCRIPTION
15–MAR–12	0	TL	ISSUED FOR CONSTRUCTION

DESIGN TL | CHK NSV | CODE CAN/CSA S6–06 | LOAD CL–625–ON
DRAWN SC | CHK DD | SITE 6–710–1 | DATE 18–JUL–11

DOC: 285380–04–091–WIP1–3054

DATE PLOTTED: 3/15/2012 11:00:48 AM
FILE LOCATION: C:\pwworking\hmmg_285380\stephen.labute@amec.com\dms09858\285380-04-091-WIP1-3054.dwg

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

Figures

Figure 3-1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures
(Figure 5.1, Ladd & DeGroot, 2004, ref. R-29)

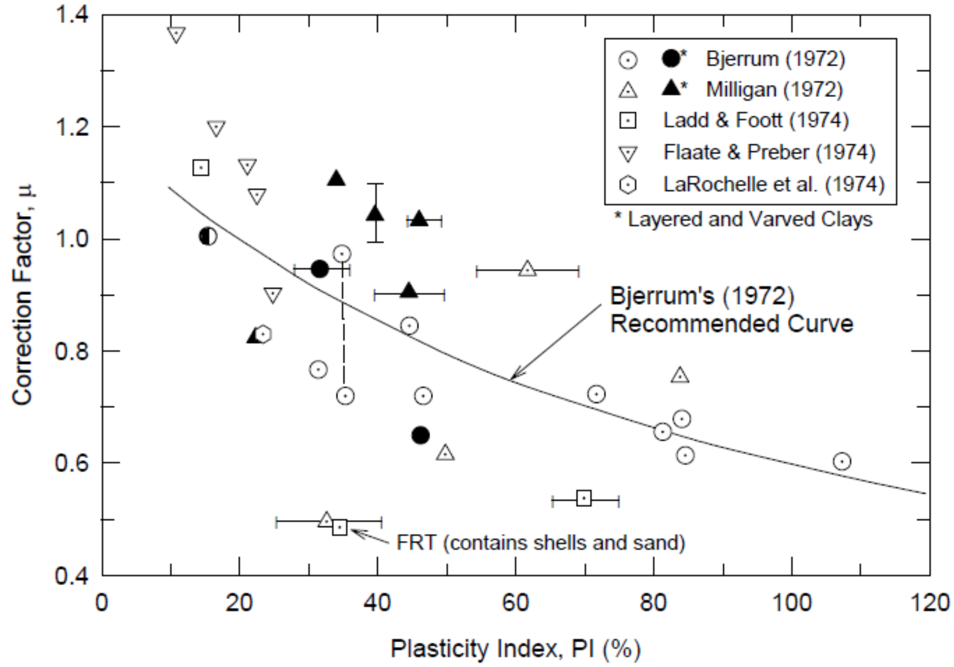
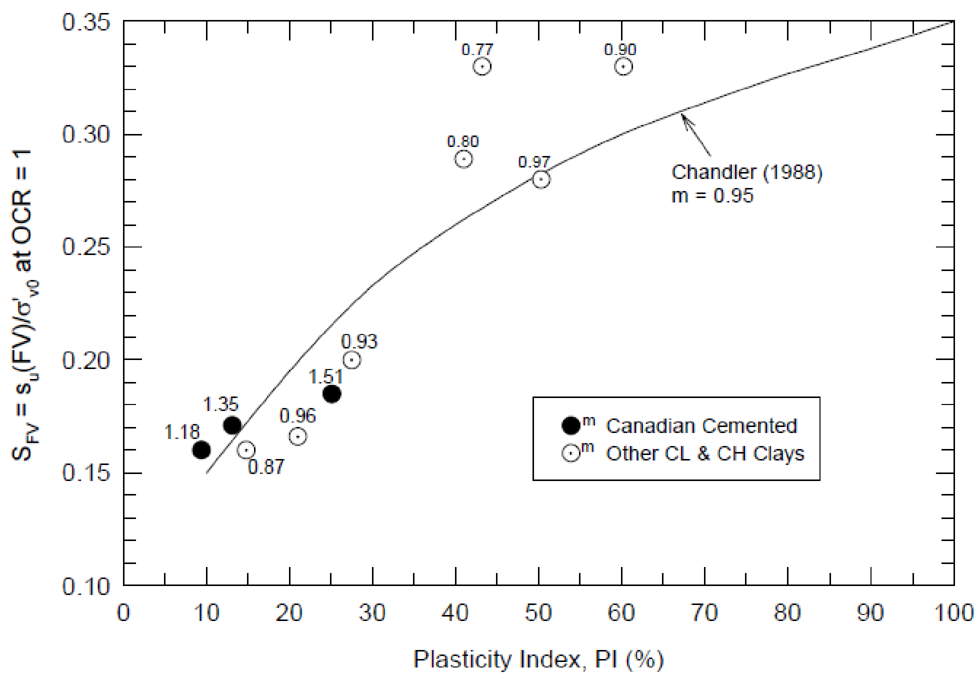
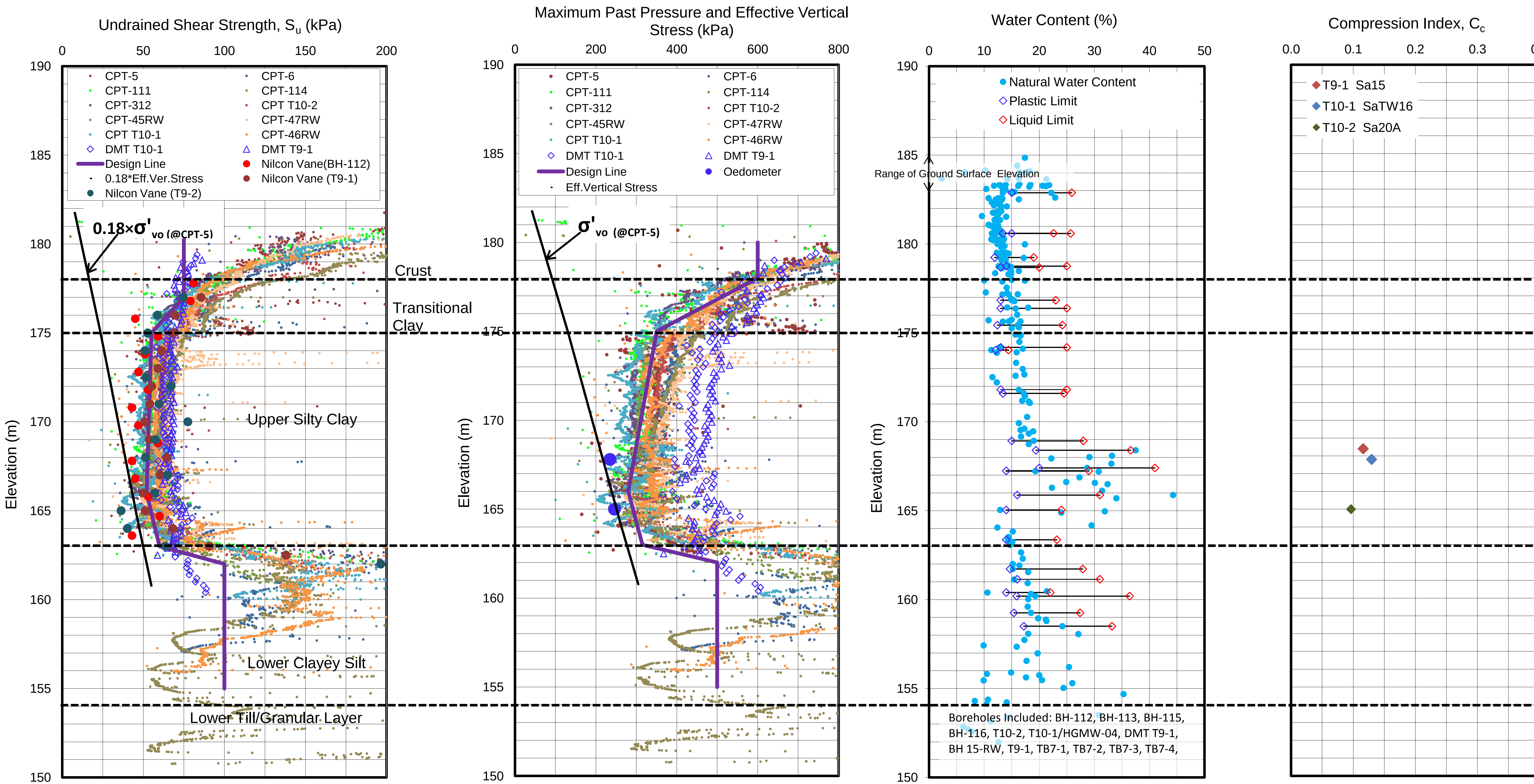


Figure 3-2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays
(Figure 5.2, Ladd & DeGroot, 2004, ref. R-29)



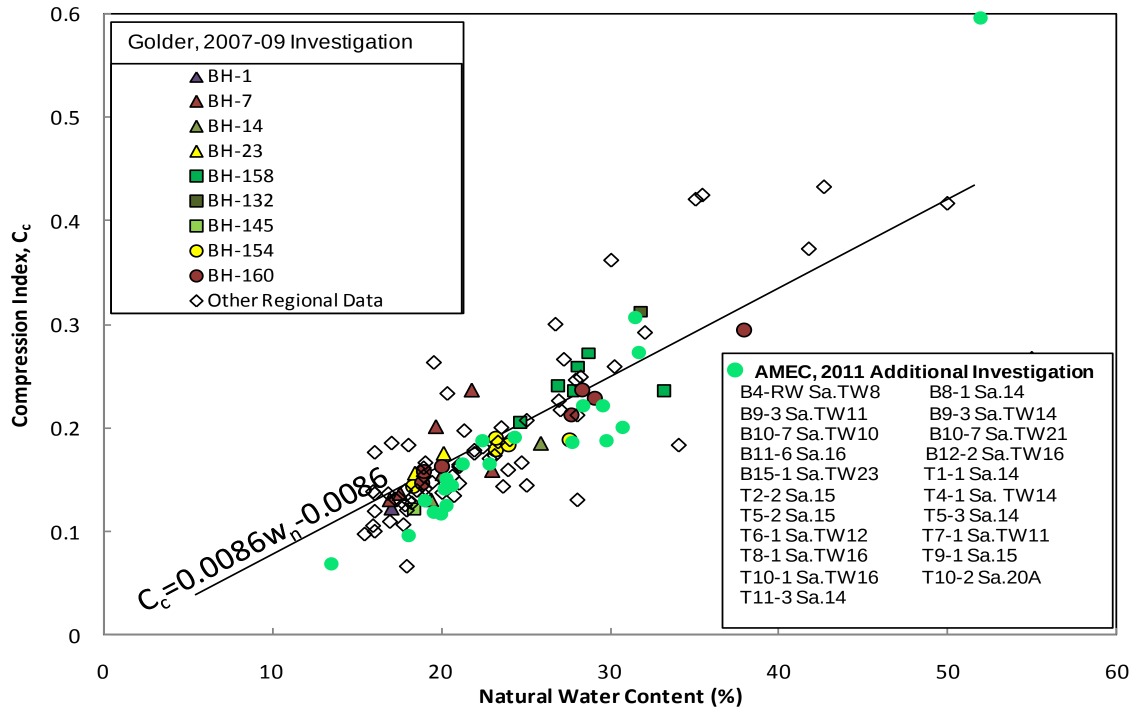


Notes:

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. Based on current geotechnical investigation by AMEC and historic information from Golder Associates.

amec Earth & Environmental	PROJECT: WINDSOR ESSEX PARKWAY					
	TITLE: SOIL PROPERTIES PROFILES STA.12+000L TO 12+800L					
CLIENT :	DATE: Mar 2012	JOB NO.: SW8801.1002	CAD FILE:	FIGURE NO.:	REV.	

Figure 4-1: Compressibility Parameters at WEP



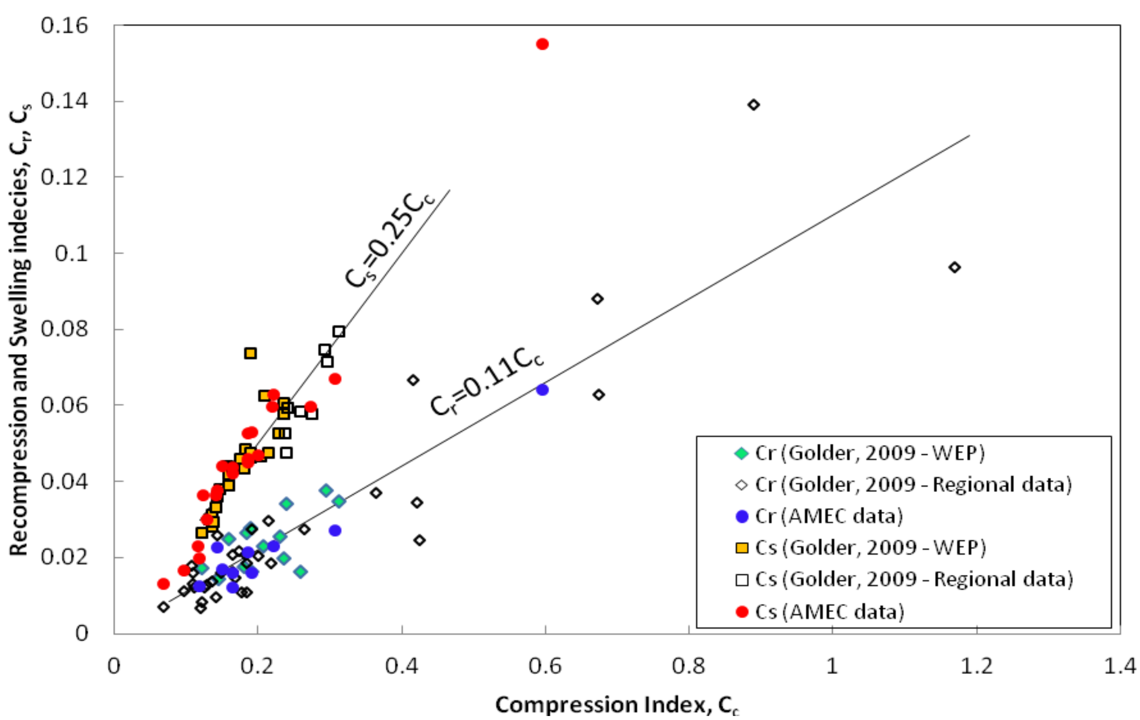


Figure 4-2: Cc versus Cα Relationship at WEP

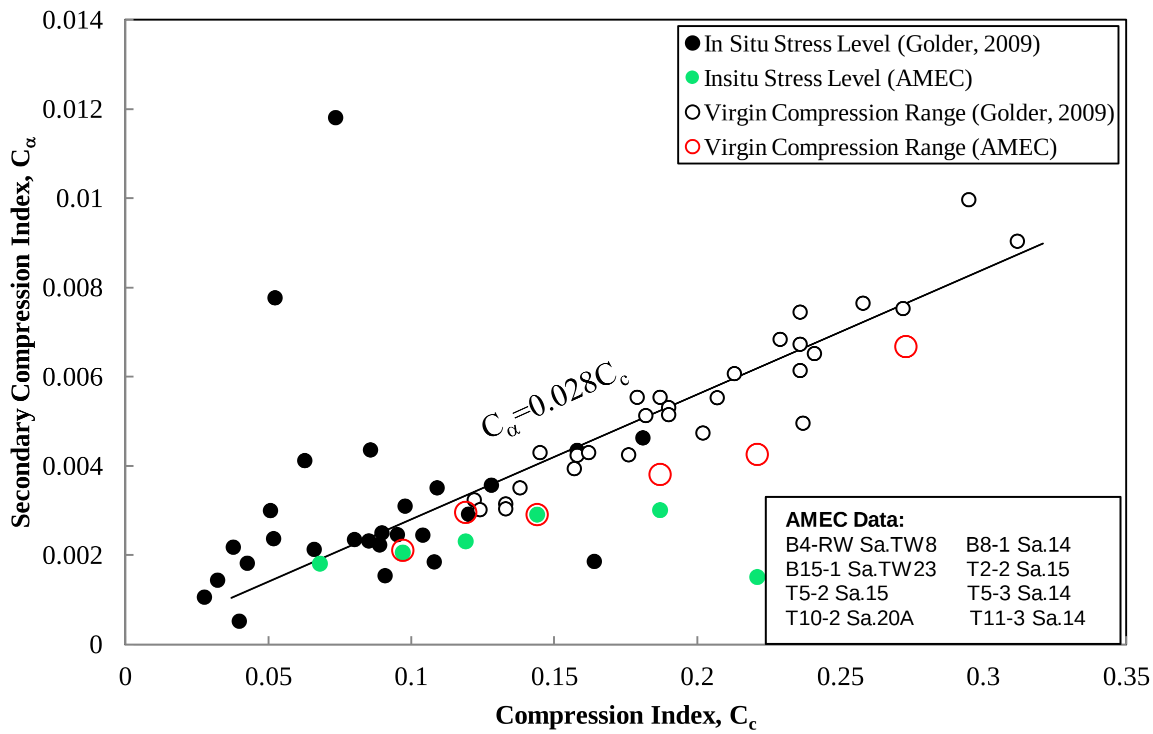
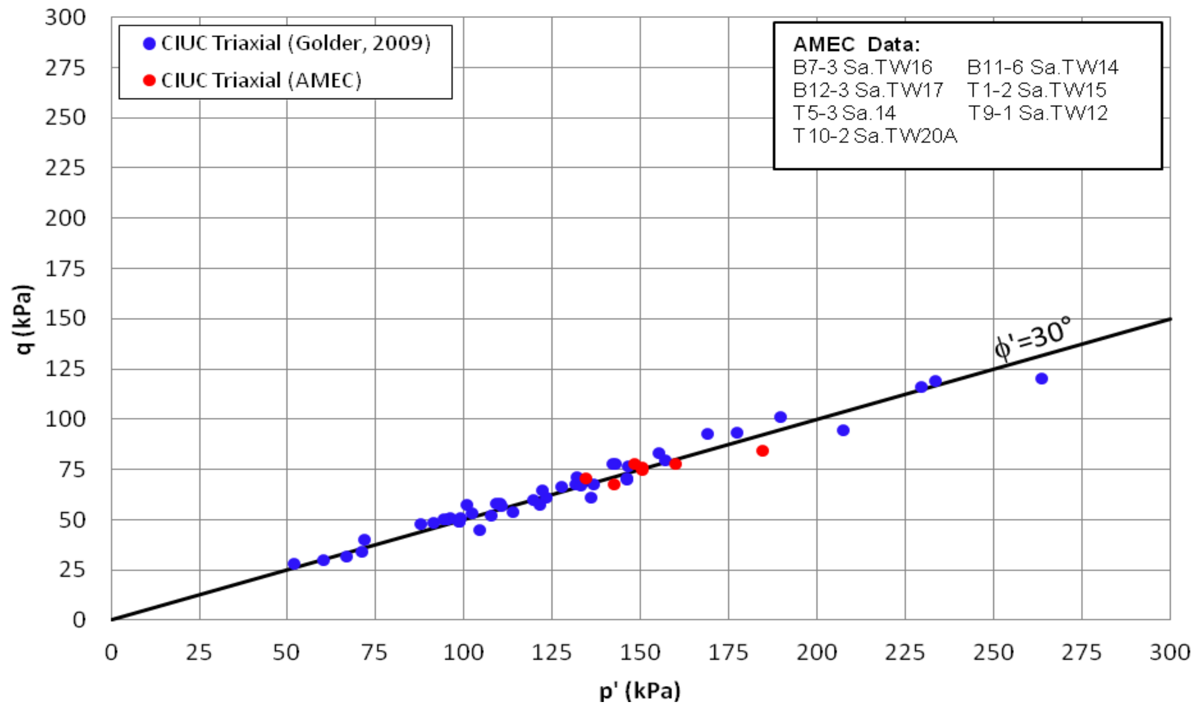


Figure 4-3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP



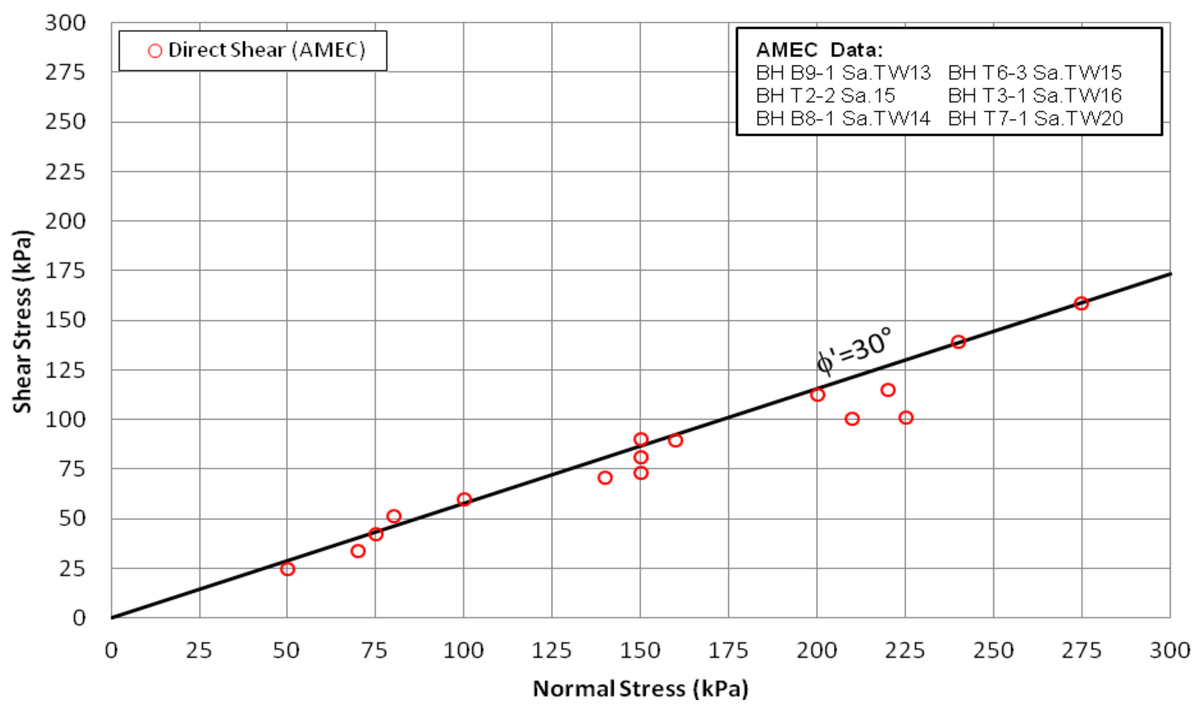
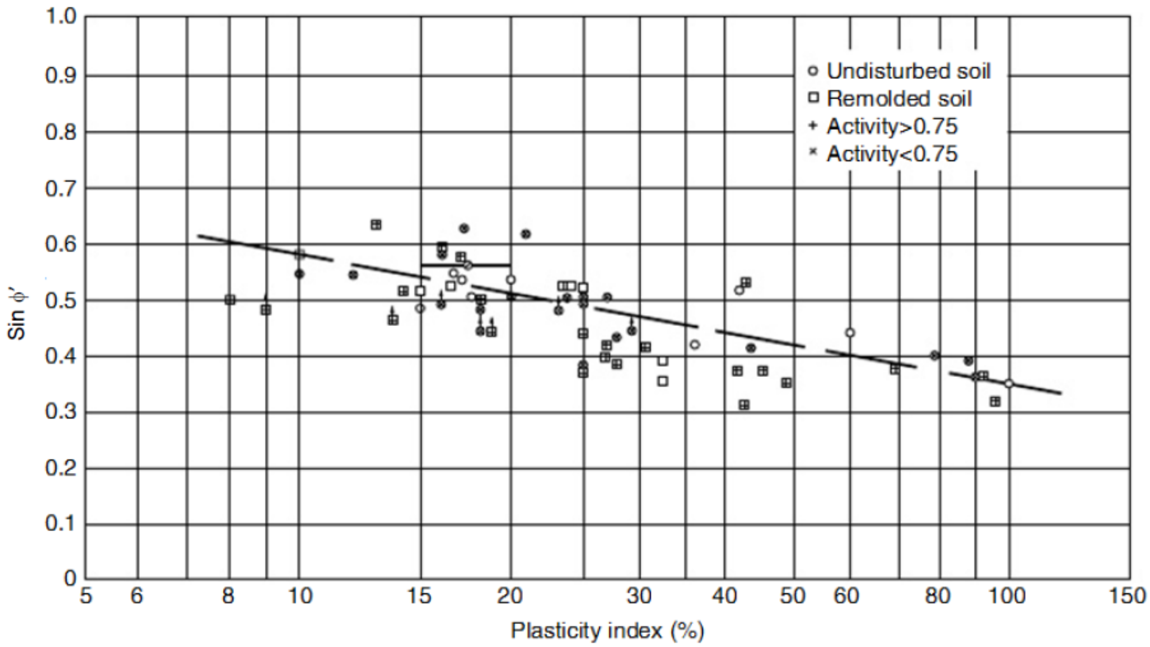


Figure 4-4: Relationship between sin ϕ′ and Plasticity Index for Normally Consolidated Soils

(Kenney, 1959)



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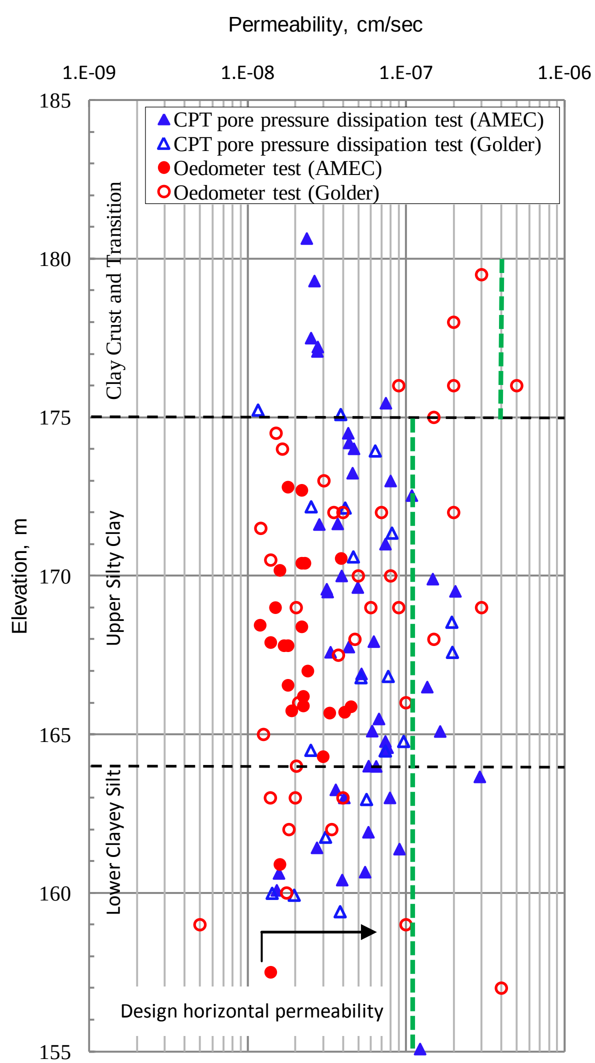
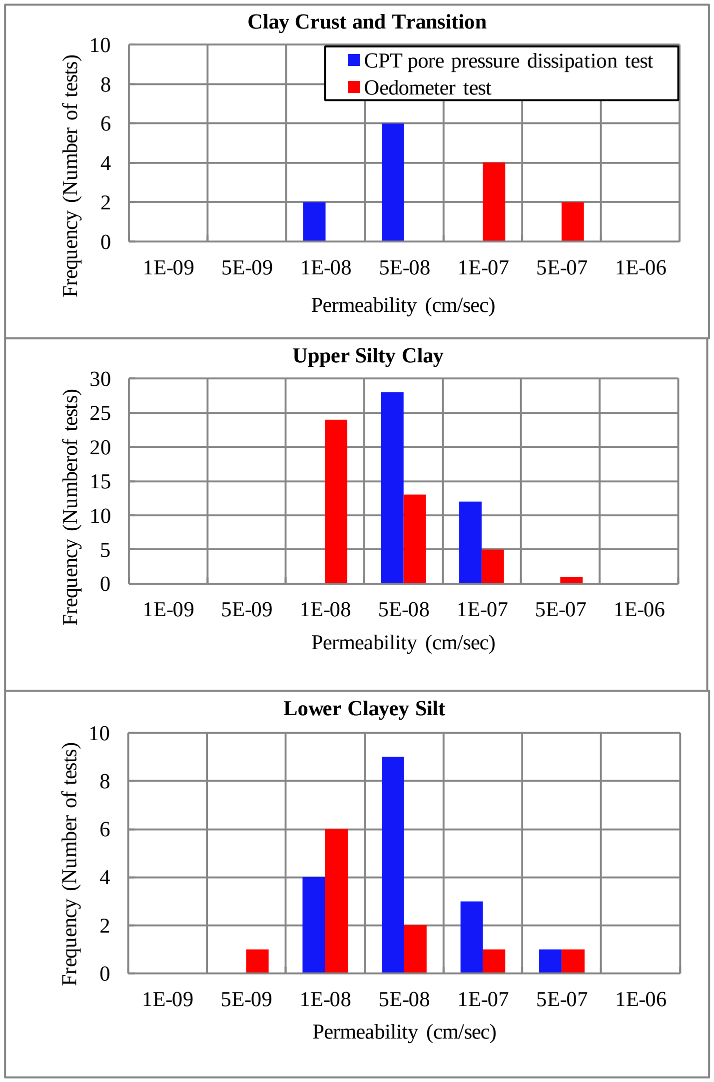
PARKWAY INFRASTRUCTURE CONSTRUCTORS

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





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

EXPLANATION OF BOREHOLE LOG

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

GENERAL INFORMATION

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

SOIL LITHOLOGY

Elevation and Depth

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

Lithology Plot

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

Description

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

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

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

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

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

Soil Sampling

Sample types are abbreviated as follows:

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

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

Field and Laboratory Testing

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

Instrumentation Installation

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

Comments

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

BEDROCK DESCRIPTION

STRENGTH CLASSIFICATION

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

JOINT SPACING CLASSIFICATION

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

ROCK QUALITY CLASSIFICATION

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

Reference: Deere et al, 1967

WEATHERING CLASSIFICATION

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

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

TERMINOLOGY

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

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

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

MTC SOIL CLASSIFICATION
Based on MTC Soil Classification Manual

MAJOR DIVISION				GROUP SYMBOL	TYPICAL DESCRIPTION	INFORMATION REQUIRED FOR DESCRIBING SOILS
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.
			PREDOMINANTLY ONE SIZE OF A RANGE OF SIZES WITH STONE INTERMEDIATE SIZES MISSING	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	
		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	
			PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	
	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
			PREDOMINANTLY ONE SIZE OR A RANGE OF SIZES WITH SOME INTERMEDIATE SIZE MISSING	SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	
		SANDS WITH FINES (APPLICABLE AMOUNT OF FINES)	NON PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES	
			PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES	

IDENTIFICATION PROCEDURE ON FRACTION SMALLER THAN 425µm

MAJOR DIVISION			DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	GROUP SYMBOL	TYPICAL DESCRIPTION	INFORMATION REQUIRED FOR DESCRIBING SOILS
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	SILT AND CLAYS	LIQUID LIMIT LESS THAN 35	NONE	QUICK	NONE	ML	INORGANIC SILTS & SANDY SILTS OR SLIGHTLY PLASTICITY, ROCK FLOUR	GIVE TYPE, NAME, IF NECESSARY, INDICATE DEGREE AND CHARACTER OF PLASTICITY, AMOUNT AND MAXIMUM SIZE OF COURSE GRAINS, COLOUR IN WET CONDITION, ODOUR, IF ANY, LOCAL OR GEOLOGIC NAME & OTHER PERTINENT DESCRICTIVE INFORMATION & SYMBOL IN PARENTHESIS.
			MEDIUM TO HIGH	NONE TO VERY SLOW	MEDIUM	CL	SILTY CLAYS (INORGANIC), GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS	
			SLIGHT TO MEDIUM	SLOW	SLIGHT	OL	ORGANIC SILT OF LOW PLASTICITY, ORGANIC SANDY SILTS	
		LIQUID LIMIT BETWEEN 35 AND 50	NONE TO SLIGHT	SLOW TO QUICK	SLIGHT	MI	INORGANIC COMPRESSIBLE FINE SANDY SILT WITH CLAY OF MEDIUM PLASTICIT, CLAYEY SILTS	
			HIGH	NONE	MEDIUM TO HIGH	CI	SILTY CLAYS (INORGANIC) OF MEDIUM PLASTICITY	
			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	

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

LABORATORY CLASSIFICATION CRITERIA

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.

$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 4;

$C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN 1 AND 3

NOT MEETING ALL GRADATION REQUIREMENTS FOR GW

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
ATTERBERG LIMITS ABOVE A- LINE WITH Ip GREATER THAN 7	

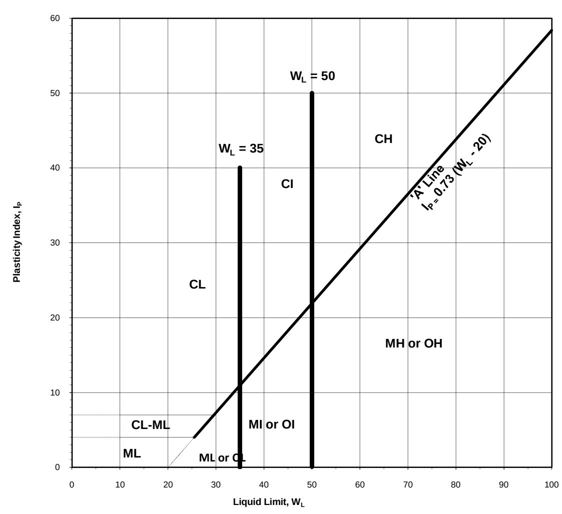
$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 6;

$C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN 1 AND 3

NOT MEETING ALL GRADATION FOR SW

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
ATTERBERG LIMITS ABOVE A- LINE WITH Ip GREATER THAN 7	

FRACTION		U.S STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		PASSING	RETAINED	PERCENT	DESCRIPTOR
GRAVEL	COARSE	75 mm	26.5 mm	40-50	AND
	FINE	26.5 mm	4.75 mm	30-40	Y/EY
SAND	COARSE	4.75 mm	2.00 mm	20-30	WITH
	MEDIUM	2.00 mm	425 µm	10-20	SOME
	FINE	425 µm	75 µm	1-10	TRACE
FINES (SILT OR CLAY BASED ON PLASTICITY)		75 µm			
OVERSIZED MATERIAL					
ROUNDED OR SUBROUNDED: COBBLES 75 mm TO 200 mm BOULDERS > 200 mm			NOT ROUNDED: ROCK FRAGMENTS > 75 mm ROCKS > 0.76 CUBIC METRE IN VOLUME		



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

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

AMEC Earth & Environmental,
a Division of AMEC American

www.amec.com

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 T10-1/HGMW-04

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4678495.6, E334122.3		ORIGINATED BY NB
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic		DATE Jul 13, 11 - Jul 15, 11	CHECKED BY MSO

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
184.9	Fill Surface
184.9 / 0.0	**FILL** 150mm Topsoil
184.7 / 0.2	**FILL** Silty Clay and Topsoil Brown-Grey
183.5 / 1.4	**CLAYEY SILT** Sandy to some sand, trace gravel, stiff; Brown; Grey

SAMPLES

NUMBER	TYPE	"N" VALUES	ELEVATION SCALE (approx.)	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	184.5		
2	SS	11	184		
3	SS	12	183		
4	SS	32	182.5		
5	SS	39	181.5		3 33 41 23
6	SS	23	181		
7	SS	18	180		4 32 42 22
8	SS	13	179.5		
9	TW	PH	178.5	22.0	3 32 43 22
	VT		178		
10	TW	PH	177		
11	TW	PH	175.5		
	VT		175		
12	TW	PH	174	21.5	3 31 44 22
13	TW	PH	172.5		
	VT		172		
14	TW	PH	171		

GROUND WATER CONDITIONS

ELEVATION SCALE: 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170

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 values: >> 1.6; 1.5; 1.5

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

WATER CONTENT (%) 10 20 30

REMARKS

-Vibrating Wire Piezometers (VWP) installed in sampled borehole observation well (OW) installed in adjacent boring at N4678497.2, E334122.3

-end of drilling July 13; continue July 14

-VWP #P10 installed at 9.1m below ground surface

Continued Next Page

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

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RECORD OF BOREHOLE No T10-1/HGMW-04

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4678495.6, E334122.3	ORIGINATED BY	NB
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	Jul 13, 11 - Jul 15, 11	CHECKED BY	MSO

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	FIELD VANE	UNIT WEIGHT γ (kN/m³)	GR	SA	SI	CL	REMARKS
	CLAYEY SILT Sandy to some sand, trace gravel, stiff *(continued)*	15	TW	PH							
			VT		1.8						
168.4 / 16.5	**CLAYEY SILT** Trace sand, trace gravel. Some silt and fine sand seams, trace pink clay nodules. Soft to firm. Grey	16	TW	PH			3	25	37	35	
		17	TW	PH							-start of wash boring
			VT		1.5						
165.4 / 19.5	**CLAYEY SILT** Some sand, trace gravel. Firm to stiff. Grey	18	TW	PH		21.9	4	27	50	18	
		19	SS	PH							-no recovery with shelby tube; sample retrieved by pushing split spoon
			VT		>>						-VWP #P22 installed at 21.6m below ground surface
162.3 / 22.6	**SILTY CLAY** And dense **SILT** seams. Firm to stiff. Grey	20	TW	PH							-TW was damaged at tip
		21	SS	59							
159.3 / 25.6	**SILTY SAND** Trace to some gravel, some clay. Compact. Grey	22	SS	23							
		23	SS	24			5	47	31	17	
		24	SS	22			17	42	27	14	

Elevation scale: 169, 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155

Continued Next Page

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

RECORD OF BOREHOLE No T10-1/HGMW-04

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4678495.6, E334122.3	ORIGINATED BY NB
DIST HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE Jul 13, 11 - Jul 15, 11	CHECKED BY MSO

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
	SILTY SAND Trace to some gravel, some clay Compact Grey *(continued)*	25	SS	79	-possibly pushing down cobbles
153.2 / 31.7	Very dense				
152.4 / 32.5	Weathered **LIMESTONE COBBLES** and **BOULDERS** (Inferred from sample fragments) Very dense	26	SS	90	
150.4 / 34.5	**LIMESTONE** Partially crystallized Fine Grained, calcite crystallization is visible Stylolites present. Fracture running parallel to the core is located between 32.49m and 32.34m Perpendicular fracture is present at location 34.56m. The Limestone is pitted between locations 32.49m and 34.32m, but the density pits and vuggs increased between 33.53m and 33.95m White to Grey	27	RC		-end of drilling July 14; continue July 15 RQD = 79% TCR = 100% SCR = 79%
		28	RC		RQD = 83% TCR = 100% SCR = 98%
150.1 / 34.8	**LIMESTONE** Fine to Medium Grained Porous, partially crystallized Limestone White to Grey				
	END OF BOREHOLE				

Water Levels in observation well:
July 29, 2011: EL. 183.7m

Piezometric Levels in VWP #P10 (EL. 175.8m):
July 29, 2011: EL. 183.4m

Piezometric Levels in VWP #P22 (EL. 163.3m):
July 29, 2011: EL. 182.4m

Elevation scale: 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143, 142, 141, 140

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

RECORD OF BOREHOLE No T10-2/HGMW-09

METRIC

W.P. RFP No. 09-54-1007	LOCATION 4678358.2N, 334191.8E		ORIGINATED BY TA
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic		DATE May 2, 11 - May 4, 11	CHECKED BY MSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.8	Ground Surface						
0.0	**TOPSOIL** Organic Clay Black						-shallow and mid-depth vibrating wire piezometers (VWP) installed in adjacent boring at (4678361.9N, 334192.4E) and observation well OW installed in adjacent boring at (4678365.8N, 334193.3E). -bedrock VWP installed in sampled borehole.
184.3 / 0.5	**CLAYEY SILT** Some sand, trace gravel Firm to Stiff Mottled Brown-Grey	1	SS	7	184		
		2	SS	8	183		
	Brown	3	SS	37	182		
		4	SS	36			
	Grey	5	SS	17	181		
		6	SS	12	180		
		7	SS	10	179		
		8	TW	PH	178	22.0	-VWP P7 installed at 6.55m below surface; 4 35 41 20
	Layers of clayey sand below approximately 7m	9	TW	PH	177		
		10	TW	PH	176 / 175	21.7	
			VT				
		11	TW	PH	174		-end of drilling May 2; continue May 3
		12	TW	PH	173 / 172	21.2	
			VT				
		13	TW	PH	171 / 170		

