

# The Windsor-Essex Parkway Project


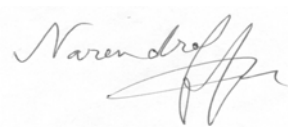

## Geotechnical Investigation and Design Report – Tunnel T- 7

### (Highway 401 Sta. 10+450L to Sta. 10+700L, LaSalle)

Geocres No. 40J6-37



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# 1 Introduction

## 1.1 Preface

The Windsor Essex Parkway (the Parkway, or the WEP) was conceived to strengthen transportation and trade links between Canada and the United States, reduce road congestion, and foster economic growth. The Parkway will connect Highway 401 to a new Canadian inspection plaza and a new international crossing over the Detroit River to Interstate 75 in Michigan, USA. It will be a six-lane highway, 11 km long with 11 tunnels and a four-lane service road that will provide full access to schools, neighbourhoods, natural areas, and shopping. Other components of the project include community and environmental features, such as: 300+ acres of green space, 20 km of recreational trails, extensive landscaping throughout the corridor, as well as noise and environmental mitigation measures. The environmental mitigation measures were based upon Permit AY-D-001-09 which was approved in February 2010.

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

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

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

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

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

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

## **1.2 Report Introduction**

This report presents the geotechnical design of Tunnel T- 7 (Villa Borghese Drive and Huron Church Line, between Station 10+450L and Station 10+700L), located in the LaSalle sector of the Windsor-Essex Parkway (WEP) project. The report includes the results of the additional geotechnical investigation carried out to support the design (i.e., the layout and configuration) available at the time of preparation of this report and addresses review comments from peer reviews and MTO.

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 (T-1 to T-11), 9 trail bridges, approximately 5.5 km length of retaining walls, 2 submerged culverts, and other structures.

The proposed 2-span Tunnel T-7 will cross Highway 401 between Sta. 10+450L and 10+700L with Huron Church Line crossing over Highway 401 near Sta. 10+600L. Realigned Highway 3 will run along and above the north abutment. Tunnel T-7 is comprised of semi-integral abutments and centre piers founded on deep end bearing piles.

The design presented in this report was generally advanced from the preliminary geotechnical design developed for the WEMG proposal in June 2010 (ref. R-47)<sup>1</sup>. 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 executable version of the Project Agreement (PA) Schedule 15-2 Part 2, Article 5.

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<sup>1</sup> 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-19, R-20 and R-29). The Essex Clay Plain was deposited during the retreat of the late Pleistocene Era ice sheets, when a series of glacial lakes inundated the area. The ice sheets generally deposited materials with a glacial till like gradation in the Windsor area. Depending on the locations of the glacial ice sheets and depths of water in the ice-contact glacial lakes, the materials may have been directly deposited at the contact between the ice sheet and bedrock or, as the lake levels rose and the ice sheets retreated and floated, the soil and rock debris within and at the base of ice may have been deposited through the lake water (i.e., lacustrine environment). It is considered that unlike typical till deposits (that have undergone consolidation and densification under the weight of the ice sheet), the majority of the “glacial till” soils in the Windsor and Detroit area were deposited through water and have a soft to firm consistency below a surficial crust layer that has become stiff to hard due to weathering and desiccation. Geologically, the deposit in the project area is considered to be slightly over-consolidated, having experienced no major overburden stresses in excess of the existing stresses.

The overburden in the St. Clair Clay Plain has variously been described as a clayey silt till, silty clay till and glaciolacustrine clay. Hudec (ref. R-29) summarized the overburden geology in Windsor as consisting of the following successive strata: desiccated lacustrine clay, normally consolidated lacustrine clay, silty Tavistock till, glaciolacustrine clay and coarse Catfish Creek till. A distinct change in overburden deposits occurs in the east-west direction along a boundary located generally along the Huron-Church Road. Whereas, the eastern part of Windsor is underlain by firm to stiff glaciolacustrine silts and clays with upper deposits of stiff sandy to silty weathered clay and hard to stiff lacustrine clay-silt crust, the western part of Windsor is characterized by a thin surficial granular deposit underlain by thin crust layer, in turn, underlain by soft to firm glaciolacustrine silts and clays.

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

The Windsor area, referred to as the Essex Domain (with respect to bedrock geology), is located in the Grenville Front Tectonic Zone (GFTZ). 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) by a seismic hazard associated to a Velocity Zone  $Z_v = 0$  and Acceleration seismic zone  $Z_a = 0$ . Zonal Velocity ratio,  $V$ , and Zonal Acceleration ratio,  $A$ , are both 0.

In accordance with the CHBDC, and based on a series of cross-hole tests completed during the background investigation program (ref. R-23), the soil profile at the site of the project generally meets the description for Soil Profile Type III (soft clay and silts greater than 12 m in depth). These cross-hole tests were completed during the background investigation program at locations distributed along the project alignment between Howard Avenue (east end) and Matchette Road (west end). The measured velocities of the shear waves were consistently over 200 m/s, with the bulk of results ranging between 200 and 300 m/s.

## 2.3 Site Conditions

Tunnel T-7 site is situated in the LaSalle sector of the Parkway (i.e., the east part of the WEP). The tunnel structure will be constructed under WEP Phase I development and will be used to carry trail traffic, Huron Church Line and Parkland over Highway 401. Highway 3 in the vicinity of Tunnel T-7 will be relocated on the north side of the proposed depressed Highway 401. Highway 401 at this location will be constructed within permanent cut. (Drawing 285380-03-60-WIP1-2701).

The topography of the lands immediately adjacent to Tunnel T-7 at Highway 401 is generally flat with elevation ranging from approximately 181.1<sup>2</sup> to 181.5 in the area of both north and south abutments. Adjacent land use is typically both residential and commercial. The elevation of the proposed top of the tunnel deck ranges from approximately 182.7 to 183.3, with the finished grades over the tunnel approximately one metre above that. A high trail embankment is present on the south side of the tunnel, rising to approximate elevation 187.3.

There are a few other structures located in the vicinity of Tunnel T-7. The Lennon drain submerged culvert is proposed just west of the tunnel, with an invert approximately 5 m below the Hwy 401 grade. Along the north abutment, the wing walls transition into HRW-20L on the west side and MSEW-21L on the east side. These structures will be discussed in separate reports.

## 2.4 Frost Depth

In accordance with MTO-SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-42) and OPSD 3090.101 the frost depth below the ground surface in Windsor area is estimated at 1.0 m<sup>3</sup>. This estimate is considered applicable to natural soils and / or conventional pavement materials where the ground surface is usually cleaned from the snow cover.

<sup>2</sup> Elevations are in metres and are referred to geodetic datum.

<sup>3</sup> Ontario Provisional Standard Drawings are included at the end of the report text.

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



### 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-17 to R-25) to develop the conceptual design and serve as part of background information for development of the WEP proposal designs. Additional geotechnical investigation has been 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-7 comprised a total of 3 boreholes, 1 Nilcon vane profile and 1 flat plate dilatometer (DMT) profile and 2 CPT tests. Four boreholes (TB5-1 – TB5-4) were also drilled near Tunnel 7 near the south side trail embankment.

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

**Table 3-1: Test Holes at and around Tunnel T-7 Site**

Reference	Boreholes	Nilcon Vane Tests	CPT	DMT
Additional Investigation (2011)	T7-1			
	T7-2	NIL 7-2		
	T7-3			
	TB5-1			
	TB5-2			
	TB5-3			
	TB5-4			
			CPT 7-1	
			CPT 7-2	
				DMT 7-1
Previous Studies (2007-09)	BH CPT-322		CPT-322	
	BH CPT-124		CPT-124	
	BH-127			

Drawing 285380-04-090-WIP1-2702 shows the locations of the test holes and an interpreted soil stratigraphic profile along the WEP centreline for the general area around Tunnel T-7 (Sta. 10+400L to 11+000L). The test hole locations and stratigraphic sections at the tunnel location are illustrated on Drawing 285380-04-090-WIP1-2703 and 285380-04-091-WIP1-2704.

##### 3.1.1 Fieldwork for Additional Investigation

The boreholes were advanced using track-mounted CME55 auger rigs owned and operated by Marathon Drilling Co. Ltd. under a contract to AMICO and under technical supervision by AMEC engineers and technicians. Boreholes were generally advanced using 200 mm OD hollow stem augers, followed by wash boring with NW casing.



Soil sampling was carried out using 50 mm diameter split spoon samplers or thin-walled Shelby tubes (70 mm diameter by 600 mm long). Soil sampling was carried out generally at 0.75 m depth interval in the top 7 to 8 m and at 1.5 m depth intervals thereafter. All samples were identified by an experienced technologist, placed in airtight containers in the field and transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing. Rock coring of the bedrock was completed using NQ or HQ sized core barrels with a length of 1.5 m.

Standard Penetration Tests (SPT, ASTM D1586) were carried out in conjunction with split spoon sampling. Field vane tests (using conventional vanes) were carried out in between sampling at selected depths. The Nilcon vane test listed in Table 3-1 was performed adjacent the corresponding borehole. Table 3-2 summarizes the depths of overburden penetration and rock coring as well as the list of instruments at each borehole location and the accompanying Nilcon vane tests. The CPT cone was pushed at a constant rate into the ground using hydraulic ram system of the drill rig (ASTM D5778). All CPT and DMT were advanced to refusal.

**Table 3-2: Overburden Thickness and Instrumentation in Boreholes**

Borehole	Location	Overburden Thickness, m	Rock Coring	VWP	MHSG
BH T7-1 (2011)	N4679413.6, E332295.2	30.3	148.9 to 145.5	172.4 161.7	172.1 161.7
BH T7-2 (2011) Nil 7-2	N4679331.1, E332388.2	33.5	148.9 to 145.5	176.9, 161.1 148.9	171.4 161.9
BH T7-3 (2011)	N4677868.8, E335104.6	32.0	149.7 to 146.4		
DMT-7-1 (2011)	N4679368.7, E332355.7	BTEO			
CPT-7-1 (2011)	N4679276.5, E332433.5	BTEO			
CPT-7-2 (2011)	N4679271.0, E332428.0	BTEO			
BH TB5-1 (2011)	N4679286.0 E332362.0	BTEO			
BH TB5-2 (2011)	N4679261.2 E332400.9	BTEO			
BH TB5-3 (2011)	N4679239.6 E332429.4	BTEO			
BH TB5-4 (2011)	N4679221.9 E332459.0	BTEO			
BH/CPT-322 (Pre-Bid)	N4679294.0, E332478.2	BTEO			
BH/CPT-124 (Pre-Bid)	N4679354.6 E332455.0	BTEO			
BH - 127 (Pre-Bid)	N4679370.9, E332251.6	30.7		145.2 to 146.7 S-Piez	

Legend: S-Piez. Standpipe piezometer  
VWP Vibrating wire piezometer  
MHSG Spider magnet heave/settlement gauge  
BTEO Borehole Terminated Early in Overburden

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

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

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

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

### 3.1.2 Laboratory and Analytical 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 tests, there was one advanced test conducted. A one dimensional consolidation test was conducted on a sample (TW11) from BH T7-1.

Selected samples of the silty clay/clayey silt obtained from Boreholes T7-1, T7-2 and T7-3 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 the laboratory geotechnical and analytical tests are indicated on borehole logs and included in Appendices C and D, respectively.

### 3.1.3 Instrumentation

Geotechnical instruments (vibrating wire piezometers – VWP and spider magnets heave/settlement gauges – MHSg) were installed at selected locations on completion of boreholes to monitor pore water pressure and deformation behaviour of the soil strata during and after construction. These are listed in Table 3.2. A brief description follows.

**Vibrating Wire Piezometers:** The VWP transducers (RST Model VW2100, 0.35 MPa for shallow to mid-depth and 0.7 MPa for deep installations) were installed in BH T7-1 and T7-2 at the selected depths and electrical wires extended to the monitoring station at the ground surface to measure pore water pressures in soil strata. The borehole was filled with a bentonite-cement mixture designed to match, as near as practical, the permeability and strength-deformation characteristics of the native soils.

**Magnetic Settlement/Heave Gauges:** Spider magnets (RST, Model SSMM100 mechanical release spider target for 25 mm pipe) were installed in Borehole T7-1 and T7-2 at selected locations and depths to permit future measurement of heave and settlement. Each magnetic torus was placed around a 25 mm diameter pipe, which was extended to above the ground surface. The spider legs grip into the surrounding soil,

which enables the magnetic torus to move up or down on the pipe as the soil settles or heaves. The locations of the magnetic torus are determined by lowering a magnetic probe inside the pipe.

The installation of the spider magnets and the grouting of the holes were carried out in accordance with the manufacturer specifications.

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

### 3.2 Data Interpretation – General Discussion

**Field Vane Test Data Correction:** The chart (Figure 3.1<sup>4</sup>) 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 (PI) of about 15. However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), the Canadian Foundations Engineering Manual suggests that the vane test data for clays with PI<20 should not be corrected (ref.R-6 and R-9). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI.

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

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

Where:

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

$Q_t$  is the corrected total cone tip resistance;

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

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

The CPT based  $S_u$  profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles. In this regard, the  $N_{kt}$  factor values used to calibrate the CPT strength profiles varied slightly for different segments of the WEP and the soil strata. Thus,  $N_{kt}$  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 15-16<sup>5</sup> and 12-13<sup>6</sup>,

<sup>4</sup> All figures are included at the end of the report text.

<sup>5</sup>  $N_{kt}$  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)

<sup>6</sup>  $N_{kt}$  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. 10+400L and Sta. 11+100T, 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. In CPT indicating pore pressures higher than cone tip resistance, the undrained shear strength was estimated from the excess pore pressures (using the  $N_u$  method).

**Pre-Consolidation Pressures from Cone Penetration Tests:** The approach used for estimating the pre-consolidation pressures from the estimated  $S_u$  profiles follows the Stress History and Normalized Soil Engineering Properties (SHANSEP) method developed at MIT (Ladd and Foott, 1974, ref. R-35) 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;

$\sigma'_{vo}$  is the vertical effective stress;

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

$S$  is the normalized strength ratio,  $S_u/\sigma'_{vo}$ , of normally consolidated soil;

$OCR$  is the overconsolidation ratio; and

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

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

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

**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-30), 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  $S = 0.18$  for this site and  $K_d$  is the horizontal stress index represented by:

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

Where:

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

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

Interestingly, the undrained shear strength ( $S_u$ ) profile inferred from the DMTs appeared as a more consistent average compared to the Nilcon vane test values within the unweathered portion of the firm cohesive soils.

The undrained shear strength ( $S_u$ ), pre-consolidation pressure ( $\sigma'_p$ ), natural water content ( $w_N$ ) and compression index ( $C_c$ ) profiles based on field and laboratory testing from boreholes, CPT and DMT carried out near Tunnel 7 are presented on Figure 3.4. Also included on this figure are  $0.18 \times \sigma'_{vo}$  curve (representing  $OCR=1$ ) and simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units. The constant 0.18 for  $S_u/\sigma'_{vo}$  for  $OCR=1$  curve is based on average plasticity index of the silty clay to clayey silt stratum and Chandler 1988 relationship (ref. R-12).

## 4 Subsurface Conditions

The general soil stratigraphy at the Tunnel T-7 borehole locations consists of the following successive strata: surficial layers of topsoil, occasional fills or pavement structure, an upper granular deposit, an extensive cohesive clayey silt to silty clay deposit below elevations ranging from approximately 179 to 180 and discontinuous lower granular deposit below about elevation 151, overlying limestone bedrock below about elevations ranging from 148 to 149. The bedrock was encountered at depths ranging from about 29.9 m to 31.5 m below the ground surface.

### 4.1 Topsoil, Surficial Fills and Upper Granular Deposit

Borehole T7-1 was put down through a pavement structure consisting of about 0.2 m thick asphalt and crushed limestone fill. The pavement structure was underlain by a black topsoil and clay fill to about 1.5 m below grade. The surficial soils in Boreholes T7-2 and T7-3 consisted of approximately 0.4 m of topsoil overlying the native sands. The thickness of the topsoil is expected to vary in quality and thickness through the project area.

A layer of upper granular deposit was encountered beneath the topsoil in Borehole T7-1 to approximately 2.1 m below grade. The upper granular deposit consisted of poorly graded fine sand.

### 4.2 Silty Clay to Clayey Silt Stratum

Cohesive silty clay to clayey silt stratum was encountered underlying the surficial topsoil or fill/granular deposits in all test holes drilled at Tunnel 7 site. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into 4 successive layers as follows: an upper desiccated firm to hard clay crust, a transition zone, an upper grey silty clay deposit and then a generally coarser lower grey clayey silt deposit. The natural water content, Atterberg limits, bulk unit weights and undrained shear strengths (from Nilcon vane tests) properties of the clay sub-strata are summarized in Table 4-1. The plasticity charts (Appendix C) suggest the silty clay stratum to be a low to medium plasticity material.

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

Property <sup>1</sup>	Clay Crust	Transition Zone	Silty Clay	Clayey Silt
Elevation Range	181 <sup>2</sup> to 177	177 to 175	175 to 163	163 to 151
Natural Water Content, $w_N$ , %	12 to 28	12 to 25	13 to 28	12 to 33
Liquid Limit, $w_L$	39	31 to 34	29 to 36	27 to 50
Plastic Limit, $w_P$	18	16 to 17	16 to 18	15 to 23
Plasticity Index, PI	21	14 to 17	13 to 21	12 to 27
Liquidity Index, LI	(-)0.01	0.09 to 0.34	0.24 to 0.39	(-)0.09 to 0.62
Unit Weight, $\gamma$ , kN/m <sup>3</sup>	N/A	20.6 to 20.9	18.6 to 20.6	18.8 to 21.6

1 – Index Properties are based on laboratory results on samples recovered from Boreholes BH 127, T7-1, T7-2, T7-3, TB5-1, TB5-2, TB5-3, TB5-4

2 – Elevation varies



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

- Crust layer: > 100 kPa
- Transition layer: 100±30 kPa to 75±20 kPa
- Upper silty clay: 75±25 kPa to 55±15 kPa
- Lower clayey silt: 80±15 kPa

The stress-strain properties and the effective shear strength properties of the silty clay stratum were based on published correlations (Kulhawy and Mayne, 1990, ref. R-33, Leroueil et al, ref. R-38 and Terzaghi et al. ref. R-46) and confirmed by tests reported in Golder's Subsurface Condition Interpretation Report (ref. R-21) 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 Figures 4.1 and 4.2 and summarized as follows:

$$C_c = 0.0086w_N - 0.0086$$

$$C_r = 0.11C_c$$

The interpreted average values used for the clay substrata for the Tunnel T-7 site are summarized as follows:

**Table 4-2: Summary of Interpreted Compressibility Properties**

Property	Clay Crust	Transition Zone	Silty Clay	Clayey Silt
Natural Water Content, $w_N$ , %	20	20	15 - 23	18
Virgin Compression Index, $C_c$	0.163	0.163	0.120 – 0.189	0.146
Recompression Index, $C_r$	0.0180	0.0180	0.0132 – 0.0208	0.0161

The modulus of elasticity has been considered to be correlated with the undrained shear strength of the material, consistent with published information (R-46) and local experience (R-21, R-22) 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	22.5	0.49	20.3	0.35
Transition Zone	19.5 to 22.5	0.49	17.6 to 20.3	0.35
Silty Clay	13.0 to 19.5	0.49	11.7 to 17.6	0.35
Clayey Silt	15 to 19.5	0.49	13.5 to 17.5	0.35

(\*) Assumed values



The effective shear strength properties applicable to the silty clay stratum were determined from triaxial and direct shear tests performed during the pre-bid and additional geotechnical investigations (Figure 4.3) and supported by published PI versus  $\sigma'$  relationships (ref. R-21 and R-32, Figure 4.4), and are summarized as follows:

Apparent cohesion, $c'$	0 kPa
Angle of internal friction, $\phi'$	$30^\circ$
Friction angle at critical state, $\Phi_c$ ,	$25^\circ$ - $26^\circ$ (**)

(\*\*) Based on triaxial tests on samples from the site (R-19)

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 stratum at BH T7-2 and T7-3 below elevations 152.7 was a discontinuous, heterogeneous non-cohesive deposit with material gradation varying from silt to gravelly sand. Based on the Standard Penetration Test (SPT) “N” value ranging generally from 32 to greater than 100, this material is considered to be in a dense to very dense state of compactness. This layer varies from 0 to 5.7 m in thickness and will vary significantly throughout the project area.