Field vane values: 1.64, 2.38

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

Continued Next Page

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

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RECORD OF BOREHOLE No T10-2/HGMW-09 METRIC

W.P. RFP No. 09-54-1007	LOCATION 4678358.2N, 334191.8E		ORIGINATED BY TA
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE May 2, 11 - May 4, 11		CHECKED BY MSO

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT Grey	14	TW	PH		20.9	
		15	SS	7	169		
168.2 / 16.6	**CLAYEY SILT** Stratified with thin layers of silt, Firm to stiff, Dark Grey	16	TW	PH	168		-end of drilling May 3; continue May 4 with wash bore with casing -VWP P21 installed at 18.71m below surface -samples 18A, 19A, 20A, 21A and vane at 22.25m completed in adjacent boring used for installation of VWP's
		17	SS	6	167		
		18A	TW	PH	166	19.6	
		19A	SS	14			
165.0 / 19.8	**CLAYEY SILT** Some sand, trace gravel, Very stiff, Grey	20A	TW	PH	165		1 19 46 34
		21A	TW	PH	164	21.8	Peak Vane >138 kPa
			VT		163		
		22	SS	20	162		
		23	SS	21	161 / 160		
	-Hairline sand-silt seams	24	SS	14	159		3 15 41 41
158.0 / 26.8	**FINE SAND** Trace to some silt, Dense, Grey, saturated	25	SS	49	158 / 157		
156.1 / 28.7	**CLAYEY FINE SAND** Some gravel, some silt, Stiff, Grey	26	SS	12	156 / 155		

Plot annotations: water content values 41.0, 44.3

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 20/12/11

RECORD OF BOREHOLE No T10-2/HGMW-09

METRIC

W.P. RFP No. 09-54-1007	LOCATION 4678358.2N, 334191.8E		ORIGINATED BY TA
DIST	HWY WEP	BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE May 2, 11 - May 4, 11		CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
154.6 / 30.2	**SANDY SILT** Some gravel, trace clay Very dense Grey	27	SS	73	154	
153.7 / 31.1	**SAND AND GRAVEL** Very dense Grey					-VWP P31 installed at 31.00 m below ground surface
153.3 / 31.5	**SANDY SILT** Some gravel Very dense Grey	28A, B	SS		153	
152.5 / 32.3	**LIMESTONE** Fine grained, cherty, bedded, highly fractured with numerous stylolites throughout, faintly to moderately porous. Pitted between 34.35m to 34.85m, light blue-grey inclusions. Light Grey	29	RC		152	-Rock Core Cu = 93.0 MPa RQD = 30% TCR = 100% SCR = 68%
		30	RC		151	RQD = 22% TCR = 100% SCR = 65%
		31	RC			RQD = 0% TCR = 39% SCR = 0%
		32	RC		150	RQD = 38% TCR = 96% SCR = 71%
149.0 / 35.8	**END OF BOREHOLE**				149	

Column headings: STRAT PLOT; 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/m³; GR SA SI CL

Elevation scale: 154, 153, 152, 151, 150, 149, 148, 147, 146, 145, 144, 143, 142, 141, 140

Water levels in observation well:
May 24, 2011: EL. 184.1m
June 4, 2011: EL. 183.9m
June 25, 2011: EL. 183.3m
July 23, 2011: EL. 182.6m

Piezometric levels in VWP #P7 (EL. 178.3m):
May 24, 2011: EL. 183.5m
June 4, 2011: EL. 183.8m
June 25, 2011: EL. 183.1m
July 23, 2011: EL. 182.4m

Piezometric levels in VWP #P21 (EL. 166.2m):
May 24, 2011: EL. 182.4m
June 4, 2011: EL. 182.7m
June 25, 2011: EL. 182.0m
July 23, 2011: EL. 181.3m

Piezometric levels in VWP #P31 (EL. 153.8m):
May 24, 2011: EL. 178.5m
June 4, 2011: EL. 178.5m
June 25, 2011: EL. 177.7m
July 23 ,2011: EL. 177.0m

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

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RECORD OF BOREHOLE No TB7A-1

METRIC

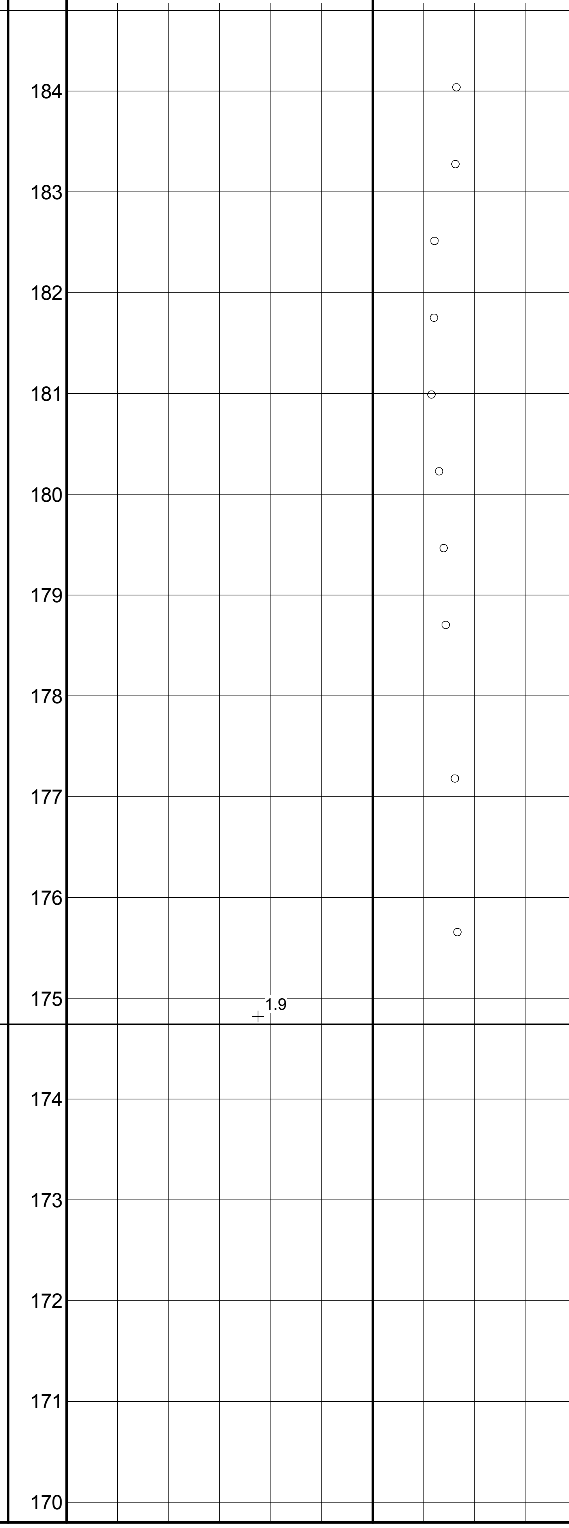
W.P. RFP No. 09-54-1007 | LOCATION N4678506.6, E334190.2 | ORIGINATED BY TA

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

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
184.8	Ground Surface				
0.0	**TOPSOIL**				
184.2 / 0.6	Mottled Brown-Grey **CLAYEY SILT** Some sand, trace gravel Stiff	1	SS	8	
	Brown - Trace fissures	2	SS	14	
	Very stiff	3	SS	26	
	Hard	4	SS	35	
	Grey, very stiff Stiff	5	SS	18	
		6	SS	12	
		7	SS	9	
		8	SS	7	
			VT		-attempt at vane shear test exceeded max of apparatus (approx. 115. kPa)
	Firm to stiff	9	SS	6	
		10	SS	3	
			VT		
174.7 / 10.1	**END OF BOREHOLE** (no refusal) Borehole dry on completion				

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



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

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RECORD OF BOREHOLE No CPT T10-1

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4678450.6, E334217.4	ORIGINATED BY	TA
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	Aug 9, 11 - Aug 9, 11	CHECKED BY	MSO

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	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
184.9	Ground Surface										
0.0	**TOPSOIL**										
184.4 / 0.5	**CLAYEY SILT** Some sand, trace gravel Stiff Mottled brown-Grey		1	SS	10		184				
			2	SS	10		183				
182.9 / 2.0	**END OF SAMPLED BOREHOLE** (Continued with CPT to refusal)						182				
							181				
							180				
							179				
							178				
							177				
							176				
							175				
							174				
							173				
							172				
							171				
							170				

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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RECORD OF BOREHOLE No CPT T10-2

METRIC

W.P. RFP No. 09-54-1007 LOCATION N46783403.2, E334089.2 ORIGINATED BY TA

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

DATUM Geodetic DATE May 2, 11 - May 2, 11 CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
185.2	Fill Surface			
0.0	**FILL** 75mm rounded gravel over 400mm silty clay			
184.7 / 0.5	**SAND** Poorly Graded Black			
184.4 / 0.8	**SANDY SILT** Some clay, trace gravel Stiff to hard Mottled brown-grey Dry to wet -Disturbed soil to approx. 1.2m Brown	1	SS	9
		2	SS	12
		3	SS	37
		4	SS	39
	Grey	5	SS	21
180.2		6	SS	17
5.0	**END OF SAMPLED BOREHOLE** (continued with CPT to refusal)			

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (185 to 171); 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

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RECORD OF BOREHOLE No DMT T10-1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678412.4, E334151.5 ORIGINATED BY LC

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

DATUM Geodetic DATE Jul 21, 11 - Jul 21, 11 CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.6	Ground Surface						
0.0 / 184.3 / 0.3	Clayey **TOPSOIL** With roots To weathered Brown-Grey Silty Clay Some sand, trace gravel					184	
	SILTY CLAY Some sand, trace gravel Dry -Brown fissures Stiff to hard Mottled Brown-Grey	1	AS			183	
		2	SS	53		182	
		3	SS	37		181	
		4	SS	23		180	
179.6 / 5.0		5	SS	13			
	END OF SAMPLED BOREHOLE (continued with DMT to refusal)					179	
						178	
						177	
						176	
						175	
						174	
						173	
						172	
						171	
						170	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

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

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RECORD OF BOREHOLE No CPT47-RW

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4678440.3, E334300.2 ORIGINATED BY TA

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

DATUM Geodetic DATE Aug 10, 11 - Aug 10, 11 CHECKED BY MSO

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.4	Ground Surface							
0.0 / 184.9	**TOPSOIL**						185	
0.5	**CLAYEY SILT** Some sand, trace gravel Firm to stiff Mottled Brown-Grey Dry		1	SS	6			
	Brown -Trace fissures		2	SS	15		184	
183.4 / 2.0	**END OF SAMPLED BOREHOLE** (continued with CPT to refusal)						183	
							182	
							181	
							180	
							179	
							178	
							177	
							176	
							175	
							174	
							173	
							172	
							171	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m^3

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

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RECORD OF CONE PENETRATION TEST CPT T10-1

METRIC

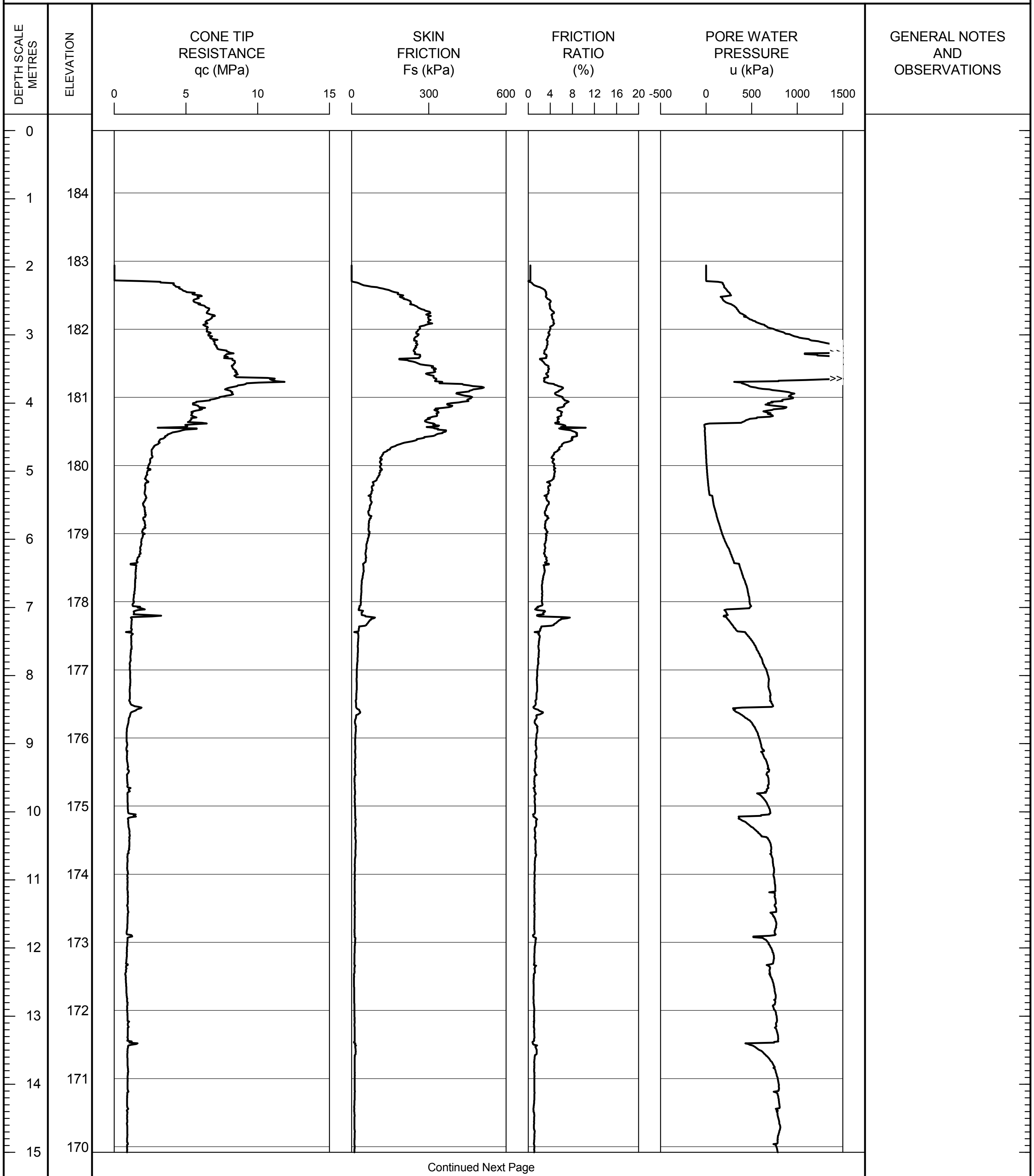
PROJECT Windsor-Essex Parkway

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

LOCATION N4678450.6; E334217.4

DATUM Geodetic

GROUND SURFACE ELEVATION: 184.9 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

CHECKED: DD

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

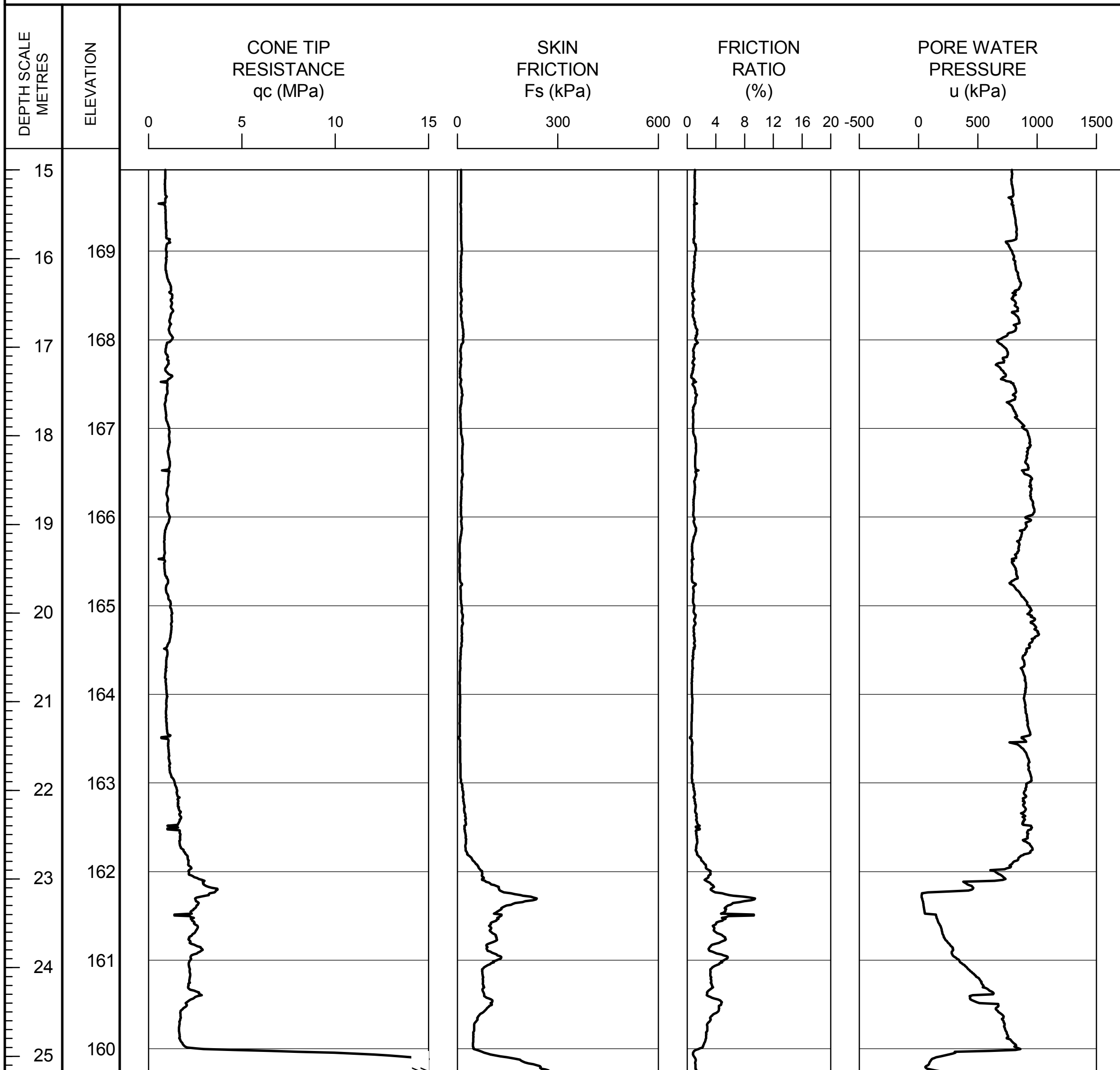
RECORD OF CONE PENETRATION TEST CPT T10-1

METRIC

PROJECT Windsor-Essex Parkway
TEST DATE 8/9/2011 - 8/9/2011
SHEET 2 OF 2

LOCATION N4678450.6; E334217.4
DATUM Geodetic

GROUND SURFACE ELEVATION: 184.9 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

OPERATOR: TA

CHECKED: DD

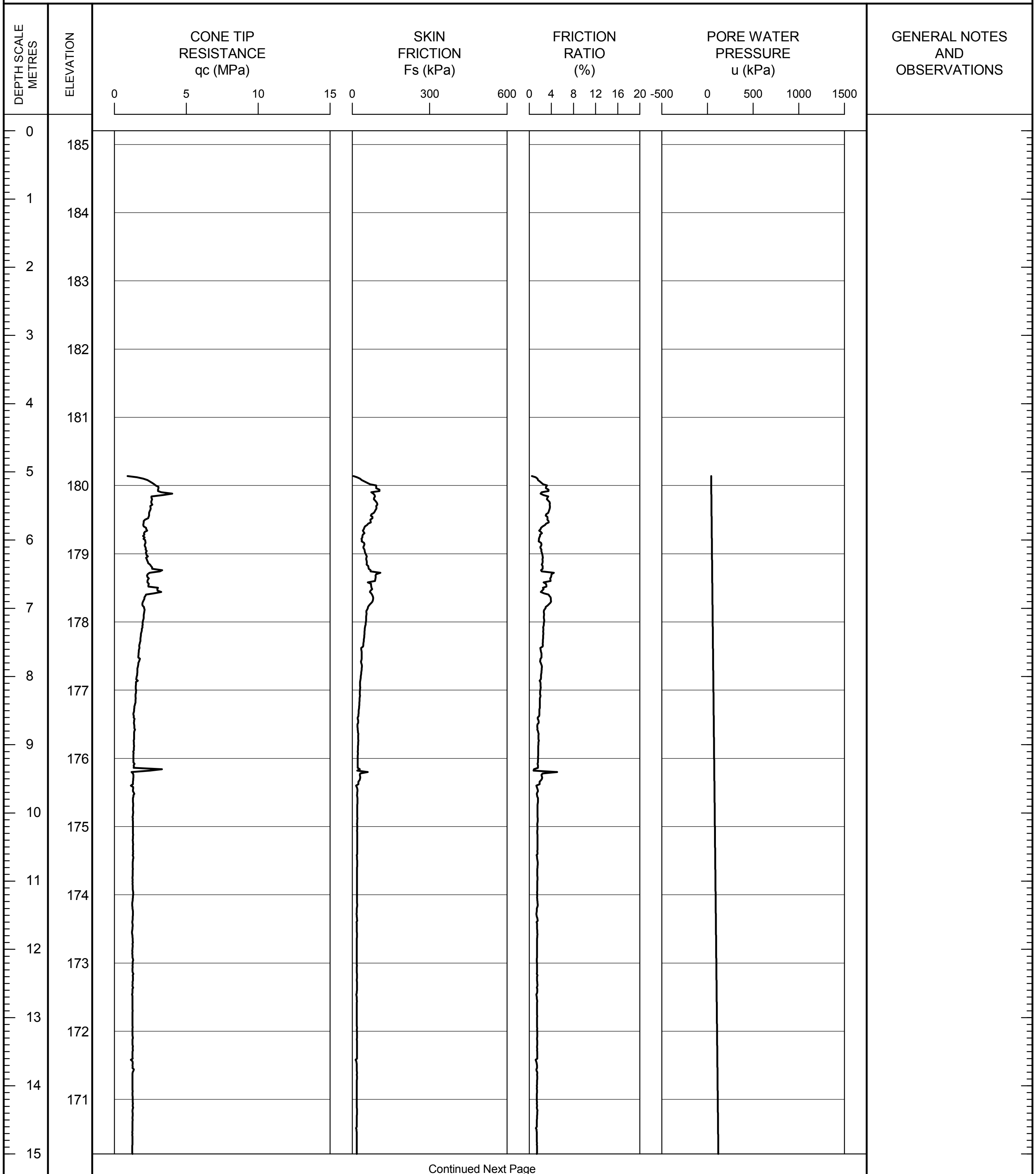
Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF CONE PENETRATION TEST CPT T10-2

METRIC

PROJECT Windsor-Essex Parkway
TEST DATE 02/05/2011 - 02/05/2011
SHEET 1 OF 2
LOCATION 4678403.2N; 334089.2E
DATUM Geodetic

GROUND SURFACE ELEVATION: 185.2 PREDRILL DEPTH: 5 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA
CHECKED: MSO

WEP CPT LOG CPT T10-2.GPJ ONTARIO MOT.GDT 02/12/11

RECORD OF CONE PENETRATION TEST CPT T10-2

METRIC

PROJECT Windsor-Essex Parkway

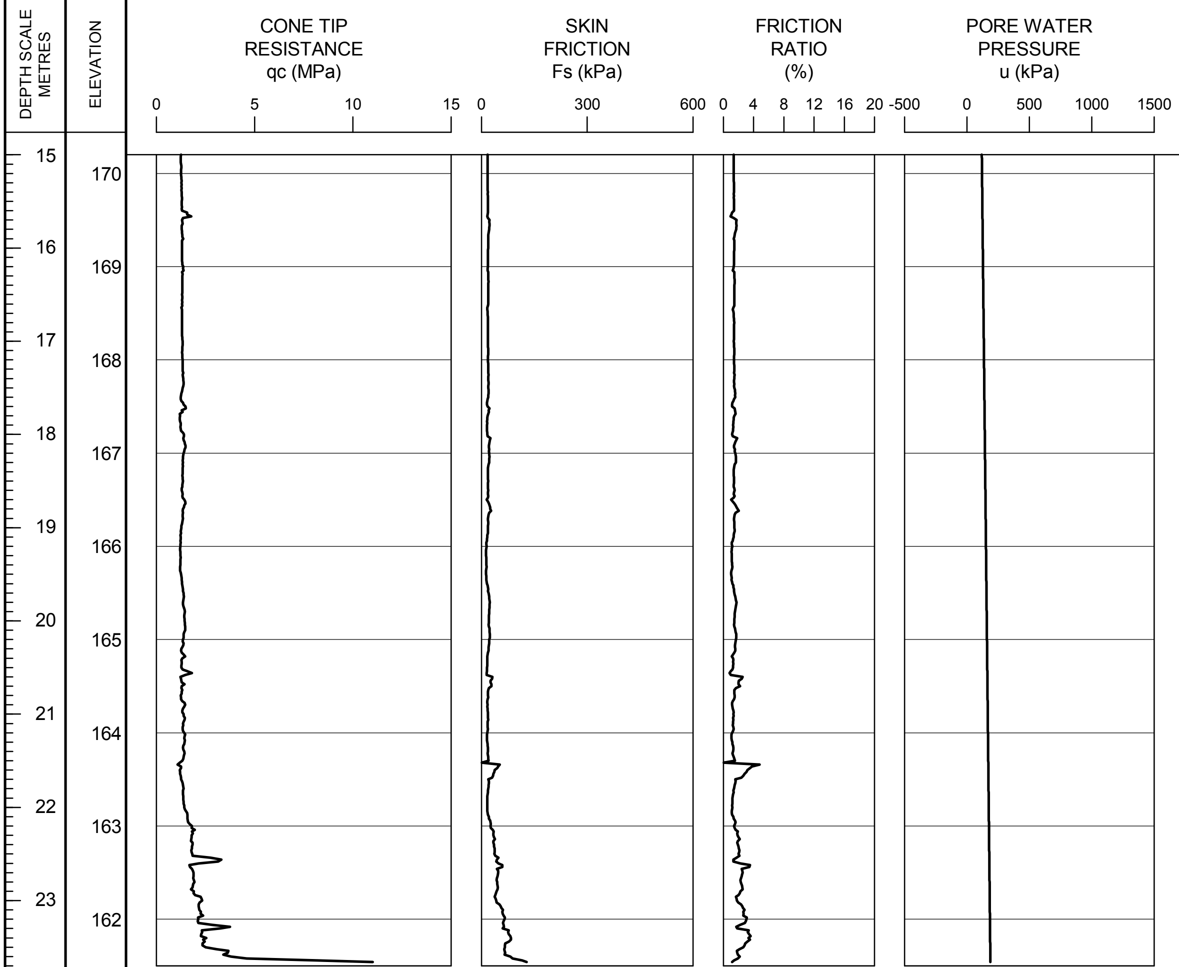
TEST DATE 02/05/2011 - 02/05/2011

SHEET 2 OF 2

LOCATION 4678403.2N; 334089.2E

DATUM Geodetic

GROUND SURFACE ELEVATION: 185.2 PREDRILL DEPTH: 5 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

OPERATOR: TA

CHECKED: MSO

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

RECORD OF CONE PENETRATION TEST CPT 47-RW

METRIC

PROJECT Windsor-Essex Parkway

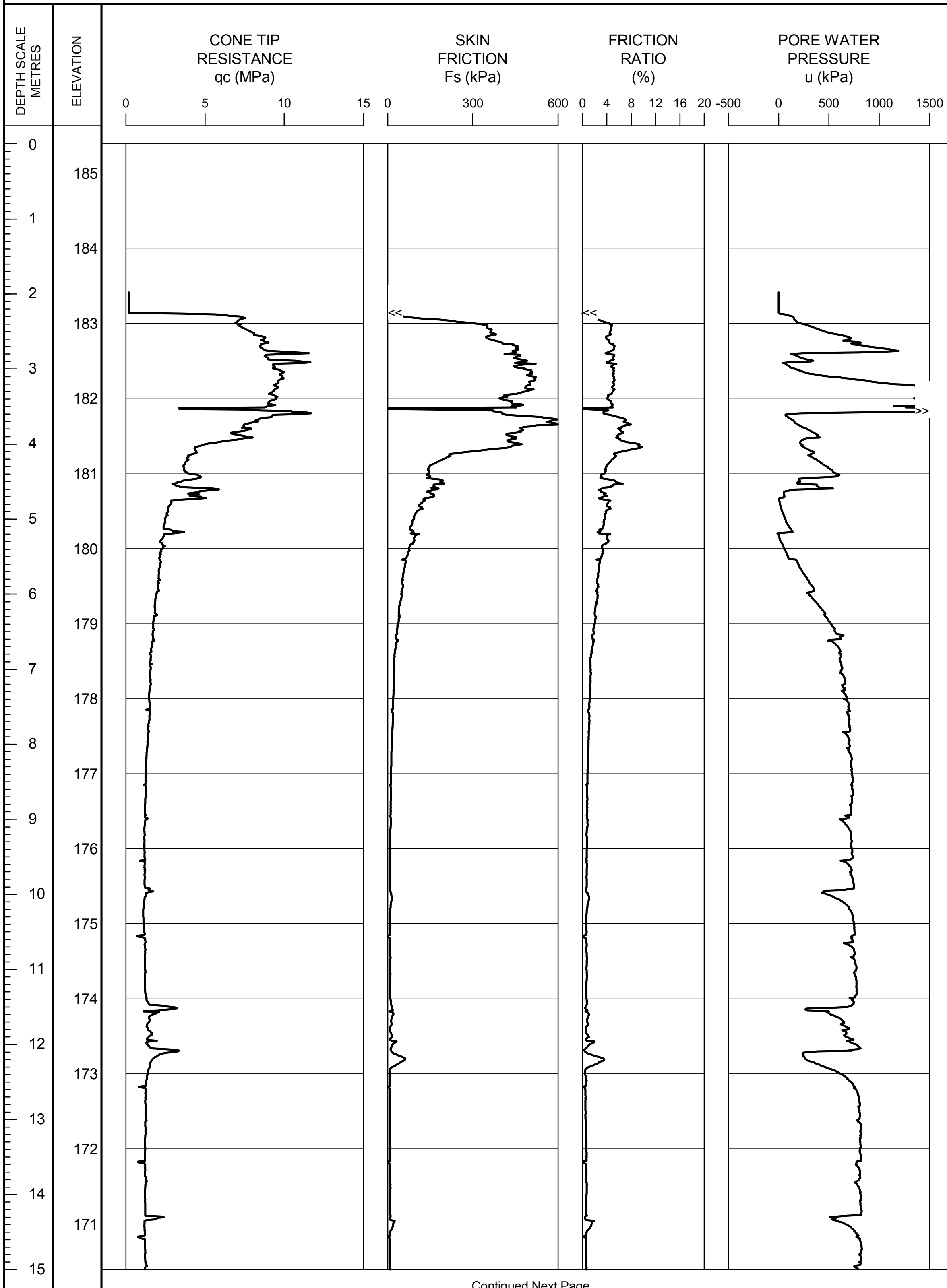
TEST DATE 8/10/2011 - 8/10/2011

SHEET 1 OF 2

LOCATION N4678440.3; E334300.2

DATUM Geodetic

GROUND SURFACE ELEVATION: 185.4 PREDRILL DEPTH: 1.98 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 47-RW

METRIC

PROJECT Windsor-Essex Parkway

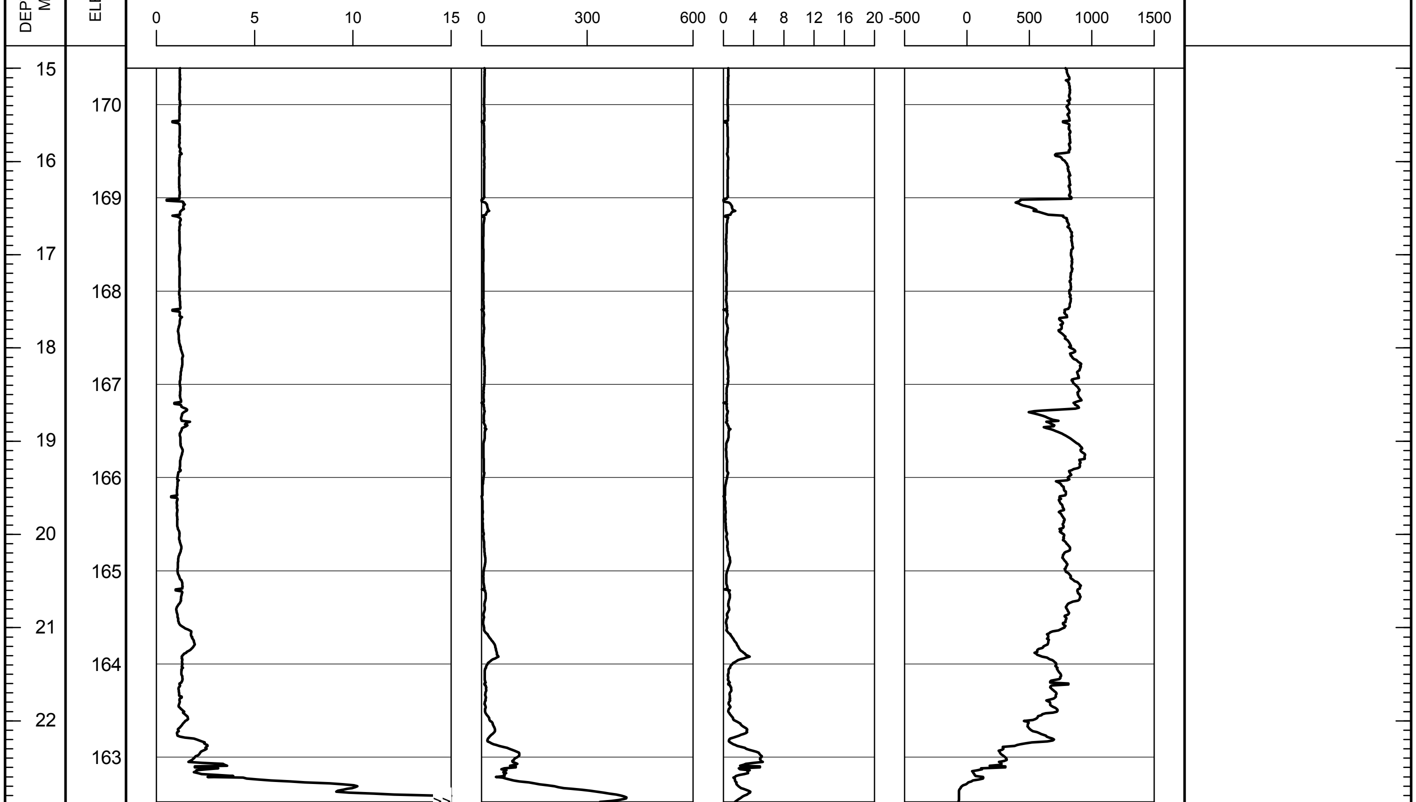
TEST DATE 8/10/2011 - 8/10/2011

LOCATION N4678440.3; E334300.2

DATUM Geodetic

GROUND SURFACE ELEVATION: 185.4 PREDRILL DEPTH: 1.98 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



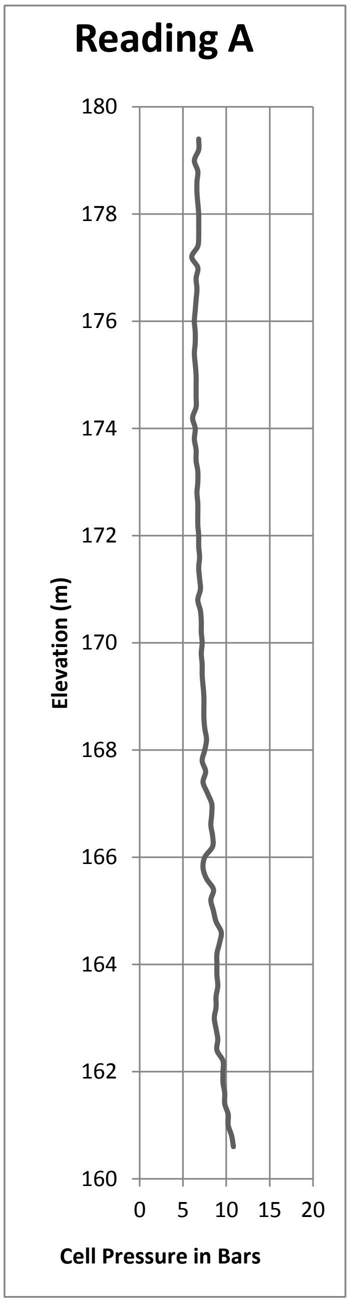
OPERATOR: TA

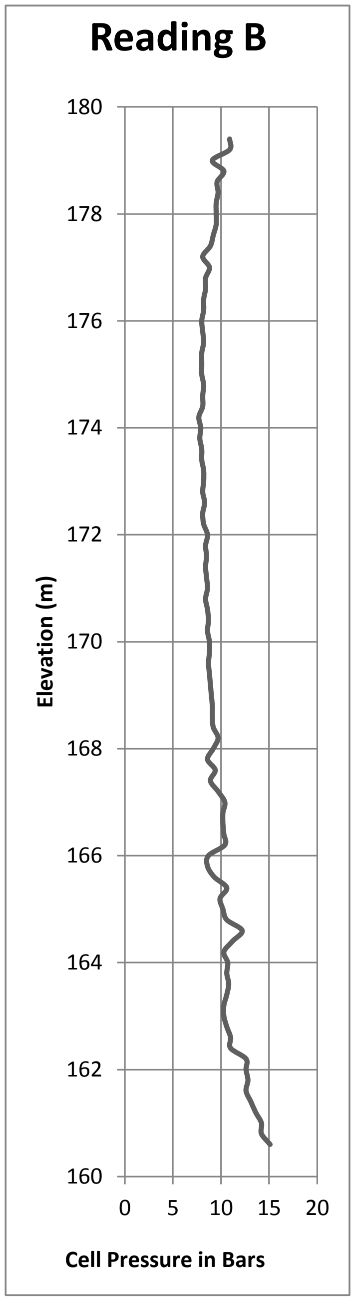
CHECKED: DD

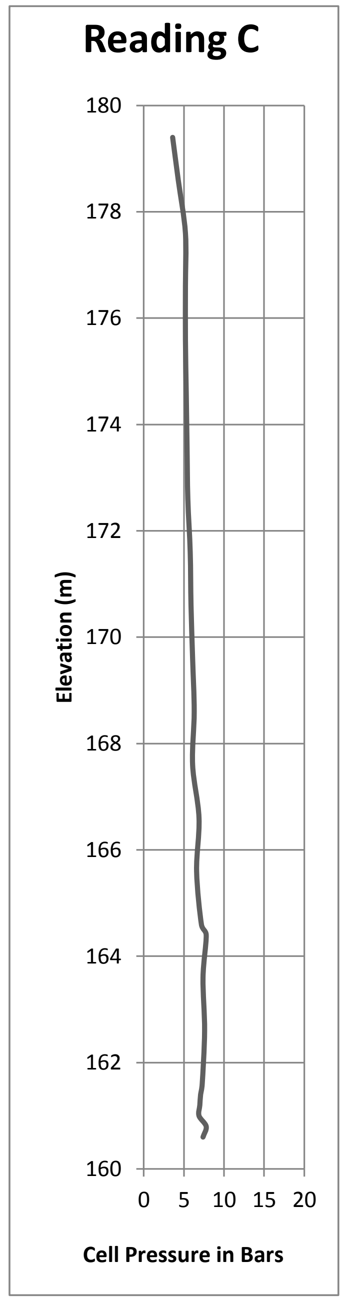
Foundation Design

RECORD OF DILATOMETER TEST DMT T10-1

Project : Windsor-Essex Parkway	Test Date: 7/21/2011	Sheet 1 of 1
Location: N 4678412.4; E 334151.5	Predrill Depth : 5.0 m	Datum Geodetic
Ground Surface Elevation : 184.6	Delta A: 0.10 Bar	Delta B: 0.33 Bar







Operator: LC

Checked: DD

Appendix B Borehole Logs from Previous Investigations

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 112

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION N 4678413.3 ;E 334221.3

ORIGINATED BY SM

DIST WEST HWY 401/3

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

COMPILED BY BRS

DATUM GEODETIC

DATE January 29, 2008 - February 12, 2008

CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION
184.58	GROUND SURFACE
0.00	TOPSOIL, silty, trace sand, trace organics Brown
184.18 0.40	CLAYEY SILT, trace sand, trace gravel Firm to very stiff Mottled brown and grey becoming grey at about elev. 181.2m

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS
1	SS	7	
2	SS	17	
3	SS	22	
4	SS	26	
5	SS	19	
6	SS	12	
7	TO	PH	CIUC
8	SS	8	
9	TO	PH	
10	TO	PH	CIUE
11	TO	PH	
12	TO	PH	
13	TO	PH	

GROUND WATER CONDITIONS: Concrete; Bentonite; Grout

ELEVATION SCALE: 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane values (sensitivity): 129.3 (1.6); 1.4; 132.1 (1.3); 1.3; 122.1 (1.3); 1.7; 1.2; 1.4; 1.2; 1.4; 1.3

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT w — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

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

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 112

METRIC

PROJECT 07-1130-207-0

W.P.

DIST WEST HWY 401/3

DATUM GEODETIC

LOCATION N 4678413.3 ;E 334221.3

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

DATE January 29, 2008 - February 12, 2008

ORIGINATED BY SM

COMPILED BY BRS

CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace sand, trace gravel Firm to very stiff Mottled brown and grey becoming grey at about elev. 181.2m	14	TO	PH	
		15	TO	PH	
		16	TO	PH	
		17	TO	PH	
		18	TO	PH	CIUC
161.64 22.94	SILTY CLAY, trace sand, trace gravel Firm Grey	19	TO	PH	CIUC
		20	TO	PH	
		21	SS	6	
158.52 26.06	SILT, trace clay Grey				
158.21 26.37	CLAYEY SILT, trace sand, trace gravel Grey	22	TO	PH	
157.76 26.82	SANDY SILT, trace gravel Grey				
157.00 27.58	CLAYEY SILT, trace sand, trace gravel Very stiff Grey				
156.36 28.22	SILT, some sand Compact Grey	23	SS	29	(85)
155.62 28.96	CLAYEY SILT, some sand, trace gravel Very stiff Grey				

Elevation scale: 169, 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155

Groundwater conditions: Grout

Field vane shear strength sensitivity values: 1.6, 1.3, 2.2, 2.0, 1.3, 1.2, 1.4, 1.5

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates
London, Ontario

PROJECT 07-1130-207-0

RECORD OF BOREHOLE No 112

METRIC

W.P.

LOCATION N 4678413.3 ;E 334221.3

ORIGINATED BY SM

DIST WEST HWY 401/3

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

COMPILED BY BRS

DATUM GEODETIC

DATE January 29, 2008 - February 12, 2008

CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, some sand, trace gravel, Very stiff, Grey	24	SS	28							(54)
153.80 30.78	SILTY SAND AND GRAVEL, with cobbles and boulders, Very dense, Grey					154					
		25	SS	101/ 101mm		153					
		26	TO	PH	Grout						
152.12 32.46	LIMESTONE, fresh, medium strong, thinly laminated to laminated, very fine to medium grained, faintly to moderately porous, Light greyish brown	27	SS	101/ 25mm		152					
	(FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	28	NQ RC		Bentonite	151	100	67	20		UC
		29	NQ RC		Sand	150 / 149	100	72	58		
		30	NQ RC		Screen	148	100	100	100		
146.39 38.19		31	NQ RC			147	100	100	100		

146.39
38.19 END OF BOREHOLE

Water level in borehole at about elev. 158.52m during drilling on February 5, 2008.

Water level measured in deep piezometer at elev. 178.28m on February 12, 2008.

Water level measured in deep piezometer at elev. 178.38m on March 20, 2008.

Water level measured in deep piezometer at elev. 177.93m on July 24, 2008.

Water level measured in deep piezometer at elev. 176.25m on September 19, 2008.

Water level measured in deep piezometer at elev. 177.54m on November 14, 2008.

Water level measured in deep piezometer at elev. 177.72m on January 28, 2009.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; 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

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

LOCATION: N 4678413.3 ;E 334221.3

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: January 29, 2008 - February 12, 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.

DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
	ROCK SURFACE	152.12 / 32.46			
MUD ROTARY, NQ ROCK CORE	LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous, light grey	150.75 / 33.83	1		
	LIMESTONE, fresh, medium strong, laminated, fine grained, vuggy to faintly porous with depth, light greyish brown, fossils present			Zone of broken material	
		149.28 / 35.30	2		
	LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous, light grey			JN,PL,Ro Cl	
		148.34 / 36.24			
	LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous, light greyish brown		3	JN,CU,Ro Cl	
		147.18 / 37.40			
	LIMESTONE, fresh, medium strong, laminated, medium grained, moderately porous, light greyish brown	146.66 / 37.92	4		
	LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, very light greyish brown	146.39 / 38.19			
	END OF DRILLHOLE				

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: SG

CHECKED: SJB

LDN_ROCK_03 07-1130-207-0-ROCK.GPJ GLDR_LDN.GDT 6/29/09 DATA INPUT: WDF

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 112A

METRIC

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

LOCATION N 4678413.3 ;E 334221.3
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
DATE February 12, 2008

ORIGINATED BY MA
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE – DESCRIPTION (SOIL CONDITIONS INFERRED FROM BOREHOLE No. 112)	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.58	GROUND SURFACE										
0.00	TOPSOIL, silty, trace sand, trace organics										
184.18 0.40	Brown CLAYEY SILT, trace sand, trace gravel Firm to very stiff Mottled brown and grey becoming grey at about elev. 181.2m					Backfill	184 183 182 181				
						Bentonite	180 179 178 177				
						Sand Piezometer	176				
175.44 9.14	END OF BOREHOLE Water level measured in shallow piezometer at elev. 181.94m on March 20, 2008. Water level measured in shallow piezometer at elev. 182.55m on July 24, 2008. Water level measured in shallow piezometer at elev. 182.50m on September 19, 2008. Water level measured in sshallow piezometer at elev. 182.39m on January 28, 2009.										

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 113

METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4678454.5 ;E 334070.3		ORIGINATED BY DJM/MA
DIST WEST	HWY 401/3	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC	COMPILED BY BRS
DATUM GEODETIC		DATE February 22, 2008 - February 28, 2008	CHECKED BY SJB

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
184.41	GROUND SURFACE
0.00 / 184.01	TOPSOIL, clayey Black
0.40	CLAYEY SILT, some sand, trace gravel with fine sand pockets Firm to stiff Brown
182.43 / 1.98	CLAYEY SILT, some sand, trace gravel Firm to very stiff Brown becoming grey at about elev. 180.8m
171.15 / 13.26	CLAYEY SILT, trace sand, trace gravel Soft to very stiff Grey

SAMPLES

NUMBER	TYPE	"N" VALUES	Elevation (approx.)	Water content / remarks	GR SA SI CL
1	SS	7	183.6		
2	SS	11	182.8	w with plastic and liquid limits	1 31 39 29
3	SS	16	182.1		
4	SS	18	181.3		
5	SS	10	180.5	w with plastic and liquid limits	1 31 42 26
6	SS	6	179.8		
7	SS	6	178.2		
8	TO	PH	176.7		
9	TO	PH	175.2		
10	TO	PH	173.7		
11	SS	9	172.1		
12	TO	PH	170.6		

Ground water conditions: Concrete (top), Bentonite, Grout; groundwater level at approx. elev. 177.7.

Field vane shear strength (kPa) / sensitivity: (>143.6) at elev. 179; 1.6 at approx. 177.6; 1.5 at approx. 176; 1.3 at approx. 174.5; 124.9, 1.9 at approx. 173; 2.1 at approx. 169.9.

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

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 113

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION N 4678454.5 ;E 334070.3

ORIGINATED BY DJM/MA

DIST WEST HWY 401/3

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

COMPILED BY BRS

DATUM GEODETIC

DATE February 22, 2008 - February 28, 2008

CHECKED BY SB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	FIELD VANE (Sensitivity)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace sand, trace gravel; Soft to very stiff; Grey	13	SS	4	169		o	
						+ 1.8		
167.95 / 16.46	SILTY CLAY, trace sand; Soft to stiff; Grey	14	SS	4	168		o	
					167	+ 2.7		
		15	SS	5	166		o	
						+ 2.6		
165.13 / 19.28	CLAYEY SILT, trace sand, trace gravel; Stiff to very stiff; Grey	16	TO	PH	165			
					164	+ 1.9		
		17	SS	9	163		o	
					162	(>143.6)		
		18	SS	19	161		o	
		19	SS	17	160		o	
		20	SS	16	159 / 158		wp–w–wl	1 15 44 40
		21	SS	14	157		o	
155.76 / 28.65	SILTY SAND, trace gravel, trace clay; Compact; Grey				156			
155.21 / 29.20	CLAYEY SILT, with silt lenses; Stiff; Grey	22	SS	12	155		o o	
154.54								

Ground water conditions: Grout; Sand

Continued Next Page

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

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London, Ontario

RECORD OF BOREHOLE No 113

METRIC

PROJECT 07-1130-207-0
W.P.
LOCATION N 4678454.5 ;E 334070.3
ORIGINATED BY DJM/MA
DIST WEST HWY 401/3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY BRS
DATUM GEODETIC
DATE February 22, 2008 - February 28, 2008
CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
29.87	SAND AND GRAVEL, trace silt Compact Grey	23	SS	25	Screen	154				
153.01 31.40	LIMESTONE, fresh, medium strong, thinly laminated to laminated, very fine to fine grained, faintly porous to porous Light grey to brown (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	24	NQ RC			153	33	0	0
		25	NQ RC		Bentonite		27	10	0	
		26	NQ RC			152	73	38	12	
		27	NQ RC		Sand	151	0	0	0	
		28	NQ RC			150 149	94	92	78	UC
148.36 36.05	END OF BOREHOLE Water level in borehole at about elev. 154.54m during drilling between February 22 and 28, 2008 Water level measured in deep piezometer at elev. 178.13m on February 28, 2008. Water level measured in deep piezometer at elev. 182.91m on March 20, 2008. Water level measured in deep piezometer at elev. 177.75m on July 22, 2008. Water level measured in deep piezometer at elev. 175.87m on September 19, 2008. Water level measured in deep piezometer at elev. 177.18m on November 11, 2008. Water level measured in deep piezometer at elev. 177.44m on January 28, 2009.									

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

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

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 113

LOCATION: N 4678454.5 E 334070.3

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: February 22, 2008 - February 28, 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
		ROCK SURFACE	153.01		153	
	MUD ROTARY / NO ROCK CORE	LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous, light grey	31.40	1		Broken Core
32		No recovery from 32.00m to 32.31m		2		
				3	152	Broken Core
33		LIMESTONE, fresh, medium strong, thinly laminated, fine grained to very fine grained, porous, light grey	151.40 / 33.01			JN,UN,Ro Cl
					151	Broken Core
						Broken Core
34		LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, grey - brown	150.70 / 33.71	4	150	
						BD,UN,SM Cl
35		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, porous, light grey	149.60 / 34.81			JN,UN,SM Cl
		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous, light grey to brown	149.18 / 35.23	5	149	JN,PL,Ro Sl
36		END OF DRILLHOLE	148.35 / 36.06			

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: SG

CHECKED: SJB

LDN_ROCK_03 07-1130-207-0-ROCK.GPJ GLDR_LDN.GDT 6/29/09 DATA INPUT: WDF

Golder Associates
London, Ontario

PROJECT 07-1130-207-0	RECORD OF BOREHOLE No 113A	1 OF 1	METRIC
W.P.	LOCATION N 4678454.5 ;E 334070.3		ORIGINATED BY DJM
DIST WEST HWY 401/3	BOREHOLE TYPE POWER AUGER, HOLLOW STEM		COMPILED BY BRS
DATUM GEODETIC	DATE February 22, 2008		CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION (SOIL CONDITIONS INFERRED FROM BOREHOLE No. 113)	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.41	GROUND SURFACE						
0.00	TOPSOIL, clayey Black						
184.01 0.40	CLAYEY SILT, some sand, trace gravel with fine sand pockets Firm to stiff Brown					184 183	
182.43 1.98	CLAYEY SILT, some sand, trace gravel Firm to very stiff Brown becoming grey at about elev. 180.8m				Bentonite Sand	182 181 180 179 178 177 176 175	
174.81 9.60	END OF BOREHOLE Water level measured in shallow piezometer at elev. 182.40m on March 20, 2008. Water level measured in shallow piezometer at elev. 182.63m on July 22, 2008. Water level measured in shallow piezometer at elev. 182.43m on September 19, 2008. Water level measured in shallow piezometer at elev. 182.50m on January 28, 2009.				Piezometer		

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

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

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-114

METRIC

PROJECT 07-1130-207-0

W.P.

DIST WEST HWY 401/3

DATUM GEODETIC

LOCATION N 4678526.7 ;E 334018.6

BOREHOLE TYPE POWER AUGER, SOLID STEM

DATE September 10, 2008

ORIGINATED BY CC

COMPILED BY SJL

CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.21	GROUND SURFACE									
0.00	TOPSOIL, silty, trace to some sand Compact Black		1	SS	20		184	~14		
0.28	CLAYEY SILT, trace to some sand, trace gravel Very stiff Mottled brown and grey becoming brown at about elev. 183.0m							~10		
			2	SS	20			~12		
			3	SS	28		183	~13		
182.38 1.83	END OF BOREHOLE Borehole dry during drilling on September 10, 2008.									

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

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-312

METRIC

PROJECT 09-1132-0080

W.P.

LOCATION N 4678319.9 ;E 334283.0

ORIGINATED BY TA

DIST WEST HWY 401 / 3

BOREHOLE TYPE POWER AUGER, SOLID STEM

COMPILED BY DMB

DATUM GEODETIC

DATE January 15, 2010

CHECKED BY

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.22	GROUND SURFACE						
0.00	TOPSOIL, clayey Black					185	
0.23	FILL, clayey silt, some sand, trace gravel, trace organics Brown and grey						
184.46 0.76	CLAYEY SILT, some sand, trace gravel, with cobbles and occasional silt partings Stiff to hard Brown	1	SS	14		184	
		2	SS	17		183	
		3	SS	38			
		4	SS	66/ 200mm		182	
						181	
180.65 4.57	END OF BOREHOLE Borehole dry during drilling on January 15, 2010.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

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

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-5

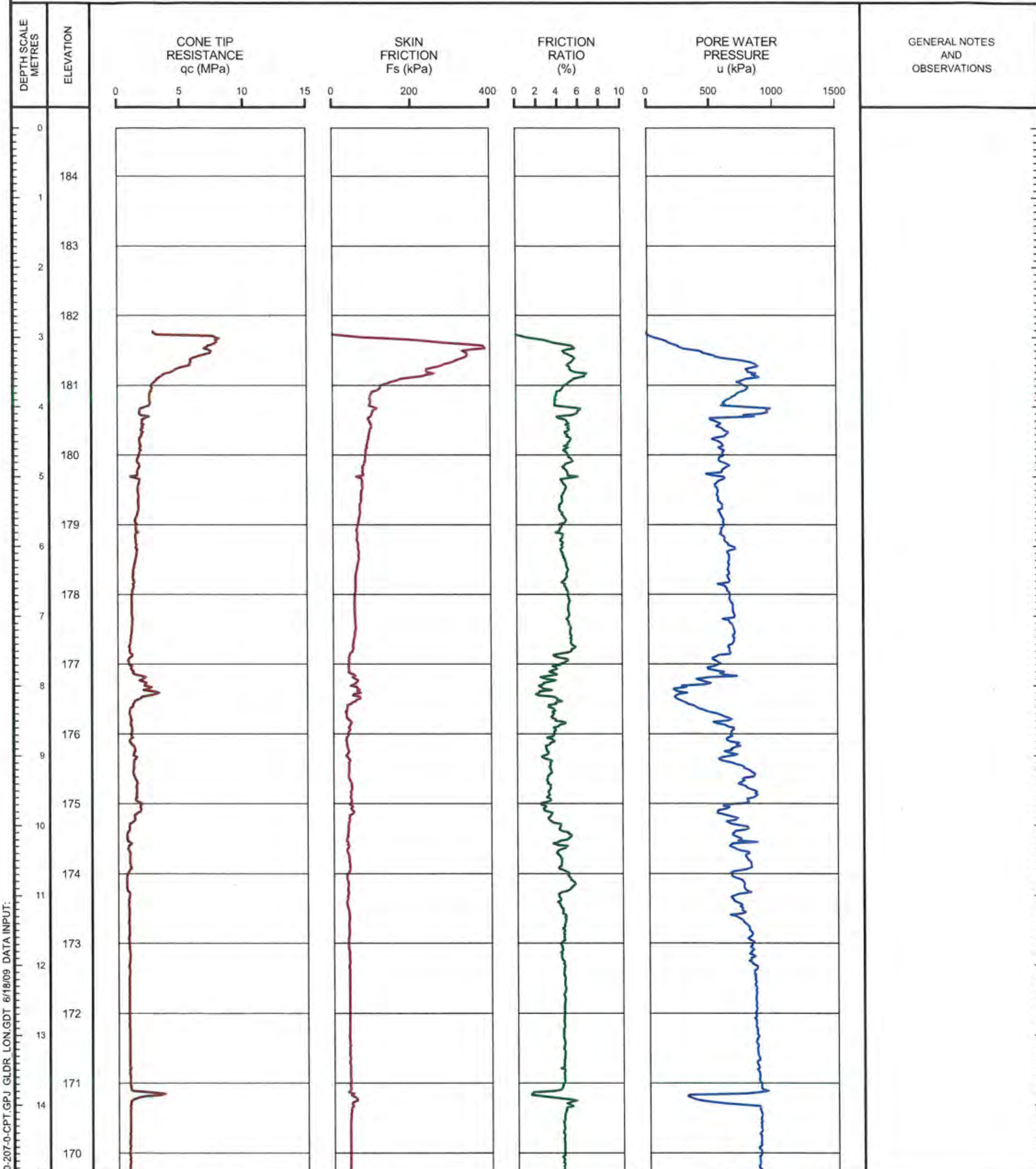
SHEET 1 OF 2

LOCATION: N 4678413.0 ;E 334220.0

TEST DATE: November 13, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 2.94m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



— CONTINUED NEXT PAGE —

DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

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

PROJECT: 07-1130-207-0

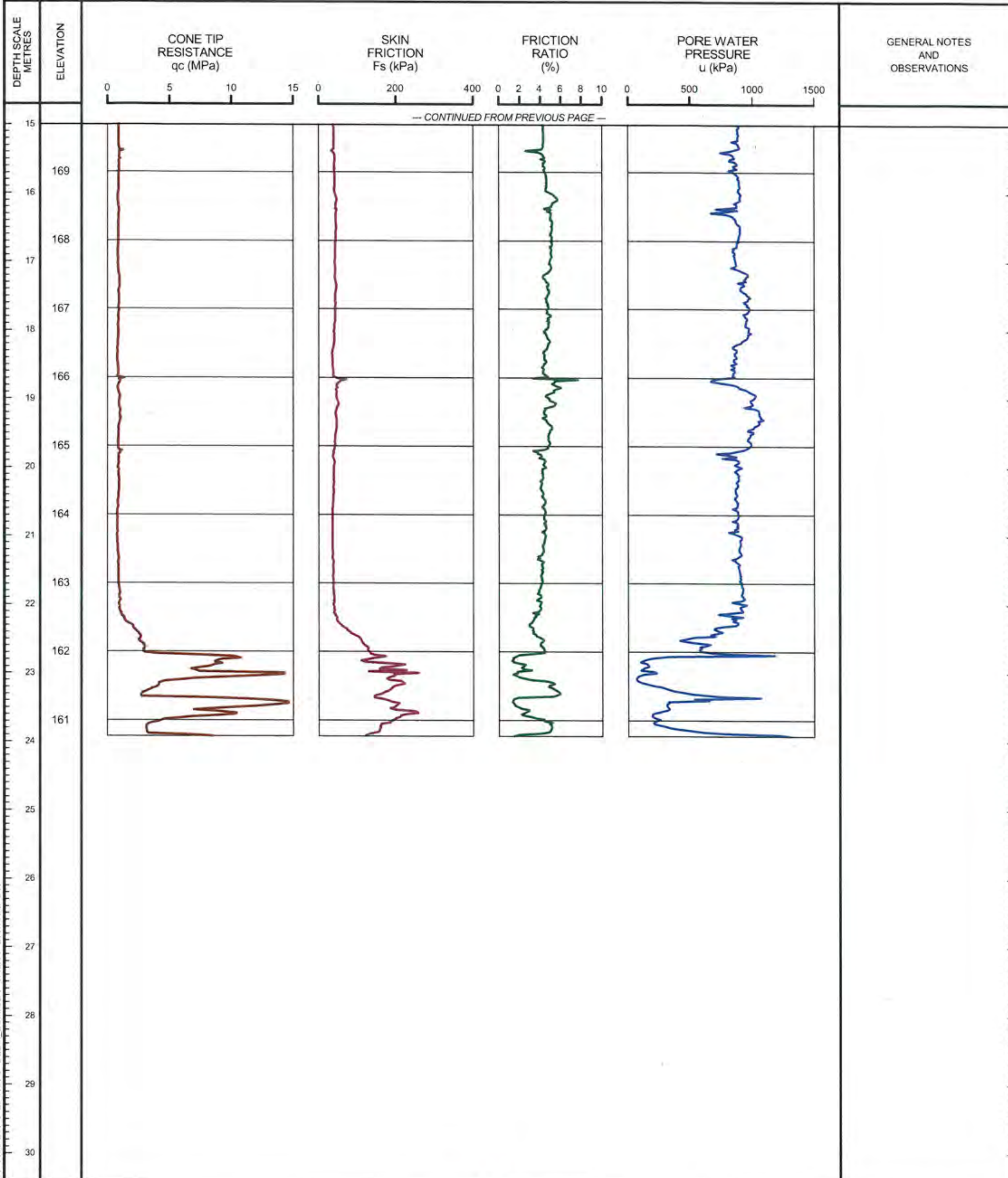
LOCATION: N 4678413.0 ;E 334220.0

RECORD OF CONE PENETRATION TEST CPT-5

TEST DATE: November 13, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 2.94m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SBS

PROJECT: 07-1130-207-0

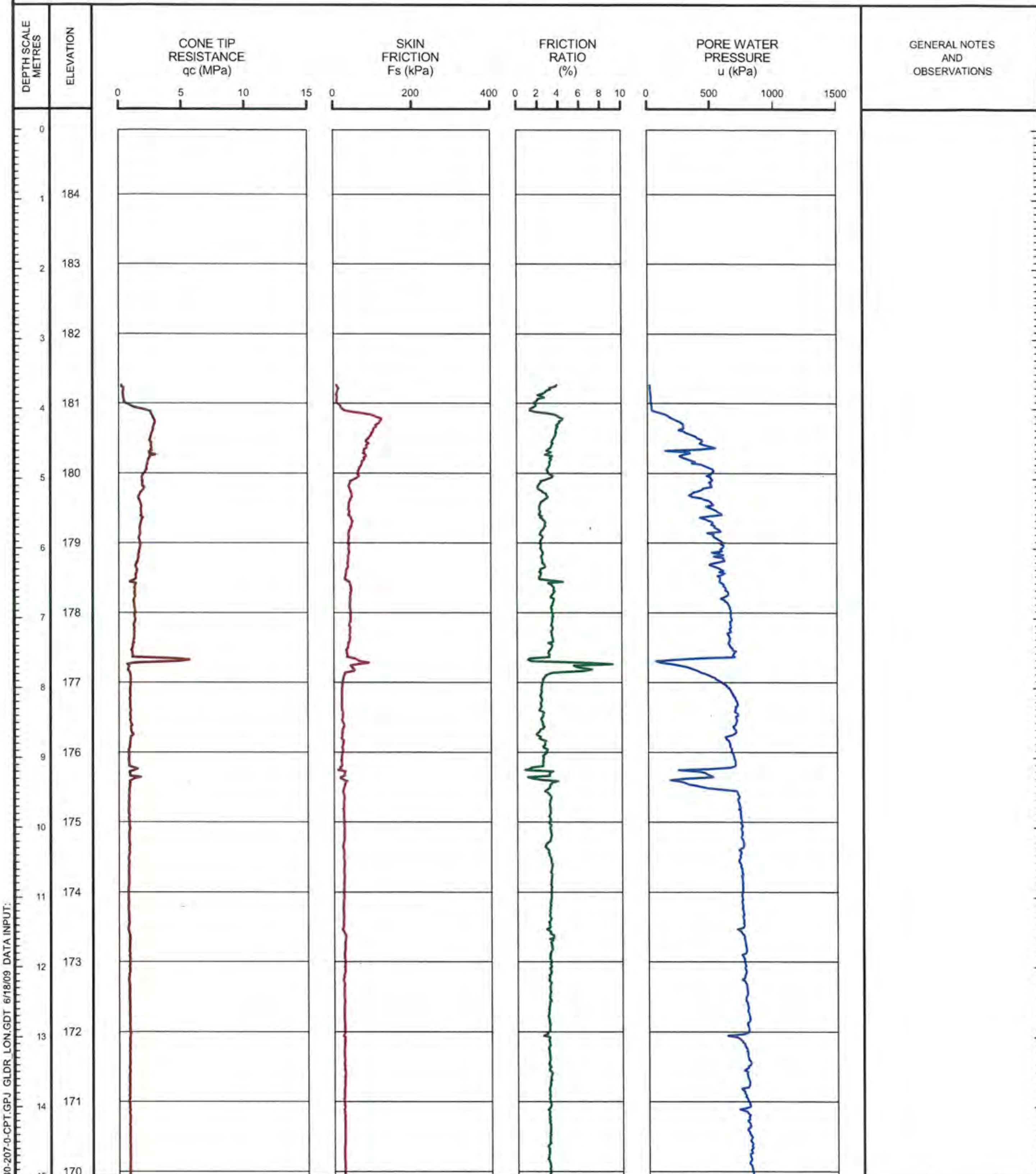
RECORD OF CONE PENETRATION TEST CPT-111

LOCATION: N 4678351.4 ;E 334347.6

TEST DATE: September 9, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 3.66m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



— CONTINUED NEXT PAGE —

DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

PROJECT: 07-1130-207-0

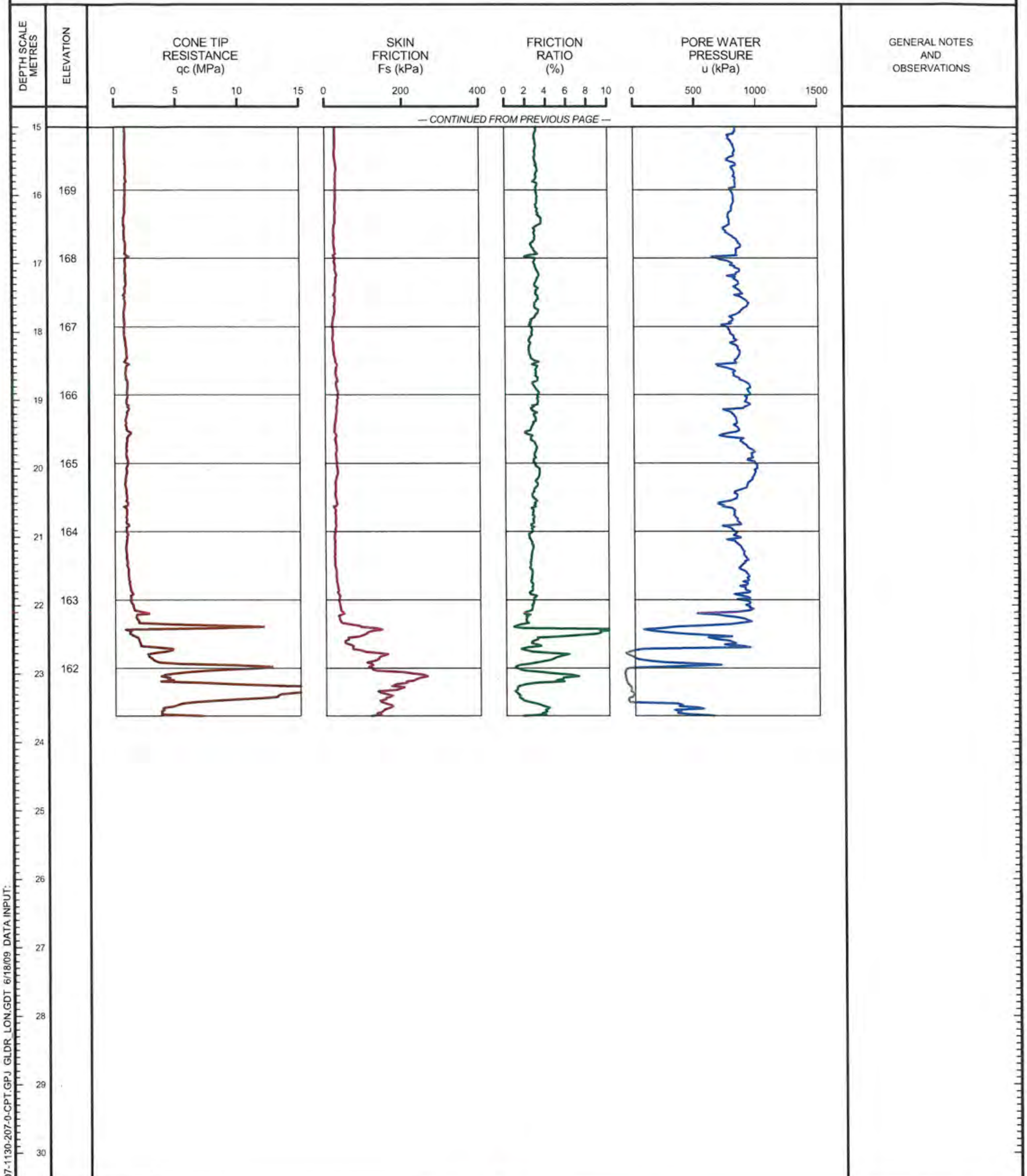
RECORD OF CONE PENETRATION TEST CPT-111

LOCATION: N 4678351.4 ;E 334347.6

TEST DATE: September 9, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 3.66m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



DEPTH SCALE

1 : 75

OPERATOR: CC

CHECKED: SJB

PROJECT: 07-1130-207-0

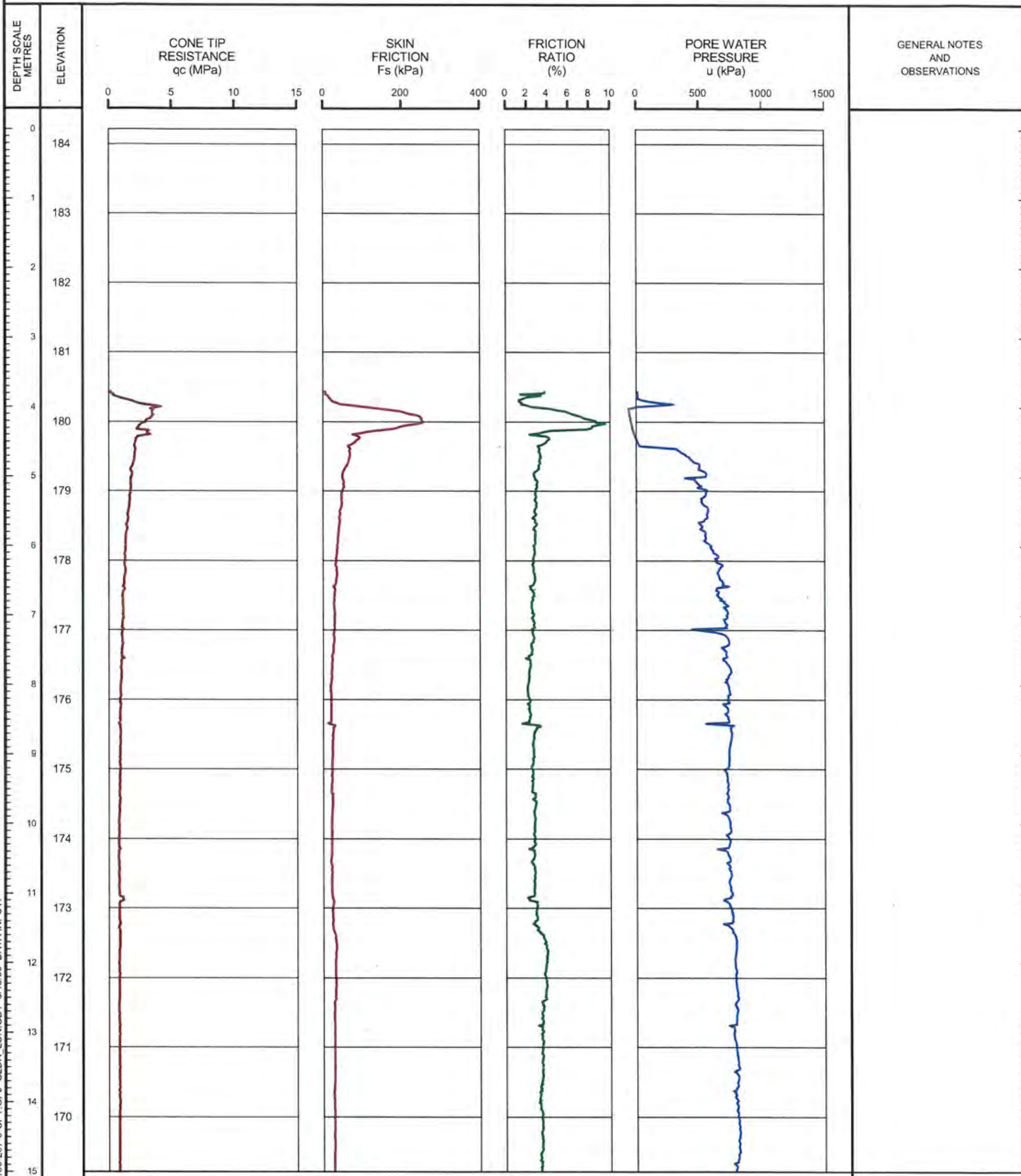
RECORD OF CONE PENETRATION TEST CPT-114

LOCATION: N 4678526.7 ;E 334018.6

TEST DATE: September 10, 2008

DATUM: GEODETIC

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



— 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:

RECORD OF CONE PENETRATION TEST CPT-114

PROJECT: 07-1130-207-0

LOCATION: N 4678526.7 ;E 334018.6

TEST DATE: September 10, 2008

DATUM: GEODETIC

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

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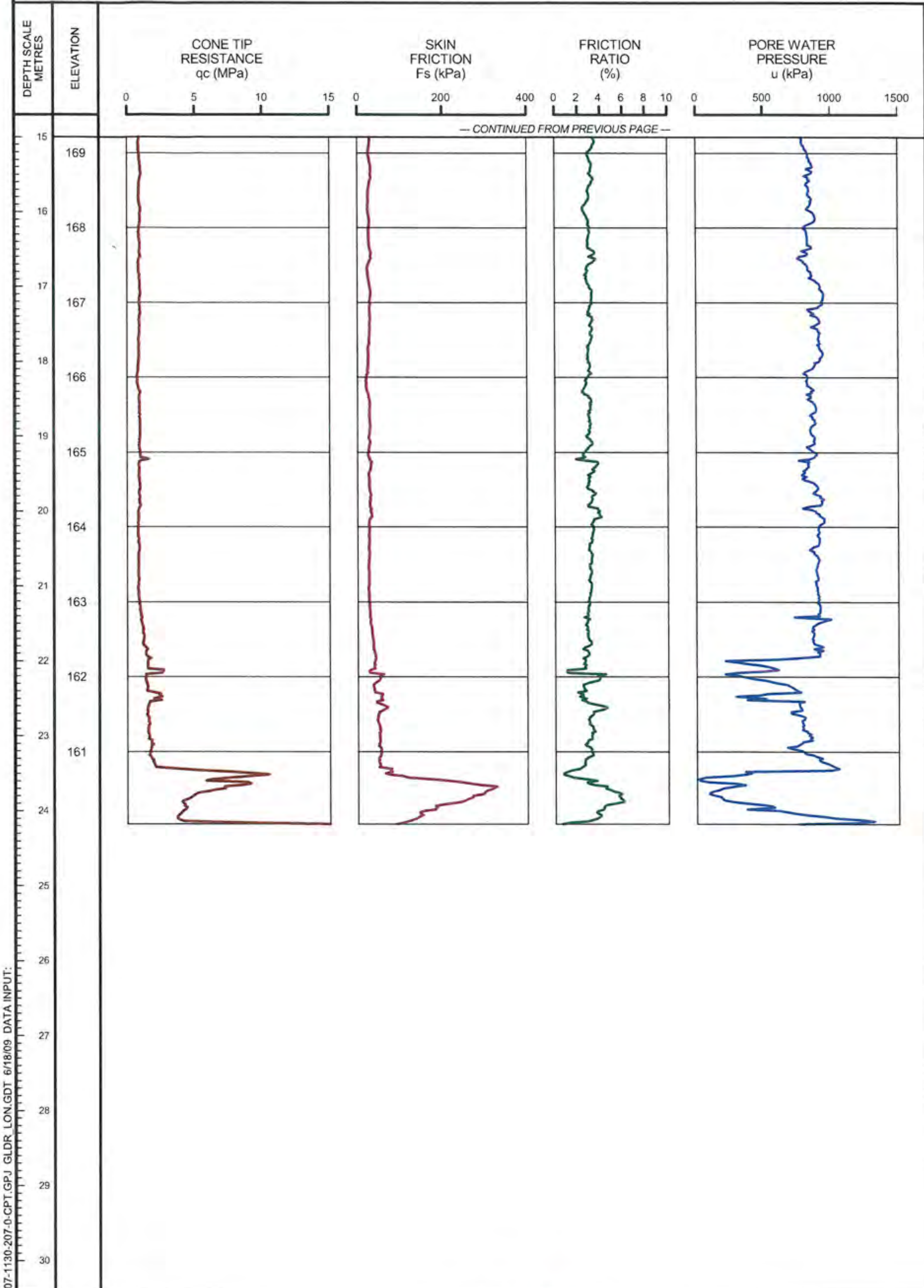


GENERAL NOTES AND OBSERVATIONS

DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

RECORD OF CONE PENETRATION TEST CPT-312

PROJECT: 09-1132-0080

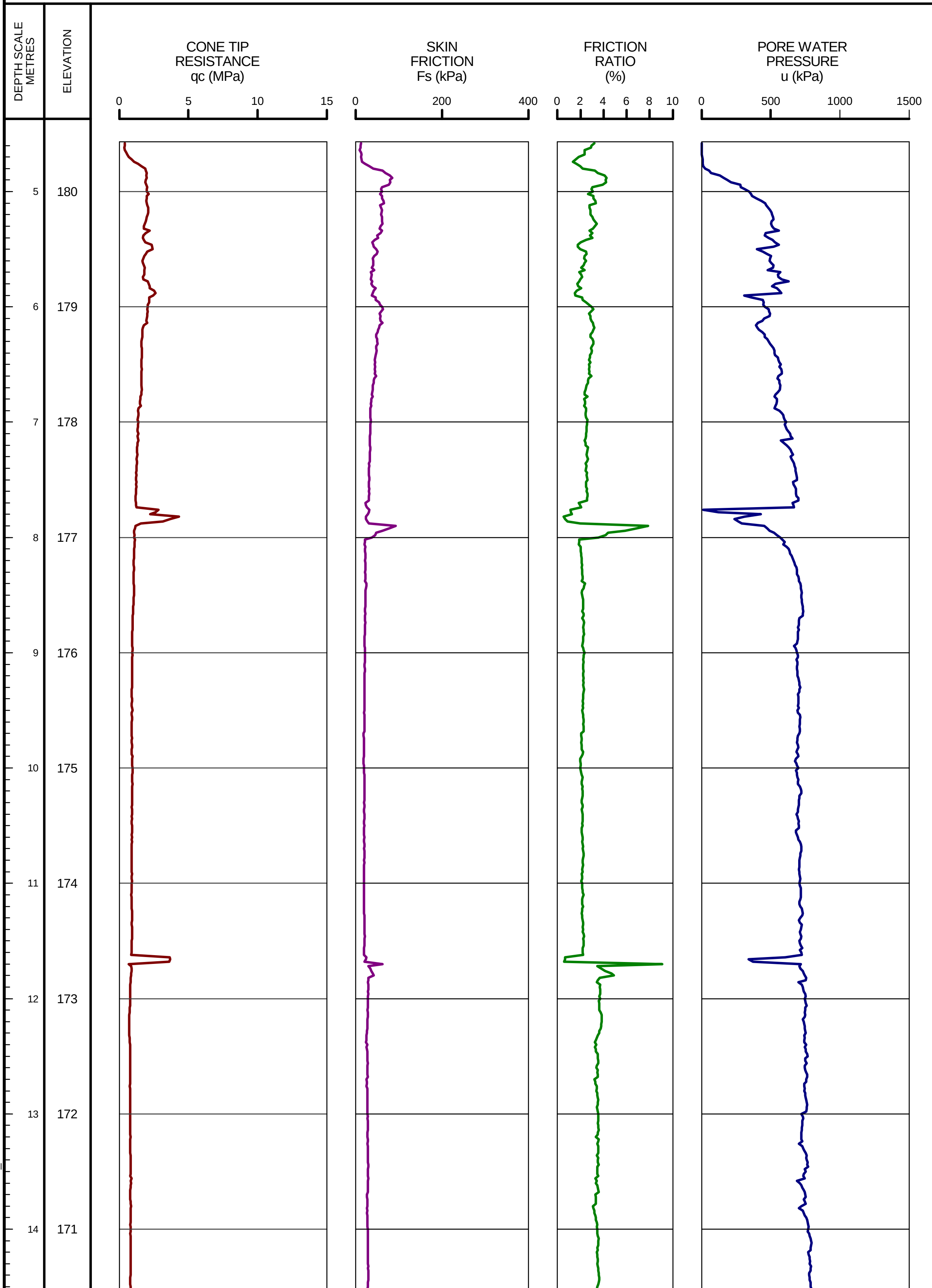
SHEET 1 OF 2

LOCATION: N 4678319.9 ;E 334283.0

TEST DATE: January 15, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 185.22m PREDRILL DEPTH: 4.57m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



GENERAL NOTES AND OBSERVATIONS

CPT PUSHED AFTER PREDRILLING SAMPLED HOLE.
FOR SOIL PROFILE, SEE RECORD OF BOREHOLE No. CPT-312.