At BH T7-1 a zone of sand mixed with weathered bedrock was encountered between 151.3 and 148.9. Standard Penetration Test data indicates this zone is in a very dense condition.

### 4.4 Bedrock

Beneath the lower granular deposit between elevations 148.9 to 149.7, light grey fairly to highly porous fine grained limestone bedrock was encountered. Where rock coring was undertaken, a white to grey, limestone bedrock was encountered. The bedrock was generally fresh, medium strong, thinly laminated, fine grained, faintly to moderately porous and moderately fractured. The Rock Quality Designation (RQD) of the recovered rock varied between 23 and 100 %, indicating a poor to excellent quality. Based on this core logging the rock mass classification was estimated to range from 2.8 to 5 for the Q-System

(Barton et. al., 1974, ref. R-3) and 53 to 58 for the Rock Mass Rating (RMR) based on ref. R-5 and indicates that the rock mass can be considered as a Fair quality rock mass based on the latter system. Photographs of rock cores recovered from the additional investigation are provided in Appendix H.

It was found during the preliminary investigations reported in Golder’s Subsurface Condition Interpretation Report (ref. R-21) 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 (ref. R-31). Additionally, based on the coefficient of variation, enough tests have been performed to characterise the compressive strength.

**Table 4-4: Bedrock Statistics**

	<b>Density (kg/m<sup>3</sup>)</b>	<b>Unit Weight (kN/m<sup>3</sup>)</b>	<b>UCS (MPa)</b>
Average	2502	24.54	85.5
Standard Deviation	96	0.94	25.4
Minimum Value	2340	22.95	35.5
Maximum Value	2660	26.09	135.3

Based on the rock mass classification and the strength properties assuming an  $m_i = 12$  for a crystalline limestone, a disturbance factor of 0.7, and a factor of safety of 3.0, an allowable bearing capacity of the rock has been calculated to range from 5.3 MPa to 13.5 MPa. The mean allowable bearing capacity is determined to be 9.2 MPa using the Hoek and Brown strength criterion for determining the bearing capacity of a fractured rock mass (ref. R-48).

#### 4.5 Groundwater Conditions

Deep standpipe and vibrating wire piezometers were installed in selected boreholes to measure the water levels within overburden and bedrock, respectively (Table 3.2).

As summarized in Table 4-5, the piezometric water levels within the overburden and the bedrock were at about elevations 179.7 to 180.5 and 177.3, respectively. These observations suggest a generally downward gradient between the overburden and the bedrock. It is recognized that these piezometric water levels (particularly in the overburden) may not have fully stabilized.

In addition, a localised artesian condition was observed in borehole T7-3 after completion of bedrock coring and prior to grouting. For a limited period of time after core retrieval artesian flow of groundwater occurred at this location. The estimated maximum head was to near elevation 184. This was the only borehole in the area to encounter such conditions. In consideration of this, other localized occurrences of artesian condition in the bedrock cannot be ruled out.

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

**Table 4-5: Summary of Measured Water Levels**

Borehole	Ground Surface Elevation	Piezometer Type	Screen / Sensor Elevation	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	Elevation
BH T7-1	181.5	VWP	172.4	Silty Clay Stratum	Aug. 6, 2011	180.5
		VWP	161.7	Silty Clay Stratum	Aug. 6, 2011	180.4
BH T7-2	181.2	VWP	176.9	Silty Clay Stratum	Aug. 6, 2011	179.7
		VWP	161.1	Silty Clay Stratum	Aug. 6, 2011	180.3
		VWP	148.9	Lower Granular	Aug. 6, 2011	177.3
BH T7-3	184.0	N/A	N/A	Bedrock	July 10, 2011	184.0
BH 127	182.3	SP	145.2 to 146.8	Bedrock	Jan. 26, 2009	177.3

Legend: VWP Vibrating Wire Piezometer  
SP Standpipe

The 100 year flood level at Tunnel T-7 is 174.26 m. This was provided by Dillon Consulting using the assumption of a pump station(s) failure (either a single pump station or all pump stations) and blocked sewer inlets in depressed sections.

#### 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 approximate concentrations of 0.5 parts per million (ppm) and can be corrosive at concentrations of about 2 ppm to 3 ppm in the groundwater. The presence of the gas was not noted by odour during the current or previous investigations at the Tunnel T-7 site.

Although the presence of the  $H_2S$  and  $CH_4$  gases was not observed during the 2011 geotechnical investigation at Tunnel T-7 site, their presence cannot be ruled out. Pumping tests were conducted at three locations across the proposed parkway to determine concentration levels of hydrogen sulphide gas in the groundwater of the area. Of these tests, TOW-2, located north of Tunnel T-7, indicated a concentration of 20.0 mg/L of  $H_2S$  gas and at TOW-3, located south of Tunnel T-4, indicated a concentration of 7.0 mg/L of  $H_2S$  gas. As Tunnel T-7 is located between TOW-2 and TOW-3,  $H_2S$  gas may be present in this area.

The understanding of the engineering behaviour (related to the impact on design and construction) of the gassy soils is rather limited. In the case of low permeability cohesive soils it is known that these soils may experience rapid drop in undrained shear strength during unloading. Due to the relatively high compressibility of the pore water fluid in gassy soils, the immediate pore water pressure response ( $\Delta U$ ) to total stress changes can be very low. This phenomena leads to reduction in effective stress and hence shear strength (ref. R-26 and R-45). It is, therefore, recommended that the design and construction methodologies should be developed in consideration of the potential presence of these gases (ref. R-15). 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

Tunnel T-7 will be constructed along the below-grade section of WEP between Sta. 10+450L and 10+700L with Huron Church Line crossing over Highway 401 near Sta. 10+600L. Tunnel T-7 is 239.6 m long with a width varying from 54 m at the west end to 57.1 m at the east end.

As shown on Drawing 285380-03-060-WP1-2701, Tunnel T-7 is 2-span deck-on-girder structure incorporating semi-integral abutments and centre pier founded on deep end-bearing HP 310x110 steel piles. False abutments using reinforced soil system (RSS) wall system and RSS wing walls are also included. A recreational trail with a trail bridge will be constructed on the south side of the tunnel. The north ends of the tunnel terminate at high wall structures. The general configurations developed for the typical abutments at Tunnel T-7 options are shown on Drawing 285380-03-061-WP1-2705.

Geotechnical designs incorporating an RSS wall with various sections of LWF (light weight backfill), approved regular backfill, granular backfill and EPS have been developed as illustrated in Appendix G). The RSS abutments will be placed over a reinforced granular mat (RGM) placed, in turn, 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 executed version of the Project Agreement Schedule 15-2 Part 2, Article 5 (PA) for the Windsor-Essex Parkway Project. The foundations' designs were as per the principles of Limit States Design (LS Method) based on Load and Resistance Factors (CHBDC, ref. R10 and Canadian Foundation Engineering Manual, ref. 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-walls was checked for all potential surfaces of sliding.

Tunnel T-7 construction is expected to involve the following sequence of earthwork, design elements and loading stages:

- Temporary excavations to about 8.5 m depth below grade;
- Installation of Reinforced Granular Mat (RGM) foundations (including drains) at north and south abutments, incorporating void forms to accommodate pile installation at later stage through RGM;
- Temporary sub-excavation to the underside of the pile cap for the centre pier;
- 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;
- Concrete fill placement within CSP;
- Construction of the pile caps, abutment stubs, piers and tunnel deck;
- Completion of the toe slope in front of the RSS wall
- Completion of final stage of backfill behind the semi-integral abutments (including EPS and/or lightweight fill, as required);
- Completion of the final topsoil placement and trail materials; and
- Completion of the pavement structure over the Highway 401 and Huron Church Line.

### 5.3 Design Soil Properties

As described in Section 3.2, the design soil properties for the silty clay deposit were interpreted from the CPT, DMT and Nilcon vane test profiles and the laboratory test results (Table 3-1). The undrained shear strength ( $S_u$ ) and preconsolidation pressure ( $\sigma'_p$ ) profiles inferred from the CPT, DMT, and Nilcon tests advanced around Tunnel T-7 and the design values obtained from these profiles are shown in Figure 3.4 and summarized in Table 5-1.

**Table 5-1: Summary of Interpreted Design Clay Strength and Consolidation History**

Clay Substratum	Elevation Range	Undrained Shear Strength ( $S_u$ ), kPa	Effective Angle of Internal Friction ( $\phi$ )°	Preconsolidation Pressure ( $\sigma'_p$ ), kPa	Over Consolidation Ratio
Clay Crust	181* to 177	75	30	550	>5
Transition Zone	177 to 175	75 to 65	30	350 to 550	>3
Silty Clay	175 to 163	65 to 44 to 50	30	220 to 350	1.15 to 2.1
Clayey Silt	163 to 151	50 to 65	30	260	1.3 to 1.4

(\*) Elevations vary

The design values of the coefficient of horizontal permeability ( $k_h$ ) and the hydraulic conductivity anisotropy ratio used for the analysis of stress and deformation response of the soils are provided in Table 5-2. These values are considered to be within range of precision of the measurements.

**Table 5-2: Design Hydraulic Conductivity Parameters for Silty Clay Stratum**

Clay Substratum	Horizontal Permeability, $k_h$	Anisotropy ratio, $k_h/k_v$ (*)	Initial Void Ratio, $e_0$
Clay Crust	$2.9 \times 10^{-4}$ m/day	1	0.54
Transition Zone	$2.9 \times 10^{-4}$ m/day	1	0.54
Silty Clay	$9.2 \times 10^{-5}$ m/day	2	0.41 to 0.62
Clayey Silt	$9.2 \times 10^{-5}$ m/day	2	0.49

(\*) Assumed

For design purposes the initial groundwater level in the overburden was considered at elevation 179.1. The effects of the 100 year flood level (174.26 m) on LWF were considered in the design.



## 5.4 Excavation and Temporary Cut Slopes

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

Excavations for abutments (including sub-excavation for RGM) are expected to encounter pavement structures, topsoil, surficial fills and granular soils and will be extended within native silty clay to depths of about 8.5 to 9.5 m (approximate elevations 172.7 to 173.7) below grade.

Basal hydrostatic uplift was calculated at abutment location based on the highest measured water level in the sandy soils above the bedrock (elevation 177.3), anticipated deepest excavation depth (RGM base at elevation 172 at the north abutment wing wall), and a silty clay/clayey silt layer thickness of 19.5 m below the deepest excavation, the factor of safety against hydrostatic uplift instability was about 1.7. The water level in the piezometers installed in Borehole T7-2, advanced for this structure and in addition piezometers installed by the contractor, should be measured on regular basis and based on the results obtained, the basal uplift hydrostatic pressure reassessed. The calculated factor of safety against hydrostatic uplift instability at pier location (excavation subgrade elevation 172.5) was also approximately 1.7. If considering the artesian condition encountered at BH T7-3 (elev. 184) and the lowest finished grade (elevation 173, drainage ditch at west end of the tunnel), the factor of safety against hydrostatic uplift is approximately 1.4. The noted factors of safety were based on the weight of the silty clay layer between the base of excavation and top of the lower granular deposit. Lenses of sandy silt and silty sand have been encountered in some boreholes along the project, embedded within the upper half of the silty clay stratum. Excavations intersecting these lenses, or approaching the top of the lenses could cause local seeps, washouts of fines, or heave of the silty clay cap.

In addition, as described in Section 4.6, presence of gassy soils near bedrock surface could potentially be encountered, and that could impact the pore water pressure and undrained shear strength condition of the lower part of the silty clay deposit. While no indications of gassy soils were recorded at this site during the background and additional investigations, in consideration of such condition and of significant soil stress relief from 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.

The above conditions warrant the installation of an adequate number of additional heave gauges and low-displacement type piezometers prior to initiation of the major excavations (details to be addressed in the report addressing the mass excavation). If significant heave and pore water pressures are indicated by the monitoring during the excavation progress, the excavation rates will have to be adjusted to allow sufficient time to dissipate the pore water pressures to safe levels. The excavation guidelines can be revised based on on-site experience.

## 5.5 Pile Foundations

### 5.5.1 Resistance to Axial Loads

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

The piles are expected to be driven to bedrock as per OPSS 903 and accordingly they will mobilize an Ultimate Limit States (ULS) axial geotechnical resistance in excess of 4000 kN. A factored geotechnical 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 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 varies between elevations 148.9 and 149.7, and the tips of piles are anticipated to be set at about that level.

In cases where some of the piles cannot be driven to bedrock due to presence of dense till lying immediately above the bedrock and a perceived risk of damaging the piles by overdriving is apparent, consideration should be given to supplementing the field testing to prove the actual mobilized resistance. If lower mobilized pile resistances are proven, options based on the most economical approaches may be considered (e.g., changes to the driving method and equipment, or addition of more piles).

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

The following general pile installation recommendations should be considered:

- The steel H piles should be installed and monitored in accordance with OPSS 903 requirements. The piles should be reinforced with Type I shoe flanges as shown in OPSD 3000.100, or approved alternatives.
- Survey of all the pile head elevations should be completed at the end of driving and just prior to forming the pile cap. Re-tapping of the piles will be necessary where uplift exceeding 5 mm is noted, or as directed by engineer.
- While unlikely to occur at this site, considering the general geologic conditions in the region, indications of natural gas venting, water, and fines washout should be monitored during driving. Provision to mitigate such occurrences (by heavy mud, grouting of the cavities, etc.) should be in place. It is recommended that the pile splicing be completed by butt-welding (OPSD 3000.150, Section A-A) to minimize the pathways for upward flow of artesian water along the piles to the surface.



- Consideration should be given to potential driving difficulties due to the presence of dense to very dense lower granular soils and potential presence of cobbles and boulders above the bedrock.
- Vibrations generated by piling should be monitored. It is not expected that the vibrations during piling will have a significant impact on the stability of temporary slopes. Nonetheless, if the vibration intensities at the toe and top of the slopes exceed 10 mm/s, appropriate mitigation measures (slope flattening or vibration dampening by dumping sand around the piles) should be considered.
- Noise monitoring should be carried out during pile driving at the site.

### 5.5.2 ULS and SLS Resistance to Lateral Loads

Both the ULS and SLS lateral load resistances are strongly dependent on the soil properties, structural configuration of the pile and pile foundation, load configuration and deformations. The SLS geotechnical resistance to lateral loads is dependent on the acceptable levels of the lateral pile deflections under the design loads and should be obtained on the basis of field load tests. In the absence of field tests, the preliminary design may be based as per Table 5-3. The ULS lateral resistance is defined as the lateral force applied to the pile shaft causing unstabilized pile displacements due to soil failure or pile structural failure. In the absence of field tests, the ULS lateral resistance may be assumed as per Table 5-3

**Table 5-3: Piles Lateral Response – Preliminary Design for Vertical Piles**

Limit State Loading	Pile Head Boundary	H-Pile Strong Axis		H-Pile Weak Axis	
		Lateral Load (kN)	Lateral Deflection (mm)	Lateral Load (kN)	Lateral Deflection (mm)
SLS	Fixed	132	5	64	2
ULS		264	20	127	9
SLS	Free	90	10	53	7
ULS		230	82	106	36

SLS - 10 mm of lateral deflection or 50% of ULS (whichever gives the lesser lateral load)

ULS - maximum bending moment = 384 kN-m (Strong Axis)

ULS - maximum bending moment = 125 kN-m (Weak Axis)

The above SLS resistance was estimated using the “p-y” model (LPile 6.0 model Ensoft 2011) for a pile considered embedded in stiff to firm silty clay. The “p-y” curves were generated using the Reese method described in the Technical manual for LPILE, using the Reese “Stiff-Clay without free water” model in conjunction with the soil parameters described in Table 5-6 below.

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

It should be noted that during driving, significant soil disturbance and damage occur around the pile shaft forming sizeable gaps between the pile and the surrounding soils. These gaps cause significant reduction of the actual SLS and ULS resistances. Where the design relies on the lateral resistance provided by the

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

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

$$k_h = n_h (z/d) \text{ for cohesionless soils, and}$$

$$= 67 (S_u/d) \text{ for cohesive soils}$$

Where:

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

$n_h$  (MPa/m) = Soil coefficient

$S_u$  (MPa) = Undrained shear strength

$z$  (m) = Depth below finished grade

$d$  (m) = Pile diameter/width

The recommended 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 <sup>(*)</sup> )		10 to 15	-
Clay Crust	>177	-	0.075 MPa
Transition Zone	177 to 175		Decreases with depth from 0.075 MPa to 0.065 MPa
Silty Clay Stratum	175 to 163	-	Decreases with depth from 0.055 MPa to 0.04 MPa
Clayey Silt Stratum	163 to 151	-	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 previously cited should be resisted fully or partially by the use of battered piles. Batter piles are considered to be more effective in resisting horizontal loads, as a part of lateral load is converted into axial load and consequently the induced bending moments are less. For ease of constructability and to limit the loss of hammer energy for pile driving, batters are usually limited to no steeper than 1H:5V.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, as indicated in Table 5-4. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed in this table.

**Table 5-5: Lateral Load Resistance Reduction Factor for Pile Groups for Horizontal Subgrade Reaction Method**

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

d = pile diameter

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

**Alternative Nonlinear ‘p-y’ Curve Method:** Alternative pile design methods may be considered using the nonlinear “p-y” interaction method or elastic continuum theory as discussed in the Canadian Foundation Engineering Manual (ref. R-6). The p-y curve represents the total lateral soil reaction pressure ‘p’ (kPa) to the pile lateral deflection ‘y’ (m) relative to the surrounding soil mass at a particular section of the pile shaft in contact with the surrounding soils. Where only pile head loads are applied and there are no lateral movements of the surrounding soil mass, y is the absolute lateral deflection. Where lateral ground movements occur, y is the relative movement between the pile and the soil. The p-y curves reflect the non-linear soil behaviour under moderate to high stress levels where the more traditional elastic modeling of the soil response is considered to be insufficient.

The general procedure for computing p-y curves is summarized in the Canadian Foundation Engineering Manual (ref. R-6). A detailed description for the generation of the p-y curves can be found in the Technical Manual for the commercial software LPILE Plus by Ensoft Inc. For a given foundation configuration, pile size, and soil stratification, the soil properties required for the generation of the p-y curves are provided in the table below. “Stiff clay” p-y curves, as given in the LPILE manual, should be developed appropriate for either static or cyclic loading conditions in the absence of free water. For p-y curves below the water table, effective unit weights in the soil mass shall be used.