--- *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

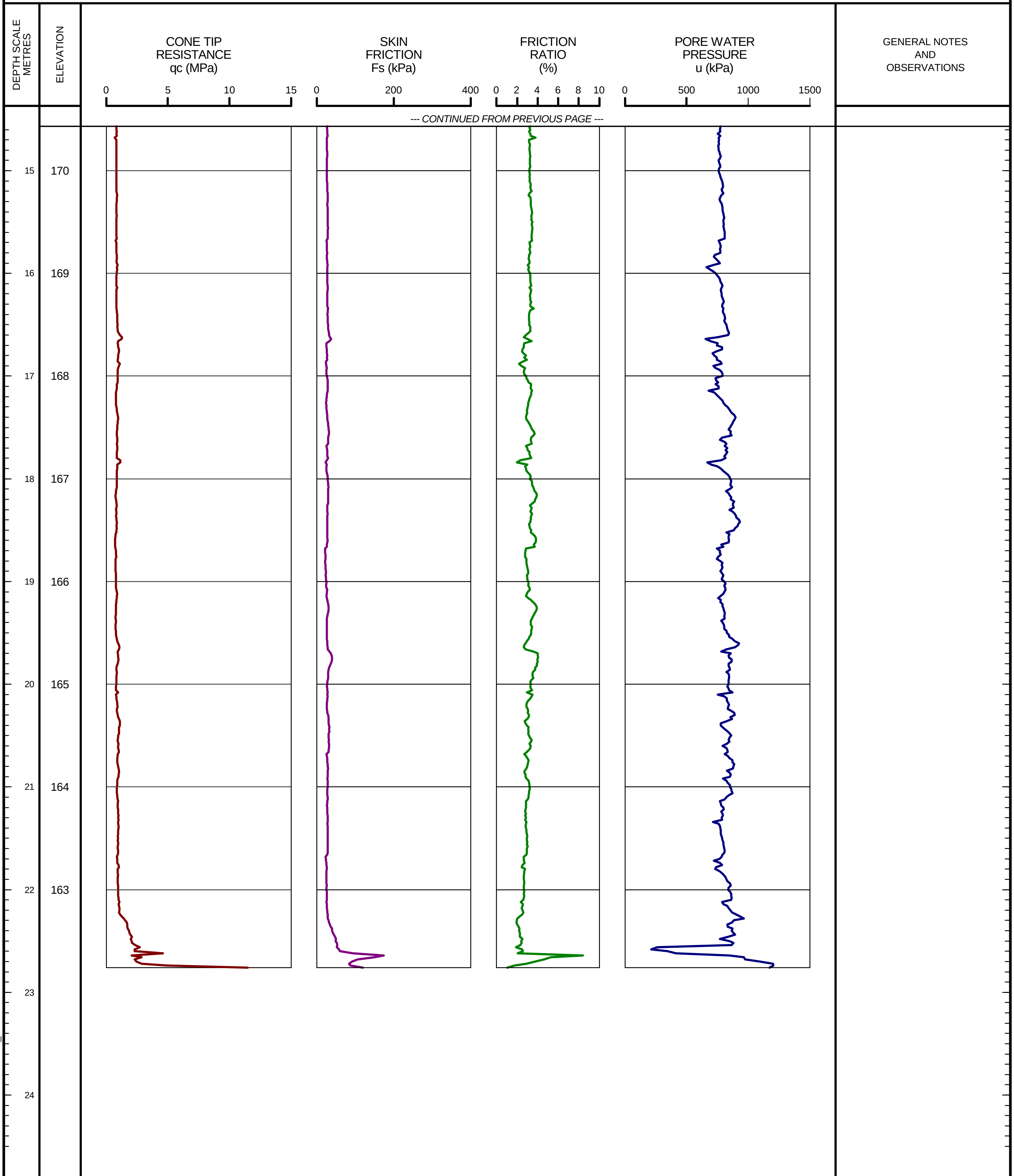
RECORD OF CONE PENETRATION TEST CPT-312

LOCATION: N 4678319.9 ;E 334283.0

TEST DATE: January 15, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 185.22m PREDRILL DEPTH: 4.57m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



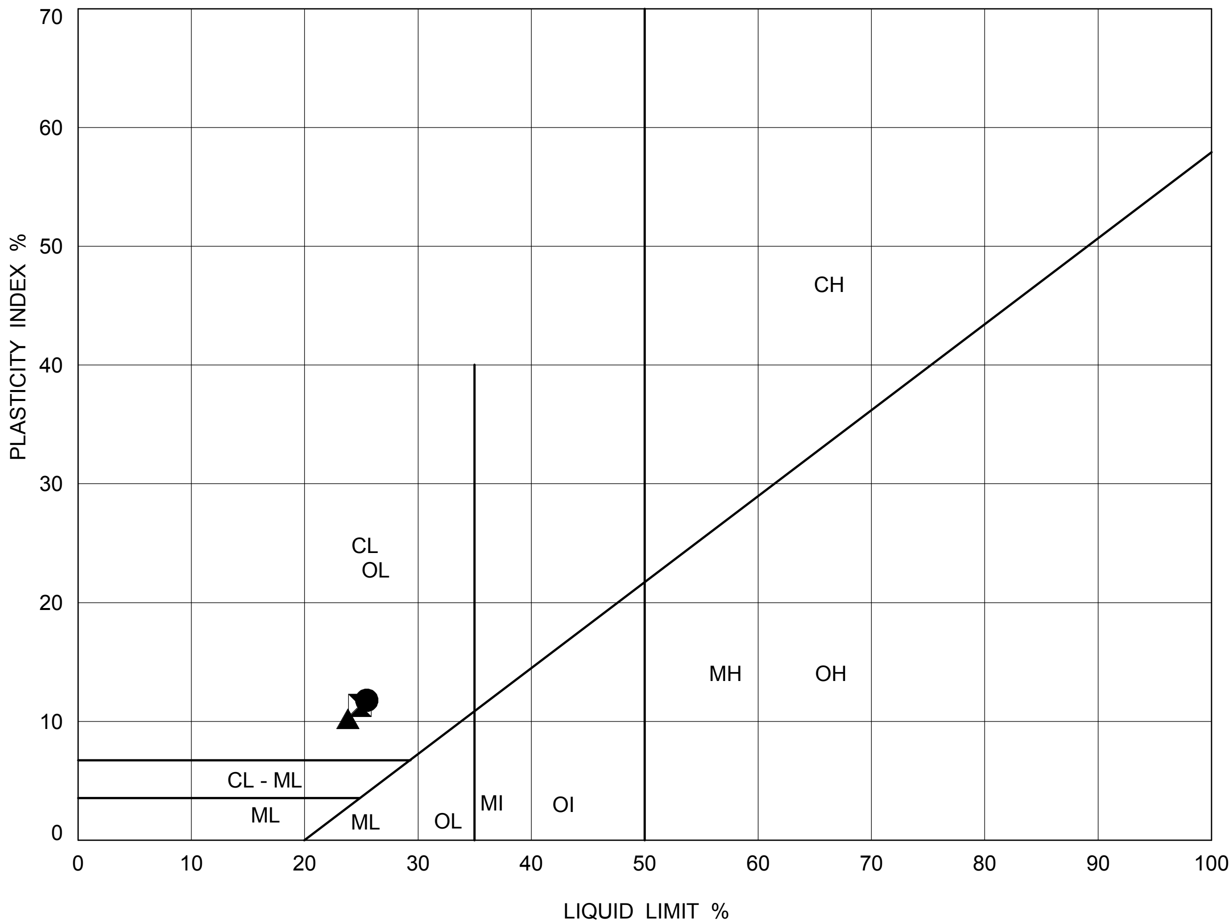
DEPTH SCALE

1 : 50

OPERATOR: TA

CHECKED:

Appendix C Laboratory Test Results



SOIL TYPE

C = Clay
M = Silt
O = Organic

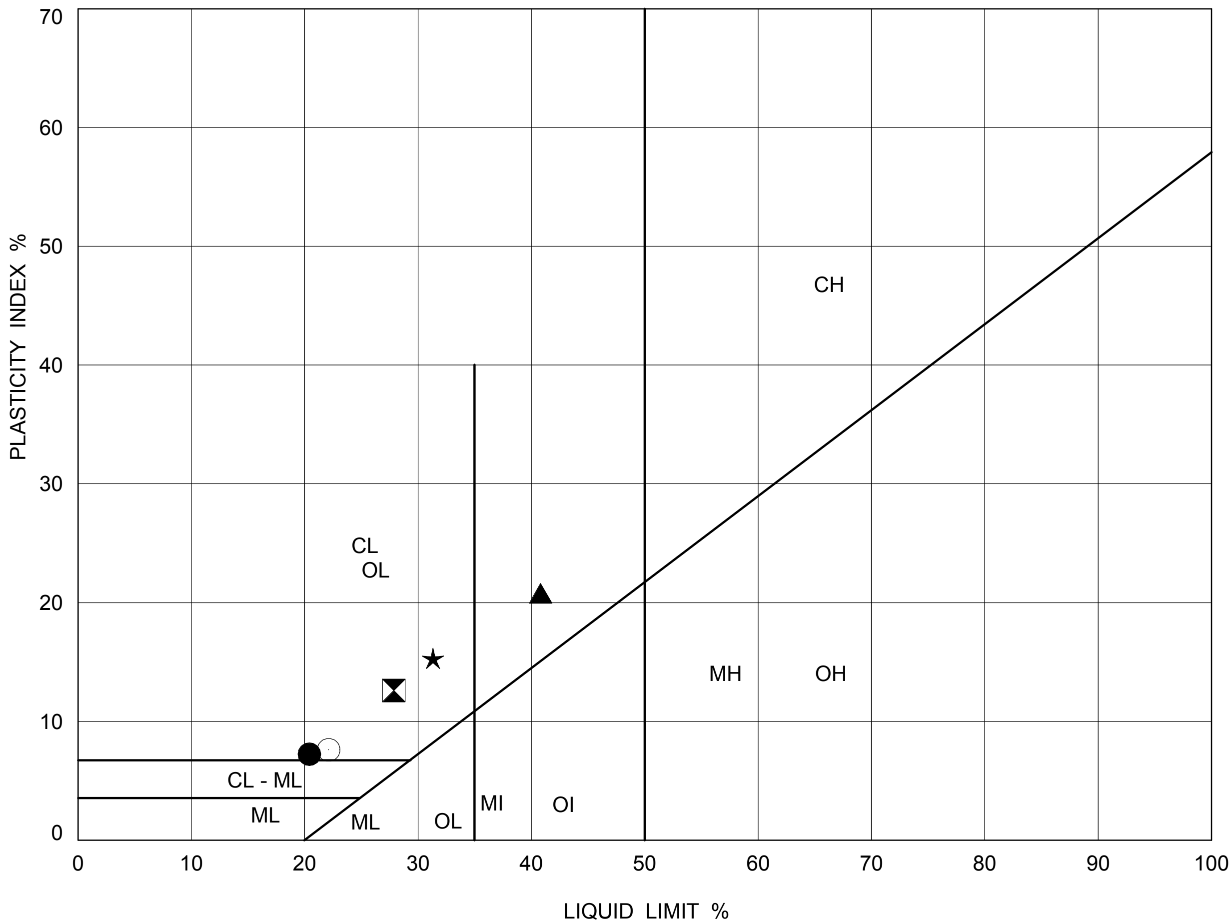
PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T10-1/HGMW-04	9	6.1	**25**	**14**	**11**
⧗	T10-1/HGMW-04	12	10.7	**25**	**13**	**12**
▲	T10-1/HGMW-04	18	19.8	**24**	**14**	**10**

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE	PLASTICITY CHART		
Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PROJECT No. SW8801.1004.101	FILE No.	
		SCALE	REV.
	DRAWN	FIGURE	C.1
	CHECK		



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

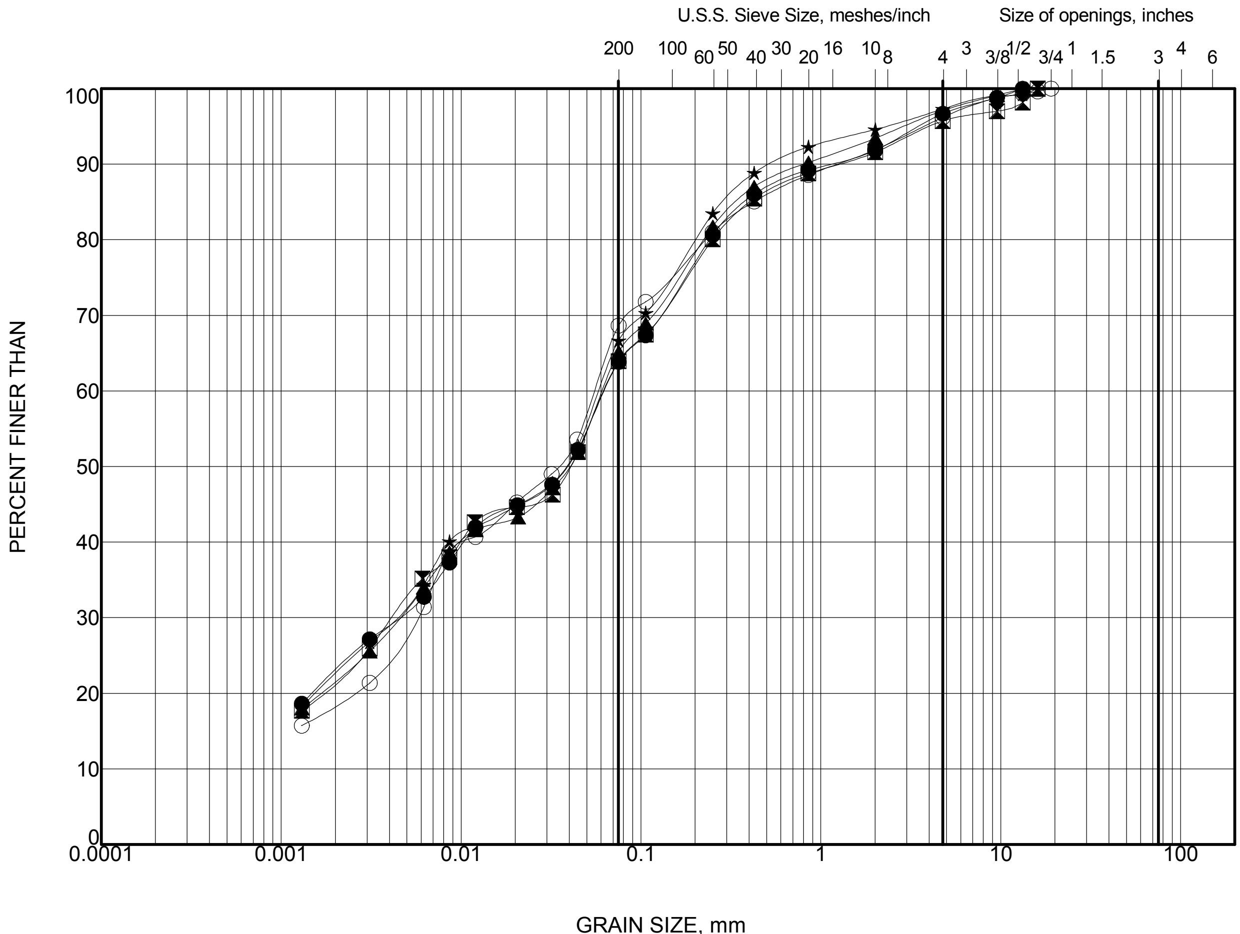
L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T10-2/HGMW-09	8	6.1	**20**	**13**	**7**
⧗	T10-2/HGMW-09	15	15.8	**28**	**15**	**13**
▲	T10-2/HGMW-09	17	17.4	**41**	**20**	**21**
★	T10-2/HGMW-09	19	18.9	**31**	**16**	**15**
○	T10-2/HGMW-09	23	24.4	**22**	**14**	**8**

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario
TITLE	PLASTICITY CHART
Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PROJECT No. SW8801.1004.101 DRAWN CHECK
FILE No. SCALE	REV.
FIGURE	C.2

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



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T10-1/HGMW-04	5	3
⧗	T10-1/HGMW-04	7	4.6
▲	T10-1/HGMW-04	9	6.1
★	T10-1/HGMW-04	12	10.7
○	T10-1/HGMW-04	18	19.8

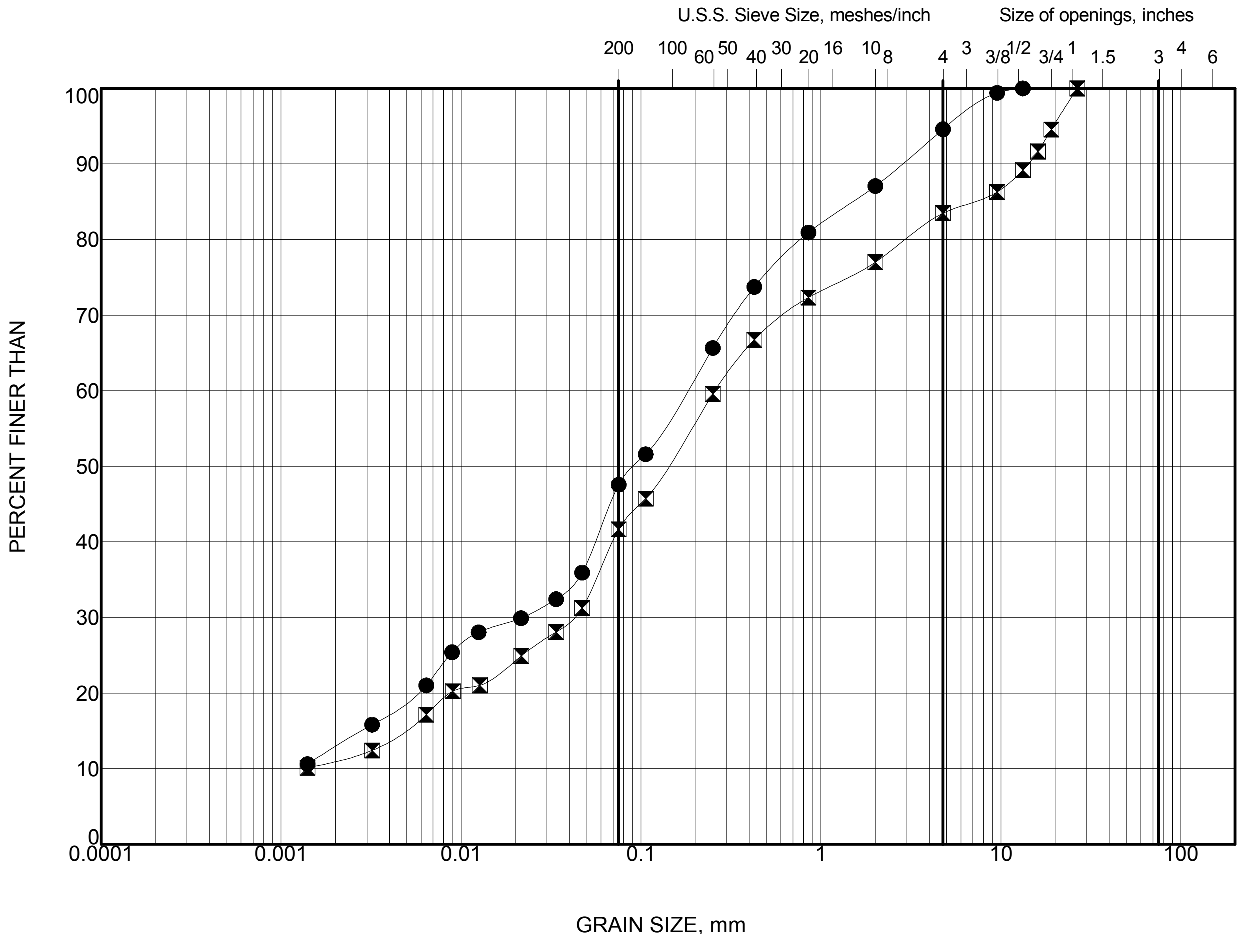
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

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

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



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T10-1/HGMW-04	23	27.4
⧗	T10-1/HGMW-04	24	29

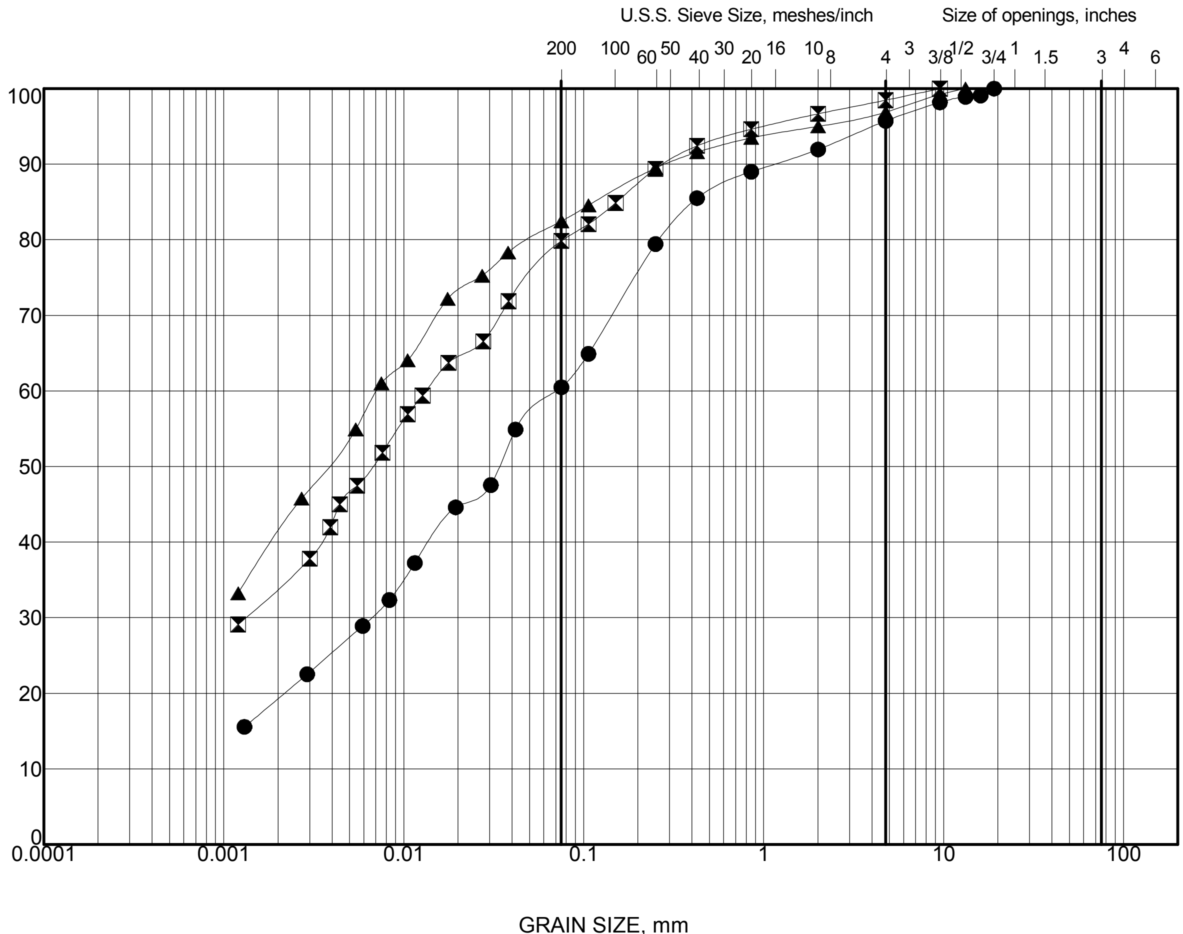
PROJECT Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE

GRAIN SIZE DISTRIBUTION

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

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



<table>
<tr><td rowspan="2">CLAY AND SILT</td><td>fine</td><td>medium</td><td>coarse</td><td>fine</td><td>coarse</td><td rowspan="2">Cobble Size</td></tr>
<tr><td colspan="3">SAND SIZE</td><td colspan="2">GRAVEL SIZE</td></tr>
</table>

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T10-2/HGMW-09	8	6.1
⧗	T10-2/HGMW-09	20.1	19.8
▲	T10-2/HGMW-09	24	25.9

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

Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PROJECT No. SW8801.1004.101	FILE No.
	DRAWN	SCALE / REV.
	CHECK	FIGURE C.5

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

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 12-Jul-11 **Sample ID:** B10-2_Sa20A **Depth(m):** 19.8 to 20.4

Test Data

Ring # :	A	Ring Height (in) =	0.755	Wt of dry filter paper (g)	0.69
Wet soil + Ring Wt (g)			208.30	Wt of ring (g)	76.58
Wet soil + Wet Paper + Ring (g)			204.96	Wet Paper (g)	2.15
Dry Soil + Dry Paper + Ring (g)			188.87	Ring Dia (in)	2.498
Initial moisture Content (%)			18.03	Final moisture Content (%)	13.11
Area of Ring (in^2)			4.90	Initial Volume (in^3)	3.7002
Initial Bulk Density (kg/m^3)			2172	Initial Dry Density (kg/m^3)	1841
Specific Gravity of Soil			2.75	Eqiv. Thick. of solids (mm)	12.844
Final Bulk Density (kg/m^3)			2537	Final Dry Density (kg/m^3)	2149
Initiall gauge reading for Load 1			0.2506	Gauge reading for last Loading	0.1829
Initial Voids Ratio			0.493	Final Void Ratio	0.359
Initial Degree of Saturation (%)			100	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	4.0	5.5	8.5	13.0	20.0	30.0	45.0
Load (tsf)	0.0416	0.0572	0.088	0.135	0.208	0.312	0.468
Gauge Reading (in)	0.2475	0.2471	0.2450	0.2418	0.2380	0.2333	0.2276
(H-Hs) mm	6.254	6.244	6.191	6.110	6.013	5.893	5.749
Voids ratio	0.487	0.486	0.482	0.476	0.468	0.459	0.448
t90 (min)			47.61	44.89	36.00	21.16	20.25
Cv (m^2/day)			0.002	0.002	0.003	0.005	0.005
k' (MPa)			1.089	1.047	1.371	1.580	1.941
Mv (mm^2 / N)			0.9182	0.9548	0.7294	0.6331	0.5151

Trial #	8	9	10	11	12	13	14
Load (kPa)	65	100.0	150.0	220.0	150.0	100.0	65.0
Load (tsf)	0.676	1.040	1.560	2.288	1.560	1.040	0.676
Gauge Reading (in)	0.22462	0.2175	0.2108	0.2043	0.2045	0.2050	0.2055
(H-Hs) mm	5.673	5.492	5.322	5.157	5.162	5.175	5.188
Voids ratio	0.442	0.428	0.414	0.401	0.402	0.403	0.404
t90 (min)	19.36	12.25	12.25	7.29			
Cv (m^2/day)	0.005	0.008	0.008	0.014			
k' (MPa)	4.913	3.584	5.379	7.714			
Mv (mm^2 / N)	0.2036	0.2790	0.1859	0.1296			

Trial #	15	16	17	18	19	20	21
Load (kPa)	45.0	30.0	20.0	13.0	20.0	30.0	45.0
Load (tsf)	0.468	0.312	0.208	0.135	0.208	0.312	0.468
Gauge Reading (in)	0.20621	0.2069	0.2078	0.2088	0.2088	0.2085	0.2078
(H-Hs) mm	5.205	5.222	5.246	5.271	5.271	5.264	5.246
Voids ratio	0.405	0.407	0.408	0.410	0.410	0.410	0.408
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: 12-Jul-11 **Sample ID:** B10-2_Sa20A **Depth(m):** 19.8 to 20.4

Trial #	22	23	24	25	26	27	28
Load (kPa)	65	100.0	150.0	220.0	330.0	490.0	740.0
Load (tsf)	0.676	1.040	1.560	2.288	3.432	5.096	7.696
Gauge Reading (in)	0.2071	0.2058	0.2045	0.2023	0.1970	0.1899	0.1817
(H-Hs) mm	5.228	5.194	5.163	5.106	4.970	4.790	4.582
Voids ratio	0.407	0.404	0.402	0.398	0.387	0.373	0.357
t90 (min)					4.84	4.62	4.41
Cv (m^2/day)					0.020	0.021	0.021
k' (MPa)					14.530	15.805	21.244
Mv (mm^2 / N)					0.0688	0.0633	0.0471

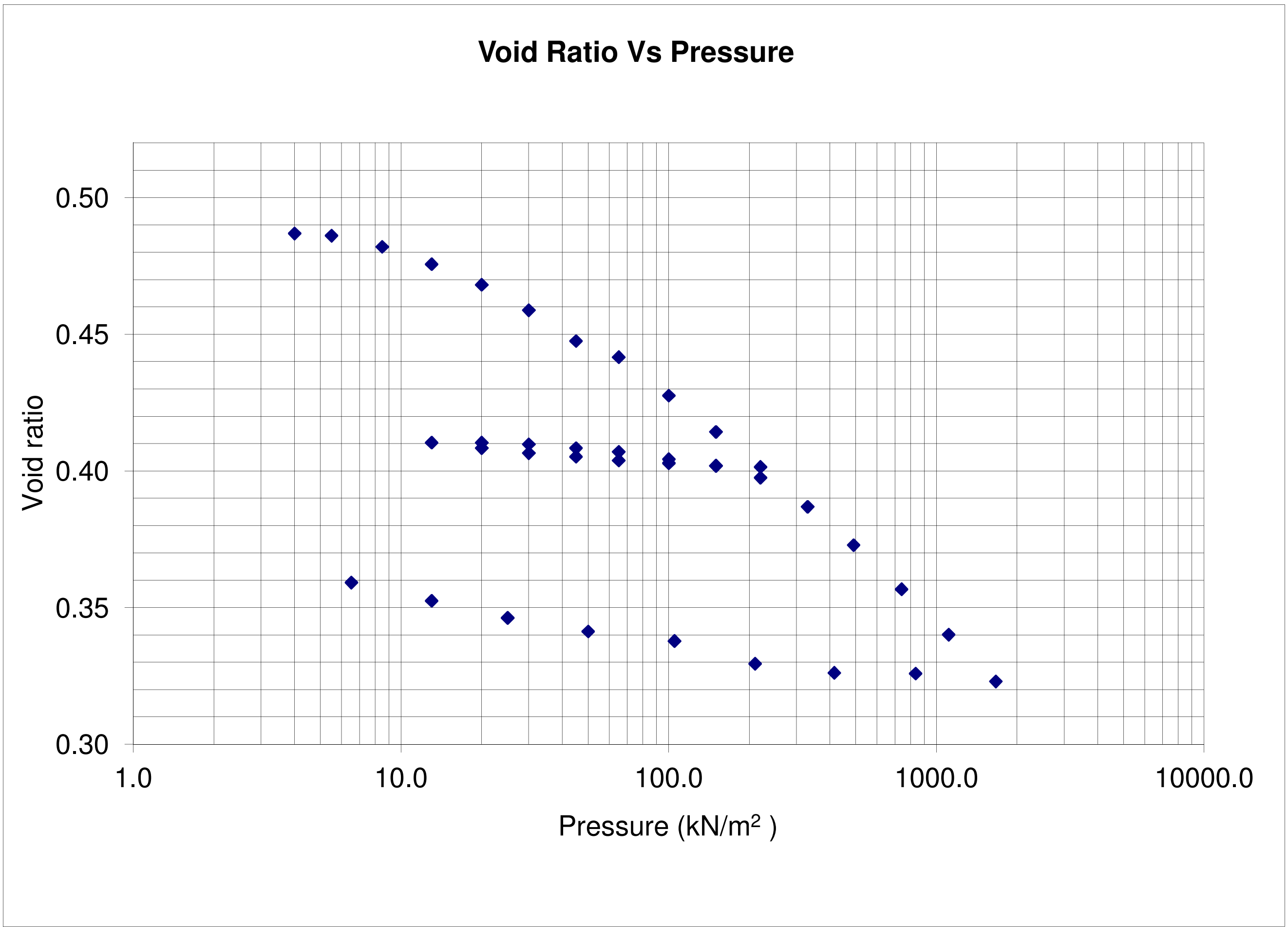
Trial #	29	30	31	32	33	34	35
Load (kPa)	1110	1665.0	835.0	415.0	210.0	105.0	50.0
Load (tsf)	11.544	17.316	8.684	4.316	2.184	1.092	0.520
Gauge Reading (in)	0.17328	0.1646	0.1661	0.1662	0.1679	0.1721	0.1739
(H-Hs) mm	4.369	4.149	4.186	4.189	4.232	4.339	4.384
Voids ratio	0.340	0.323	0.326	0.326	0.330	0.338	0.341
t90 (min)	4.00	2.89					
Cv (m^2/day)	0.023	0.031					
k' (MPa)	30.220	43.431					
Mv (mm^2 / N)	0.0331	0.0230					

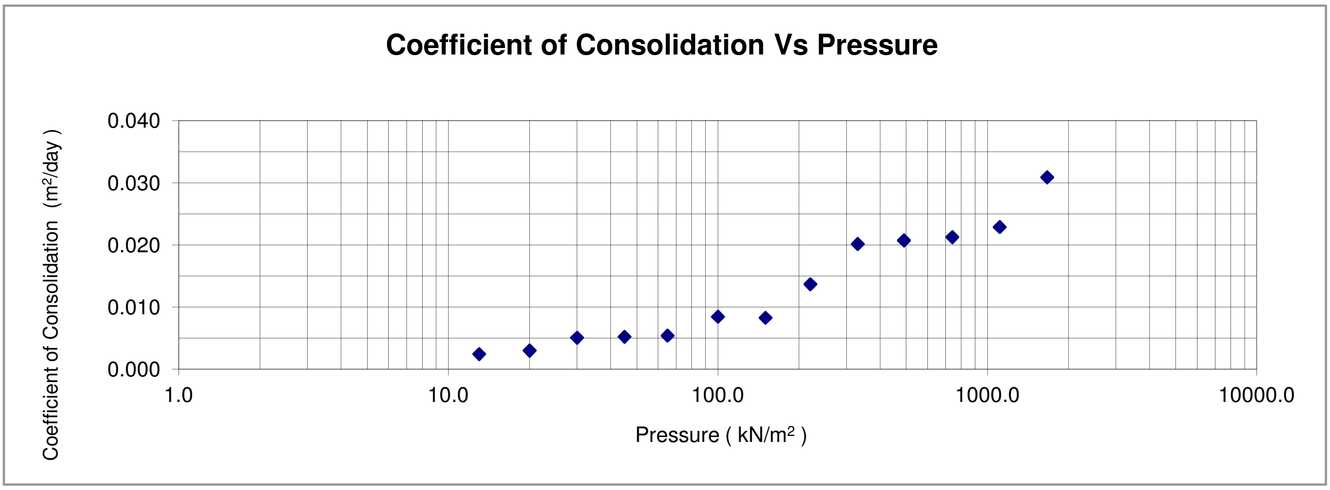
Trial #	36	37	38				
Load (kPa)	25	13.0	6.5				
Load (tsf)	0.26	0.135	0.068				
Gauge Reading (in)	0.1764	0.1796	0.1829				
(H-Hs) mm	4.448	4.528	4.614				
Voids ratio	0.346	0.353	0.359				
t90 (min)							
Cv (m^2/day)							
k' (MPa)							
Mv (mm^2 / N)							

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP
Job No.: SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 12-Jul-11
Sample ID: B10-2_Sa20A
Depth(m): 19.8 to 20.4

σ'_v versus e and c_v





ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 12-Jul-11 **Sample ID:** B10-2_Sa20A **Depth(m):** 19.8 to 20.4

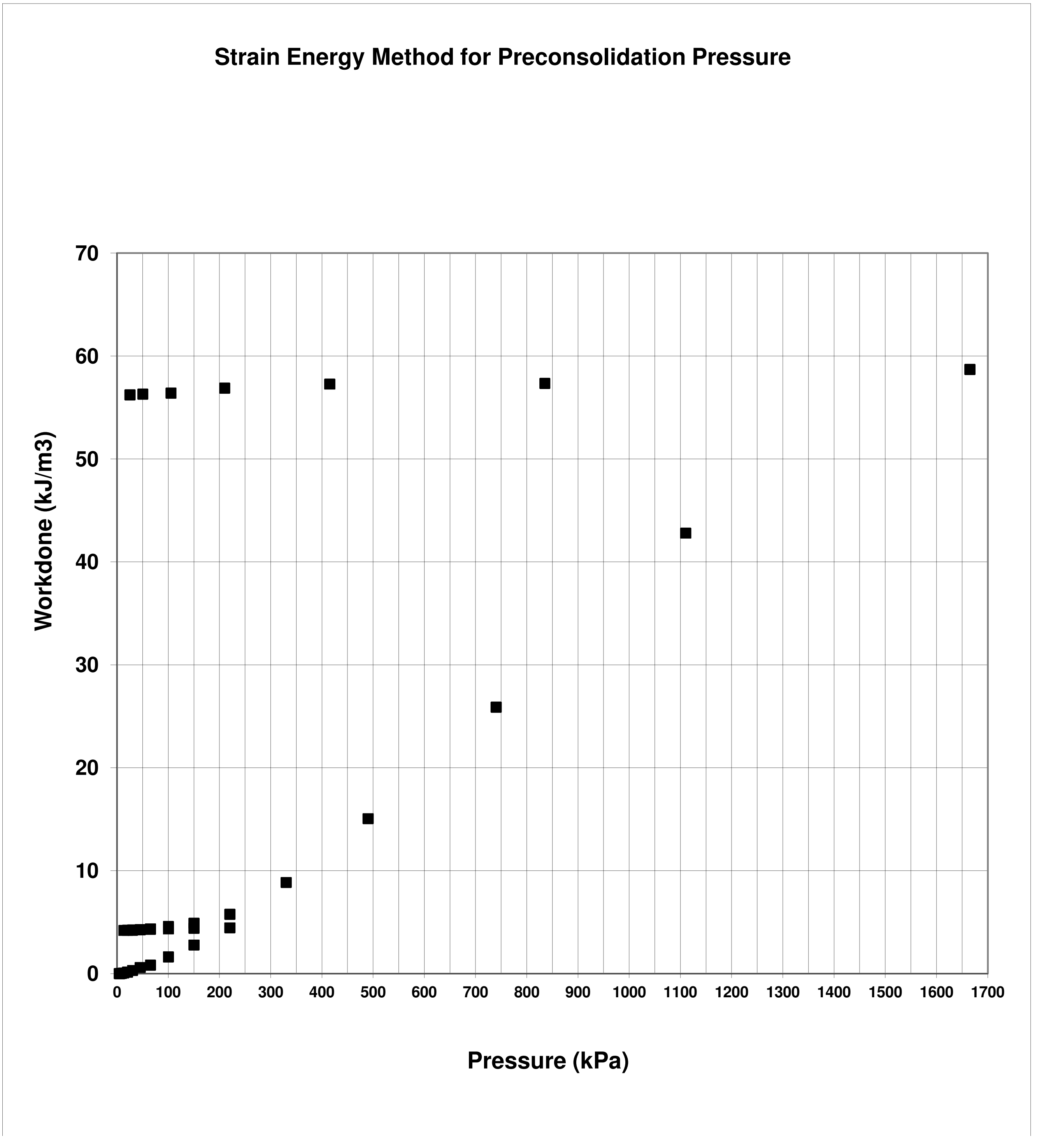
Strain Energy Data

Presssure (kN/m^2)	c_v (m^2/day)	Void ratio
4.0		0.487
5.5		0.486
8.5		0.482
13.0	0.002	0.476
20.0	0.003	0.468
30.0	0.005	0.459
45.0	0.005	0.448
65.0	0.005	0.442
100.0	0.008	0.428
150.0	0.008	0.414
220.0	0.014	0.401
150.0		0.402
100.0		0.403
65.0		0.404
45.0		0.405
30.0		0.407
20.0		0.408
13.0		0.410
20.0		0.410
30.0		0.410
45.0		0.408
65.0		0.407
100.0		0.404
150.0		0.402
220.0		0.398
330.0	0.020	0.387
490.0	0.021	0.373
740.0	0.021	0.357
1110.0	0.023	0.340
1665.0	0.031	0.323
835.0		0.326
415.0		0.326
210.0		0.330
105.0		0.338
50.0		0.341
25.0		0.346
13.0		0.353
6.5		0.359

Presssure (kN/m^2)	Height mm	Total Work (kJ/m^3)
4.0	19.177	0.000
5.5	19.167	0.003
8.5	19.114	0.022
13.0	19.032	0.068
20.0	18.936	0.152
30.0	18.816	0.309
45.0	18.672	0.598
65.0	18.596	0.821
100.0	18.415	1.623
150.0	18.245	2.780
220.0	18.080	4.451
150.0	18.085	4.404
100.0	18.098	4.346
65.0	18.110	4.308
45.0	18.128	4.271
30.0	18.145	4.248
20.0	18.169	4.226
13.0	18.194	4.203
20.0	18.194	4.203
30.0	18.186	4.219
45.0	18.169	4.273
65.0	18.151	4.354
100.0	18.117	4.590
150.0	18.086	4.906
220.0	18.029	5.767
330.0	17.893	8.858
490.0	17.713	15.056
740.0	17.505	25.893
1110.0	17.292	42.805
1665.0	17.072	58.706
835.0	17.109	57.348
415.0	17.112	57.288
210.0	17.155	56.893
105.0	17.262	56.411
50.0	17.307	56.313
25.0	17.371	56.243
13.0	17.451	56.198
6.5	17.537	56.182

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP
Job No.: SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 12-Jul-11
Sample ID: B10-2_Sa20A
Depth(m): 19.8 to 20.4



ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project:	**WEP**			**Job No.:**	**SW8801.1004.101**
Client:	**Hatch Mott MacDonald Limited**				
Date:	**12-Jul-11**	**Sample ID:**	**B10-2_Sa20A**	**Depth(m):**	**19.8 to 20.4**

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 15-Aug-11 **Sample ID:** T10-1_TW16 **Depth(m):** 18.9

Test Data

Ring # :	B	Ring Height (in) =	0.760	Wt of dry filter paper (g)	0.69
Wet soil + Ring Wt (g)			206.91	Wt of ring (g)	76.52
Wet soil + Wet Paper + Ring (g)			204.58	Wet Paper (g)	2.14
Dry Soil + Dry Paper + Ring (g)			186.73	Ring Dia (in)	2.498
Initial moisture Content (%)			19.06	Final moisture Content (%)	14.97
Area of Ring (in^2)			4.90	Initial Volume (in^3)	3.7247
Initial Bulk Density (kg/m^3)			2136	Initial Dry Density (kg/m^3)	1794
Specific Gravity of Soil			2.72	Eqiv. Thick. of solids (mm)	12.735
Final Bulk Density (kg/m^3)			2230	Final Dry Density (kg/m^3)	1873
Initiall gauge reading for Load 1			0.2591	Gauge reading for last Loading	0.2040
Initial Voids Ratio			0.516	Final Void Ratio	0.406
Initial Degree of Saturation (%)			100	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	3.5	5.5	8.0	12.5	18.5	27.5	42.5
Load (tsf)	0.0364	0.0572	0.083	0.130	0.192	0.286	0.442
Gauge Reading (in)	0.2590	0.2580	0.2574	0.2540	0.2497	0.24555	0.2412
(H-Hs) mm	6.567	6.542	6.525	6.439	6.329	6.225	6.114
Voids ratio	0.516	0.514	0.512	0.506	0.497	0.489	0.480
t90 (min)			16.81	25.00	19.36	9.00	20.25
Cv (m^2/day)			0.007	0.005	0.006	0.012	0.005
k' (MPa)			2.919	1.013	1.046	1.648	2.562
Mv (mm^2 / N)			0.3426	0.9877	0.9560	0.6070	0.3903

Trial #	8	9	10	11	12	13	14
Load (kPa)	62.5	95.0	140.0	210.0	140.0	95.0	62.5
Load (tsf)	0.65	0.988	1.456	2.184	1.456	0.988	0.650
Gauge Reading (in)	0.23675	0.2309	0.2256	0.2187	0.2191	0.2197	0.2211
(H-Hs) mm	6.002	5.853	5.719	5.543	5.553	5.569	5.604
Voids ratio	0.471	0.460	0.449	0.435	0.436	0.437	0.440
t90 (min)	16.00	12.25	10.89	8.41			
Cv (m^2/day)	0.007	0.009	0.010	0.012			
k' (MPa)	3.350	4.098	6.213	7.370			
Mv (mm^2 / N)	0.2985	0.2440	0.1609	0.1357			

Trial #	15	16	17	18	19	20	21
Load (kPa)	42.5	27.5	18.5	12.5	8.0	12.5	18.5
Load (tsf)	0.442	0.286	0.192	0.130	0.083	0.130	0.192
Gauge Reading (in)	0.22248	0.2233	0.2251	0.2268	0.2288	0.2286	0.2279
(H-Hs) mm	5.639	5.660	5.706	5.749	5.801	5.795	5.777
Voids ratio	0.443	0.444	0.448	0.451	0.456	0.455	0.454
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: 15-Aug-11 **Sample ID:** T10-1_TW16 **Depth(m):** 18.9

Trial #	22	23	24	25	26	27	28
Load (kPa)	27.5	42.5	62.5	95.0	140.0	210.0	315.0
Load (tsf)	0.286	0.442	0.650	0.988	1.456	2.184	3.276
Gauge Reading (in)	0.2268	0.2252	0.2238	0.2219	0.2200	0.2166	0.2103
(H-Hs) mm	5.749	5.709	5.672	5.623	5.575	5.490	5.331
Voids ratio	0.451	0.448	0.445	0.442	0.438	0.431	0.419
t90 (min)							5.76
Cv (m^2/day)							0.017
k' (MPa)							12.016
Mv (mm^2 / N)							0.0832

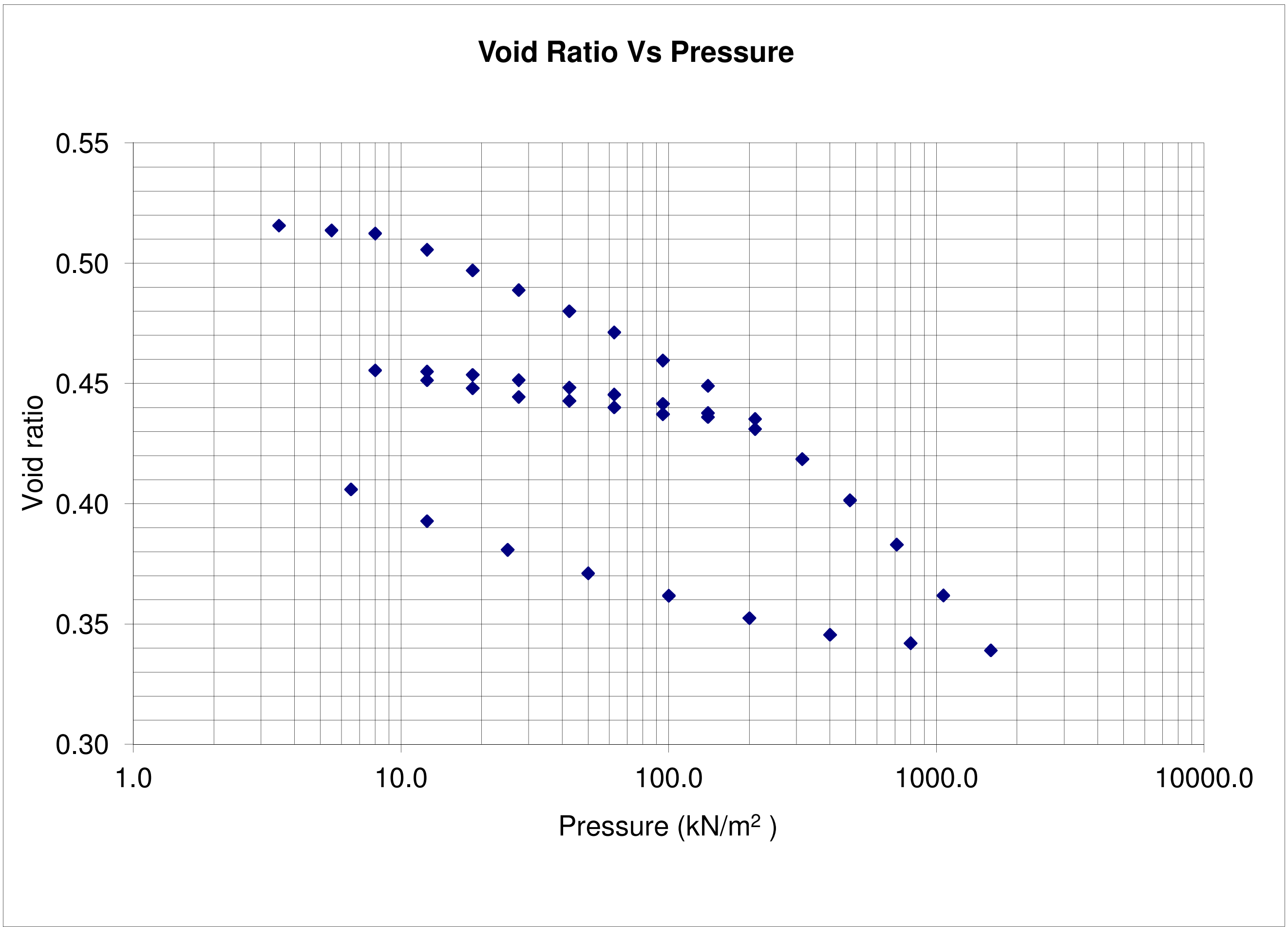
Trial #	29	30	31	32	33	34	35
Load (kPa)	475	710.0	1060.0	1595.0	800.0	400.0	200.0
Load (tsf)	4.94	7.384	11.024	16.588	8.320	4.160	2.080
Gauge Reading (in)	0.20175	0.1925	0.1819	0.1705	0.1720	0.1737	0.1772
(H-Hs) mm	5.113	4.878	4.609	4.318	4.356	4.400	4.489
Voids ratio	0.401	0.383	0.362	0.339	0.342	0.346	0.352
t90 (min)	7.84	6.76	3.80	5.06			
Cv (m^2/day)	0.013	0.014	0.025	0.018			
k' (MPa)	13.263	17.851	22.895	31.904			
Mv (mm^2 / N)	0.0754	0.0560	0.0437	0.0313			

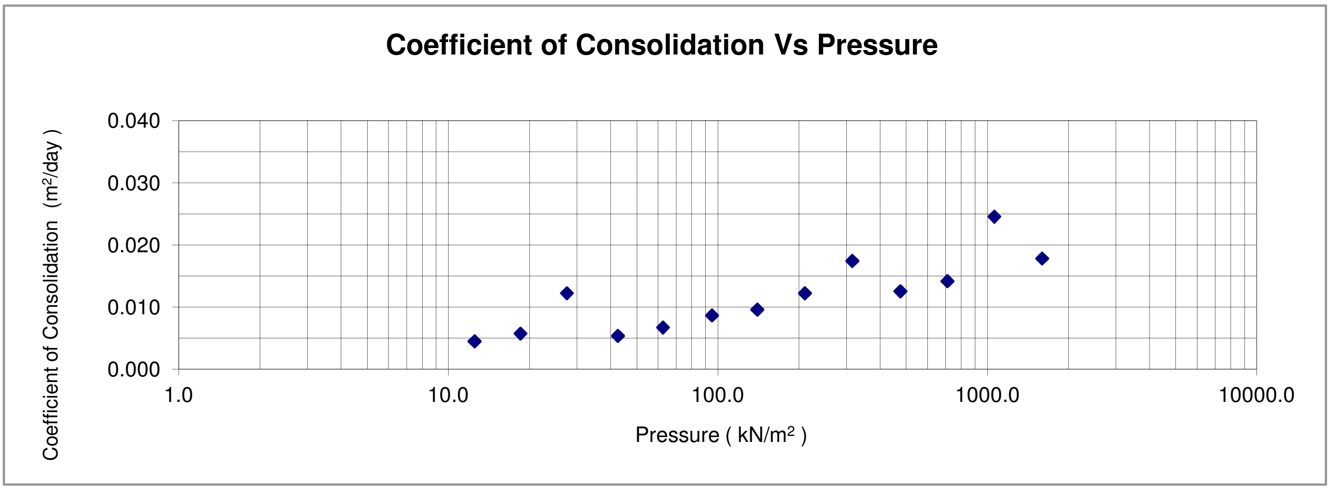
Trial #	36	37	38	39	40		
Load (kPa)	100	50.0	25.0	12.5	6.5		
Load (tsf)	1.04	0.520	0.260	0.130	0.068		
Gauge Reading (in)	0.1819	0.1865	0.1914	0.1974	0.2040		
(H-Hs) mm	4.607	4.726	4.851	5.002	5.170		
Voids ratio	0.362	0.371	0.381	0.393	0.406		
t90 (min)							
Cv (m^2/day)							
k' (MPa)							
Mv (mm^2 / N)							

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP
Client: Hatch Mott MacDonald Limited
Date: 15-Aug-11 **Sample ID:** T10-1_TW16
Job No.: SW8801.1004.101
Depth(m): 18.9

σ'_v versus e and c_v





ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 15-Aug-11 **Sample ID:** T10-1_TW16 **Depth(m):** 18.9

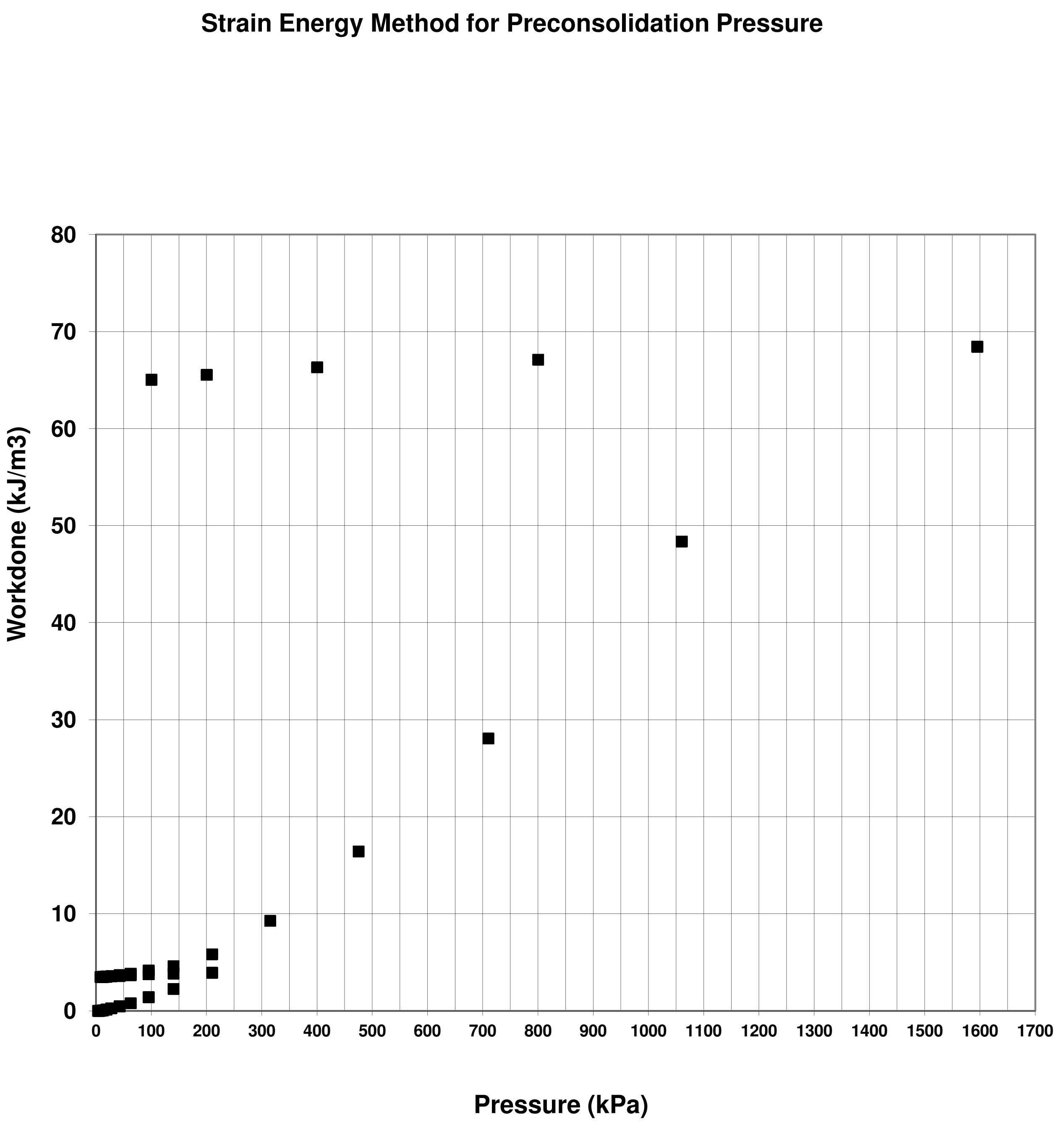
Strain Energy Data

Presssure (kN/m^2)	c_v (m^2/day)	Void ratio
3.5		0.516
5.5		0.514
8.0		0.512
12.5	0.005	0.506
18.5	0.006	0.497
27.5	0.012	0.489
42.5	0.005	0.480
62.5	0.007	0.471
95.0	0.009	0.460
140.0	0.010	0.449
210.0	0.012	0.435
140.0		0.436
95.0		0.437
62.5		0.440
42.5		0.443
27.5		0.444
18.5		0.448
12.5		0.451
8.0		0.456
12.5		0.455
18.5		0.454
27.5		0.451
42.5		0.448
62.5		0.445
95.0		0.442
140.0		0.438
210.0		0.431
315.0	0.017	0.419
475.0	0.013	0.401
710.0	0.014	0.383
1060.0	0.025	0.362
1595.0	0.018	0.339
800.0		0.342
400.0		0.346
200.0		0.352
100.0		0.362
50.0		0.371
25.0		0.381
12.5		0.393
6.5		0.406

Presssure (kN/m^2)	Height mm	Total Work (kJ/m^3)
3.5	19.304	0.000
5.5	19.279	0.006
8.0	19.262	0.012
12.5	19.176	0.057
18.5	19.067	0.146
27.5	18.962	0.272
42.5	18.851	0.477
62.5	18.739	0.790
95.0	18.590	1.414
140.0	18.456	2.265
210.0	18.280	3.927
140.0	18.291	3.842
95.0	18.306	3.777
62.5	18.341	3.675
42.5	18.376	3.608
27.5	18.397	3.582
18.5	18.443	3.543
12.5	18.486	3.519
8.0	18.538	3.491
12.5	18.532	3.495
18.5	18.514	3.518
27.5	18.486	3.570
42.5	18.446	3.683
62.5	18.409	3.841
95.0	18.360	4.154
140.0	18.312	4.614
210.0	18.227	5.834
315.0	18.068	9.285
475.0	17.850	16.432
710.0	17.615	28.081
1060.0	17.346	48.371
1595.0	17.055	68.449
800.0	17.093	67.109
400.0	17.137	66.329
200.0	17.226	65.555
100.0	17.344	65.039
50.0	17.463	64.782
25.0	17.588	64.648
12.5	17.739	64.566
6.5	17.907	64.536

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP
Client: Hatch Mott MacDonald Limited
Date: 15-Aug-11
Sample ID: T10-1_TW16
Job No.: SW8801.1004.101
Depth(m): 18.9



ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project:	WEP	**Job No.:**	SW8801.1004.101
Client:	Hatch Mott MacDonald Limited		
Date:	15-Aug-11	**Sample ID:** T10-1_TW16	**Depth(m):** 18.9

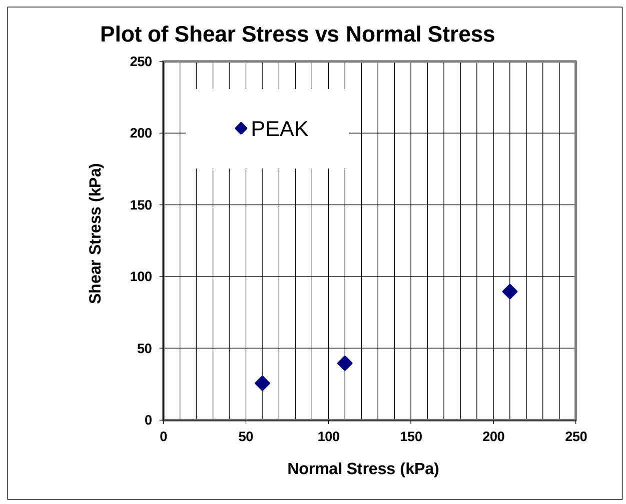
DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**24 August 2011**
Sample ID.:	**T10-1_Sa16**	**Tested By:**	**FC/SB**
Lab No.:	**AdS056_2011**	**Checked By:**	**SB**

Specimen ID	**1**	**2**	**3**
Date of Test	**15-Aug-11**	**18-Aug-11**	**23-Aug-11**
Normal Stress (kPa)	**60**	**110**	**210**
Rate of displacement (mm/min)	0.02	0.02	0.03
Initial thickness of specimen (mm)	24.10	24.10	24.10
Initial diameter of specimen (mm)	63.30	63.30	63.30
Initial moisture content (%)	18.9	19.6	18.7
Density (kN/m^3)	20.8	20.7	21.1
Final moisture (%)	19.4	18.4	16.8

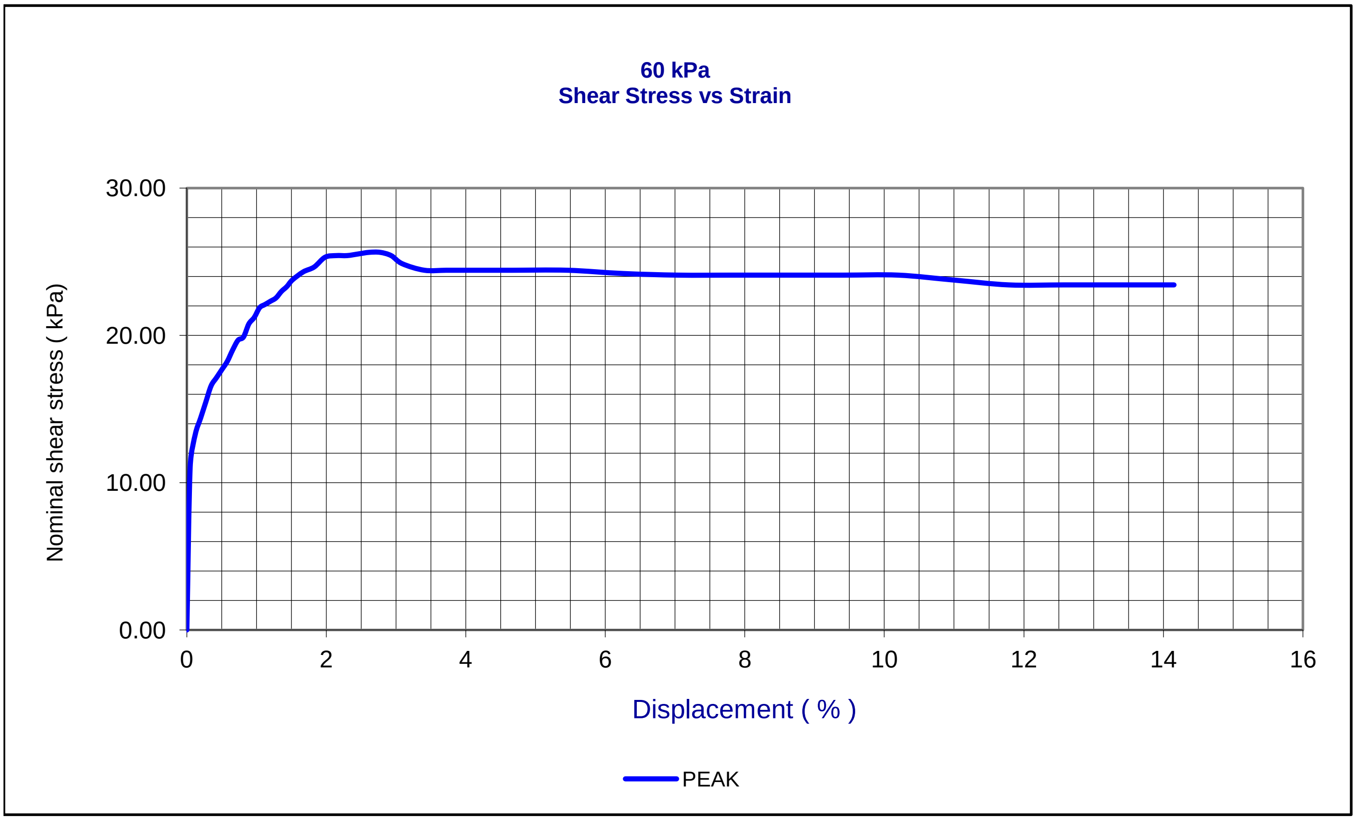
Specimen ID	**Normal Stress**	**Peak Shear Stress**
	kPa	**kPa**
1	60.0	25.6
2	110.0	39.6
3	210.0	89.5

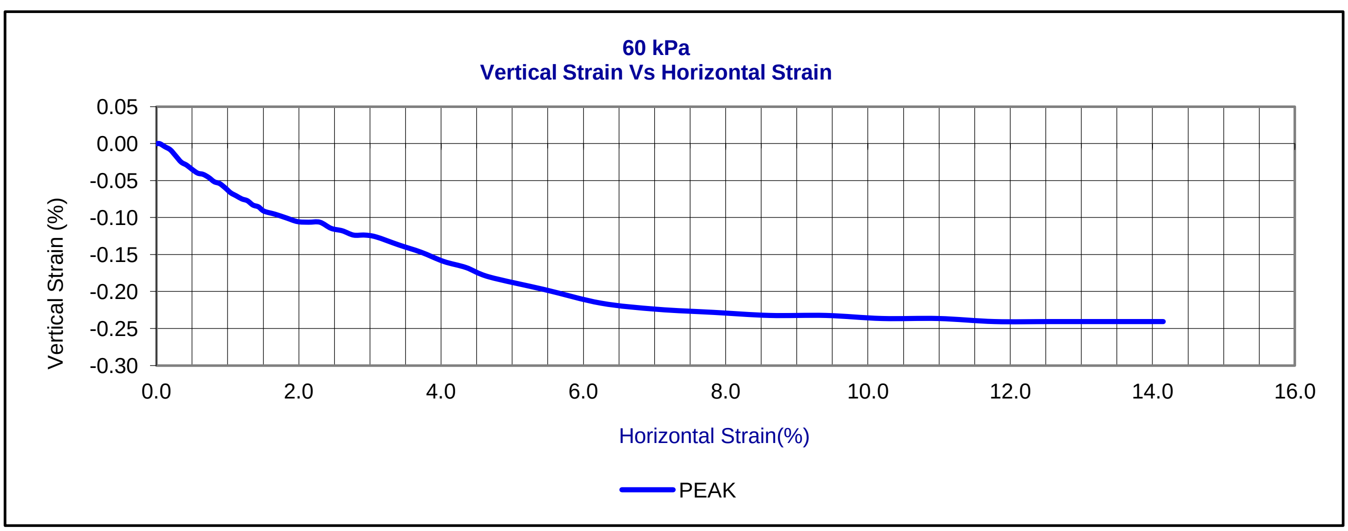
Note: Test specimens were inundated with water.



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

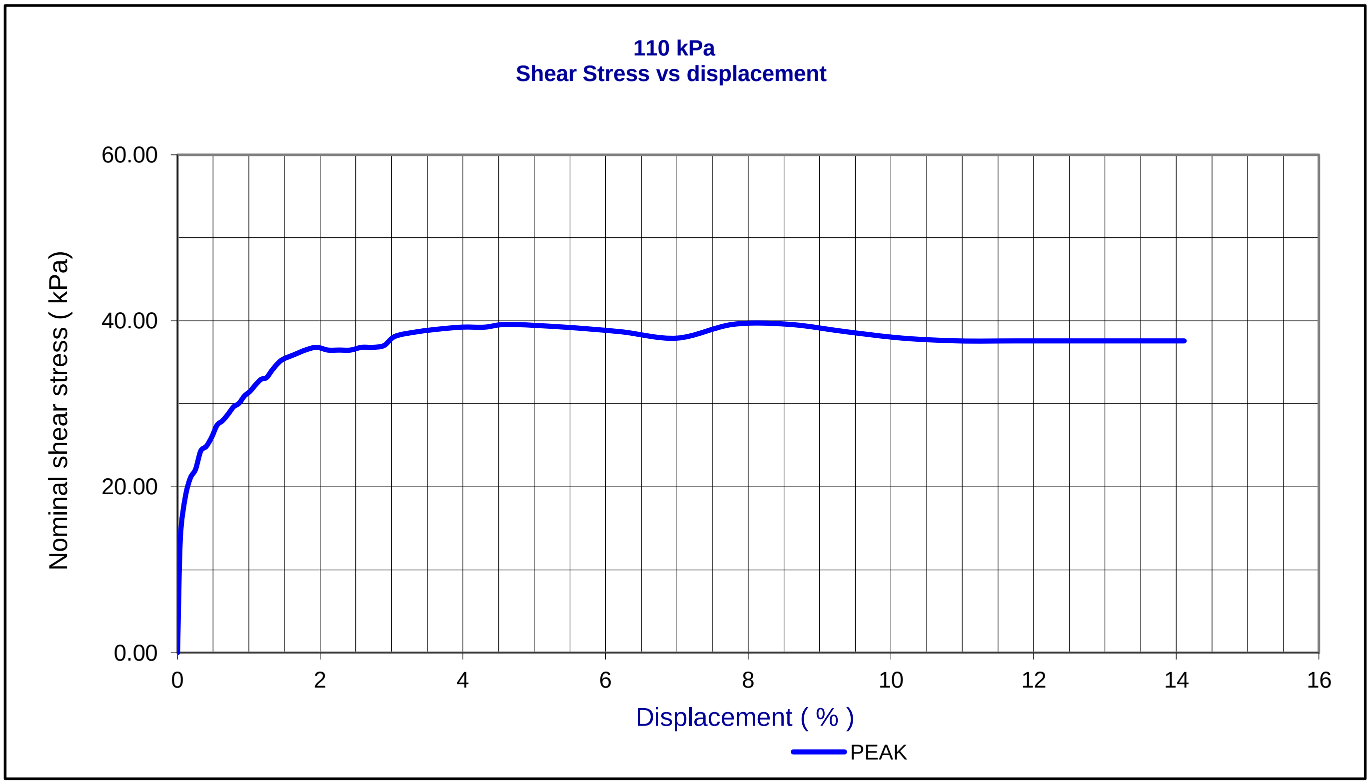
Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**24-August-2011**
Sample ID.:	**T10-1_Sa16**	**Tested By:**	**FC/SB**
Lab No.:	**AdS056_2011**	**Checked By:**	**SB**

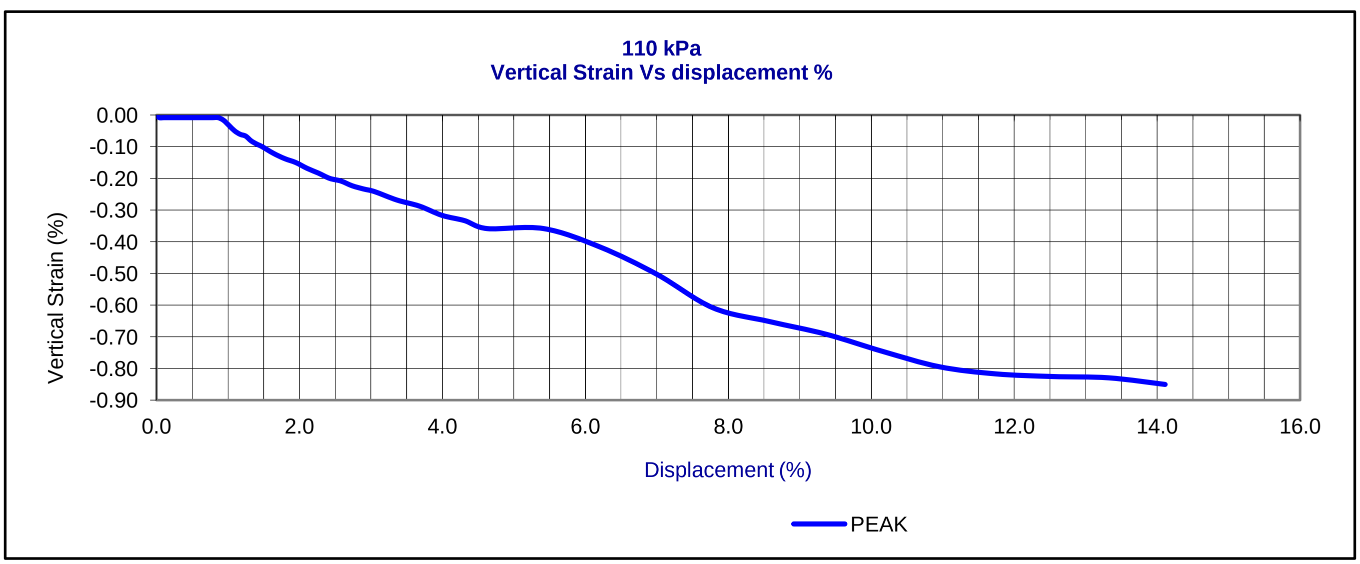




DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

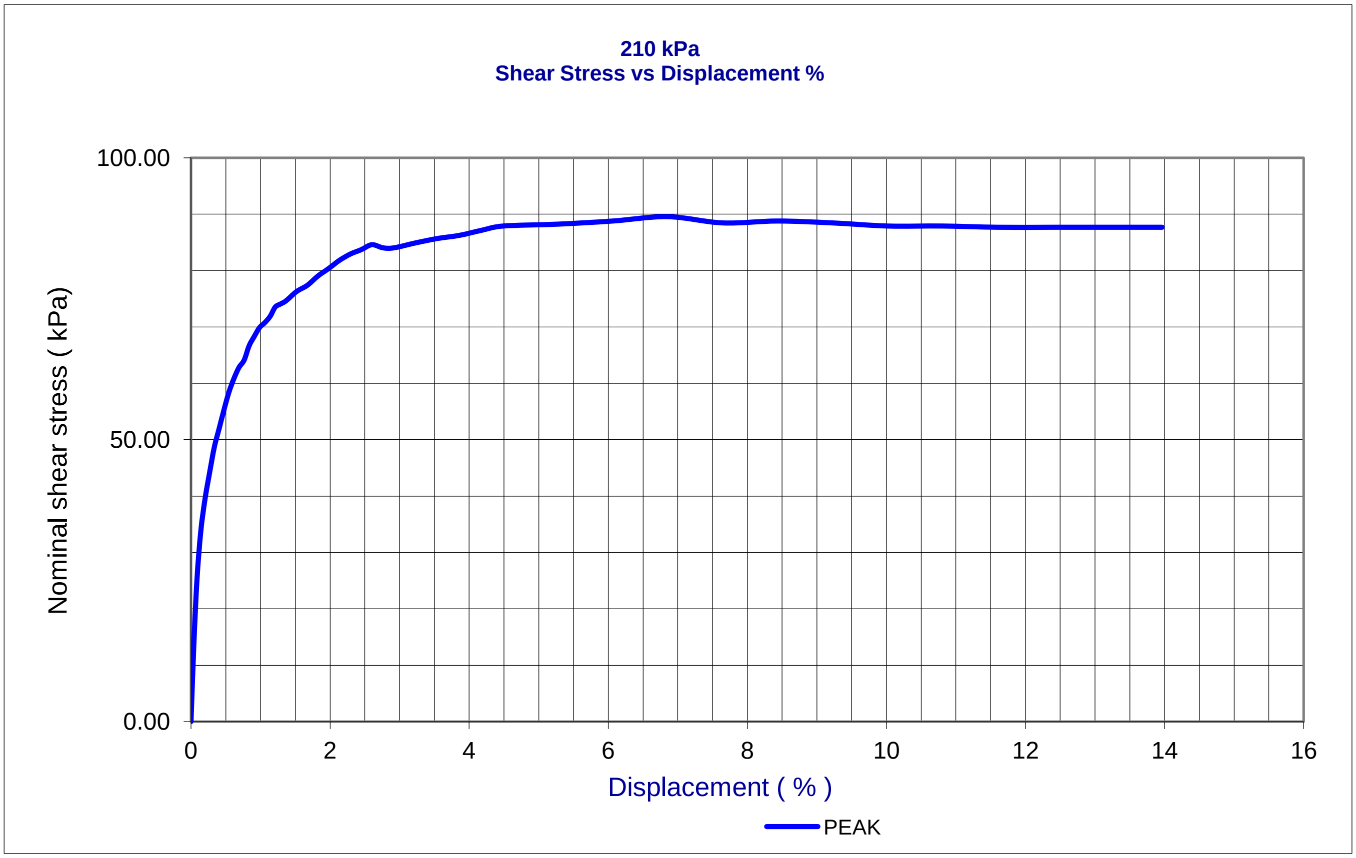
Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**24-August-2011**
Sample ID.:	**T10-1_Sa16**	**Tested By:**	**FC/SB**
Lab No.:	**AdS056_2011**	**Checked By:**	**SB**

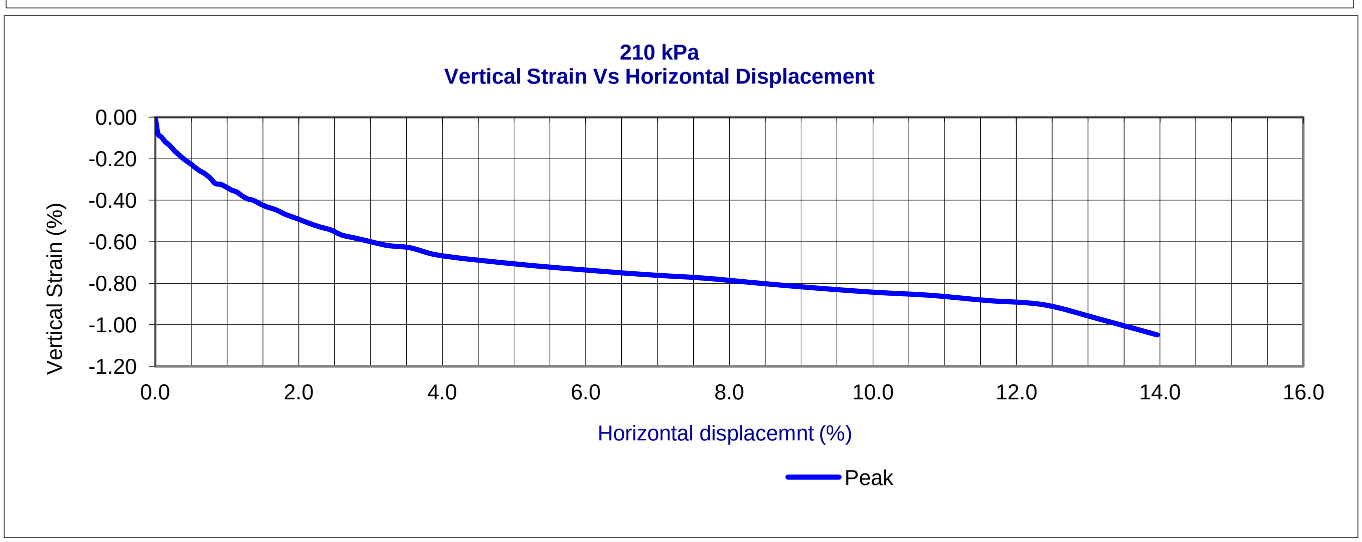




DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

Project:-	**WEP**	**Job#:**	**SW8801.1004.101**
Client:-	**Hatch Mott MacDonald Limited**	**Date:**	**24 August 2011**
Sample ID.:	**T10-1_Sa16**	**Tested By:**	**FC/SB**
Lab No.:	**AdS056_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: 14-MAY-11
Report Date: 19-MAY-11 14:29 (MT)
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1005225

Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 092960

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

		L1005225-1	L1005225-2	L1005225-3	L1005225-4	L1005225-5
	Sample ID	L1005225-1	L1005225-2	L1005225-3	L1005225-4	L1005225-5
	Description	SOIL	SOIL	SOIL	SOIL	SOIL
	Sampled Date	13-MAY-11	13-MAY-11	13-MAY-11	13-MAY-11	13-MAY-11
	Sampled Time					
	Client ID	BH: T2-1 SA#24 @ 100'	BH: T5-1 SA#25 @ 100'	BH: T5-3 SA#11 @ 35'	BH: B7-1 SA#6 @ 15'	BH: B7-2 SA#29 @ 100'
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	9.11	22.3	20.8	17.3	22.0
	pH (pH units)	7.83	7.83	8.01	7.95	7.84
	Redox Potential (mV)	330	246	234	232	238
	Resistivity (ohm cm)	2250	1900	1830	2380	1530
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20	<0.20	<0.20	<0.20	<0.20
Anions and Nutrients	Sulphate (mg/kg)	389	582	520	451	897

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID Description Sampled Date Sampled Time Client ID	L1005225-6 SOIL 13-MAY-11 BH: T10-2 SA#7 @ 17.5'	L1005225-7 SOIL 13-MAY-11 BH: T2-2 SA#11 @ 35'	L1005225-8 SOIL 13-MAY-11 BH: T5-2 SA#24 @ 90'	L1005225-9 SOIL 13-MAY-11 BH: B7-3 SA#11 @35'	
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	12.0	14.9	9.76	18.0	
	pH (pH units)	7.86	7.84	7.89	8.13	
	Redox Potential (mV)	228	230	260	275	
	Resistivity (ohm cm)	3620	2330	1860	2130	
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20	<0.20	<0.20	<0.20	
Anions and Nutrients	Sulphate (mg/kg)	180	444	387	390	

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:

092960

GLOSSARY OF REPORT TERMS

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

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

Quality Control Report

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

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT	**Soil**							
Batch R2189775								
WG1279866-2 LCS								
% Moisture			92		%		70-130	16-MAY-11
WG1279866-1 MB								
% Moisture			<0.10		%		0.1	16-MAY-11
PH-WT	**Soil**							
Batch R2191152								
WG1281781-1 CVS								
pH			99		%		80-120	18-MAY-11
WG1281781-2 DUP		**L1005225-1**						
pH		7.83	7.85		pH units	0.26	20	18-MAY-11
REDOX-POTENTIAL-WT	**Soil**							
Batch R2191357								
WG1282006-1 DUP		**L1005225-2**						
Redox Potential		246	254		mV	3.2	25	19-MAY-11
RESISTIVITY-WT	**Soil**							
Batch R2191349								
WG1282003-1 CVS								
Resistivity			101		%		70-130	19-MAY-11
WG1282003-2 DUP		**L1005225-2**						
Resistivity		1900	1840		ohm cm	3.4	25	19-MAY-11
SO4-WT	**Soil**							
Batch R2191378								
WG1280978-3 LCS								
Sulphate			96		%		60-140	18-MAY-11
WG1280978-1 MB								
Sulphate			<20		mg/kg		20	18-MAY-11
SULPHIDE-WT	**Soil**							
Batch R2190625								
WG1281156-1 CVS								
Sulphide			104		%		50-120	18-MAY-11
WG1281151-2 DUP		**L1005225-4**						
Sulphide		<0.20	<0.20	RPD-NA	mg/kg	N/A	20	18-MAY-11
WG1281151-1 MB								
Sulphide			<0.20		mg/kg		0.2	18-MAY-11

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential							
	1	13-MAY-11	19-MAY-11 15:26	24	147	hours	EHTL
	2	13-MAY-11	19-MAY-11 15:27	24	147	hours	EHTL
	3	13-MAY-11	19-MAY-11 15:29	24	148	hours	EHTL
	4	13-MAY-11	19-MAY-11 15:30	24	148	hours	EHTL
	5	13-MAY-11	19-MAY-11 15:31	24	148	hours	EHTL
	6	13-MAY-11	19-MAY-11 15:32	24	148	hours	EHTL
	7	13-MAY-11	19-MAY-11 15:33	24	148	hours	EHTL
	8	13-MAY-11	19-MAY-11 15:34	24	148	hours	EHTL
	9	13-MAY-11	19-MAY-11 15:35	24	148	hours	EHTL

Legend & Qualifier Definitions:

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

Notes*:
Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1005225 were received on 14-MAY-11 09:45.