**Table 5-6: Soil Parameters for p-y Curve Calculation**

Soils Around the Piles	Elevation Range	Design Bulk Unit Weight (kN/m <sup>3</sup> )	Undrained Shear Strength, $S_u$ (kPa)	$\epsilon_{50}$
Clay Crust	Above 177	22	75	0.005
Transition Zone	177 to 175	22	65	0.007
Silty Clay	175 to 163	19.5	50	0.010
Clayey Silt	163 to 151	19.5	65	0.007

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

The obtained p-y curves may require 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  $B_m = 0.75$  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  $\beta_{ki}$  = 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 Resistance Reduction Factor for Pile Groups for p-y Method**

Relative Pile Position	Pile Spacing Ratio, s/d	$\beta_{ki}$
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)	$\leq 4$	$0.70(s/d)^{0.26} \leq 1$
Trailing piles in line (piles behind the leading pile)	$\leq 7$	$0.48(s/d)^{0.38} \leq 1$

The modifier factor applies to the “p” values.

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

### 5.5.3 Soil Pile Interaction Assessment

**Downdrag Loads (Negative Skin Friction – NSF):** Potential for downdrag loads on piles was estimated with respect to the anticipated ground movements (rebound and settlements) that are assumed to occur during and following excavation of the overburden of up to 9 m to accommodate the future depressed highways, followed by partial re-placement of fills to construct the tunnel abutments.

Soil stress-deformation analyses described later in Section 5.6.2 were conducted using the SIGMA/W software. The net estimated ground vertical movement (settlement/heave) at various times after excavation in the vicinity of the pile shaft are presented in Appendix F. The analyses indicated the following:

- Ground settlements are expected to occur along the pile shaft during construction of the RSS wall, tunnel roof and completion of the associated backfill and continue for approximately 9 months at the north and south abutments except for the section of the south abutment in the vicinity of the high west approachway embankment to TB-5 where the consolidation time may increase to about 20 months after completion of the trail embankment; and
- Ground rebound is expected to occur after the completion of the ground surface settlements.

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

- a) Maximum transient downdrag of 800 kN during RSS construction and backfill above the abutments; and
- b) Residual (long-term) downdrag decreasing to less than 200 kN.

The large transient downdrag can be reduced by pile staging after the completion of the RSS structure.

The structural stress in the pile can be controlled by delaying the placement of the topsoil over the tunnel deck after substantial completion of the backfill above the abutments.

The downdrag loads do not affect the geotechnical resistance.

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

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

No downdrag is anticipated at the pier piles.

**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 filled with concrete around the pile shaft) within the RSS wall. Below the RSS wall, the pile section was HP section. The arrangements of pile cap, pile and RSS wall are shown on Drawings No. 285380-03-060-WIP1-2701 and 285380-03-061-WIP1-2705.
- The ground lateral movement (Figures F5-1 and F5-2) along the pile shaft anticipated to occur after the installation of the piles was estimated using the stress-deformation analysis described below in Section 5.6.2.
- The pile head was assumed to be a free head.
- The above soil deformation field was imposed as “loads” along the pile shaft. The calculation was conducted using the “p-y” model (LPile 5.0 model Ensoft 2010). The “p-y” curves were generated using the Reese method described in the Technical manual for LPILE, using the soil parameters indicated in Table 5-5:

**Table 5-8: Assumed Soil Properties for Pile Interaction Assessment**

Material	Soil Model in L-Pile	Effective Unit Weight, kN/m <sup>3</sup>	$\phi^\circ$	$n_h$ , MPa/m	$S_u$ , kPa
RSS Fill (LWF Option)	Sand (Reese)	14	35	5	N/A
RSS Fill (Regular Fill Option)	Sand (Reese)	21	35	10	N/A
RGM Granular(*)	Sand (Reese)	21	30	2	N/A
Silty Clay Stratum below Elevation 178	Stiff Clay without Free Water (Reese)	10	N/A	N/A	50

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

- The earth pressures from backfill and surcharge loads against the pile cap were not considered in the analyses.
- The shear force, bending moment and displacement within the pile were calculated from LPile model.

Based on the above approach 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 5-9: North Abutment Piles – Estimated Unfactored Loads on Pile Shaft due to Soil Deformations**

Location	Maximum Induced Bending Moment, kN-m (*)	Depth of Maximum Bending Section below Pile Cap, m	Lateral Load Transferred by Pile Shaft to RSS Wall, kN	Deflection of the Pile at underside of RSS Wall, mm
RSS Wall	39	6.5	43	1

(\*) Free Pile Head – H-Pile Strong Axis

Based on the above approach and anticipated lateral ground displacement, the estimated maximum unfactored bending moment in the shaft was 40 kN-m for the strong axis pile loadings and a free head condition. The shear force diagram indicated that the maximum shear force transferred by the pile shaft to the surrounding RSS wall was 45 kN. These results should be considered in the structural design of the piles and in the design of RSS structural components. These bending moments, shear forces and deflections are in addition to those caused by tunnel loads applied to the piles. The maximum computed moment in the pile under assumed pile head load equal to the conventional SLS resistance (90 kN) was 95 kN-m for the strong axis pile loadings. Accordingly, a potential combination of the maximum bending stresses from pile head shear force and ground displacement field would lead to a maximum bending moment of 135 kN-m, which is less than the yield moment of the pile.

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

**Pile Cap/Abutment Stem Anchoring:** It is understood that anchoring of the abutment stem within the backfill above the RSS wall is intended using embedded soils reinforcement connected to the pile cap. The detailed design of the anchoring is to be provided by the supplier of the reinforced soil system.



The reinforced soil material to be placed above the RSS wall within the height of the pile cap should be an approved high quality light weight granular fill compatible with the reinforcing materials and meeting also the PA requirements.

Unit weight:	12 kN/m <sup>3</sup>
Friction Angle ( $\Phi$ ):	35°
$K_a$ :	0.27

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

$$p_h = K_r K_a \sigma_v + \Delta \sigma_H \quad (\text{FHWA-NHI-10-024, ref. R-33})$$

where:

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

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

Regular Backfill:	21 kN/m <sup>3</sup>
Lightweight Fill (LWF):	12 kN/m <sup>3</sup>
Extruded Polystyrene (EPS):	0.5 kN/m <sup>3</sup>

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

All the property values discussed above are unfactored.

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



ELL = 8 kN/m & 5 kN/m	Earth pressure from Live Loads, LL=16 kPa for roadway (north abutment) and LL=9 kPa for trails (south abutment).
EDL = 11 kN/m & 17 kN/m	Earth pressure from Dead Surcharge load above the pile cap (for north and south abutment respectively).
EB = 8 kN/m & 9 kN/m	Earth pressure due to backfill behind the pile cap (for north and south abutment respectively).

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 geotechnical configurations developed for Tunnel T-7 (north and south abutments) are shown in Figure G1, Appendix G. The north and south abutments generally comprise RSS walls founded on a RGM foundation, approved clay backfill, EPS and LWF. Approved granular fill will be utilized for the pavement structure.

Along the south abutment at the Trail Bridge TB-5 location, the trail was modelled using approved granular fill and EPS. For analyses purposes, the tunnel diaphragm and pile cap (abutment stem) were assumed to be restrained (full restraint of lateral earth pressures) by the tunnel deck structure and soil reinforcement strips behind the pile cap.

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

The design assessments were based on (a) assumed strength and deformation properties of the proprietary components (RSS, RGM, EPS and LWF), which will have to be confirmed by proprietary suppliers, and (b) the assumed external loads and backfill properties. The final design of the abutment may require adjustments based on the proprietary components and structural design. In general, the RSS wall is to be designed and constructed in accordance with MTO's RSS Design Guidelines and Special Provisions SP599S22 and SP599S23.

The properties of the proprietary products used in the geotechnical analyses are summarized in Table 5-10:

**Table 5-10: Assumed Proprietary Product Properties**

Material	Unit Weight (kN/m <sup>3</sup> )	Limit Equilibrium (Slope/W Models)		Stress Deformation (Sigma/W Models)	
		Friction Angle	Apparent Cohesion (kPa)	E (MPa)	Poisson's Ratio, $\mu$
RSS (with LWF)	12.0	35°	50	40	0.35
RSS (with Approved Granular Fill)	21.0	35°	50	50	0.35
RGM	22.0	35°	50	50	0.35
EPS	0.5	0°	15	10	0.20

The properties assumed for the backfill materials are also given in Table 5-11:

**Table 5-11: Assumed Backfill Material Properties**

Backfill Material	Unit Weight, kN/m <sup>3</sup>	Undrained Shear Strength, kPa	Drained Angle of Internal Friction	Modulus of Elasticity, E, MPa	Poisson's Ratio, $\mu$
Compacted Clay Fill	21	50	30°	22.5	0.49 (undrained), 0.35 (drained)
Compacted Granular Fill	21	N/A	35°	50	0.35
Light Weight Fill (LWF)	12	N/A	35°	40	0.35

### 5.6.1 Global Stability

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

Figures E.1 to E.9 illustrate the stability models for sections of the north and south abutments of the tunnel. The global stability analyses have been carried out for both short-term and long-term loading conditions. Slope reinforcement associated with the presence of the piles was not considered in the stability models (conservative approach). A surcharge of 12 kPa was applied at the top of ground surface at the north abutment under travelled area and 9 kPa at the south abutment (landscape area) for both short and long term models.

The calculated factors of safety are as indicated in Table 5-12.

**Table 5-12: Calculated Factors of Safety**

Abutment	Factor of Safety for Loading Condition			Figure
	Short-Term	End-of-Construction	Long-Term (Drained)	
North Wall – Sta. 10+450L	1.45 (1.27*)	1.50 (1.36)	1.57 (1.49**)	E-1, E-2, & E-3
South Wall – Sta. 10+575L	1.44 (1.27*)	1.49 (1.31)	1.60 (1.48)	E-4, E-5, & E-6
South Wall – Sta. 10+450L	1.59 (1.32)	1.76 (1.47)	1.60 (1.48)	E-7, E-8, & E-9

**Legend:**

The FS values given outside and within the parentheses correspond to circular and non-circular failure surfaces

(\*) Base and Subbase materials over Highway 401 subgrade must be in place before completion of the top 1 m backfill (pavement / topsoil) over the abutments in order to improve the FS for the short-term critical non-circular slip surface

(\*\*) Incorporates a drainage blanket between the native silty clay and clay backfill extending from the RGM subdrain to elevation 178.5

“Short-term” condition refers to temporary undrained conditions when the structure is completed and in operation, however, the pavement box inside the tunnel is not present.

“End-of-Construction (EOC)” indicates undrained conditions when the structure and pavement are fully completed and in operation.

“Long-Term” condition refers to drained conditions for the complete structure in full operation after the complete stabilization of the pore water pressures.

### 5.6.2 Stress Deformation Analyses

Stress-deformation analyses (SDA) were carried out by finite element modeling using SIGMA/W software. The main purpose of the SDA was to assess the deformations of the soil mass supporting and surrounding the tunnel structure. Numerical studies indicated that the incorporation of the structural elements (piles, pile cap and tunnel roof) has little effects on the deformation response of the soil mass and causes modeling difficulties to the pore water dissipation process. As such, the structural elements (deck, girders, pile caps and piles) were not included in the model. The presence of the deck, girders and pipe caps were simulated with appropriate boundary restraints.

The configurations of the SDA models are presented in Figures F.1 and F.2. The calculation model typically assumed the following loading steps:

- Definition of the initial (in-situ) stress condition for level ground;
- Bulk excavation to the subgrade level under the highway pavement (90 day duration, Day 1 to 90);
- Construction of the RGM and pile installation (7 day duration, Day 90 to 97);
- Structure Construction. Construction of the RSS wall, pile cap, tunnel deck and associated backfill to underside of pavement structure (13 day duration, Day 97 to 110);
- Completion of the pavement structure for Highway 3 and Highway 401 (Day 111); and
- Dissipation of excess pore pressure (long-term).

The stratigraphy and selection of the soil properties (except for the RSS structure and pavement box) was based on the design soil properties discussed at Section 5.3. The RSS structure, RGM and pavement were assumed as a homogeneous elastic materials described in Table 5-9 and 5-10.

The SDA were carried out using a coupled stress – pore pressure effective stress-based model. The long-term phreatic surface was assumed to correspond to the initial groundwater level at elevation 179.1 m and follow the excavation and subgrade surfaces. Elastic-plastic Mohr-Coulomb models were used for all soil layers except for the unweathered firm to stiff silty clay which was described by the Modified Cam-Clay model. Hydraulic conductivity properties described in Table 5-2 were assigned to the different soil layers.

The construction stages represented by excavation, completion of the RGM, completion of the RSS and the entire abutment followed by the placement of the pavement box were assumed to occur over relatively short durations. Hence, the state of stress and deformations at the end of each of the first 4 stages largely corresponds to undrained conditions. After the completion of the entire construction, the model is allowed to dissipate the excess pore-pressures over a period of time until a steady-state condition of the pore water is achieved.

The SIGMA model was developed for the north and south abutment at Sta. 10+575L where the height of the retained soils measured from the top of finished grade to the bottom of the RSS is the highest and will provide the upper limits for the deformation estimates. The results for the SDA analyses on the north side are provided on Figures F1-1 to F1-4. The results for the SDA analyses on the south side are provided on Figures F2-1 to F2-4.

### 5.6.3 Serviceability Limit States (SLS) Performance

The SLS performance was assessed on the basis of the SDA described above in Section 5.6.2. SDA analyses reports and deformation charts are provided in Appendix F. The estimated deformations for the North and South Abutments are summarized as follows:

**Table 5-13: Summary of Calculated Cumulative Deformations for the North Abutment**

Parameter	End of RSS Construction (Day 110)	End of Construction (Day 111)	After 27 years	After 40 years
<b>Settlements on Top of Highway 3 at Distances from the Face of RSS of:</b>				
2 m (face of Diaphragm)	N/A	-15 mm (*)	-10 mm	-5 mm
10 m	N/A	-25 mm (*)	-15 mm	-10 mm
20 m	N/A	-15 mm (*)	-15 mm	-15 mm
30 m	N/A	-5 mm (*)	-5 mm	-5 mm
Settlement of top of RSS face	-25 mm (*)	-30 mm	-20 mm	-20 mm
Maximum Heave (rebound) at Highway 401 C/L	N/A	45 mm (*)	65 mm	80 mm

N/A Not Applicable (area located within the temporary excavation).

Note: Distances measured perpendicular to the tunnel abutment. Long-term condition implies full pore pressure dissipation.

(\*) Indicates calculated settlement compensated during construction of structures

The deformations are rounded to closest 5 mm.

-(ve) indicates settlement

**Table 5-14: Summary of Calculated Cumulative Deformations for the South Abutment**

Parameter	End of RSS Construction (Day 110)	End of Construction (Day 111)	After 27 years	After 40 years
<b>Settlements on Top of Approach Way at Distances from the Face of RSS of:</b>				
2 m (face of Diaphragm)	N/A	-15 mm (*)	-20 mm	-20 mm
10 m	N/A	-25 mm (*)	-30 mm	-30 mm
20 m	N/A	-30mm (*)	-45 mm	-45 mm
30 m	N/A	-30 mm (*)	-45 mm	-45 mm
Settlement of top of RSS face	-30 mm (*)	-30 mm	-30 mm	-30 mm
Maximum Heave (rebound) at Highway 401 C/L	N/A	45 mm (*)	75 mm	75 mm

N/A Not Applicable (area located within the temporary excavation).

Note: Distances measured perpendicular to the tunnel abutment.

(\*) Indicates calculated settlement compensated during construction of structures

The deformations are rounded to closest 5 mm.

-(ve) indicates settlement

The ground movements generated by the construction loads are anticipated to effectively stabilize within approximately 25 to 40 years following completion of construction.

SLS lateral performance of the north RSS wall is illustrated on Figure F1-6. Lateral deformations of the wall will be less than 10 mm, with rotations less than 0.5% SLS lateral performance of the south RSS

wall is illustrated on Figure F2-6. Lateral deformations of the wall will also be less than 10 mm, with rotations less than 0.5%.

Due to the relatively smooth changes in the geometry of the tunnel along the north abutment, the above settlement changes along Highway 401 are anticipated to be gradual in the longitudinal direction. However, given the presence of the TB-5 embankment (and associated bridge) and Huron Church Line along the south abutment, the deformations along the south abutment are expected to vary significantly. The section analysed is expected to represent the largest deformations.

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

The settlements discussed above do not include deformations caused by seasonal temperature and moisture variations and due to the effects of the long-term compression of the backfill materials that are expected to be nominal. In this regard, it is recommended that stringent compaction control be exercised to minimize the magnitude of backfill compression.

#### 5.6.4 RSS Wall External Stability

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

The design of the RSS and RGM may be based on the following ultimate subgrade bearing capacities:

**Table 5-15: Subgrade Ultimate Bearing Capacity**

Abutment	Assumed Lowest Subgrade Elevation	Loading Condition	$q_u$ (kPa)
North Side (with 1.5 m thick RGM)	172.8 <sup>(1)</sup>	Short-Term (Undrained)	280 <sup>(2)</sup>
		Long-Term (Drained)	960 <sup>(3)</sup>
South Side (with 1.5 m thick RGM)	174.1 <sup>(1)</sup>	Short-Term (Undrained)	280 <sup>(2)</sup>
		Long-Term (Drained)	760 <sup>(3)</sup>

(1) Below RGM

(2) Based on an average cohesion of 52-55 kPa within the assumed zone of influence of the RSS wall foundation

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

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

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

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



$c'$  = cohesion/adhesion at sliding interface

$\delta$  = friction angle at sliding interface

$V$  = vertical force (kN)

$H_f$  = design horizontal load (kN)

Allowance for buoyancy where applicable should be made.

The following soil properties at the interfaces between the RSS, RGM and silty clay subgrade can be used in the design:

**Table 5-16: Soil Properties for use in Base Sliding**

Interface	Undrained (Short-Term)		Drained (Long-Term)	
	$\delta$ , degrees	$c$ , kPa	$\delta'$ , degrees	$c'$ , kPa
RSS to RGM	35	0	35	0
RGM to Silty Clay	0	50	30	0

Based on geotechnical analyses discussed in Sections 5.6.1 to 5.6.4, preliminary abutment configurations and dimensions were determined (Table 5-17). As noted previously in Section 5.6, the abutment configurations and dimensions indicated in these analyses are preliminary and are to be finalized by proprietary suppliers. The final design of the abutments is to be developed in consultation with the suppliers of the proprietary components.

**Table 5-17: Abutment Tentative Dimensions**

Abutment	Assumed Total Height <sup>(1)</sup> , m	RGM <sup>(2)</sup> Size (Thickness x Min. Width at Base), m	LWF <sup>(2)</sup> required behind and/or top of RSS wall	EPS <sup>(2)</sup> , m <sup>3</sup> /m	RSS Structure Size (Width x Height) <sup>(3)</sup> , m
North	9.9	1.5 x 8.0	Yes	44	6.5 x 5.0
South <sup>(4)</sup>	8.4	1.5 x 7.5	No	6	6.0 x 3.7

(1) Measured from top of finished grade at to the base of the RSS structure.

(2) In general, the use of RGM, LWF and EPS is required to meet the ULS design compliance for undrained short-term condition.

(3) The RSS supplier may require wider structures to meet the internal design requirement. The effects of a wider structure on bearing capacity will need to be assessed.

(4) Based on Sta. 10+450L

## 5.7 RGM Foundation Loads

A RGM foundation comprising Granular A or B Type II was considered under the RSS false abutment walls to improve the load distribution to the bearing soils and satisfy the bearing capacity requirements for undrained conditions at the north and south abutments. 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 at a 45° angle. The following loads were estimated to act

on top of the RGM on the basis of conventional calculation of the bearing pressures under gravity retaining walls:

**Table 5-18: Estimated Load on RGM**

RGM Thickness	Abutment Location	Average unfactored bearing pressure, kPa
1.5 m	North Wall	155
1.5 m	South Wall	130

Based on the above load on RGM, an estimated unfactored horizontal tensile load of 40 kN per meter of RGM was estimated across the entire height of 1.5 m at the north abutment and 32 kN across the entire height of 1.5 m at the south abutment. For cost estimates, a factored (1.4) tensile load can be accommodated by 2 layers of UX1400HS, or equivalent for the north and south abutments.