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

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

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

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

ALS Environmental

CHAIN OF CUSTODY / ANALYTICAL SERVICES REQUEST FORM

C of C # 092960
PAGE 1 OF 1

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

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

COMPANY NAME: Amec E+E
OFFICE: Windsor
PROJECT MANAGER: Shone MacLeod
PROJECT #: SW8401.1001.101
PHONE: 519 735-2499 FAX: 519 735 [illegible]
ACCOUNT #:
QUOTATION#: Q28643 PO#:

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

REPORT FORMAT / DISTRIBUTION
EMAIL ✓ FAX ____ BOTH ____
SELECT: PDF ____ DIGITAL ____ BOTH ____
EMAIL1 Shone.MacLeod@amec.com
EMAIL2

ANALYSIS REQUEST: Corrosion Package

PLEASE INDICATE FILTERED, PRESERVED OR BOTH (F, P, F/P)
SUBMISSION #: L1005225
ENTERED BY: PStasAny
DATE/TIME ENTERED: 16-MAY-11
BIN #:

SAMPLING INFORMATION

Date (dd-mm-yy)	Time (24 hr) (hh:mm)	COMP	GRAB	WATER	SOIL	OTHER	SAMPLE DESCRIPTION TO APPEAR ON REPORT	NUMBER OF CONTAINERS	Corrosion Package	COMMENTS	LAB ID
May 13, 11					X		BH: T2-1 Sa#24 @ 100'		✓		-1
"					X		BH: T5-1 Sa#25 @ 100'		✓		-2
"					X		BH: T5-3 Sa#11 @ 35'		✓		-3
"					X		BH: B7-1 Sa#6 @ 15'		✓		-4
"					X		BH: B7-2 Sa#29 @ 100'		✓		-5
"					X		BH: T10-2 Sa#7 @ 17.5'		✓		-6
"					X		BH: T2-2 Sa#11 @ 35'		✓		-7
"					X		BH: T5-2 Sa#24 @ 90'		✓		-8
"					X		BH: B7-3 Sa#11 @ 35'		✓		-9

SPECIAL INSTRUCTIONS/COMMENTS

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

SAMPLE CONDITION
FROZEN ☐
COLD ☐
COOLING INITIATED ☑
AMBIENT ☐
MEAN TEMP: 13.8
OBSERVATIONS Yes ☐ No ☐
If yes add SIF
INIT: AP

SAMPLED BY: Justin Palmer DATE & TIME: May [illegible]
RECEIVED BY: DATE & TIME:
RELINQUISHED BY: DATE & TIME:
RECEIVED AT LAB BY: Austin P DATE & TIME: 14/5/11 09:45

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

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

Appendix E Slope Stability Analyses

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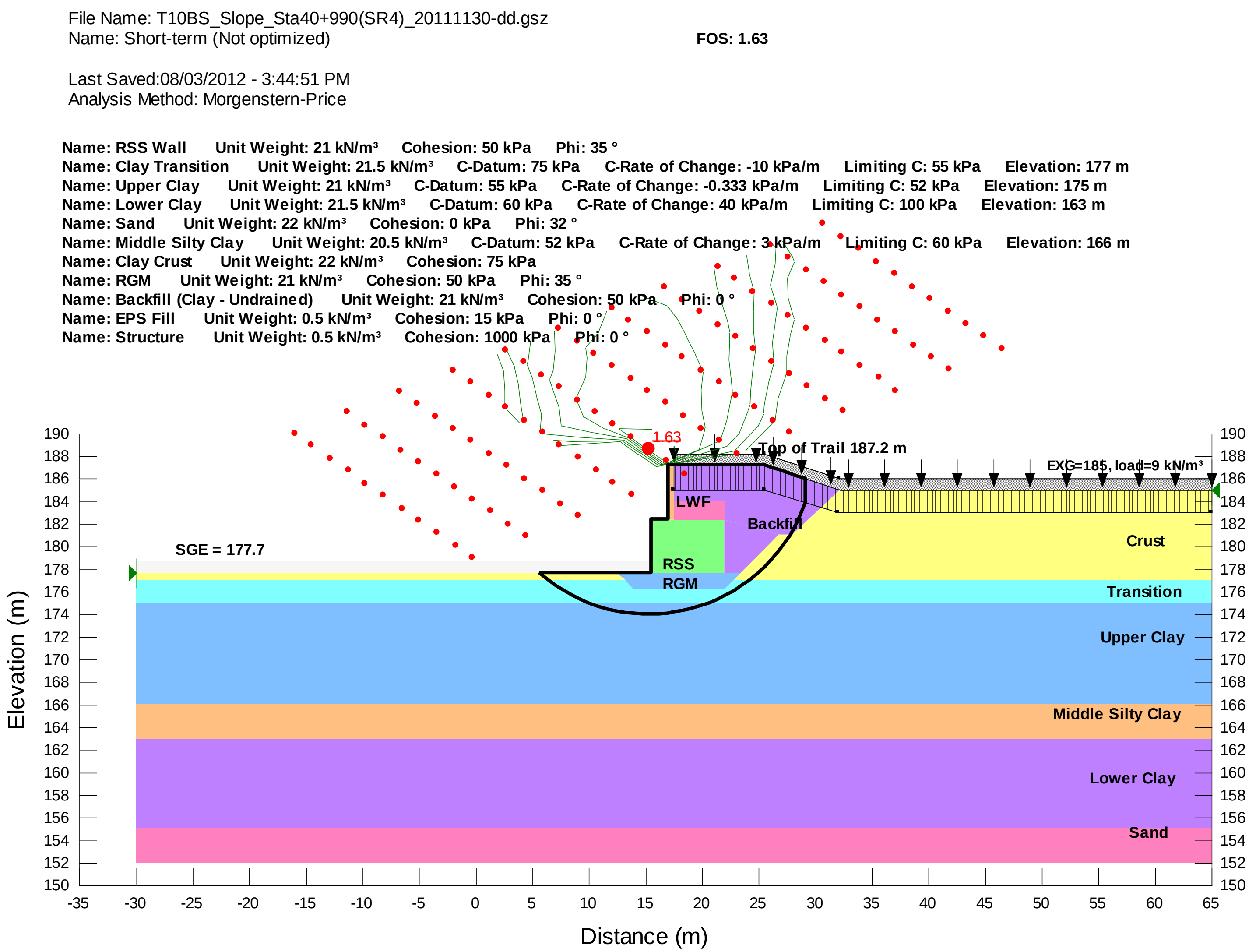
PARKWAY INFRASTRUCTURE CONSTRUCTORS

Figure E-1: Global Stability Result – South Abutment Sta 40+990 – Short-Tem (Undrained properties)



Project:	Windsor-Essex Parkway	**Date:**	March/2012
Document:	Geotechnical Investigation and Design Report Tunnel T-10B (40+840 to 41+000 – SR4)	**Rev:**	0
Doc No.:	285380-04-119-0004 (Geocres No. 40J3-12)	**Page No.:**	1 of 9

Figure E-2: Global Stability Result – South Abutment Sta 40+990 – End of Construction Loading (Undrained properties)

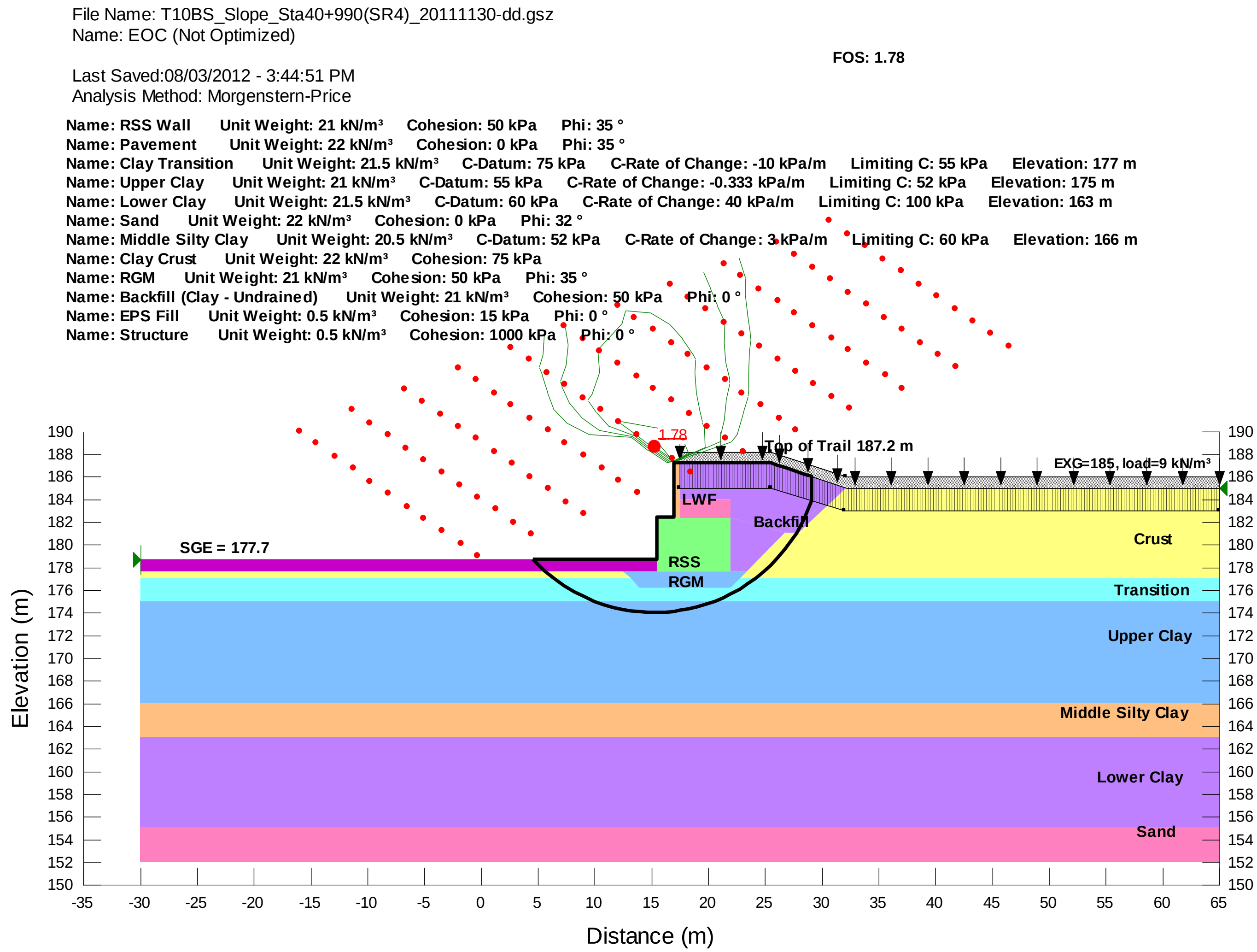
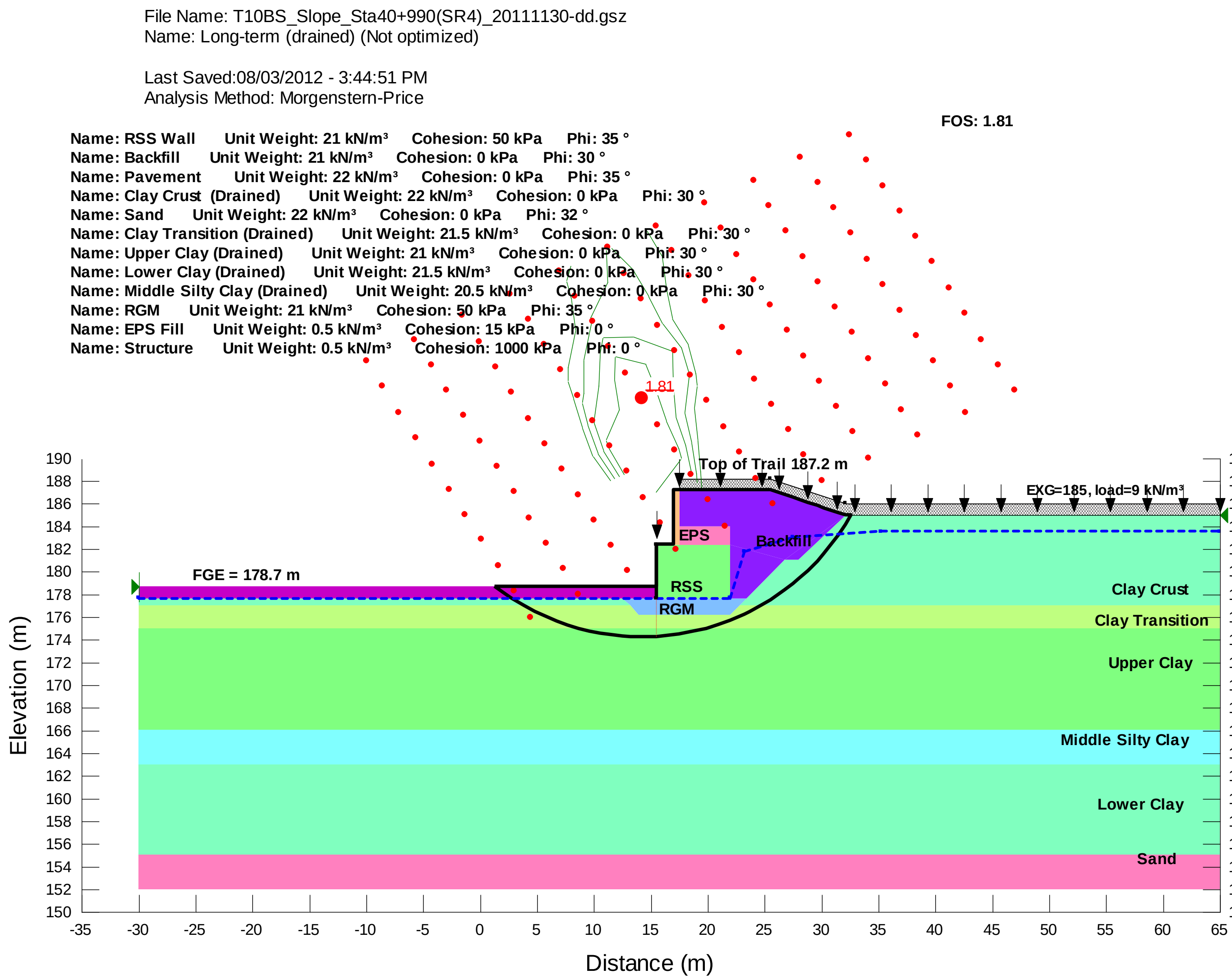


Figure E-3: Global Stability Result – South Abutment Sta 40+990 – Long-term Loading (Drained properties)



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PARKWAY INFRASTRUCTURE CONSTRUCTORS

Figure E-4: Global Stability Result – North Abutment Sta. 40+915 – Short Term (Undrained properties)

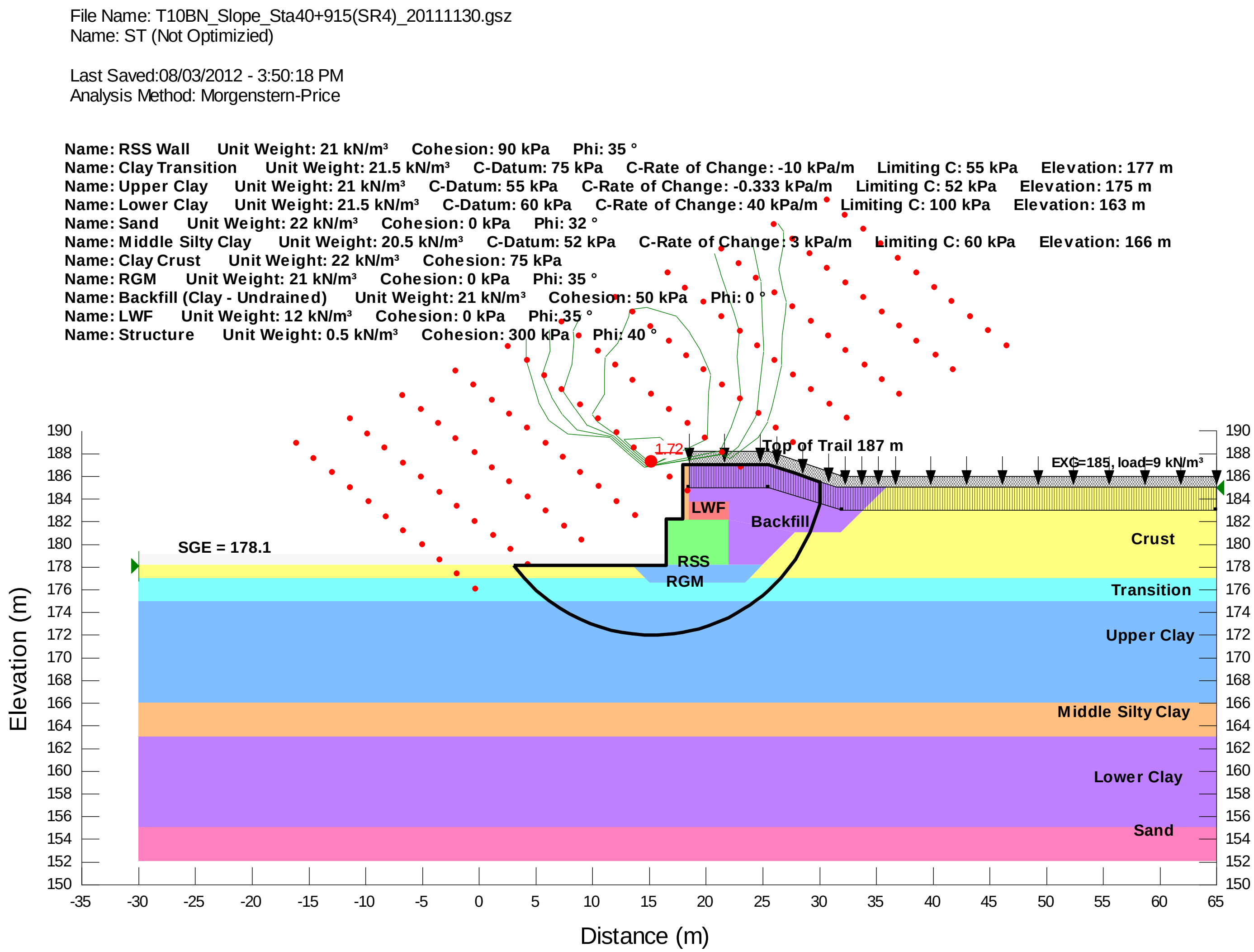


Figure E-5: Global Stability Result – North Abutment Sta. 40+915 – End of Construction Loading (Undrained properties)

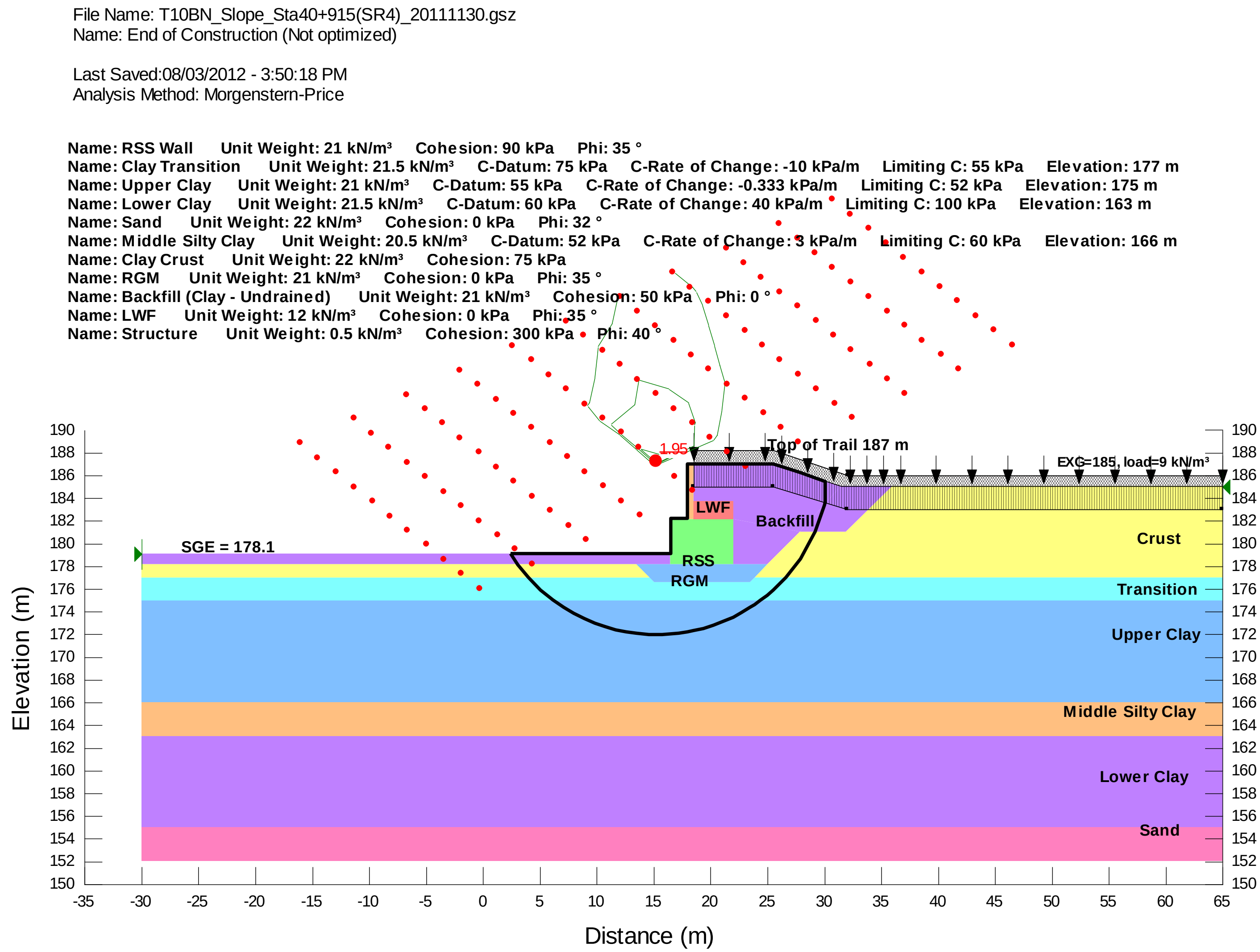
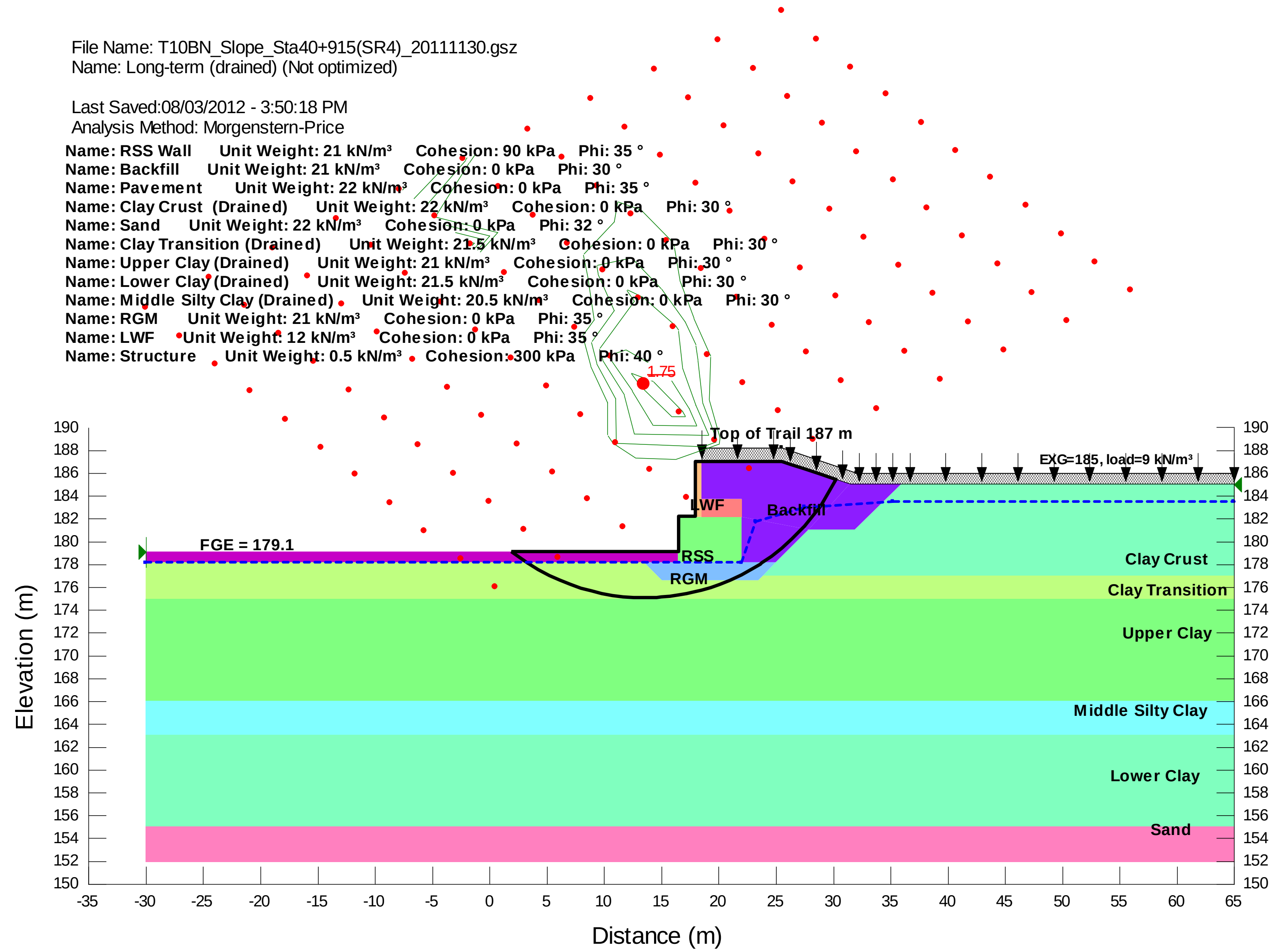


Figure E-6: Global Stability Result – North Abutment Sta. 40+915 – Long-Term (Drained properties)



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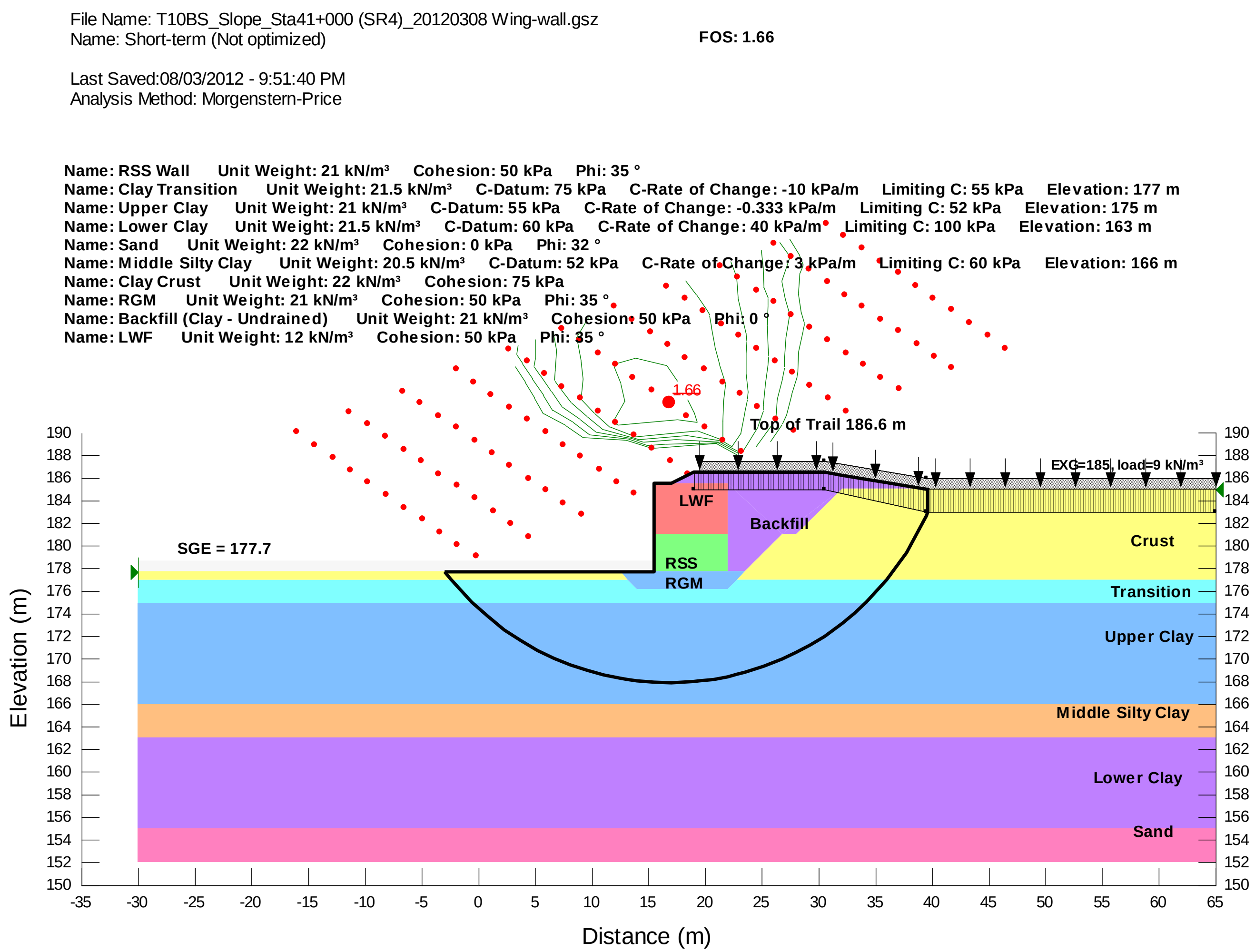
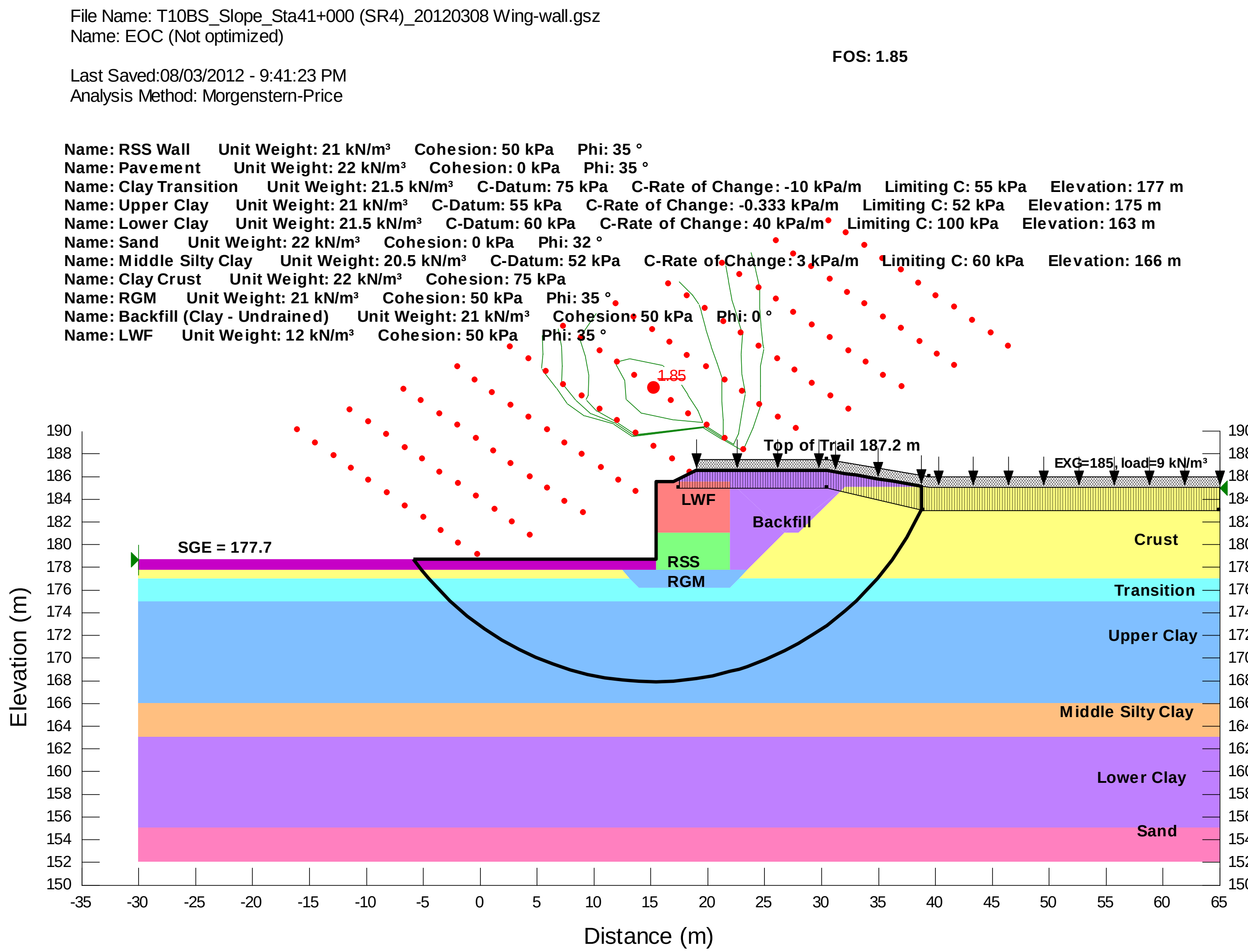
PARKWAY INFRASTRUCTURE CONSTRUCTORS

Figure E-7: Global Stability Result – Wing Wall Southeast– Short Term Loading (Undrained properties)



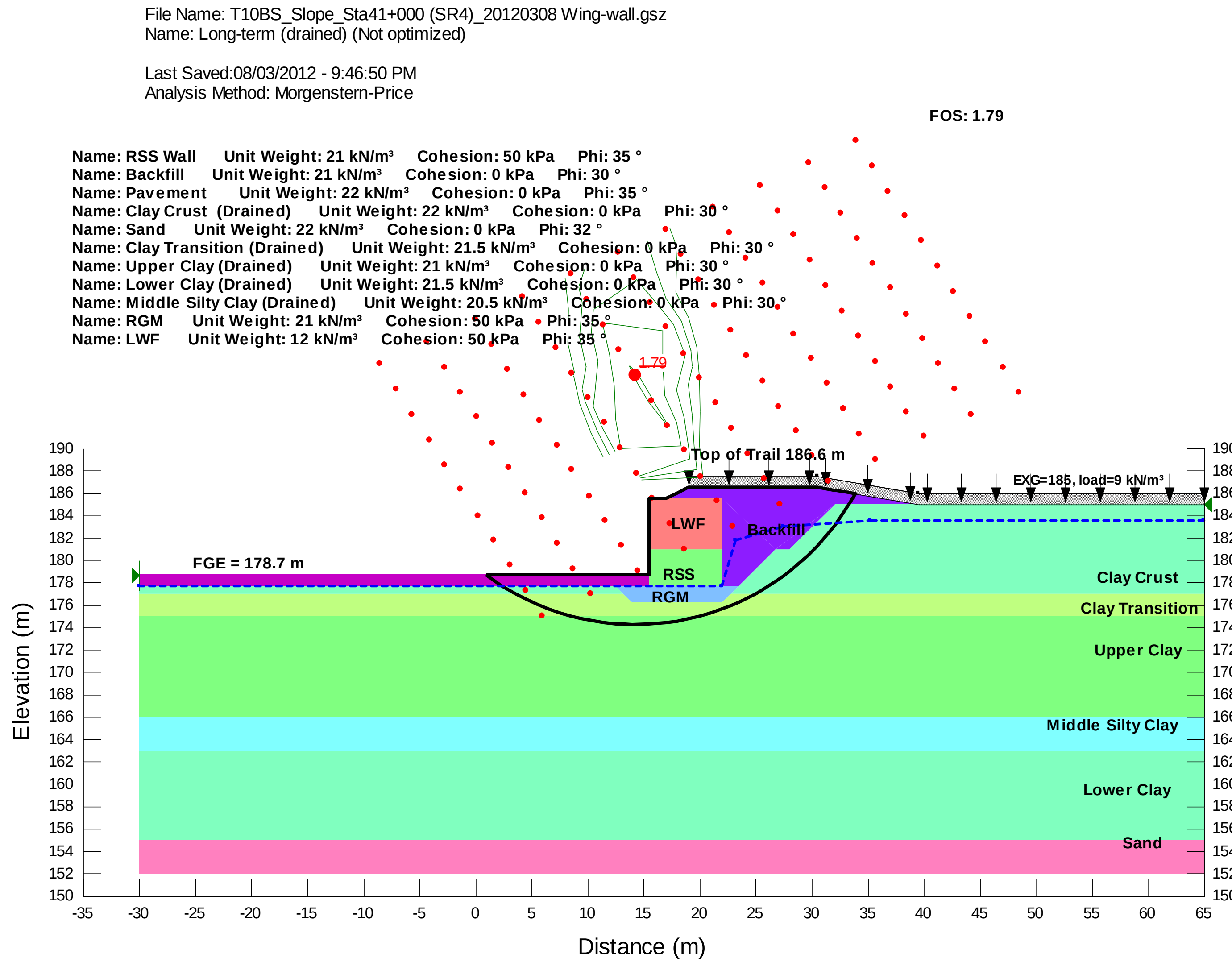
Project:	Windsor-Essex Parkway	**Date:**	March/2012
Document:	Geotechnical Investigation and Design Report Tunnel T-10B (40+840 to 41+000 – SR4)	**Rev:**	0
Doc No.:	285380-04-119-0004 (Geocres No. 40J3-12)	**Page No.:**	7 of 9