The above loads are for the use by the RGM suppliers to assist in the RGM's internal design.

## 5.8 Wing Walls

High RSS wing walls aligned parallel to the tunnel centre line are indicated at the north abutment. The south abutment has return wall perpendicular to the abutment and a lower wing wall parallel to the tunnel center line that tapers down away from the structure. Global stability analyses were carried out on RSS wing walls and return wall for the walls located at the northwest and southwest corners of the structure (which are considered to be the highest on each side). The calculated factors of safety, summarized in Table 5-19, are in excess of 1.3 against global instability. Figures E10 to E18 in Appendix E illustrate the stability models for the wing walls.

The northwest wing wall has been designed with a 2.0 m thick RGM, LWF within the RSS wall and no EPS. The south west wing wall has been designed with a 1.5 m thick RGM, granular fill within the RSS and no EPS. The south west return wall has been designed with LWF.

Similar to the abutment RSS walls, the RSS wing walls have been preliminarily checked for ULS bearing and sliding resistances.

**Table 5-19: Calculated Factors of Safety for Wing Walls against Global Instability**

Wing Wall Location	Wall Height	RGM Thickness	Preliminary Width of RSS Wing Wall	Calculated Factor of Safety		
				Undrained Condition		Drained Condition
				Short-Term	EOC	Long-Term
North West and North East (High Wall)	9.0 m	2.0 m	10.0 m	1.45 (1.25*)	1.5 (1.29)	1.69 (1.58)
South Tapered Walls	3.9 m	1.5 m	4.0 m	1.66 (1.43)	1.86 (1.57)	1.71 (1.55)
South Return Walls	4.9 m	-	4.0 m	1.46 (1.33)	1.67 (1.55)	2.09 (2.00)

Legend:

(\*)The FS values given outside and within the parentheses correspond to circular and non-circular failure surfaces. Base and Subbase materials over Highway 401 subgrade must be in place before completion of the pavement of Hwy 3 over the north abutment in order to improve the Fs for the short-term critical non-circular slip surface

“Short-term” condition refers to temporary undrained conditions when the structure is completed and in operation. However, the 401 pavement box is not present. For the return wall this includes the construction of the tapered wall but not the 401 pavement box.

“End-of-Construction (EOC)” indicates undrained conditions when the structure and pavement are fully completed and in operation.

“Long-Term” condition refers to drained conditions for the complete structure in full operation after the complete stabilization of the pore water pressures.

## 5.9 Backfilling

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

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

Heavy compaction equipment should not be used immediately adjacent to the walls of the structure. The backfill adjacent the structure walls should be placed in thin (maximum 100 mm thick) loose lifts and compacted using light rollers or other compactors approved by the Engineer. Effects of backfill compaction activities should be simulated as live load over and above the static lateral earth pressure for structural design in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06.

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

Earth pressures on abutments and wing walls may be calculated on the basis of the following parameters:

**Table 5-20: Soil Parameters for Earth Pressure Calculations**

Soil Parameter	Group I Soils	Group II Soils	Group III Soils
Fill Unit Weight, kN/m <sup>3</sup>	22	21	20.5
Friction Angle Range	33-35	29-32	22-30
Coefficients of Static Lateral Earth Pressure:			
'Active' or Unrestrained, $K_a^{(*)}$	0.27 to 0.30	0.31 to 0.35	0.33 to 0.45
'At Rest' or Restrained, $K_o^{(*)}$	0.43 to 0.46	0.47 to 0.52	0.50 to 0.62
'Passive', $K_p^{(*)}$	3.3 to 3.7	2.9 to 3.2	2.2 to 3.0

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

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

Legend:

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

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

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

$$K_o = (1 - \sin\phi)(1 + \sin\beta)$$

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

Where:  $\phi$  = Friction angle of backfill material,

$\beta$  = Slope of the backfill surface.

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

## 5.10 Permanent Subdrainage System

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

Use of free draining granular soils within the reinforced soil mass of the RSS structures and the RGM will ensure that these structures will act as a “natural” drain conveying the seepage resulted from the phreatic groundwater and infiltrations from surface precipitations toward the toe of the wall facing and base of the

RGM. In order to prevent accumulation and stagnation of groundwater within the RGM, the subgrade should be graded to direct the collected groundwater to subdrains discharging to manholes or sumps.

Depending on the grain size of the LWF and RGM materials, a filter layer may also be required at the interface between the native soils and imported granular materials. The LWF should be wrapped in geotextile.

A simplified steady-state model (Appendix I) was used to estimate seepage rates associated with the long-term drawdown of the groundwater along a typical cross-section of Tunnel T-7. SEEP/W 2007 software was used for this analysis. Groundwater recharge from infiltrations from ground surface sources was also considered. The rates of recharge were estimated on the basis of saturated hydraulic conductivity of the

soils in conjunction with the assumption that no mounding of the long-term groundwater should occur. A ground surface infiltration rate of  $6 \times 10^{-6}$  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 2 litres/day per meter length of the tunnel. This is an approximate estimate and the actual quantities could differ significantly from this magnitude. The above flow rate does not include additional seepage that may occur from other external sources, like runoff from ground surface, perched groundwater, or accidental water main breaks.

## 6 Other Geotechnical Recommendations

### 6.1 Construction Dewatering

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

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

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

All surface water should be directed away from all open excavations.

### 6.2 General Construction Requirements

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

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

- All excavation works should be carried out in accordance with the guidelines outlined in Occupational Health and Safety Act (OHSA) and OPSS 902. The native undisturbed soils may be classified as Type 3 soils. The excavations below the original ground levels may intersect water bearing backfill within trenches of active and/or abandoned utilities. In these cases, Type 4 soil conditions may occur and should be addressed accordingly.
- The silty clay soils at the project site are highly susceptible to rapid deterioration when exposed to elements, weathering and/ or subjected to direct construction traffic.
- Temporary slopes, permanent slopes, and subgrade areas must be appropriately protected at all times against surface erosion due to runoff, desiccation, freeze-thaw effects, etc.



- To protect the integrity of subgrade for foundations and pavements, the final excavation lift above the design elevation should not be less than 500 mm and should be carried out only when the Contractor is ready to prepare and cover the subgrade with the materials specified in the design the same day the final excavation is exposed and approved. No construction traffic should be permitted over subgrade without approved protective covers.
- The final excavation of the layer above the design subgrade is to be carried out using buckets equipped with smooth lips. Once exposed, the subgrade must be immediately inspected. Upon approval, the subgrade should be immediately protected. Depending on the type of construction, geofabrics, granular mats, a skim coat (minimum 75 mm thick) of lean concrete protection (mud mat), etc. should be used.
- As indicated earlier, pore pressures, heave/settlement behaviour and presence of gassy soils below the excavation should be monitored diligently during excavation. If the presence of gassy soils is evidenced (for example, dissolved gas bubbles coming out of solution and softening of the excavation face), the excavation should be carried out in small (say 1 m) depth increments and sufficient time to dissipate the pore pressures should be allowed at each excavation stage.
- Regular monitoring and inspection of the condition of temporary slopes and excavation base for signs of instability, deterioration, sloughing, etc should be carried out by qualified personnel. Appropriate mitigation measures should be implemented.
- Regular monitoring of geotechnical instrumentation should be carried out by qualified personnel. Appropriate mitigation measures should be implemented.
- Excavations should be limited in size in the area and appropriate monitoring of the nearby residences should take place. Monitoring should consist of a precondition survey and regular surveying during and after construction of the nearby utilities, residences, etc.
- Rip/rap, or otherwise coarse rockfill cover are considered to have half the insulation effect as offered by soil deposits/cover, and therefore, the depth of frost penetration will have to be increased proportionally.

**Fill Staging:** Prior to placement of fill, including the trail embankment, higher than above the top of the RSS wall, the tunnel deck must be completed.

### 6.3 Instrumentation and Monitoring during Construction

As mentioned earlier in Section 5.4, a program of site instrumentation and monitoring of the temporary works during construction should be implemented by the Contractor in addition to the limited instrumentation already installed during the geotechnical investigation.

Details and recommendations for additional instrumentation, monitoring program, as well as guidelines for alert levels, interpretation and contingencies are provided in a separate report 285380-04-118-0001

The Contractor is responsible for planning, installation and maintenance of instrumentation as well as the completion of monitoring of the response of the excavations (ground movement) during construction.

Detailed plans and procedures should be submitted to HMQ for approval at least 3 month prior to commencement of the monitoring of the works.

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

## 6.4 Corrosion Potential

Analytical testing was carried out on samples of the sandy silt/silty sand obtained in three at Tunnel T-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 and metallic elements:

**Table 6-1: Results of Analytical Testing on Soils**

Location of Soil Samples	Elevation of Soil Sample	pH	Redox Potential, mV	Resistivity, ohm.cm	Sulphide, mg/kg	Sulphate, mg/kg
Borehole T7-1 (Sample 9)	173.3	7.98	130	2960	<0.20	200
Borehole T7-2 (Sample 10)	172.1	8.03	131	2440	<0.20	360
Borehole T7-3 (Sample 12)	168.1	7.94	120	2500	<0.20	307

The reported results of laboratory testing indicate that based on CSA A23.1, concrete in contact with the tested soil material would have a negligible degree of exposure to sulphate attack (ref. R-10).

Based on the measured electrical resistivity, pH, redox potential, sulphide contents etc., the tested soil would be considered noncorrosive to buried metallic elements (ref. R-2). A corrosion specialist should review the test result and confirm their adequacy.

## 6.5 Construction Quality Control

To ensure that construction is carried out in a manner consistent with the intent of the recommendations set forth in this report, a construction quality control program, including geotechnical inspection, field and laboratory 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 potential 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.

## 6.6 Lennon Submerged Culvert

The Lennon submerged culvert is to cross Highway 401 just west of Tunnel T-7 near Highway 401 station 10+425L. The geotechnical aspects of Lennon culvert are being addressed under a separate cover.

These include stability and estimates of potential soil ground deformations adjacent to the drain excavations.

The base of the submerged culvert excavation is approximately 166.5 m, this is 6 m deeper than the invert of the tunnel. The culvert is 20-25 m away from the end of tunnel and 10-15 m away from the end of the wing wall.

The tunnel structure is supported on piles in the vicinity of the culvert; however the RSS walls may be impacted by the construction of the drain. Theoretical analysis indicates that the excavation for the submerged culvert if carried out after the completion of the main 401 excavation would have nominal impact at a distance greater than 15 m. However, considering the proximity of the RSS abutment and wing walls, careful consideration must be given to the construction approach and adequate monitoring during the excavation and construction of the drain (assuming that the drain is constructed after substantial completion of the tunnel). Close consultation between the Contractors and Engineers should take place regarding the staging and construction methods intended.

## 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 provides recommendations current to the issue date only. The geotechnical analyses and optimisations of the project are ongoing as design configurations and material requirements are updated and provided to us. These analyses will be updated and reported at a later date.

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

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

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

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

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

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

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

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

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

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

## 8 Closure

The geotechnical report for Tunnel T-7 was prepared by Thomas Ring, EIT of AMEC and Mr. Wayne Hurley, P.Eng. of TBT Engineering and checked by Dr. Dan Dimitriu, P.Eng. (lead designer). The project was executed under the technical direction of Dr. Narendra S. Verma, P.Eng. who also provided the senior review of the report. Mr. Matt Oldewening, P.Eng. managed the geotechnical investigation and Mr. Brian Lapos, P.Eng. was the project manager.

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

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Yours truly,

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## 9 References

- R-1. Aas, G., Lacasse, S., Lunne, T. and Hoeg, K., 1986, Use of in situ tests for foundation design on clays. Proc. ASCE Spec. Conf. In Situ '86, ASCE GSP 6, 1-30.
- R-2. American Water Works Association, 2005, ANSI/AWWA C105/A21.5-05 American National Standard for Polyethylene Encasement for Ductile-Iron Pipe Systems.
- R-3. Barton, N. R., Lien, R. and Lunde, J., (1974). Engineering Classification of Rock Masses for the Design of Tunnel Support, Rock Mech. 6(4), 189-239.
- R-4. Bhushan, Kul, Amante, Carlos V. and Saaty, Ramzi, 2000, Soil improvement by precompression at a tank farm site in Central Java, Indonesia, Feb. 14.
- R-5. Bieniawski, Z.T. (1976). Rock mass classification in rock engineering. In exploration for rock engineering, Proc.. of the Symp. on Exploration for Rock Engineering (ed. Z.T. Bieniawski) A.A. Balkema, Rotterdam, 1, 97-106. Cape Town.
- R-6. Bjerrum, L. 1972, Embankments on soft ground: SOA Report. Proc. Specialty Conference on Performance of Earth and Earth-Supported Structures, ASCE, Purdue, 2, 1-54.
- R-7. Byrne, P., Cheung, H. and Yan, L., 1987, Soil Parameters for Deformation Analysis of Sand Masses, Canadian Geotechnical Journal, vol. 24, pp. 366-376
- R-8. Campanella, R.G. and Howie, J.A., 2005, Guidelines for the Use, Interpretation and application of seismic piezocone test data, A Manual on Interpretation of Seismic Piezocone Test Data for Geotechnical Design, June.
- R-9. Canadian Geotechnical Society, 2006, Canadian Foundation Engineering Manual (CFEM), 4th Edition.
- R-10. Canadian Standard Association, 2006, Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-06 S6.1.06.
- R-11. Canadian Standard Association, 2009, Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete CAN/CSA-A23.
- R-12. Chandler, R.J., 1988, The in-situ measurement of the undrained shear strength of clays using the field vane: SOA paper. Vane Shear Strength Testing in Soils Field and Laboratory Studies, ASTM STP 1014, 13-44.
- R-13. Demers, D. and Leroueil, S., 2002, Evaluation of preconsolidation pressure and the overconsolidation ratio from piezocone tests of clay deposits in Quebec. Canadian Geotechnical Journal, 39(1), 174-192.
- R-14. Department of the Navy, 1986, Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2, Naval Facilities Engineering Command.
- R-15. Dittrich, J.P., Rowe, R.K. Becker, D.E. and Lo, K.Y., 2010, Influence of ex-solved gases on slope performance at the Sarnia approach cut to the St. Clair Tunnel, Canadian Geotechnical Journal, 47, 971-984.

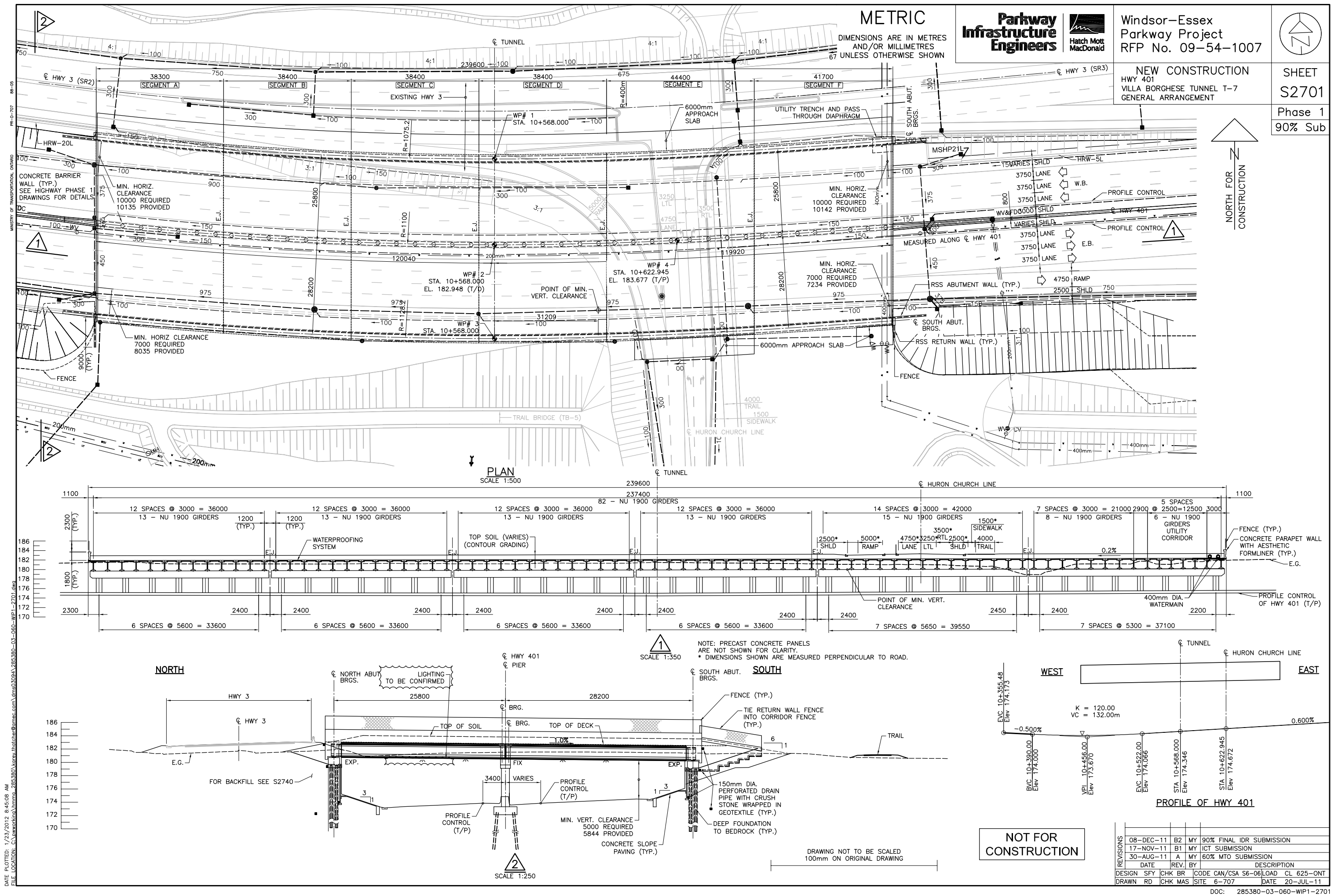
- R-16. Duncan, J.M. et al, 1980, Strength, Stress-Strain and Bulk Modulus Parameters for Finite Element Analyses of Stresses and Movements in Soil Masses, Report UCB/GT/80-01
- R-17. Golder Associates Ltd., 2006, Structure Settlement Study, Highway 401 Reconstruction, GWP 64-00-00, MOT Ontario, SW Region, Geocres No. 40J2-79, November, 2006
- R-18. Golder Associates Ltd., 2007, Preliminary foundation investigation and design report, Detroit River International Crossing Bridge Approach Corridor, Geocres No. 40J6-18, October.
- R-19. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Geocres No. 40J6-27, June.
- R-20. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Subsurface Conditions – Baseline Report, Geocres No. 40J6-28, June.
- R-21. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Subsurface Conditions – Interpretation Report, Geocres No. 40J6-28, Revision December.
- R-22. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 1 – Soil Chemistry Data, Geocres No. 40J6-27, February.
- R-23. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 2 – In Situ Cross Hole and Vertical Seismic Profile Testing, Geocres No. 40J6-27, March.
- R-24. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 3 – Supplementary Cone Penetration Testing, Geocres No. 40J6-27, February.
- R-25. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 4 – Supplementary Geotechnical Investigation, March.
- R-26. Grozic, J.L., Nadim, F, and Kvalstad, T.J., 2005, On the undrained shear strength of gassy clays, Computers and Geotechnics, Elsevier, 483-490.
- R-27. Grozic, J.L., Robertson, P.K., and Morgenstern, N.R., 1999, The behaviour of loose gassy sand, Canadian Geotechnical Journal, 36, 482-492.
- R-28. Holtz, R.D. and Kovacs, W.D., 1981, An Introduction to Geotechnical Engineering, Prentice-Hall Inc., chapter 11
- R-29. Hudec, P.P., 1998, Geology and Geotechnical Properties of Glacial Soils in Windsor.
- R-30. ISSMGE Committee TC16, 2001, The Flat Dilatometer tests (DMT) in soil investigations Report, by the International Conference on In situ Measurements of Soil Properties, Bali, Indonesia.
- R-31. International Society for Rock Mechanics (ISRM), 1978. Suggested methods for the quantitative description of discontinuities in rock masses. Int. J Rock Mech. Min. Sci. & Geomech. Abstr. 15, 319-368.
- R-32. Kenney, T.C., 1959, “Discussion”, Journal of the Soil Mechanics and Foundations Division, ASCE, vol. 85, no. SM3, pp. 67-79
- R-33. Kulhawy, F.H. and Mayne, P.W., 1990, Manual on Estimating Soil Properties for Foundation Design, Report EPRI-EL6800, Palo Alto, CA, Electric Power Research Institute.

- R-34. Ladd, C.C., 1971, Strength Parameters and Stress-Strain Behaviour of Saturated Clays, Research Report R71-23, Soils Publication 278, Dept. of Civil Engineering, M.I.T., Cambridge, Mass.
- R-35. Ladd, C.C., and Foott, R. 1974, New design procedure for stability of soft clays, Journal of the Geotechnical Engineering Division, 100(GT7), 763-786.
- R-36. Ladd, C.C., Foott, R., Ishihara, K., Schlosser, F., and Poulos, H.G. 1977, Stress-deformation and strength characteristics: SOA report. Proc., 9th Int. Conf. on Soil Mechanics and Foundation Eng., Tokyo, 2, 421-494.
- R-37. Ladd, Charles C. and DeGroot, Don J., 2004, Recommended practice for soft ground site characterization: Arthur Casagrande Lecture, 12th Pan-American Conference on Soil Mechanics and Geotechnical Engineering, MIT Cambridge, MA USA, June 22-25, 2003, Revised May 9.
- R-38. Leroueil, S., Magnan, J-P., and Tavenas, F., 1990, Embankments on Soft clays, Ellis Horwood.
- R-39. Leroueil, S., Demers, D., and Saihi, F., 2001, Considerations on stability of embankments on clay, Soils and Foundations, Japanese Geotechnical Society, Vol. 41, No. 5, 117-127, Oct.
- R-40. Lo, K.Y. and Hinchberger, S.D., 2006, Stability analysis accounting for macroscopic and microscopic structures in clays, Keynote Lecture, Proceeding 4th International Conference on Soft Soil Engineering, Vancouver, Canada, pp 3-34, Oct. 4-6.
- R-41. Lunne, T., Robertson, P.K., and Powel, J., 1997, Cone Penetration Testing in Geotechnical Practice.
- R-42. Ministry of Transportation Ontario, 1990, Pavement Design and Rehabilitation Manual, SDO-90-01.
- R-43. Quigley, Robert M., 1980, Geology, mineralogy, and geochemistry of Canadian soft soils: a geotechnical perspective, National Research Council of Canada, Canadian Geotechnical Journal, Vol. 17, pp. 261-285.
- R-44. Schofield, A.N. and Wroth, C.P., 1968, Critical State Soil Mechanics, McGraw Hill Book Company, London, p. 157
- R-45. Sobkowicz, J.C. and Morgenstern, N.R., 1984, The undrained equilibrium behaviour of gassy sediments, Canadian Geotechnical Journal, Vol. 21, pp. 439-448.
- R-46. Terzaghi, K., Peck, R.B., and Mesri, G., 1990, Soil Mechanics in Engineering Practice, John Wiley and Sons, NY.
- R-47. Windsor-Essex Mobility Group, 2010, Design Submission, Section 5.1.3 – Geotechnical Design.
- R-48. Wyllie, D.C., 1999, Foundations on Rock, 2nd edn, Taylor and Francis, London, UK, 401 pp.

## Drawings

Project: Windsor-Essex Parkway  
Document: Geotechnical Investigation and Design Report– Tunnel T-7  
(Hwy. 401 Sta. 10+450L to Sta. 10+700L, LaSalle) Geocres No. 40J6-37  
Doc No.: 285380-04-119-0028

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



DATE PLOTTED: 1/23/2012 8:47:21 AM  
FILE LOCATION: C:\pwworking\mining-285380\wpcad\dwg\285380-03-061-WP1-2705.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

PR-12-707

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

Parkway  
Infrastructure  
Engineers



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



NEW CONSTRUCTION  
HWY 401  
VILLA BORGHESE TUNNEL T-7  
FOUNDATION LAYOUT

SHEET  
S2705

Phase 1  
90% Sub

NOTES:

- FOR GENERAL NOTES SEE SHEET S2702.
- THIS DRAWING TO BE READ IN CONJUNCTION WITH THE RETAINED SOIL SYSTEM WALLS AND ABUTMENT LAYOUT DRAWINGS.

PILE NOTES:

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

CONSTRUCTION SEQUENCE - ABUTMENTS:

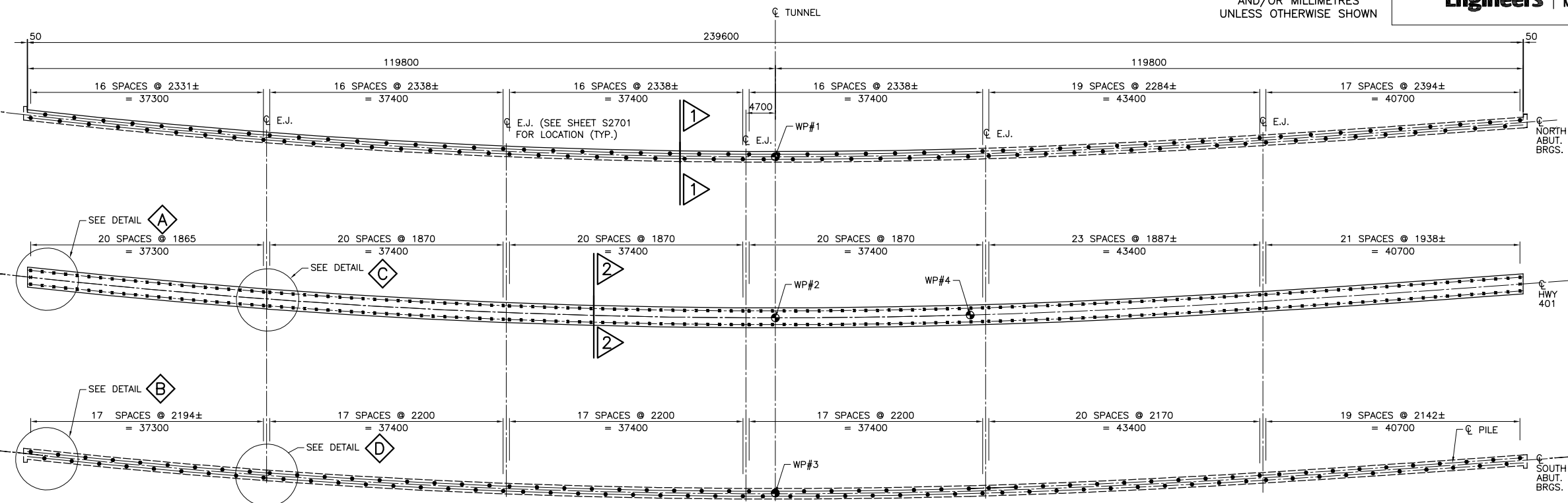
- EXCAVATE FOR TUNNEL OPENING.
- PROVIDE SUITABLE PROTECTION DURING CONSTRUCTION.
- EXCAVATE TO UNDERSIDE OF RGM.
- INSTALL RGM.
- DRIVE PILES.
- PLACE 500mm DIA. GALVANIZED CSP PIPES AND PROVIDE BLOCKING OF PILES IN CSP PIPES.
- CONSTRUCT RETAINED SOIL SYSTEM WALLS AND BACKFILL TO UNDERSIDE OF ABUTMENTS. BACKFILL AS PER S 2740.
- FILL 500mm DIA. CSP PIPES WITH CONCRETE.
- CONSTRUCT ABUTMENTS TO UNDERSIDE OF BEARING PEDESTALS.
- CONTRACTOR TO PROVIDE SUITABLE STABILITY DURING CONSTRUCTION.
- SEE ABUTMENT DETAILS DRAWING FOR CONTINUATION.

CONSTRUCTION SEQUENCE - PIER:

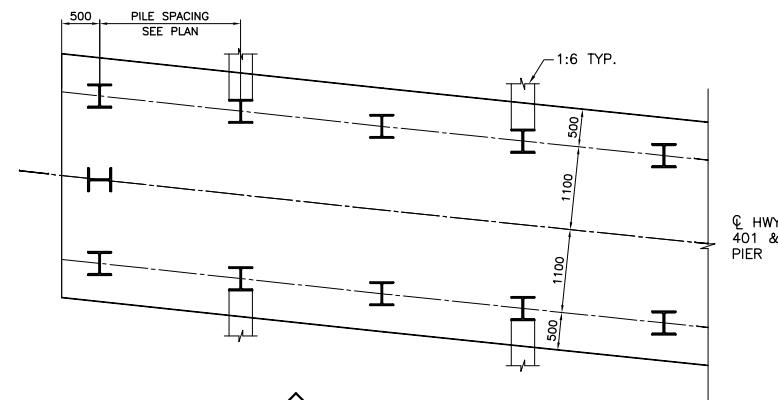
- EXCAVATE TO UNDERSIDE OF FOOTING.
- DRIVE PILES.
- PLACE FOOTING PAD.
- CONTRACTOR TO PROVIDE SUITABLE STABILITY DURING CONSTRUCTION.
- CONSTRUCT PIER FOOTING TO TOP OF FOOTING
- SEE PIER DETAILS DRAWING FOR CONTINUATION.

APPLICABLE STANDARD DRAWINGS:

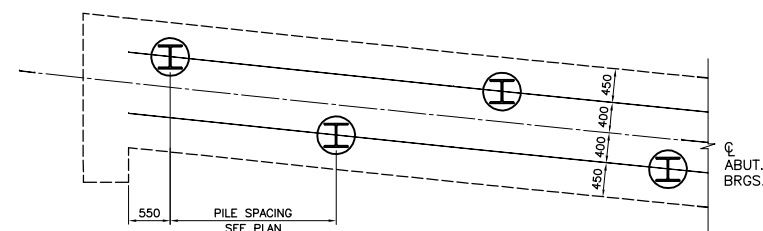
OPSD-3000.100 FOUNDATION PILES - STEEL H--PILE DRIVING SHOE  
OPSD-3000.150 FOUNDATION PILES - STEEL H--PILE SPLICE



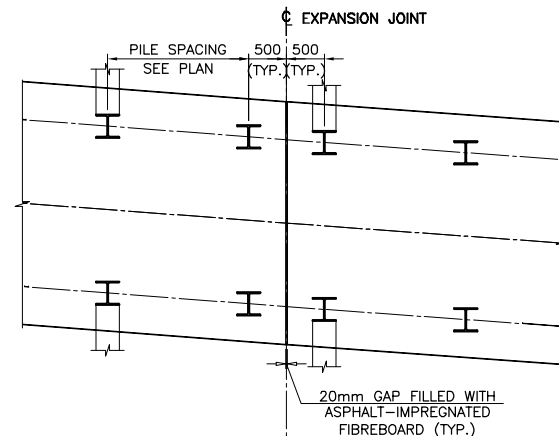
PLAN  
SCALE 1:400



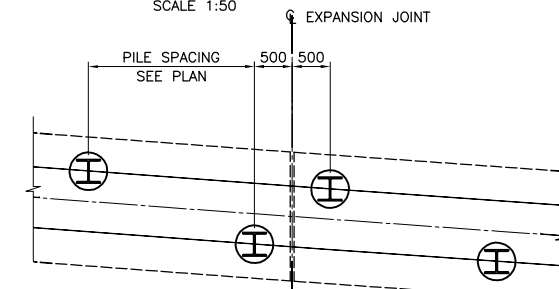
A  
SCALE 1:50



B  
SCALE 1:50



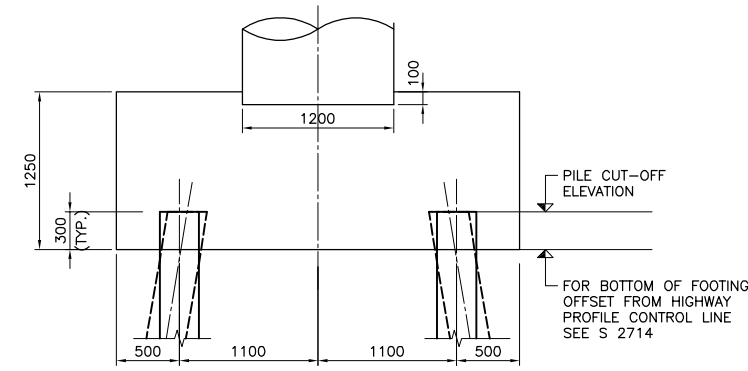
C  
SCALE 1:50



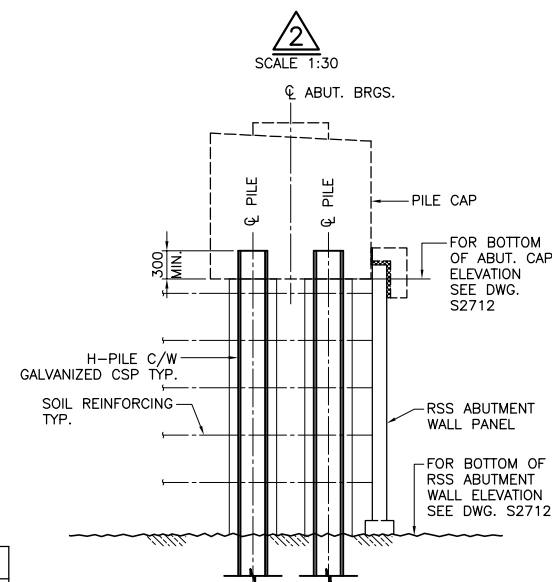
D  
SCALE 1:50

PILE DATA			
LOCATION	No. REQUIRED	LENGTH (m)	BATTER
N. ABUTMENT	106	32.3	VERTICAL
PIER	132	26.3	VERTICAL
	130	26.6	1:6
S. ABUTMENT	113	31.8	VERTICAL

WORKING POINT DATA		
LOCATION	NORTHING	EASTING
WP #1	4 679 365.452	332 382.417
WP #2	4 679 343.781	332 368.416
WP #3	4 679 320.196	332 353.179
WP #4	4 679 327.217	332 394.871



2  
SCALE 1:30  
CL ABUT. BRGS.



1  
SCALE 1:40

DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

NOT FOR  
CONSTRUCTION

REVISIONS	DATE	REV.	BY	DESCRIPTION
08-DEC-11	B2	MY		90% FINAL IDR SUBMISSION
17-NOV-11	B1	MY		ICT SUBMISSION
30-AUG-11	A	MY		60% MTO SUBMISSION
DESIGN	SFY	CHK	MAS	CODE CAN/CSA S6-06 LOAD CL 625-ONT
DRAWN	RD	CHK	BR	SITE 6-707 DATE 20-JUL-11

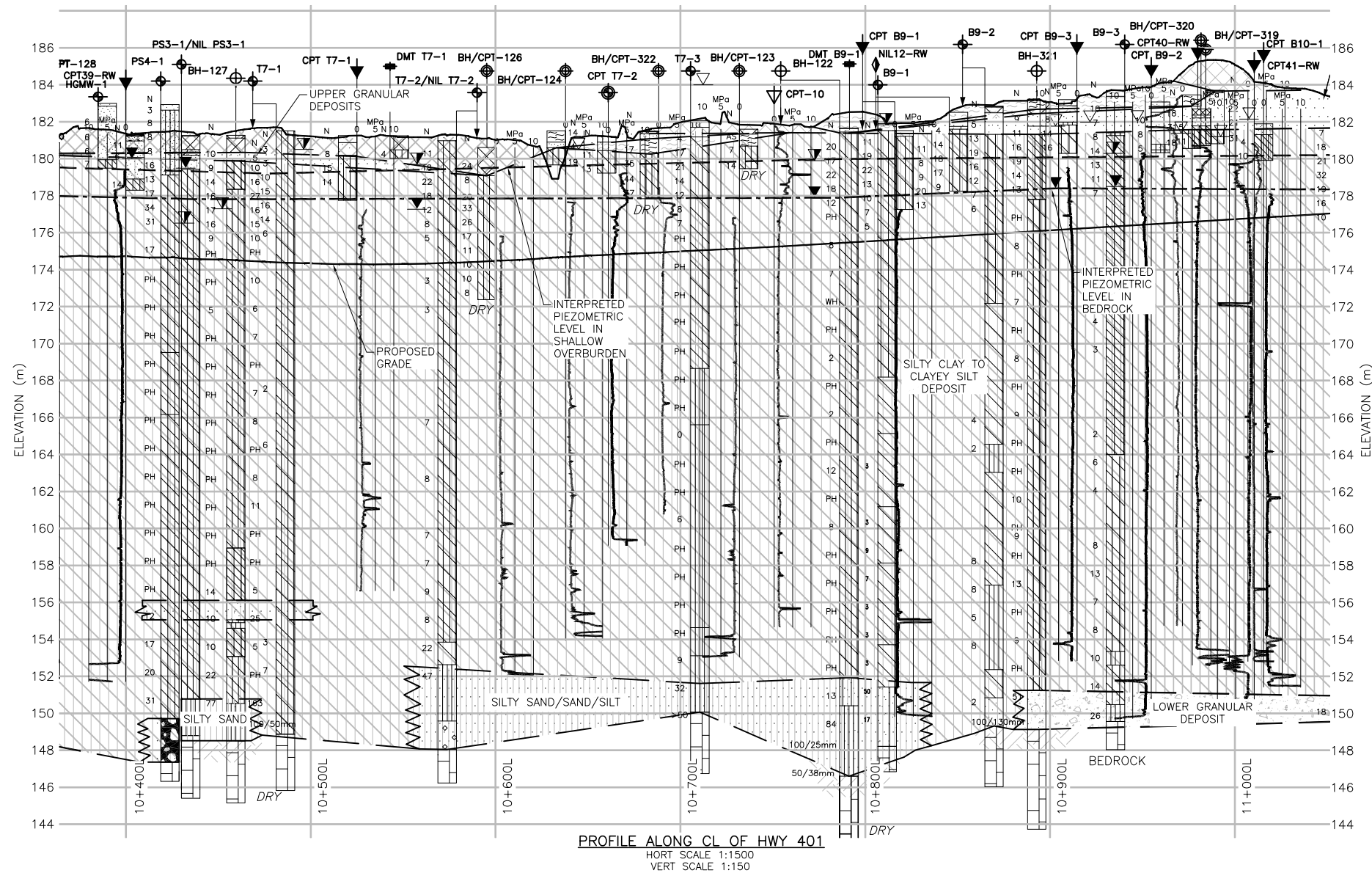
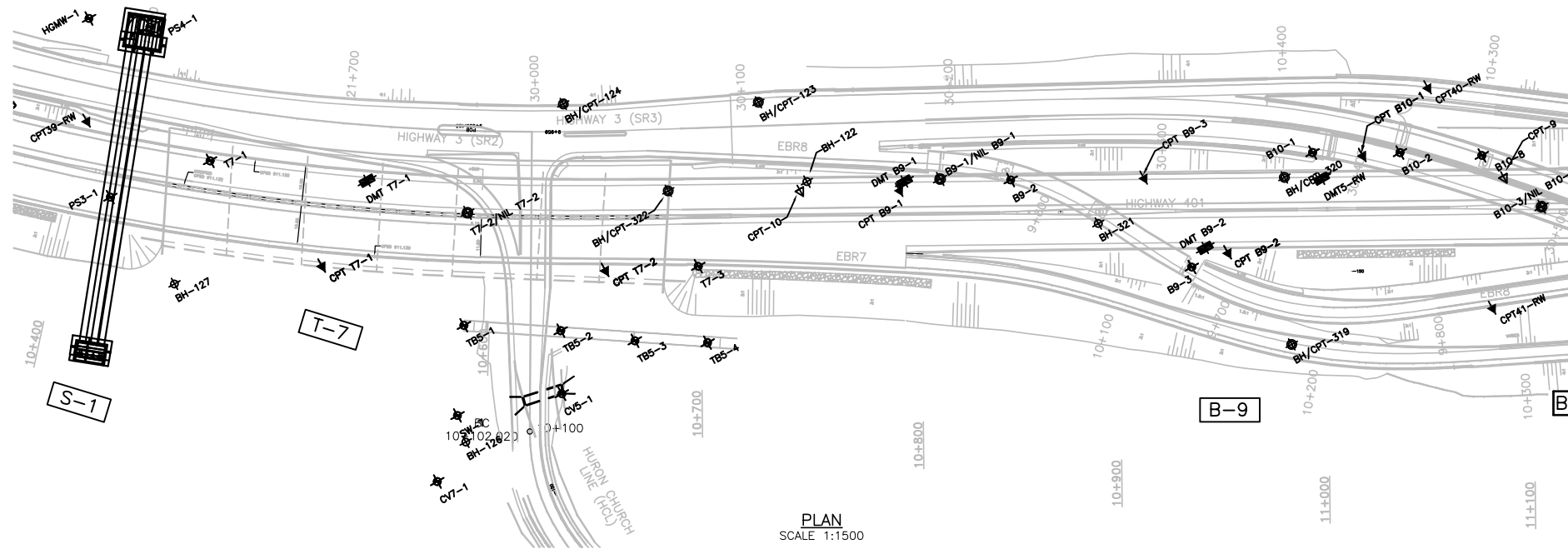
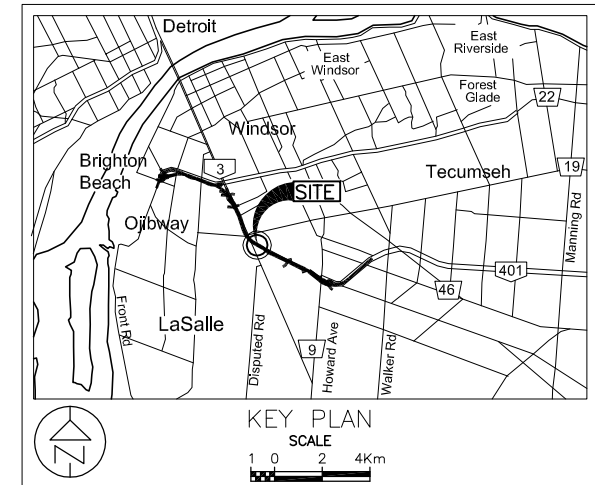
DOC: 285380-03-061-WP1-2705