Figure E-8: Global Stability Result – Wing Wall Southeast – End of Construction Loading (Undrained properties)



Project:	Windsor-Essex Parkway	**Date:**	March/2012
Document:	Geotechnical Investigation and Design Report Tunnel T-10B (40+840 to 41+000 – SR4)	**Rev:**	0
Doc No.:	285380-04-119-0004 (Geocres No. 40J3-12)	**Page No.:**	8 of 9

Figure E-9: Global Stability Result – Wing Wall Southeast – Long-Term (Drained properties)



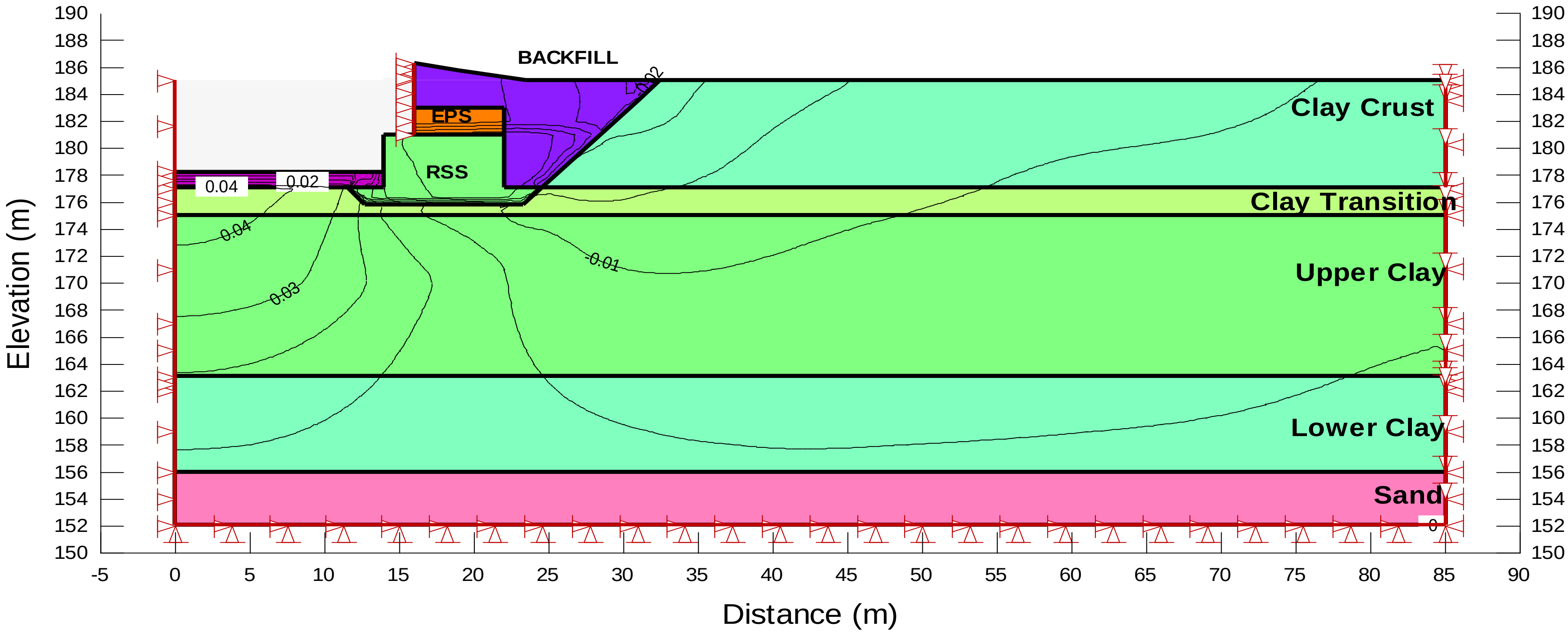
Project:	Windsor-Essex Parkway	**Date:**	March/2012
Document:	Geotechnical Investigation and Design Report Tunnel T-10B (40+840 to 41+000 – SR4)	**Rev:**	0
Doc No.:	285380-04-119-0004 (Geocres No. 40J3-12)	**Page No.:**	9 of 9

Appendix F Stress Deformation Analyses

Tunnel T10-B South Abutment
Stress-Deformaton Analysis
Cummulative Heave/Settlement (m)
End of Excavation
18/07/2011

WEP- SW8801.1002.101

Name: RSS Backfill Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Backfill Young's Modulus (E): 20000 kPa Poisson's Ratio: 0.49 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³
Name: Pavement Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Effective Young's Modulus (E'): 31500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Sand Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition (Drained) Effective Young's Modulus (E'): 17550 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Upper Clay (Drained) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.012 Initial Void Ratio: 0.61 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clay (Drained) Effective Young's Modulus (E'): 26000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: EPS Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m³ Poisson's Ratio: 0.49



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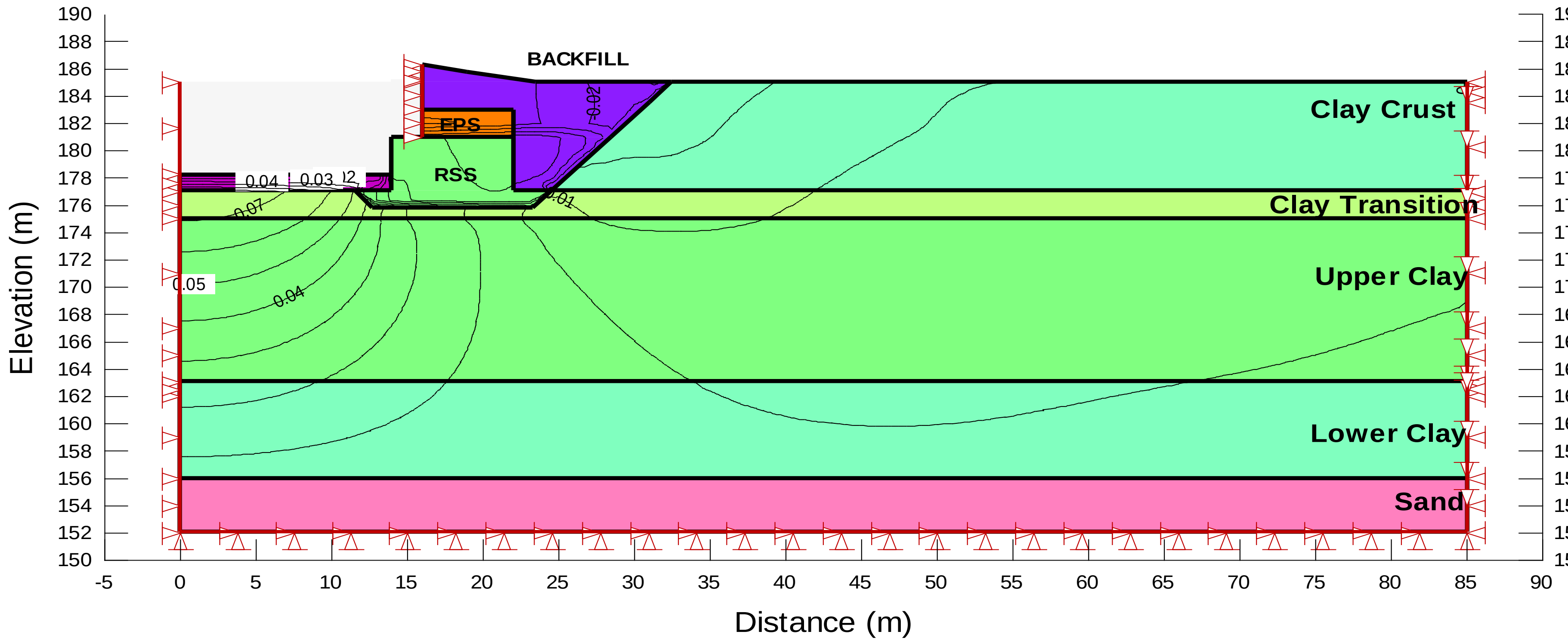
PARKWAY INFRASTRUCTURE CONSTRUCTORS

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	CUMMULATIVE HEAVE/SETTLEMENT AT EOC
DATE: Jul 2011	JOB NO.:
CAD FILE:	FIGURE NO.: F.1
REV.:	

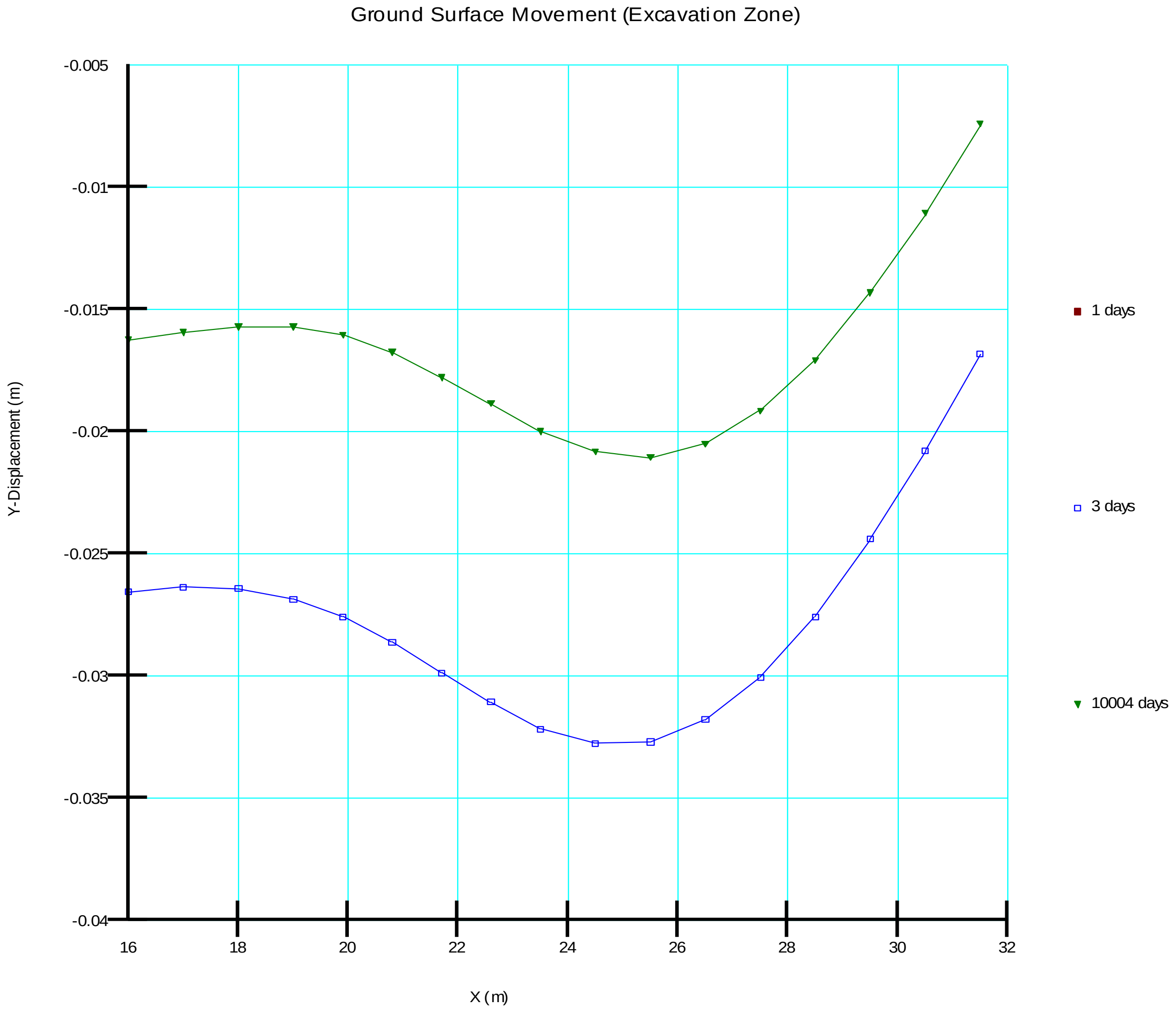
Tunnel T-10B South Abutment
Stress-Deformaton Analysis
Cummulative Heave/Settlement (m)
Long-Term
18/07/2011

WEP- SW8801.1002.101

Name: RSS Backfill Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Backfill Young's Modulus (E): 20000 kPa Poisson's Ratio: 0.49 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³
Name: Pavement Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Effective Young's Modulus (E'): 31500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Sand Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition (Drained) Effective Young's Modulus (E'): 17550 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Upper Clay (Drained) O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.012 Initial Void Ratio: 0.61 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clay (Drained) Effective Young's Modulus (E'): 26000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: EPS Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m³ Poisson's Ratio: 0.49



PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	CUMMULATIVE HEAVE/SETTLEMENT LONG-TERM
DATE: Jul 2011	JOB NO.:
CAD FILE:	FIGURE NO.: F.2
REV.:	



Legend:
1 days = End of Excavation (EOE)
3 days = Completion of Abutment
10004 days = Long-term Condition

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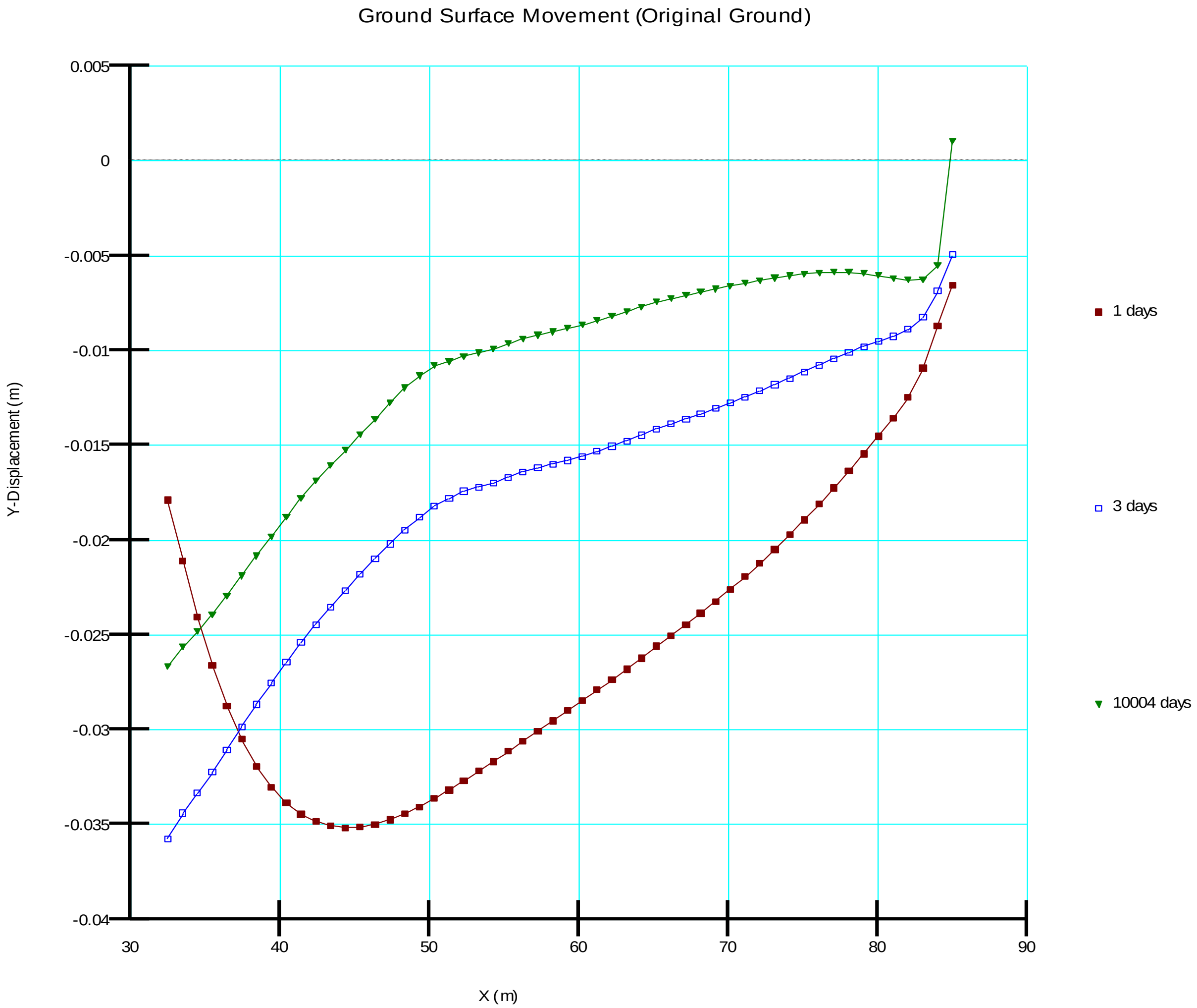
PARKWAY INFRASTRUCTURE CONSTRUCTORS

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: GROUND SETTLEMENT AT APPROACH EMBANKMENT - EXCAVATION ZONE				
DATE: Jul 2011	JOB NO.:	CAD FILE:	FIGURE NO.: F.3A	REV.



Legend:
1 days = End of Excavation (EOE)
3 days = Completion of Abutment
10004 days = Long-term Condition

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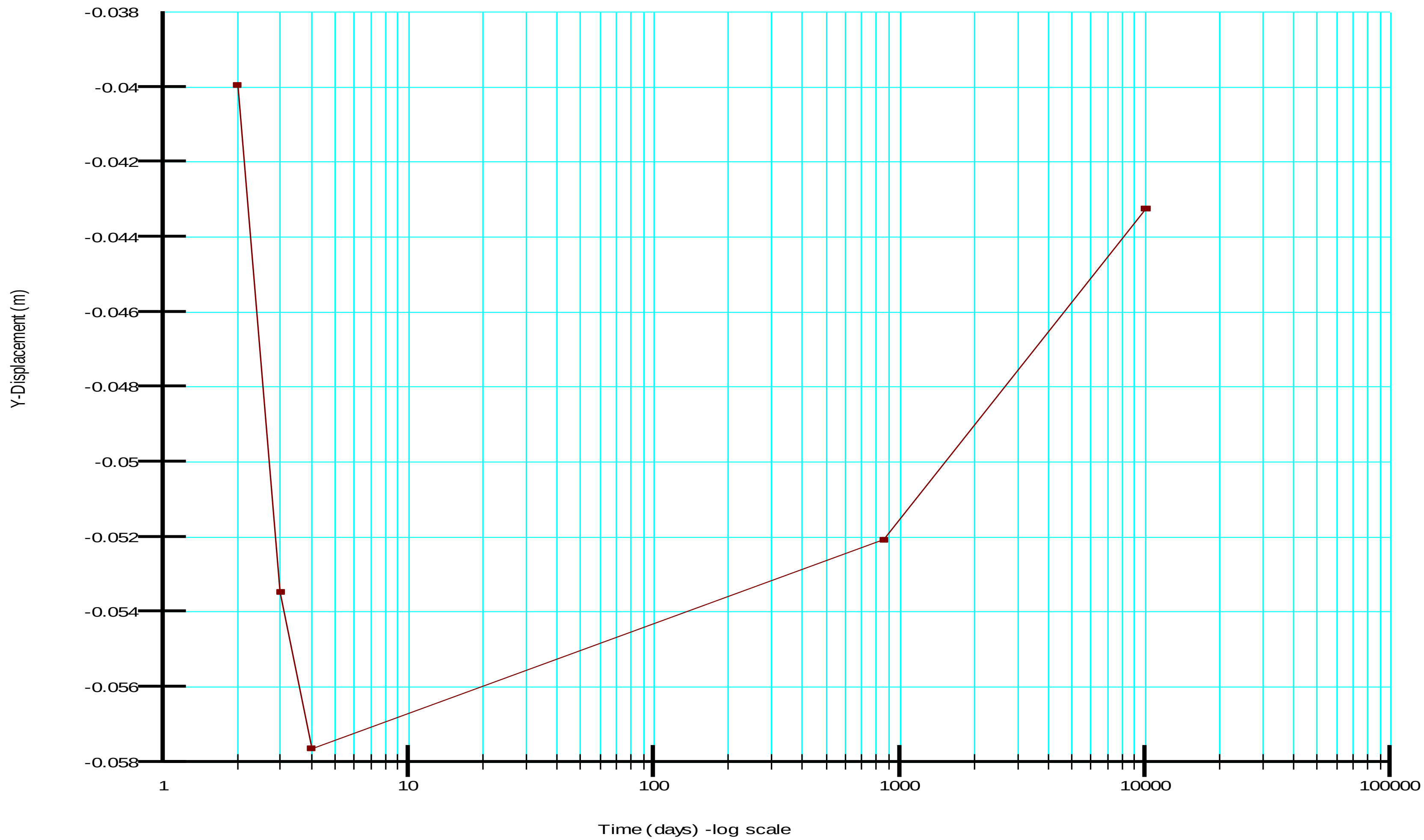
PARKWAY INFRASTRUCTURE CONSTRUCTORS

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: GROUND SETTLEMENT AT APPROACH EMBANKMENT - ORIGINAL GROUD SOUTH ABUTMENT				
DATE: Jul 2011	JOB NO.:	CAD FILE:	FIGURE NO.: F.3B	REV.

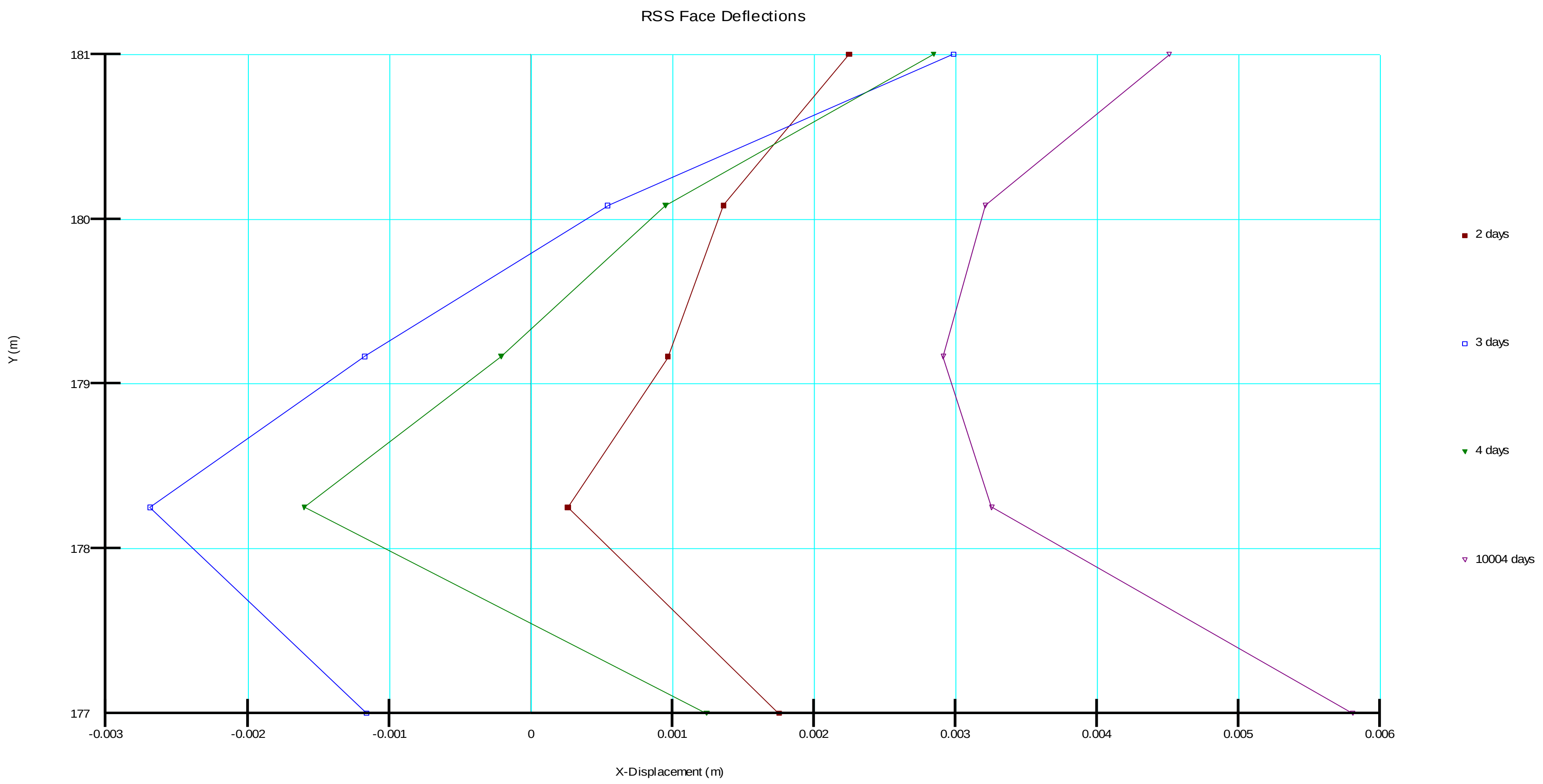
Top Of RSS Facing Movement



2 days = RSS Completion
3 days = Completion of Abutment
4 days = End of Construction (EOC)
858 = Intermediate result
10004 days = Long-term Condition

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	TOP OF RSS WALL FACING MOVEMENT

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.4	REV.
Jul 2011				



Legend:
2 days = RSS Completion
4 days = End of Construction
5 days = EOC
10004 days = Long-term Condition

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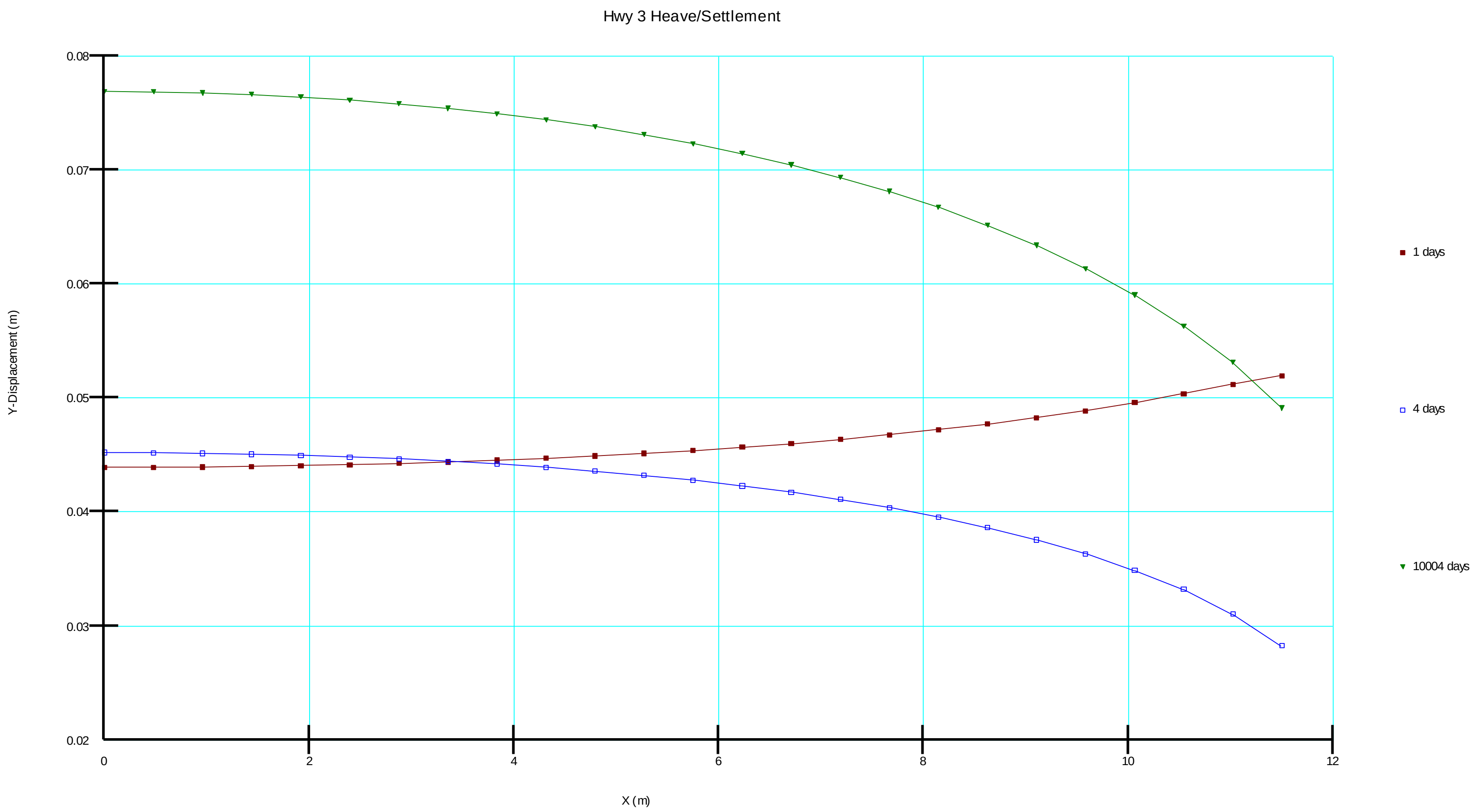
PARKWAY INFRASTRUCTURE CONSTRUCTORS

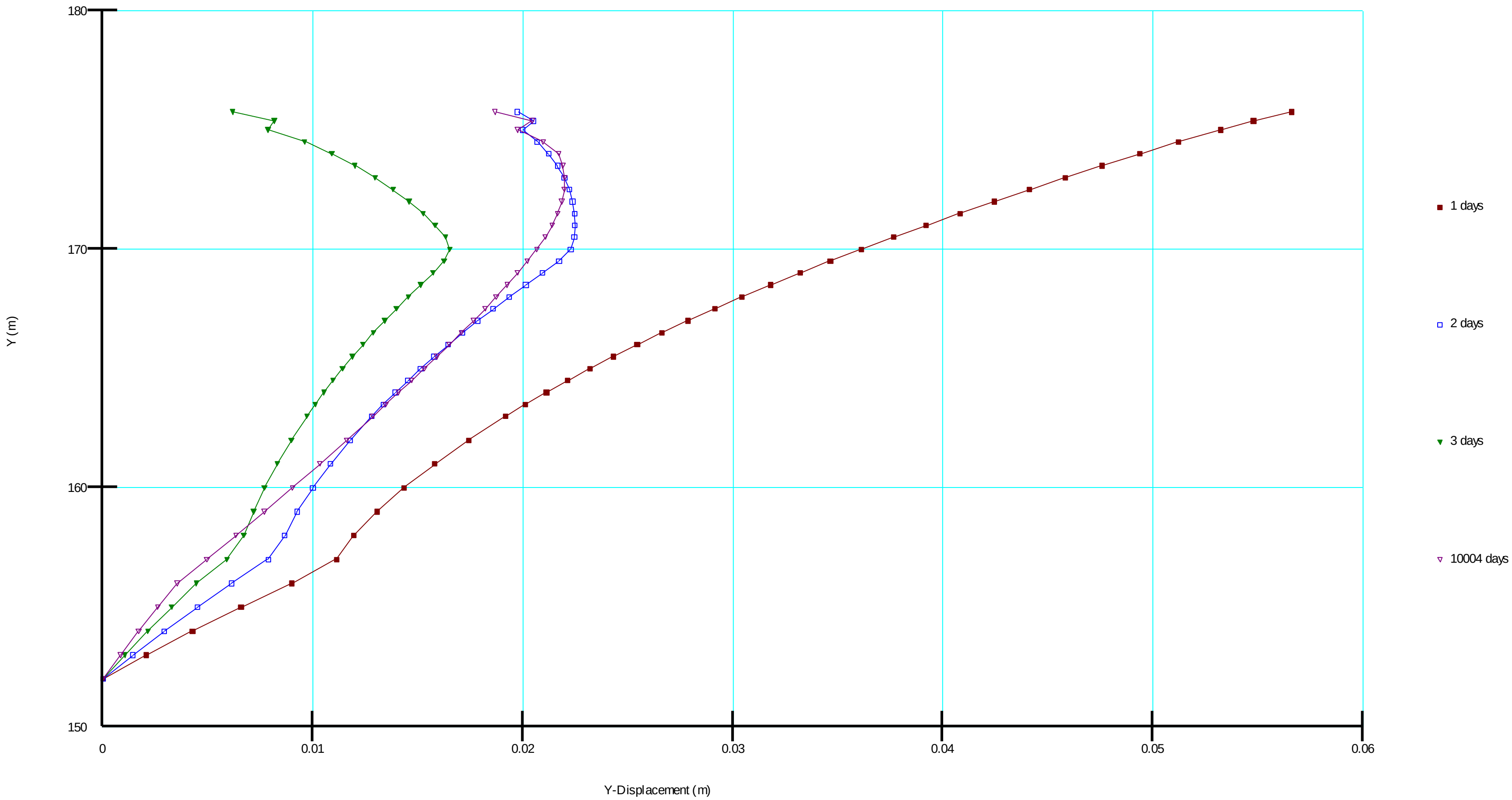
PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: LATERAL DEFLECTION OF RSS WALL TUNNEL T-!0B				
DATE: Jul 2011	JOB NO.:	CAD FILE:	**FIGURE NO.: F.5**	REV.



1 day = EOE
4 days = EOC
10054 = Long-Term Condition

Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PARKWAY INFRASTRUCTURE CONSTRUCTORS	PROJECT: **WINDSOR ESSEX PARKWAY**			
		TITLE: HIGHWAY 3 SETTLEMENT/HEAVE TUNNEL T-10B			
		DATE: Jul 2011	JOB NO.:	CAD FILE:	**FIGURE NO.: F.6** REV.:

Soil Profile Settlement

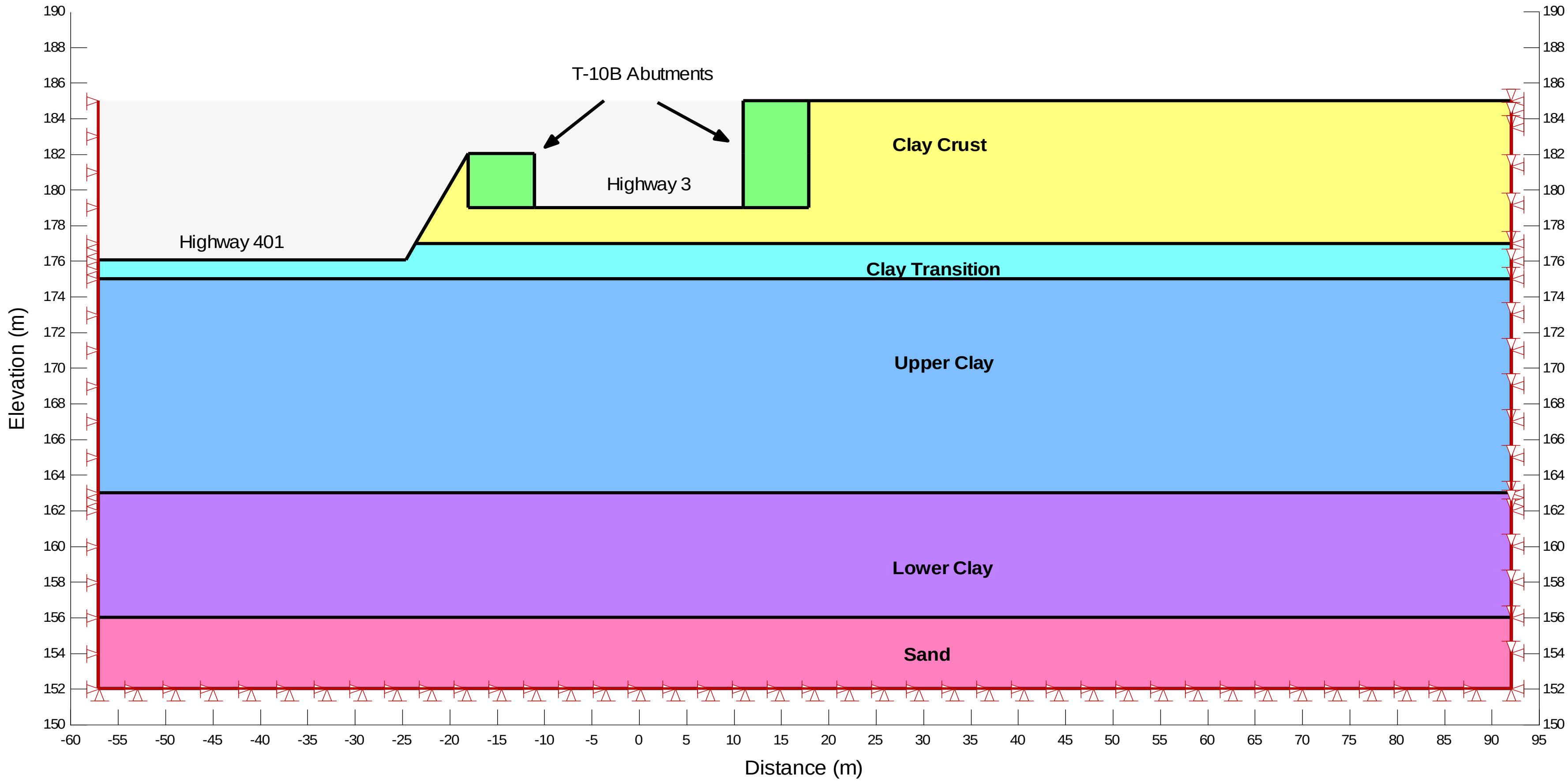


1 day = EOE
2 days = RSS Completion
3 days = Completion of Abutment
10004 = Long-Term Condition

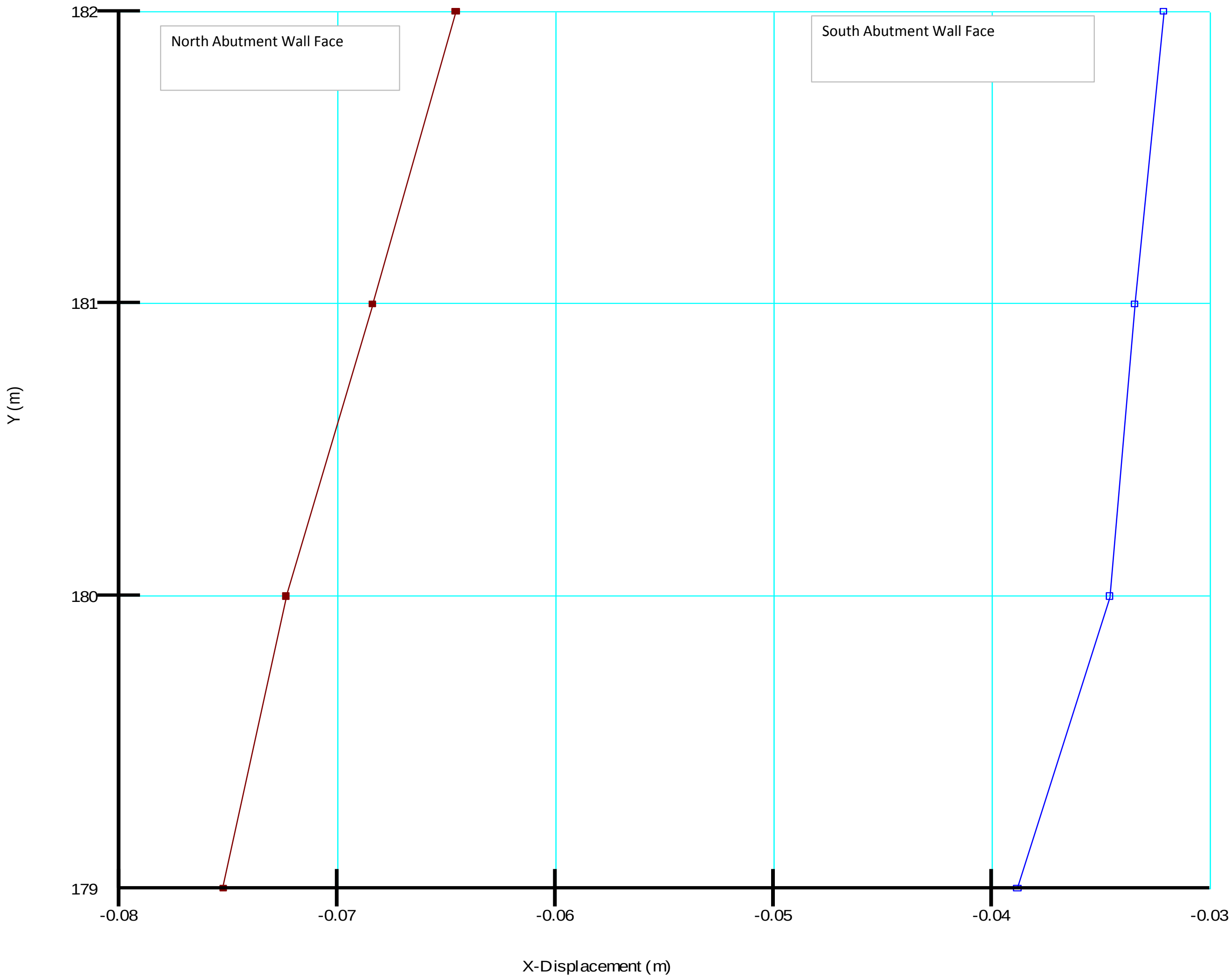
Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PARKWAY INFRASTRUCTURE CONSTRUCTORS	PROJECT: WINDSOR ESSEX PARKWAY			
		TITLE: SOIL SETTLEMENT PROILE ALONG PILE LOCATION			
		DATE: Jul 2011	JOB NO.:	CAD FILE:	FIGURE NO.: F.7	REV.