**Parkway Infrastructure Engineers** | **amec**   
 **Hatch Mott MacDonald**

SHEET

Phase 1



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

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

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

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

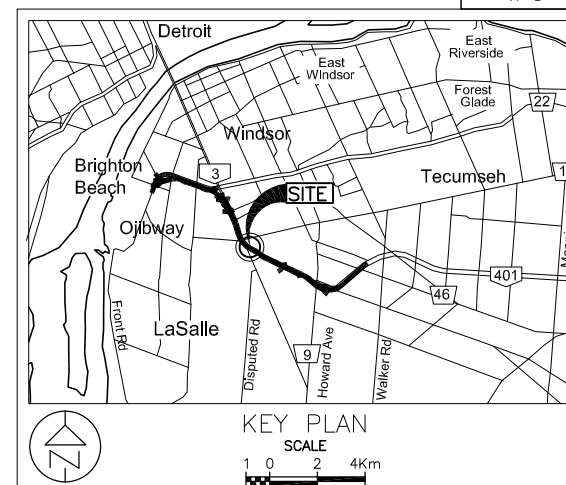
NEW CONSTRUCTION  
HWY 401  
VILLA BORGHESE TUNNEL T-7  
BOREHOLE LOCATIONS & SOIL STRATA

SHEET



G2703

Phase 1

IFC

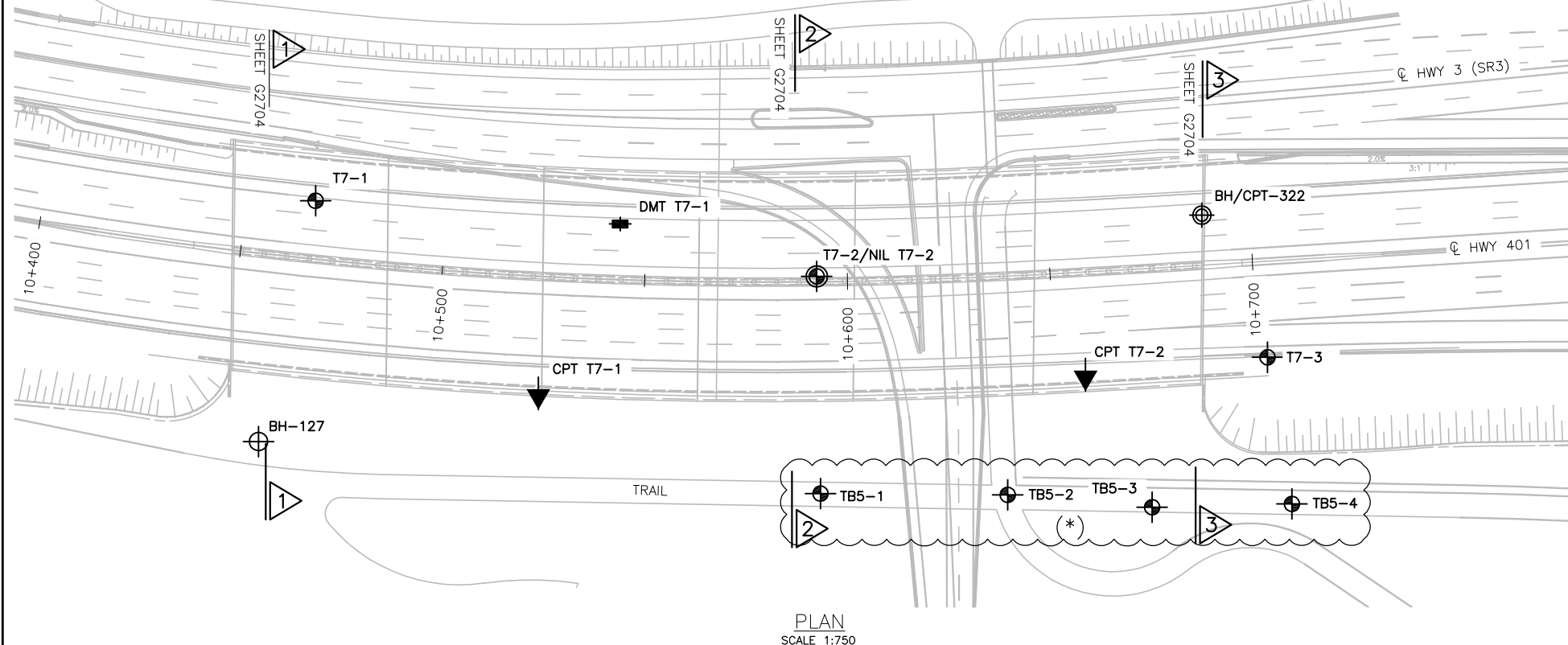


### LEGEND

- |     |   |  |
|-----|---|--|
|     | BOREHOLE<br>CURRENT INVESTIGATION                                     |  |
|     | BOREHOLE AND NILCON VANE<br>CURRENT INVESTIGATION                     |  |
|     | SW/SP HOLE (HYDROGEOLOGY)<br>CURRENT INVESTIGATION                    |  |
|     | NILCON VANE<br>CURRENT INVESTIGATION                                  |  |
|     | CPT - CURRENT INVESTIGATION   |  |
|     | DMT - CURRENT INVESTIGATION   |  |
|     | BOREHOLE<br>PREVIOUS INVESTIGATION                                    |  |
|     | BOREHOLE, CPT AND NILCON VANE<br>PREVIOUS INVESTIGATIONS              |  |
|     | CPT -PREVIOUS INVESTIGATION   |  |
| N   | SPT N-VALUE   |  |
| 16  | BLOWS/0.3m UNLESS<br>OTHERWISE STATED<br>(STD. PEN. TEST, 475 J/BLOW) |  |
|     | P - VIBRATING WIRE PIEZOMETER   |  |
| DRY | BOREHOLE DRY DURING DRILLING  |  |
|     | WATER LEVEL DURING DRILLING   |  |
|     | WATER LEVEL (SHALLOW PIEZO)   |  |
|     | WATER LEVEL (DEEP PIEZO)  |  |
|     |   |  MHSG - MAGNETIC<br>HEAVE/SETTLEMENT<br>GAUGE |
|     |   |   |

## NOTES












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

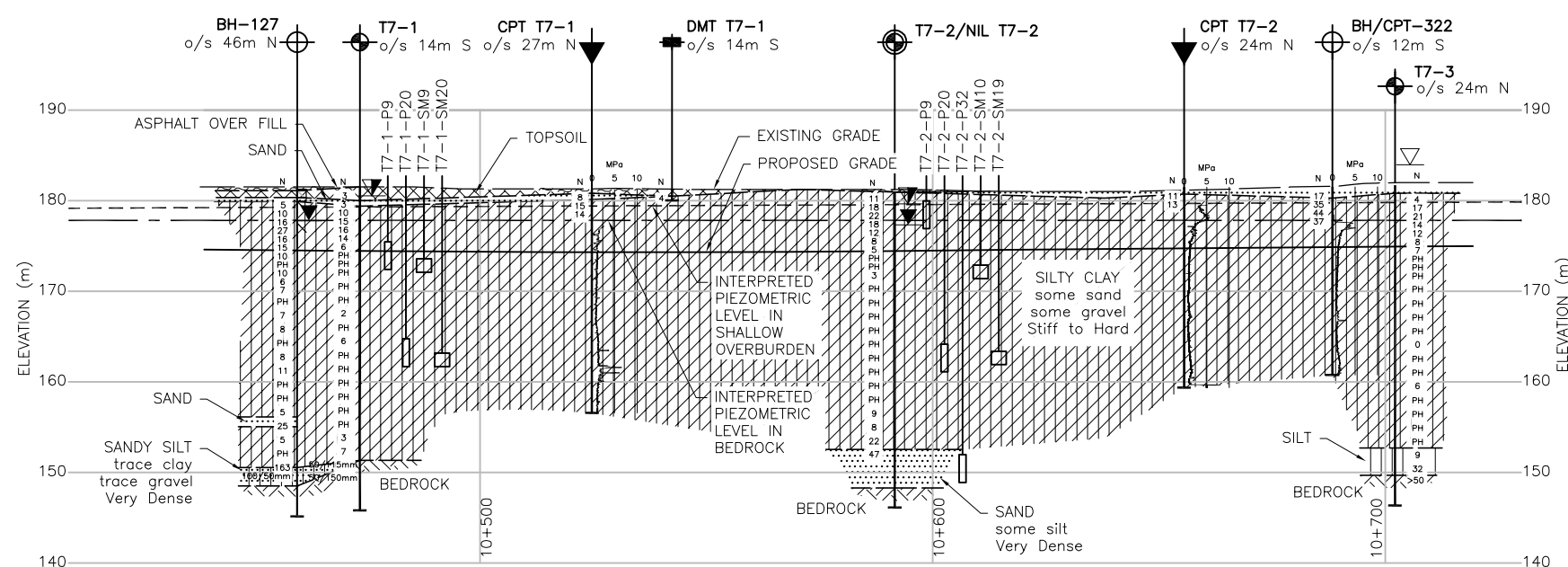


## LIST OF ABBREVIATIONS

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

### MATERIAL LEGEND

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



PROFILE ALONG CL OF TUNNEL

HORT SCALE 1:750  
VERT SCALE 1:375

(\*) TB5-1 THRU TB5-4 NOT INCLUDED FOR CLARITY

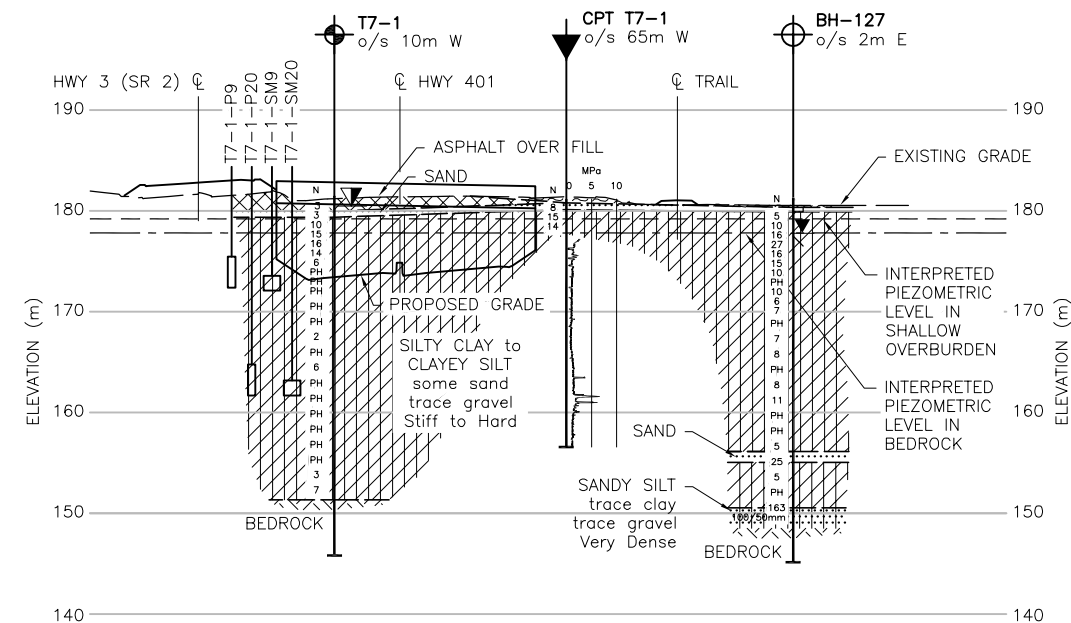
DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

No.	ELEVATION	CO-ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC TEST HOLES			
T7-1	181.5	4679413.6	332295.2
T7-2/NIL T7-2	181.2	4679331.1	332388.2
T7-3	181.7	4679255.7	332473.2
TB5-1	181.0	4679286.0	332362.0
TB5-2	180.8	4679261.2	332400.9
TB5-3	181.3	4679239.6	332429.4
TB5-4	181.7	4679221.9	332459.0
CPT T7-1	181.2	4679345.0	332316.9
CPT T7-2	181.2	4679276.9	332433.5
DMT T7-1	181.5	4679368.7	332355.7
PREVIOUS TEST HOLES			
BH/CPT-322	181.50	4679294.0	332478.2
BH/CPT-124	181.51	4679354.6	332455.6
BH-127	181.27	4679370.9	332251.6

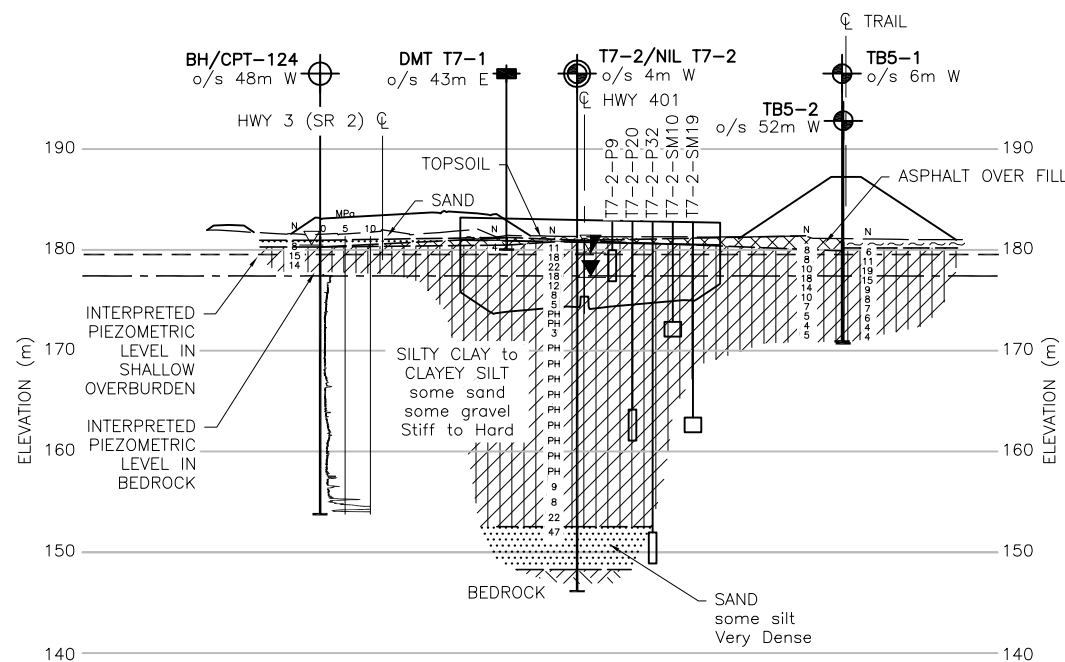
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	17-MAY-12	0	TR	ISSUED FOR CONSTRUCTION				
	DATE	REV.	BY	DESCRIPTION				
DESIGN	WH	CHK	NSV	CODE	CAN/CSA S6-06	LOAD	CL-625-ONT	
DRAWN	MM	CHK	WH	SITE	6-707	DATE	18-JUL-11	

DOC: 285380-04-090-WIP1-2703

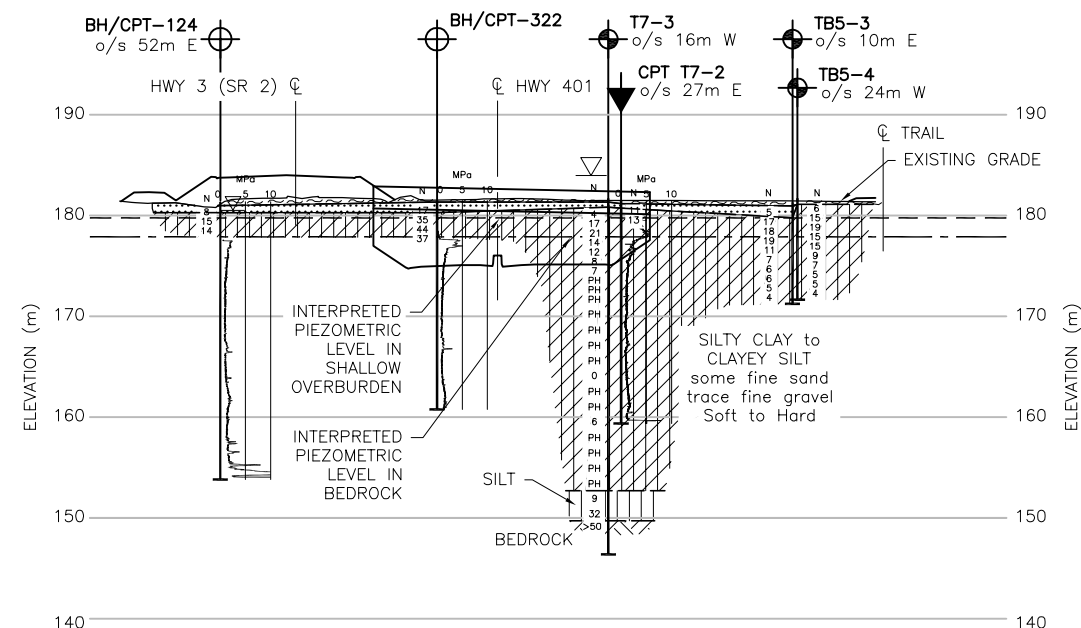
## METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWNWindsor-Essex  
Parkway Project  
RFP No. 09-54-1007NEW CONSTRUCTION  
HWY 401  
VILLA BORGHESE TUNNEL T-7  
SOIL STRATIGRAPHYSHEET  
G2704Phase 1  
IFC

1  
HORIZONTAL SCALE 1:750  
VERTICAL SCALE 1:375



2  
HORIZONTAL SCALE 1:750  
VERTICAL SCALE 1:375



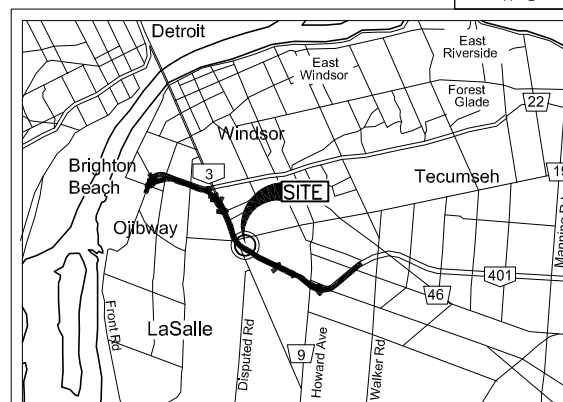
3  
HORIZONTAL SCALE 1:750  
VERTICAL SCALE 1:375

## LIST OF ABBREVIATIONS

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

## MATERIAL LEGEND

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



KEY PLAN  
SCALE 1:0 2 4Km

## LEGEND

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

## NOTES

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

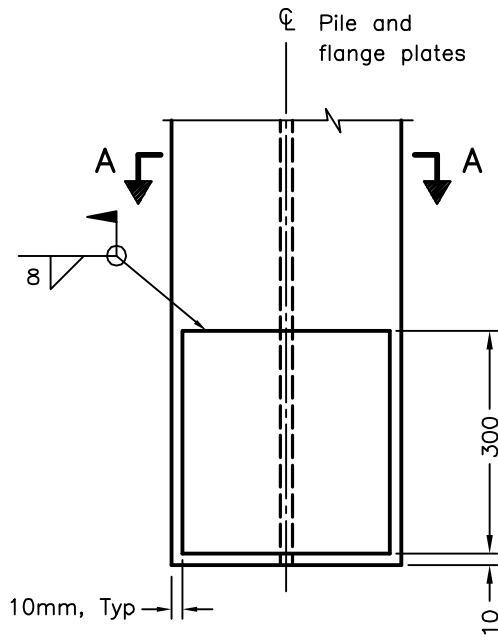
DRAWING NOT TO BE SCALED  
100mm ON ORIGINAL DRAWING

REVISIONS	DATE	REV.	BY	DESCRIPTION
1	17-MAY-12	0	TR	ISSUED FOR CONSTRUCTION
DESIGN	WH	CHK	NSV	CODE CAN/CSA S6-06/LOAD CL-625-ONT
DRAWN	MM	CHK	WH	SITE 6-707 DATE 18-JUL-11

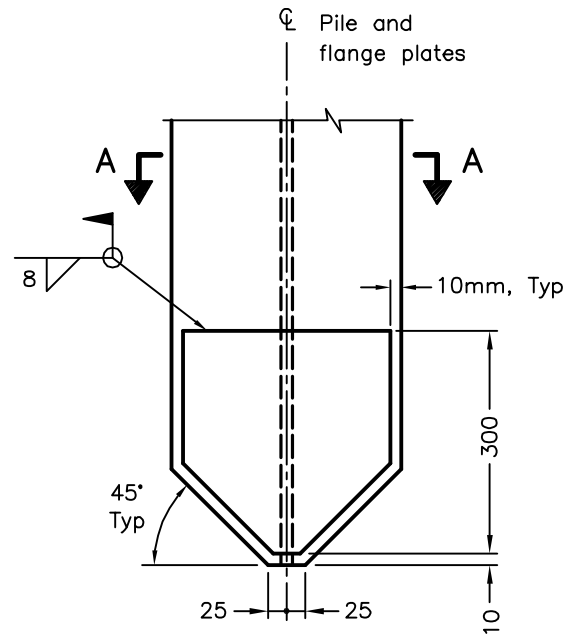
## Applicable OPSDs

Project: Windsor-Essex Parkway  
Document: Geotechnical Investigation and Design Report– Tunnel T-7  
(Hwy. 401 Sta. 10+450L to Sta. 10+700L, LaSalle) Geocres No. 40J6-37  
Doc No.: 285380-04-119-0028

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

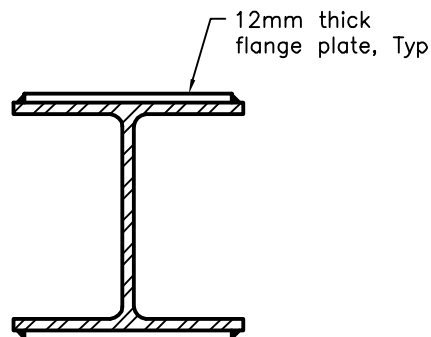


TYPE I



TYPE II

### ELEVATION



PILE DRIVING SHOE  
SECTION A-A

#### NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

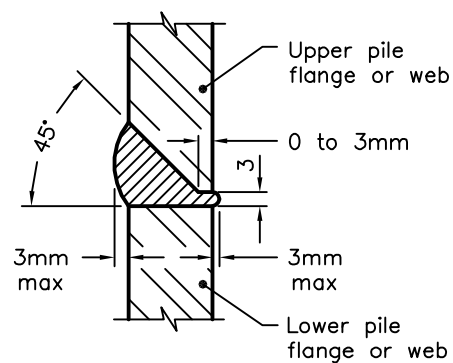
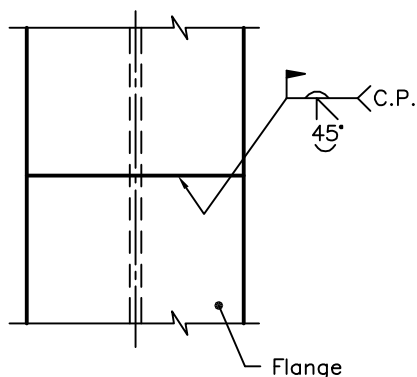
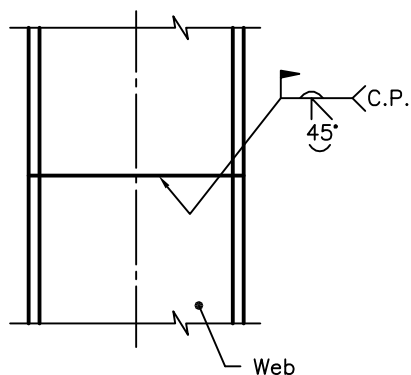
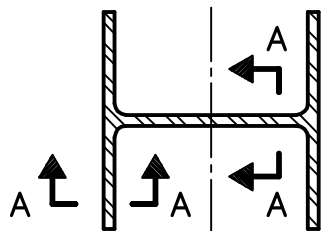
Rev 2

FOUNDATION  
PILES

STEEL H-PILE DRIVING SHOE

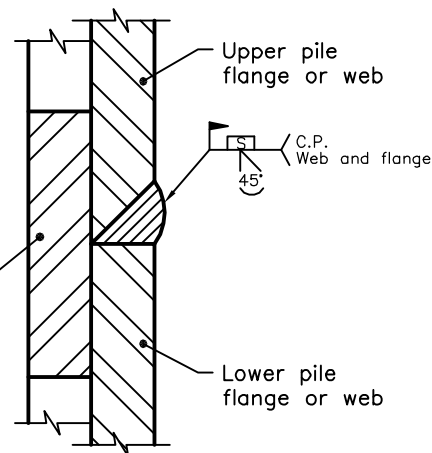
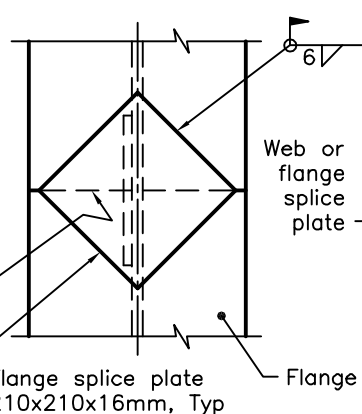
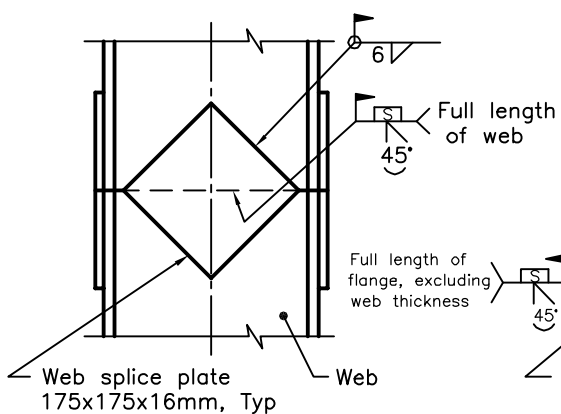
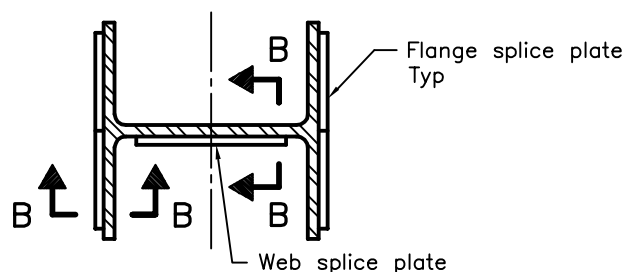
OPSD 3000.100





**BUTT WELD**

**SECTION A-A**



**BUTT WELD WITH SPLICE PLATES**

**SECTION B-B**

**NOTES:**

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

Rev

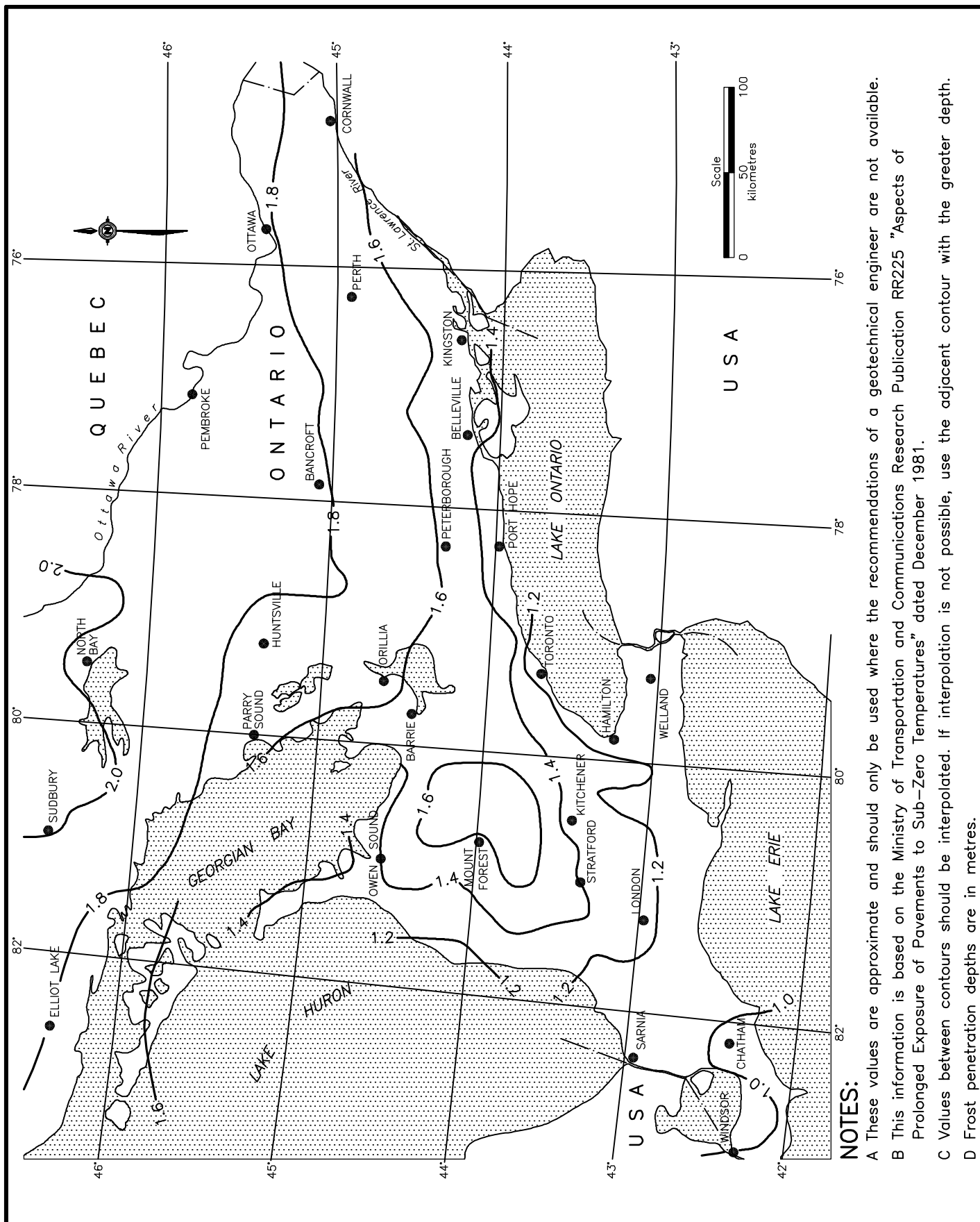
1

**FOUNDATION  
PILES  
STEEL H-PILE SPLICE**

**OPSD 3000.150**







**NOTES:**

- A These values are approximate and should only be used where the recommendations of a geotechnical engineer are not available.
- B This information is based on the Ministry of Transportation and Communications Research Publication RR225 "Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures" dated December 1981.
- C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.
- D Frost penetration depths are in metres.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