Simplified Highway 401 Excavation Model
Tunnel T-10A and T-10B Interaction
Station 40+850
Last Solved Date: 23/07/2011

Name: RSS Backfill Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Clay Transition Young's Modulus (E): 19500 kPa Poisson's Ratio: 0.49 Cohesion: 65 kPa Phi: 0 ° Unit Weight: 21.5 kN/m³
Name: Upper Clay Young's Modulus (E): 14400 kPa Poisson's Ratio: 0.49 Cohesion: 48 kPa Phi: 0 ° Unit Weight: 21 kN/m³
Name: Low er Clay Young's Modulus (E): 29000 kPa Poisson's Ratio: 0.49 Cohesion: 95 kPa Phi: 0 ° Unit Weight: 21.5 kN/m³
Name: Sand Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Crust Young's Modulus (E): 35000 kPa Poisson's Ratio: 0.49 Cohesion: 75 kPa Phi: 0 ° Unit Weight: 22 kN/m³



Parkway Infrastructure Engineers amec Hatch Mott MacDonald	PARKWAY INFRASTRUCTURE CONSTRUCTORS	PROJECT: WINDSOR ESSEX PARKWAY
		TITLE: Simplified Hgiway 401 Excavation Model TUNNEL T-10A and T-10B Inteaction
		DATE: Jul 2011 · JOB NO.: · CAD FILE: · FIGURE NO.: F.8 · REV.



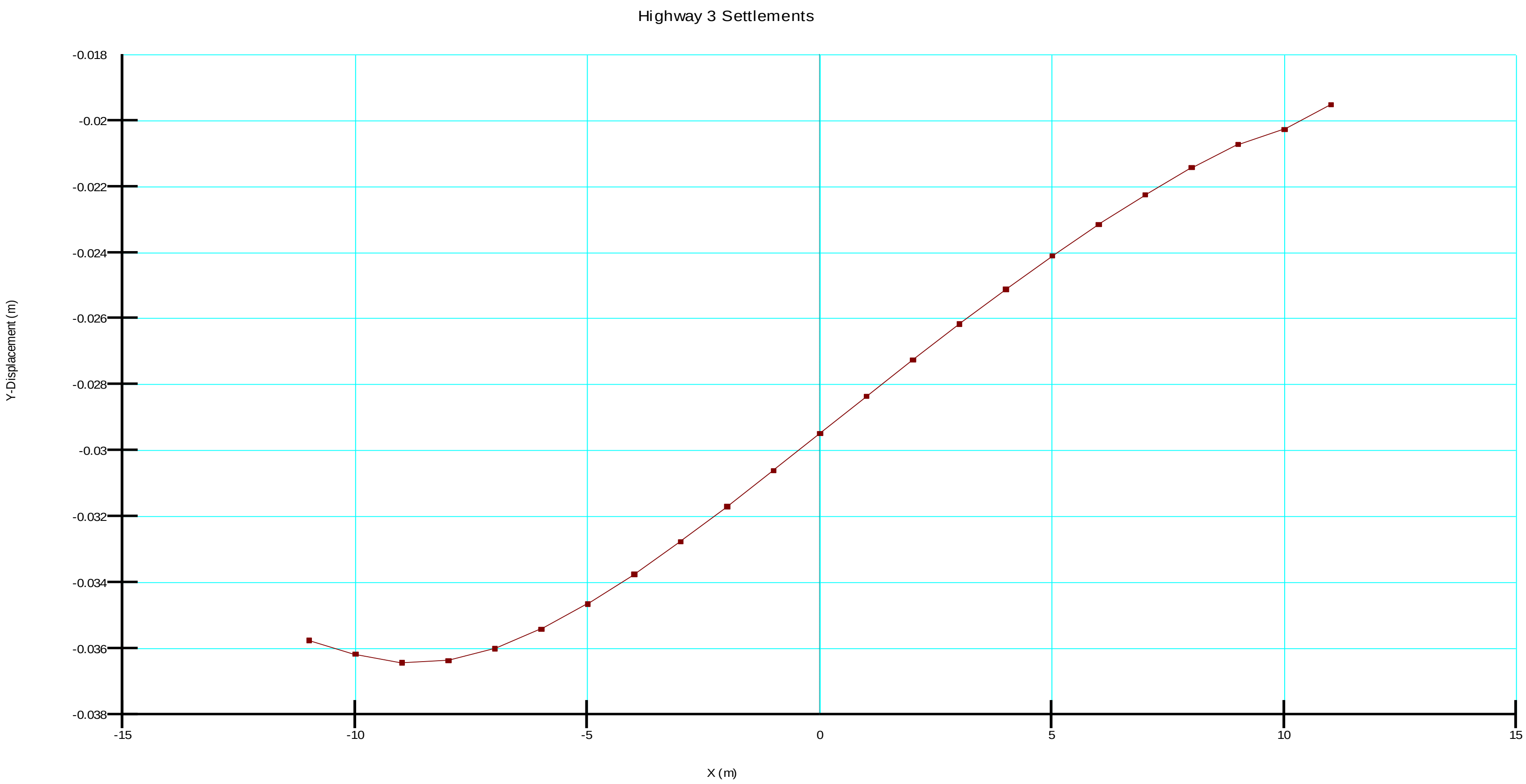
Based on Simplifed Model presented in Figure F.7

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

PARKWAY INFRASTRUCTURE CONSTRUCTORS

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	LATERAL WALL MOVEMENT AT TUNNEL T-10B based on TUNNEL T-10A excavation

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.9	REV.
Jul 2011				



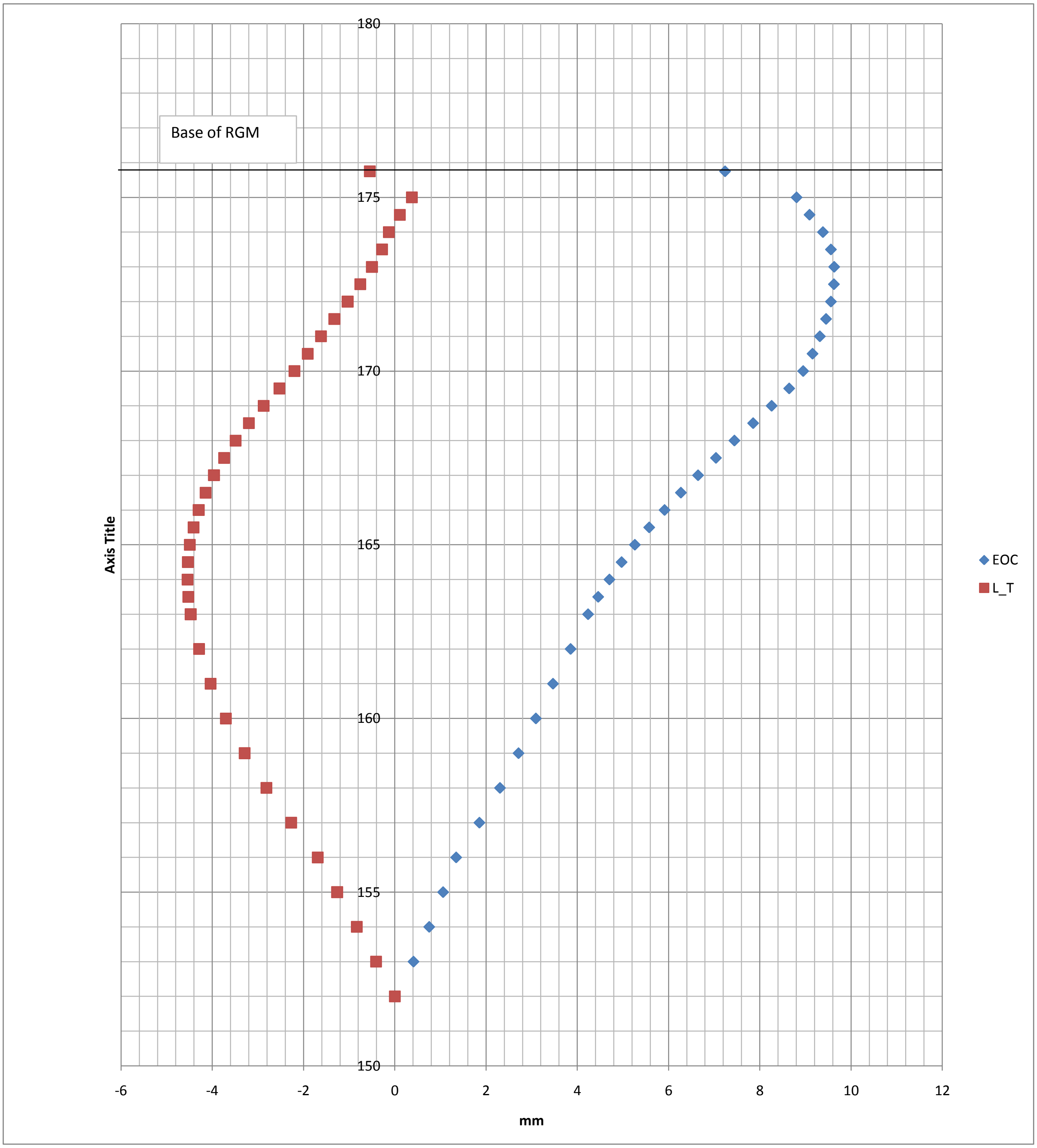
Based on Simplifed Model presented in Figure F.8

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PARKWAY INFRASTRUCTURE CONSTRUCTORS

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: Highway 3 SOIL SETTLEMENTS AT TUNNEL T-10B based on				
DATE: Jul 2011	JOB NO.:	CAD FILE:	FIGURE NO.: F.9	REV.

Appendix G Pile Analysis Results



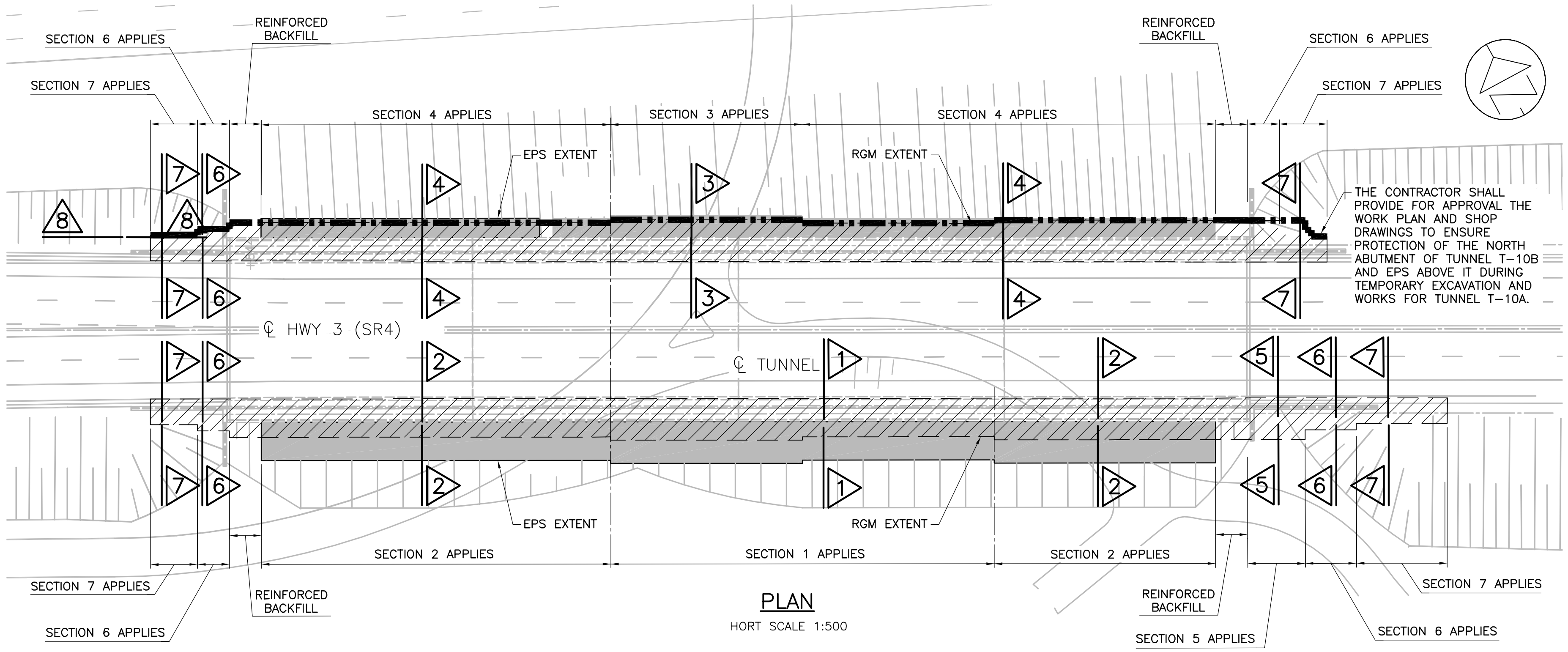
Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald | PARKWAY INFRASTRUCTURE CONSTRUCTORS

Project	WINDSOR ESSEX PARKWAY		
TITLE	L-Pile Analysis - Soil Deformations from Sigma		
Date July 22, 2011	JOB NO SW8801.1004.101	FIGURE G.1	REV

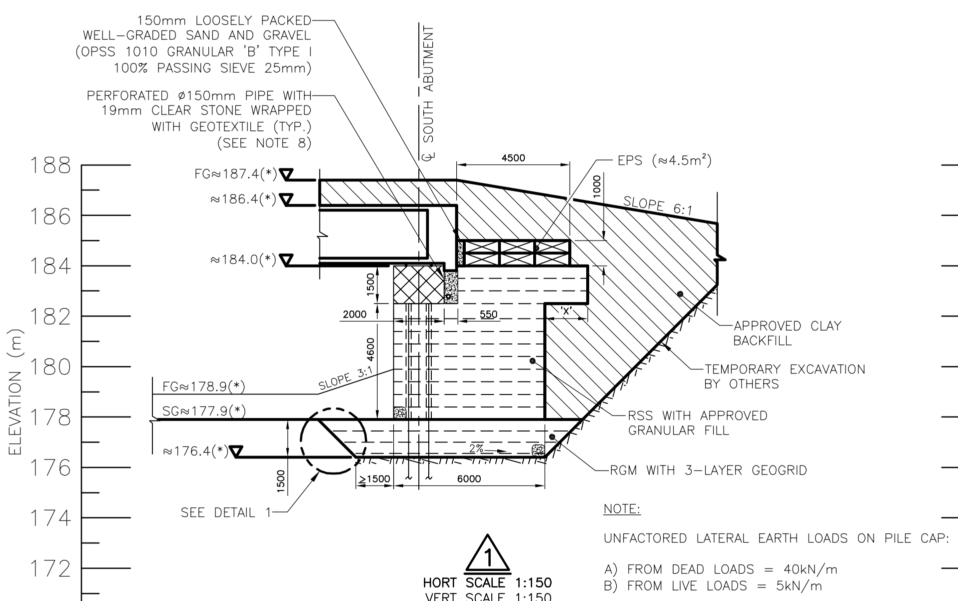
Appendix H Photographs

Photograph 1: Borehole T10-2 – Rock Core. Elevation 152.3 to 148.8

Appendix I: Conceptual Drawings



PLAN
HORT SCALE 1:500



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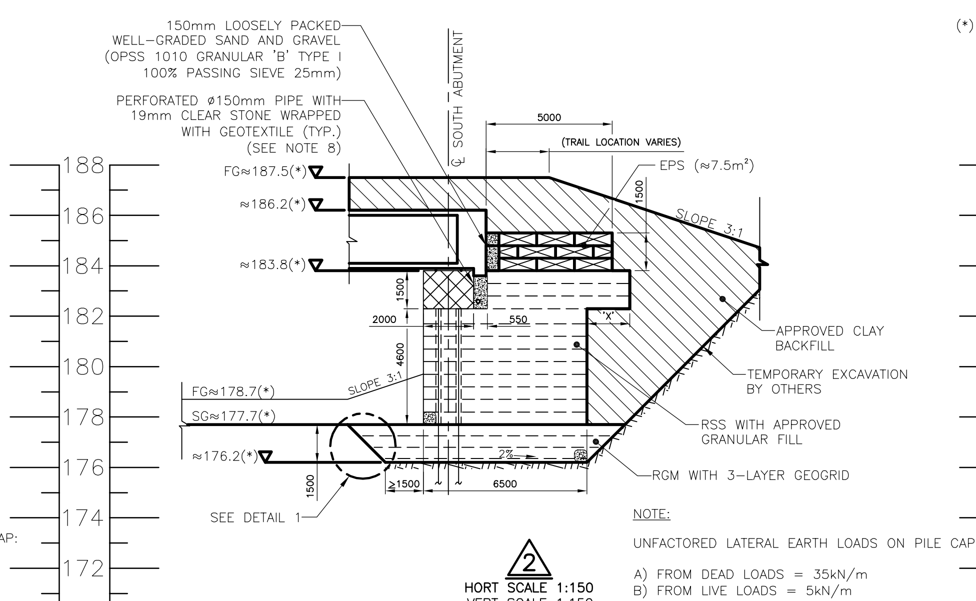
VERT SCALE 1:150
SOUTH ABUTMENT (STA 40+950) HWY 3-SR4



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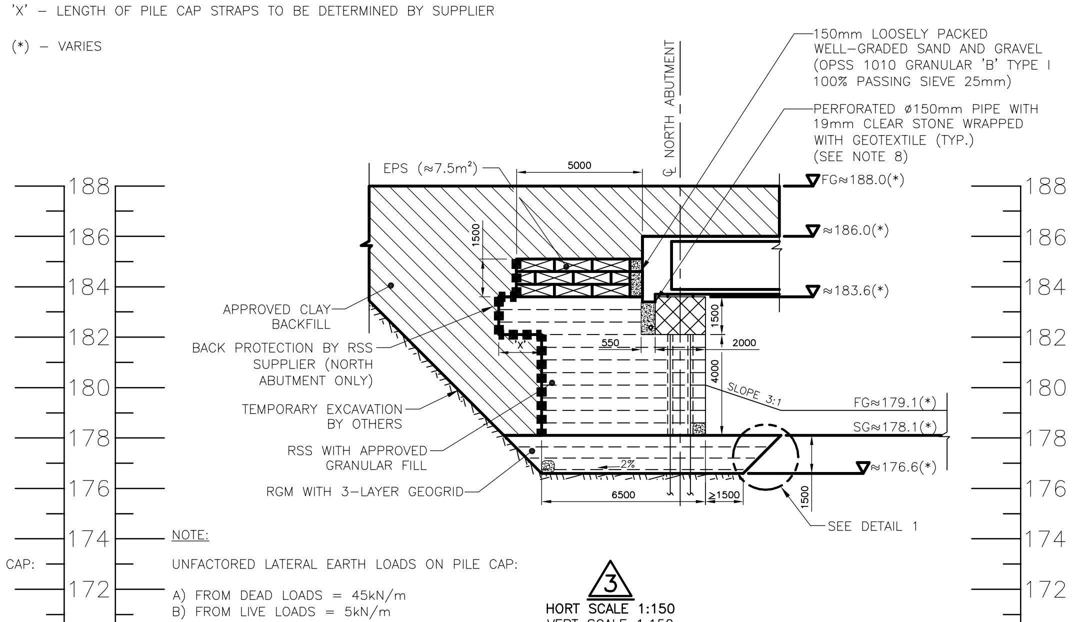
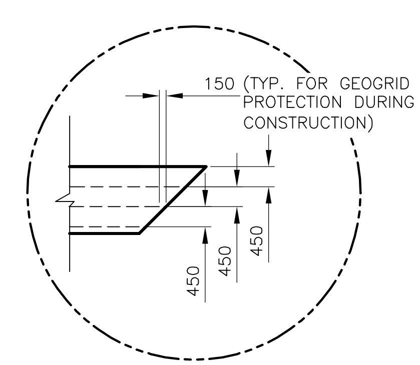
SOUTH ABUTMENT (STA 40+990) HWY 3-SR4



HORT SCALE 1:150
VERT SCALE 1:150
NORTH ABUTMENT (STA 40+915) HWY 3-SR4



DETAIL 1

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-10B BASED ON GEOTECHNICAL DESIGN ANALYSES.
3. THE ILLUSTRATED RSS WALL WIDTH AND RGM DIMENSIONS REPRESENT THE MINIMUM DIMENSIONS BASED ON GLOBAL STABILITY REQUIREMENTS. THE DESIGN OF THE RSS WALL AND REINFORCED SOIL STRUCTURES BY OTHERS.
4. TUNNEL ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGN WERE INTERPRETED FROM INFORMATION INDICATED ON STRUCTURAL DRAWINGS AVAILABLE IN JANUARY 2012. ABUTMENT ELEVATIONS VARY ALONG TUNNEL T-10B.
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. FOR DETAILS REGARDING CONSTRUCTION, SEE APPLICABLE CONSTRUCTION NOTES.
8. HIGHEST DRAIN ELEVATION WITH RGM SHALL BE AT LEAST 1m BELOW TOP OF RGM.
9. BACKFILL ABOVE THE RSS SHALL BE PLACED ONLY AFTER THE PAVEMENT SUBGRADE ADJACENT TO THE TOE OF THE RGM IS FULLY RESTORED AND PROTECTED.
10. FINAL 1m BACKFILL ABOVE NORTH ABUTMENT OF TUNNEL T-10B TO BE DELAYED AFTER SUBSTANTIAL COMPLETION OF THE SOUTH ABUTMENT OF TUNNEL T-10A.
11. BACK PROTECTION OF THE REINFORCED SOIL ZONES (RSS AND PILE CAP STRIP AREA) REQUIRED FOR THE NORTH ABUTMENTS TO ALLOW FUTURE EXCAVATION FOR TUNNEL T-10A.

LEGEND:

RSS – REINFORCED SOIL STRUCTURE

LWF – LIGHT WEIGHT FILL

RGM – REINFORCED GRANULAR MAT (GRANULAR 'B' TYPE II COMPACTED TO 100% WITH 3 GEOGRID SHEETS UX1400HS, OR EQUIVALENT)

EPS – EXPANDED POLYSTYRENE

'X' – LENGTH OF PILE CAP STRAPS TO BE DETERMINED BY SUPPLIER

(*) – VARIES

DRAWING NOT TO BE SCALED
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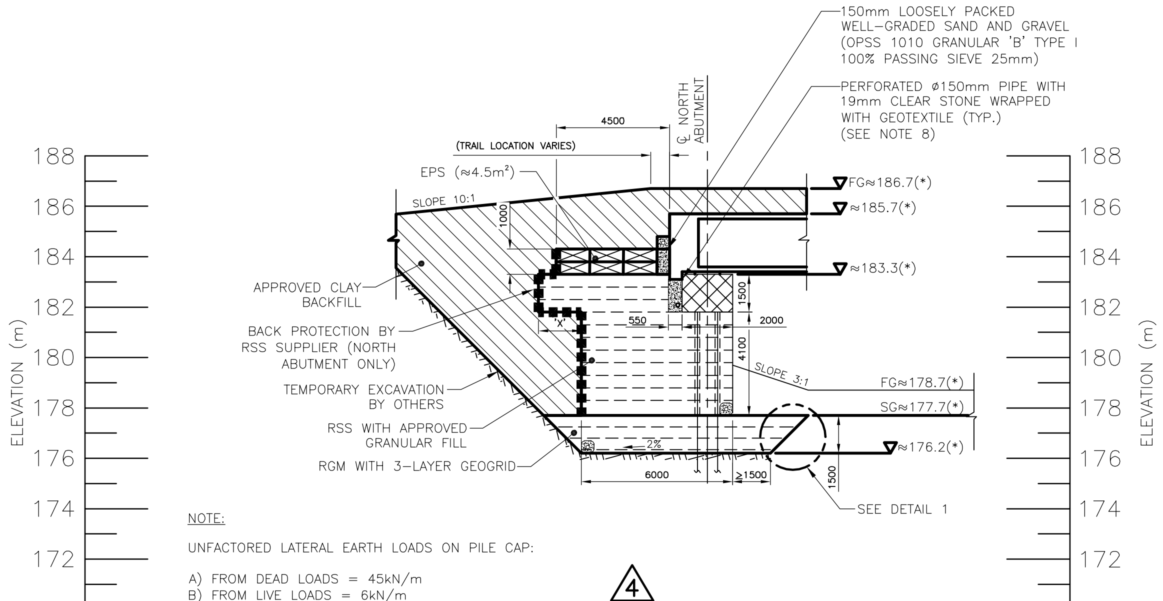
TUNNEL T-10B
HEARTHWOOD TUNNEL
PLAN AND SECTIONS OF ABUTMENTS AND WINGWALLS

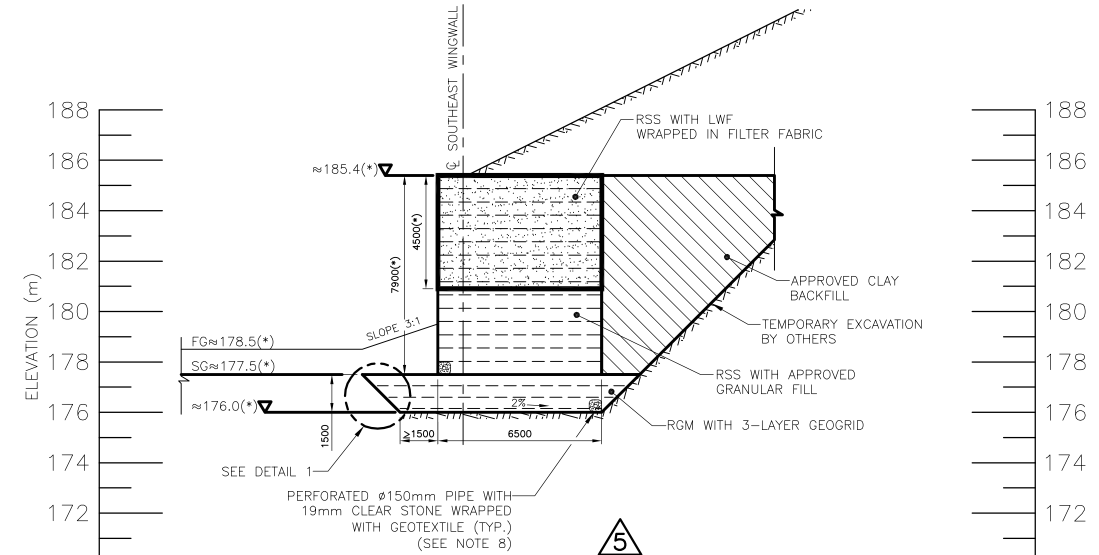
DWG. BY: SJL	CHK. BY: TL	FIGURE NO.: I.1
DATE: Mar-12	SHEET: 1 OF 1	

DOC: 285380-04-094-WIP1-3084_FIG I.1



4
HORT SCALE 1:150
VERT SCALE 1:150

NORTH ABUTMENT (STA 40+990) HWY 3-SR4



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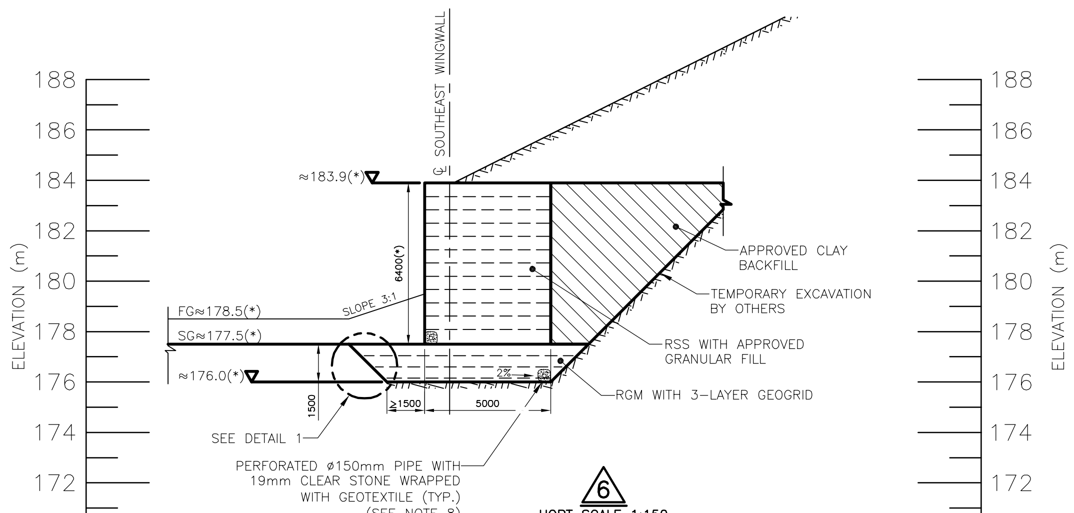
VERT SCALE 1:150

SOUTHEAST WINGWALL (LONG PANEL) HWY 3-SR4



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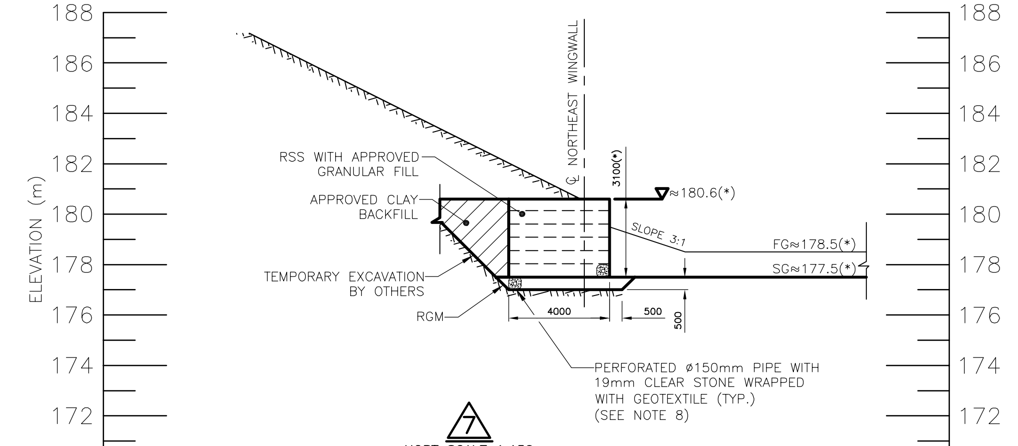
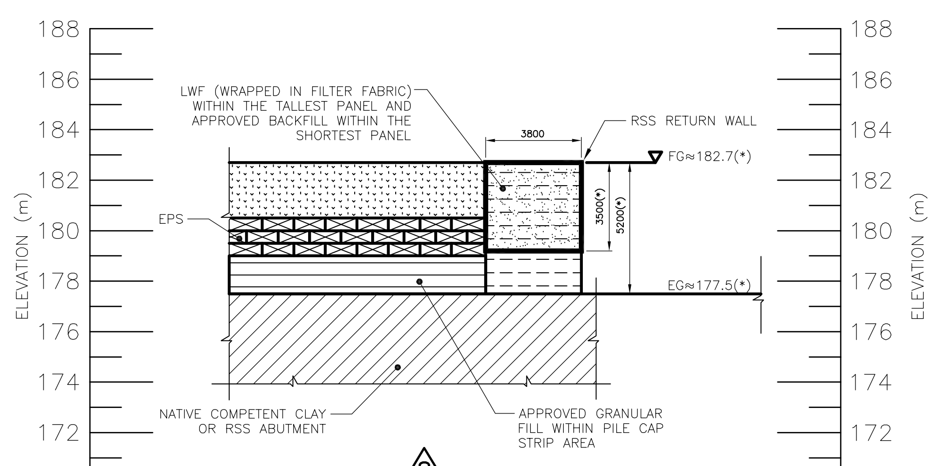
HORT SCALE 1:150
VERT SCALE 1:150

SOUTHEAST WINGWALL (SHORT PANEL) HWY 3-SR4

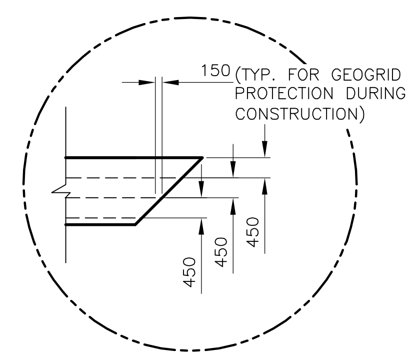


7
HORT SCALE 1:150
VERT SCALE 1:150

NORTHEAST WINGWALL (SHORT PANEL) HWY 3-SR4



8
HORT SCALE 1:150
VERT SCALE 1:150



DETAIL 1

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

DOC: 285380-04-094-WIP1-3084_FIG I.2

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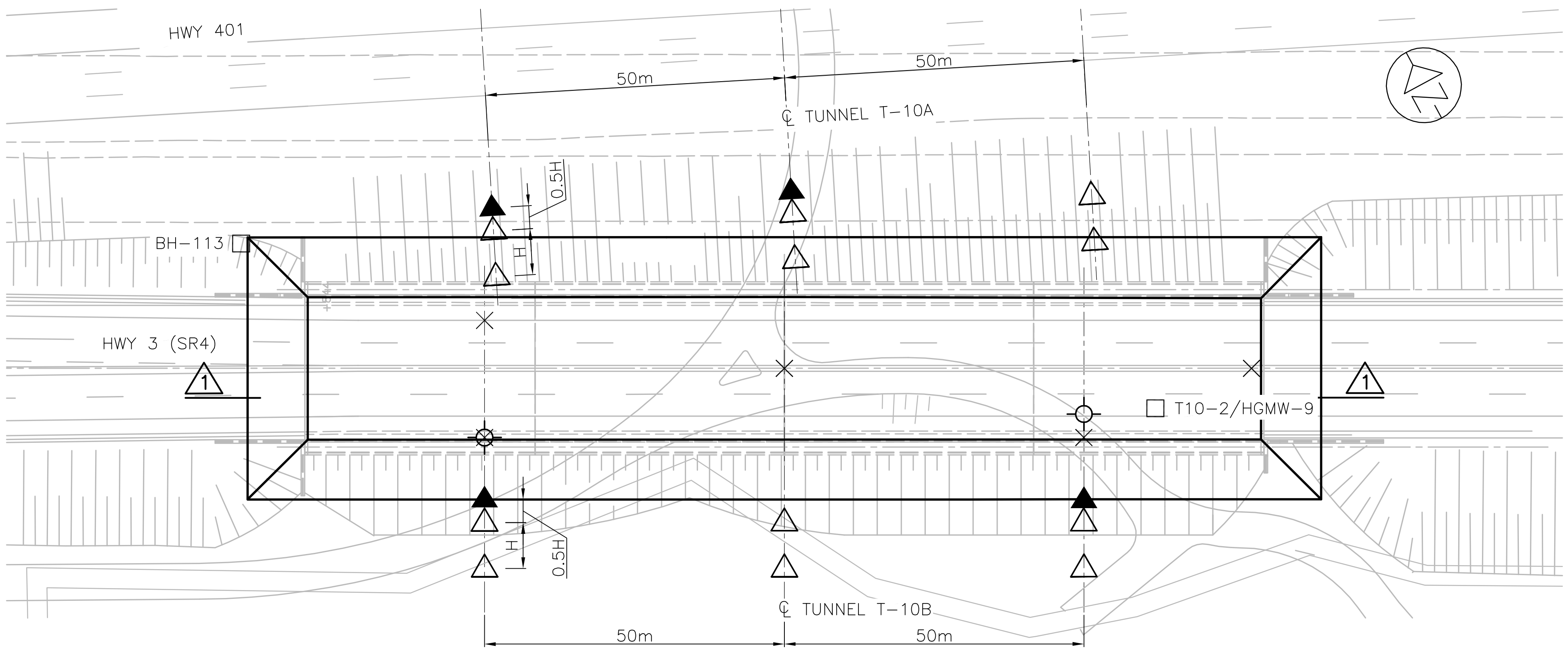
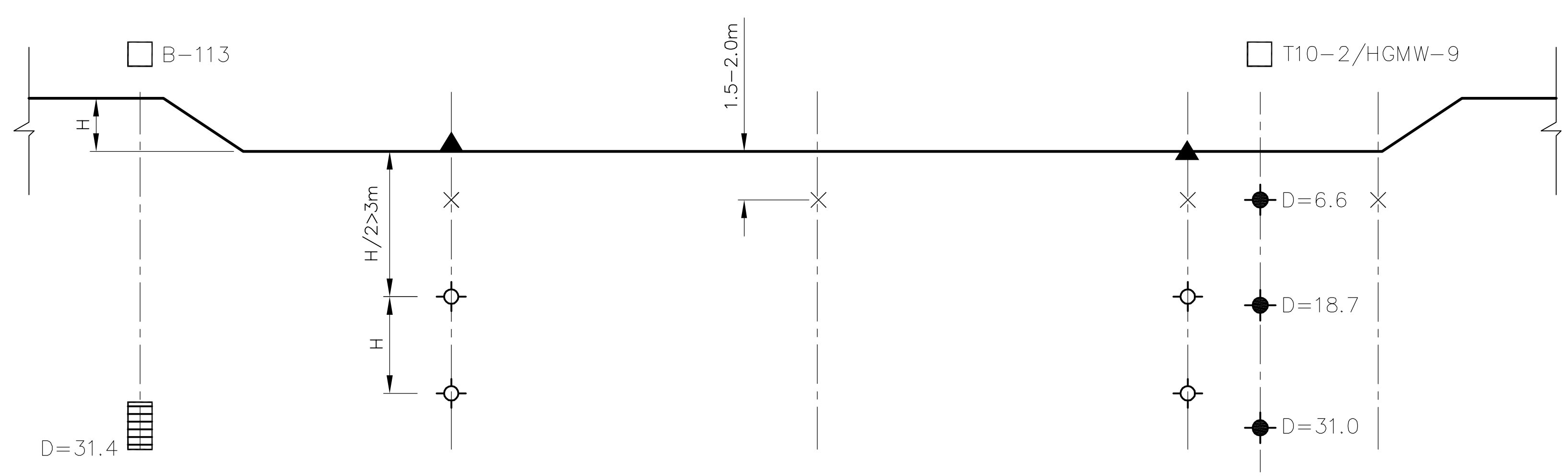
TUNNEL T-10B
HEARTHWOOD TUNNEL
PLAN AND SECTIONS OF ABUTMENTS AND WINGWALLS

DWG. BY: SJL	CHK. BY: TL	FIGURE NO.: I.2
DATE: Mar-12	SHEET: 1 OF 1	

Appendix J: Instrumentation Plan



PLAN



SECTION

1

LEGEND:

- PROPOSED HEAVE GAUGE
- PROPOSED PIEZOMETER (LOW DISPLACEMENT)
- EXISTING PIEZOMETER (VIBRATING WIRE)
- PROPOSED SURVEY PINS
- PROPOSED INCLINOMETER
- EXISTING INCLINOMETER
- EXISTING INSTRUMENTED BOREHOLES
- EXISTING STANDPIPE WELL

NOTE:

INSTRUMENTATION AT NORTH ABUTMENT LOCATIONS TO BE FIELD FIT BASED ON TUNNEL T-10A TEMPORARY EXCAVATION.

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

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TUNNEL T-10B
INSTRUMENTATION FOR EXCAVATION MONITORING DURING CONSTRUCTION

DWG. BY: SJL	CHK. BY: TL	FIGURE NO.: J.1
DATE: Mar-12	SHEET: 1 OF 1	

DOC: 285380-04-096-WIP1-3085_FIG J.1

Appendix K: Seepage Model

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PARKWAY INFRASTRUCTURE CONSTRUCTORS

Figure K-1: Simplified Seepage Model Results