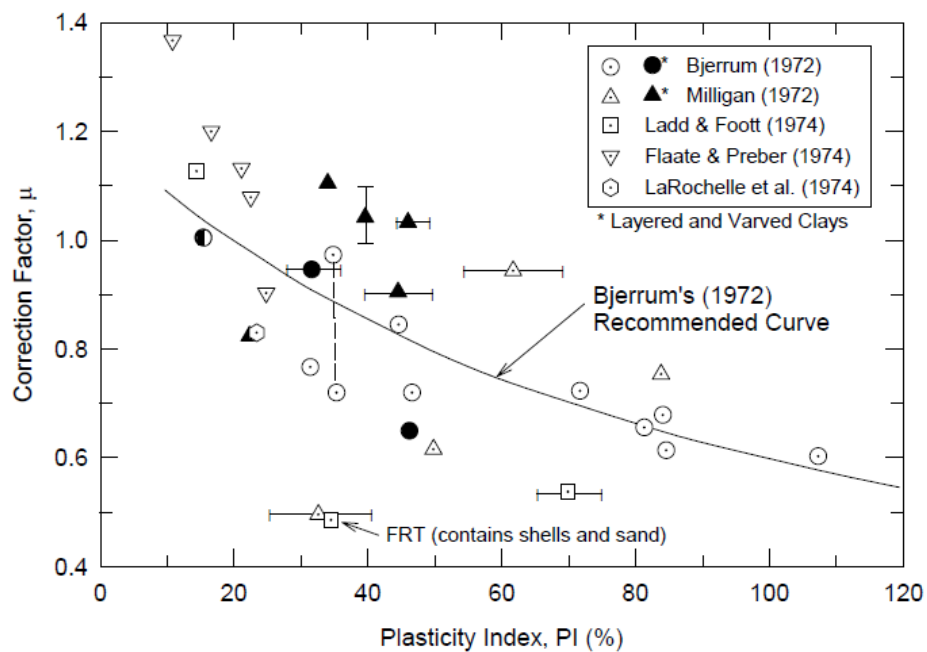
Rev 1

**FOUNDATION  
FROST PENETRATION DEPTHS  
FOR SOUTHERN ONTARIO**

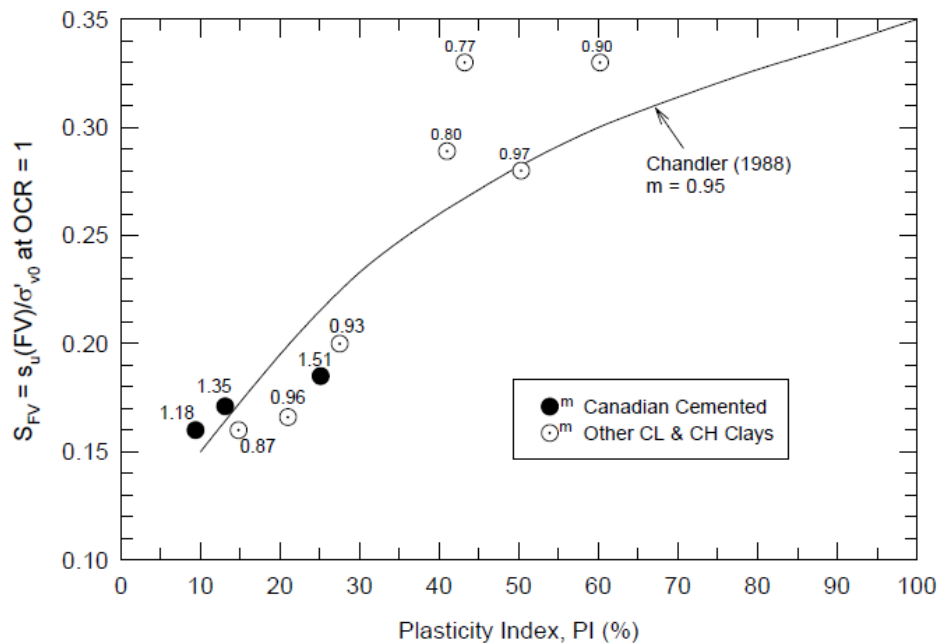


**OPSD 3090.101**

## Figures



**Figure 3.1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures**  
(Figure 5.1, Ladd & DeGroot, 2004)



**Figure 3.2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays**  
(Figure 5.2, Ladd & DeGroot, 2004)

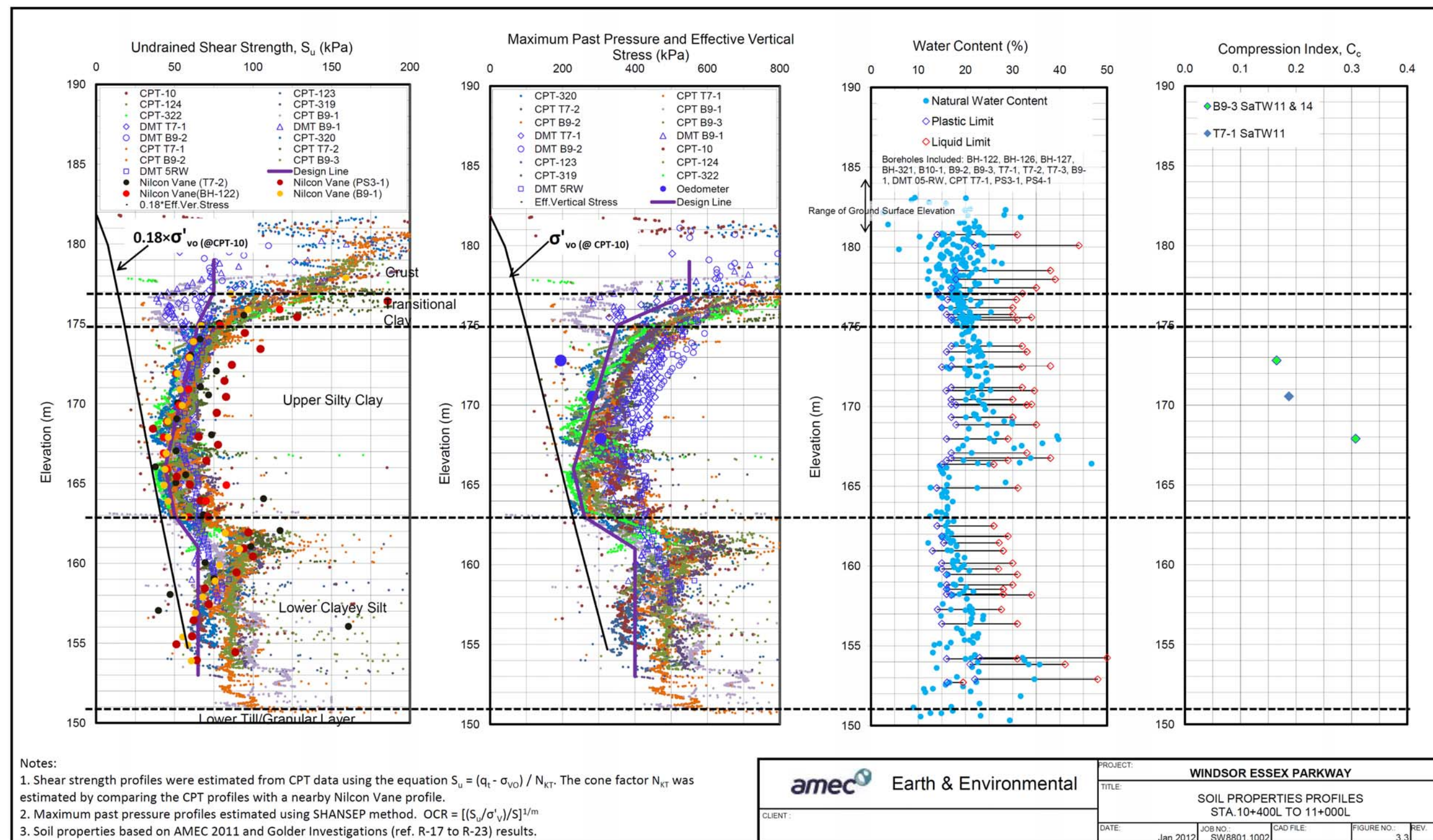
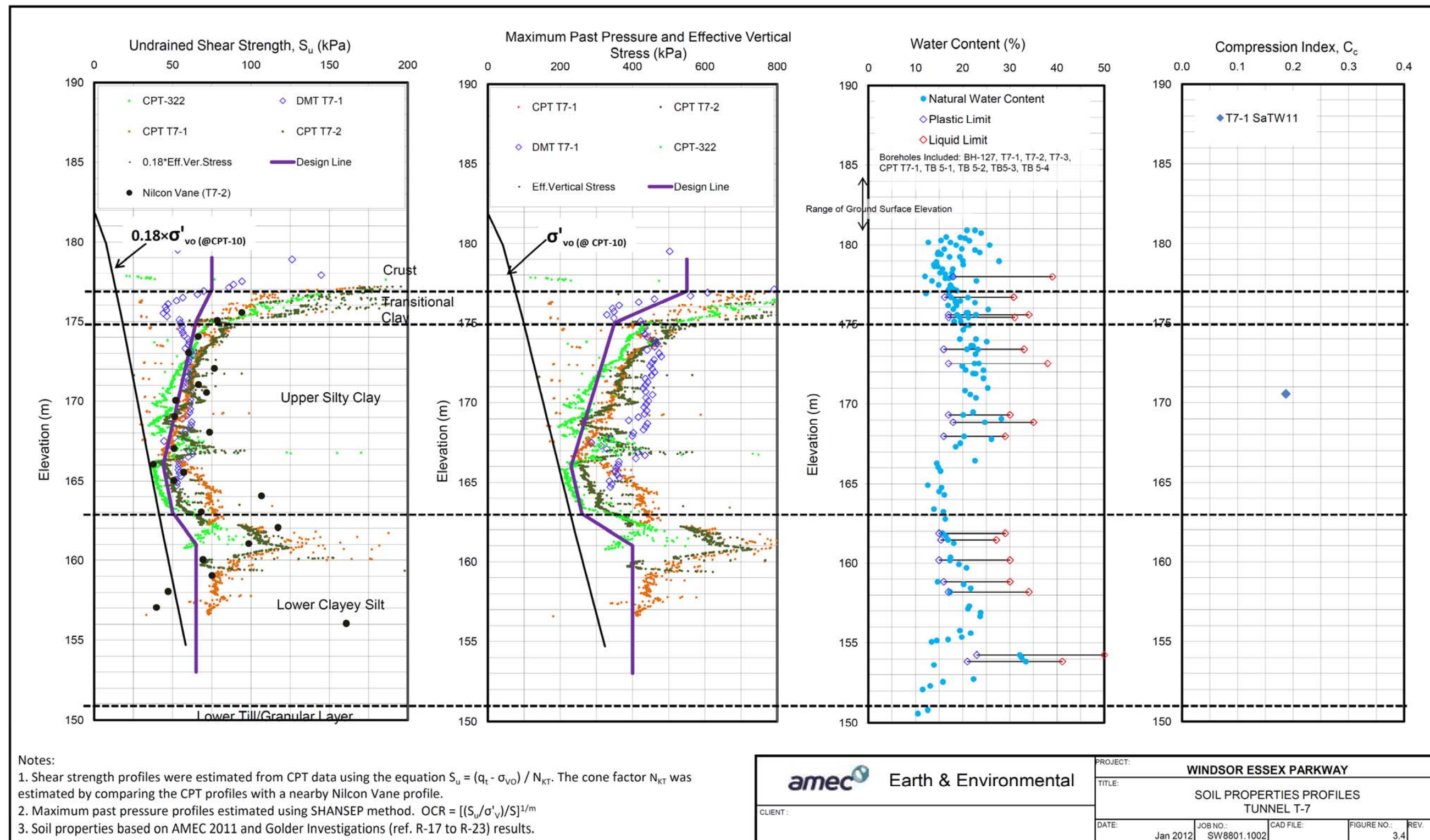


Figure 3.3: Soil Properties Profiles Sta. 10+400L to Sta. 11+000L





**Figure 3.4: Soil Properties Profiles, Tunnel T-7**

**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Tunnel T-7  
 (Station 10+450L to Station 10+700L) Geocres No. 40J6-37  
**Doc No.:** 285380-04-119-0028

**Date:** May / 2012  
**Rev:** 0  
**Page No.:** Figures 4 of 8



**Figure 4-1: Data Summary of Compression Indices  $C_c$ ,  $C_s$  and  $C_r$**

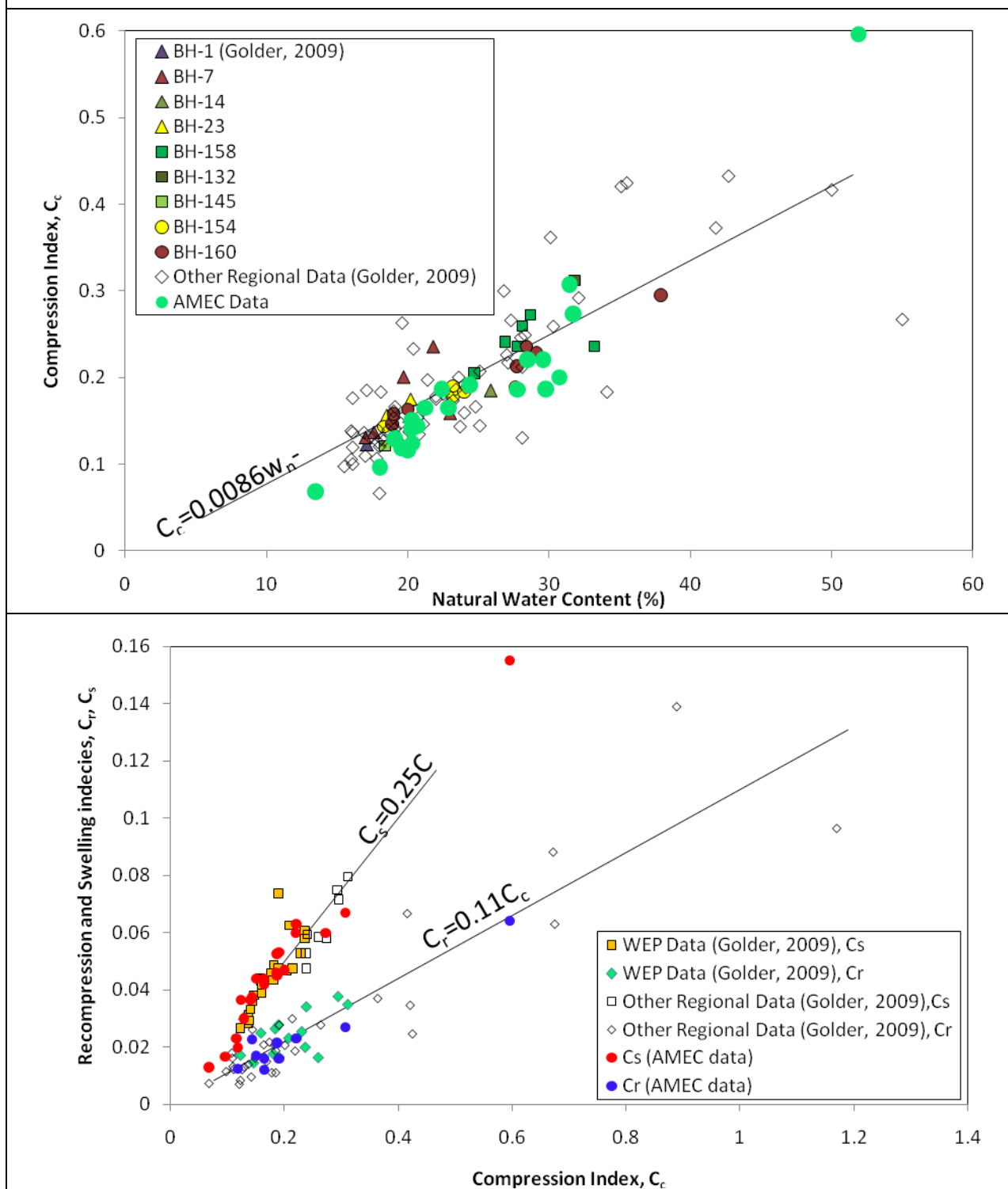


Figure 4-2: Data Summary of Compression Indices  $C_c$  and  $C_\alpha$

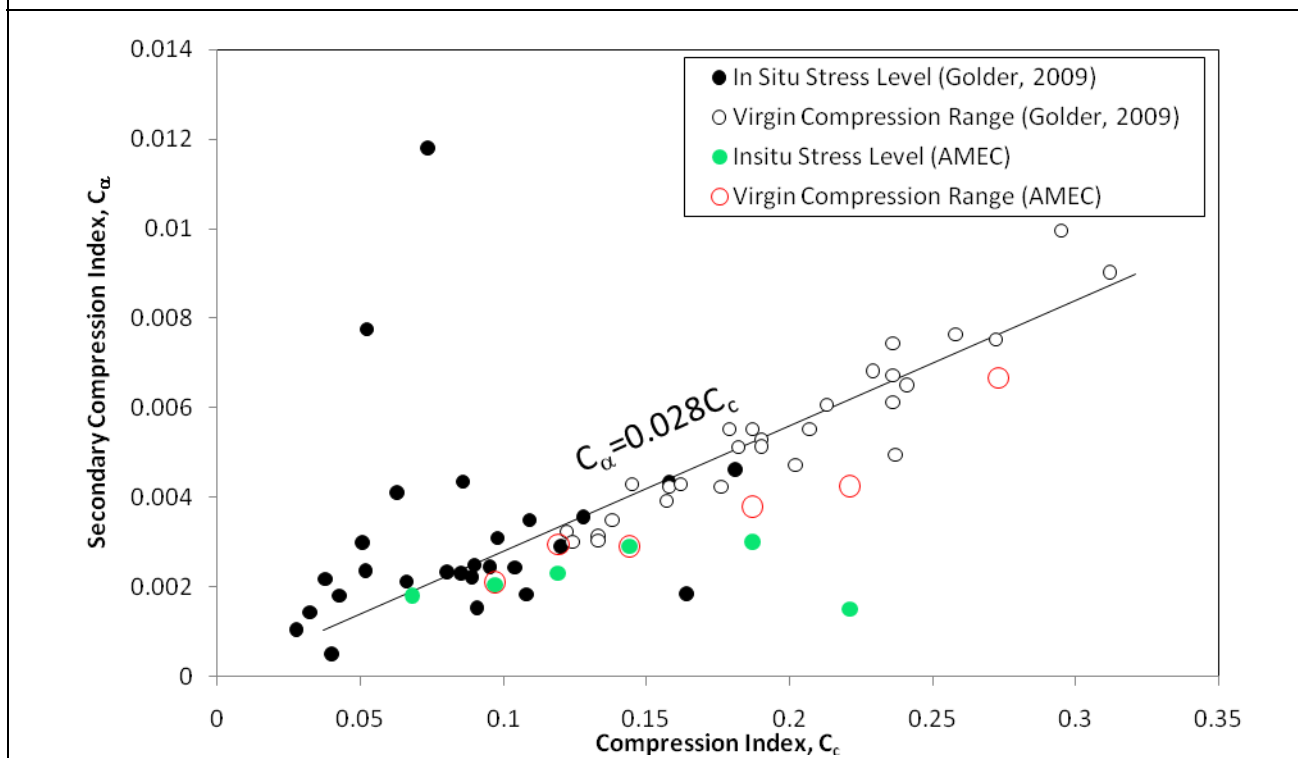


Figure 4-3: Data Summary of Effective Friction Angle ( $\phi'$ )

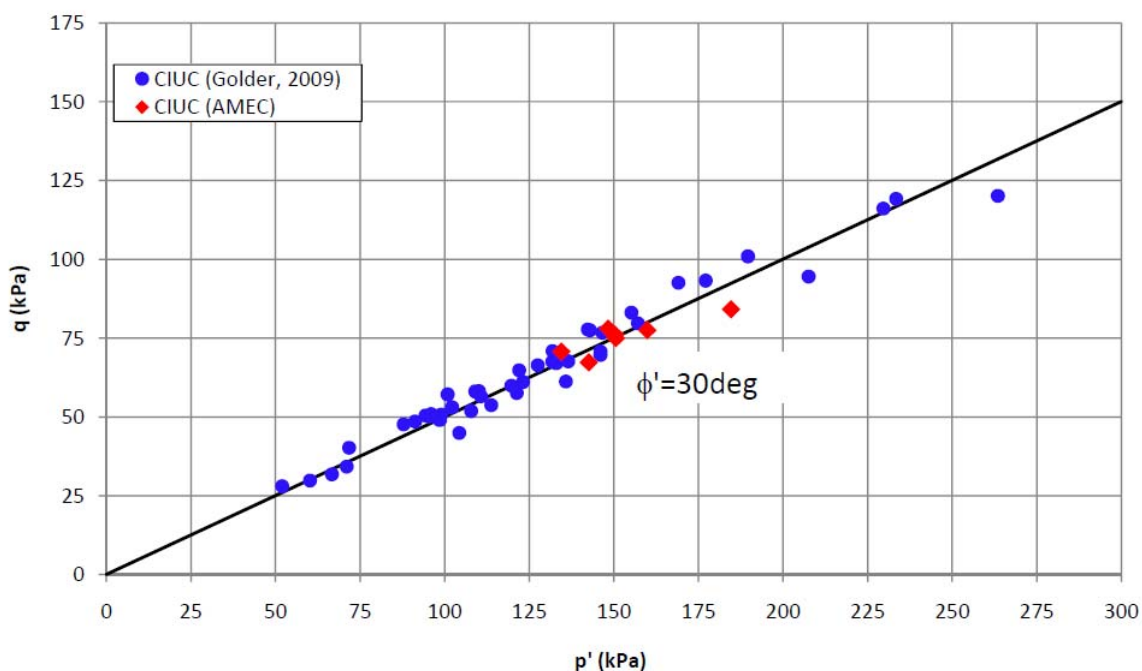


Figure 4-4: Relationship between  $\sin \phi'$  and Plasticity Index for Normally Consolidated Soils

(Kenney, 1959)

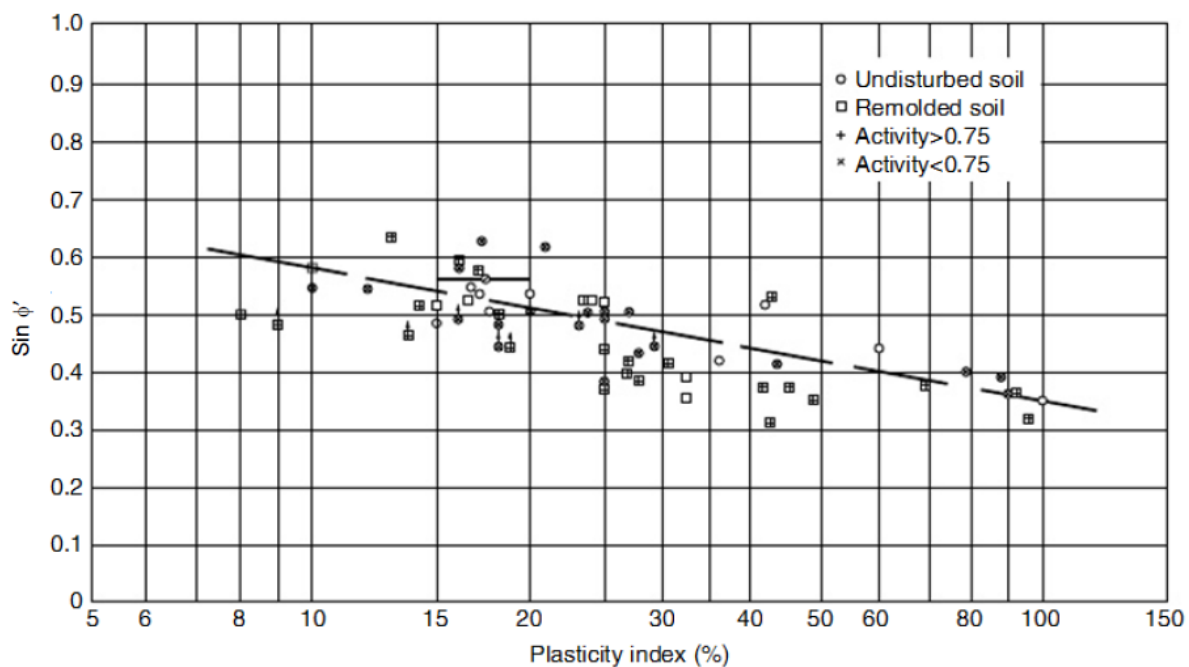
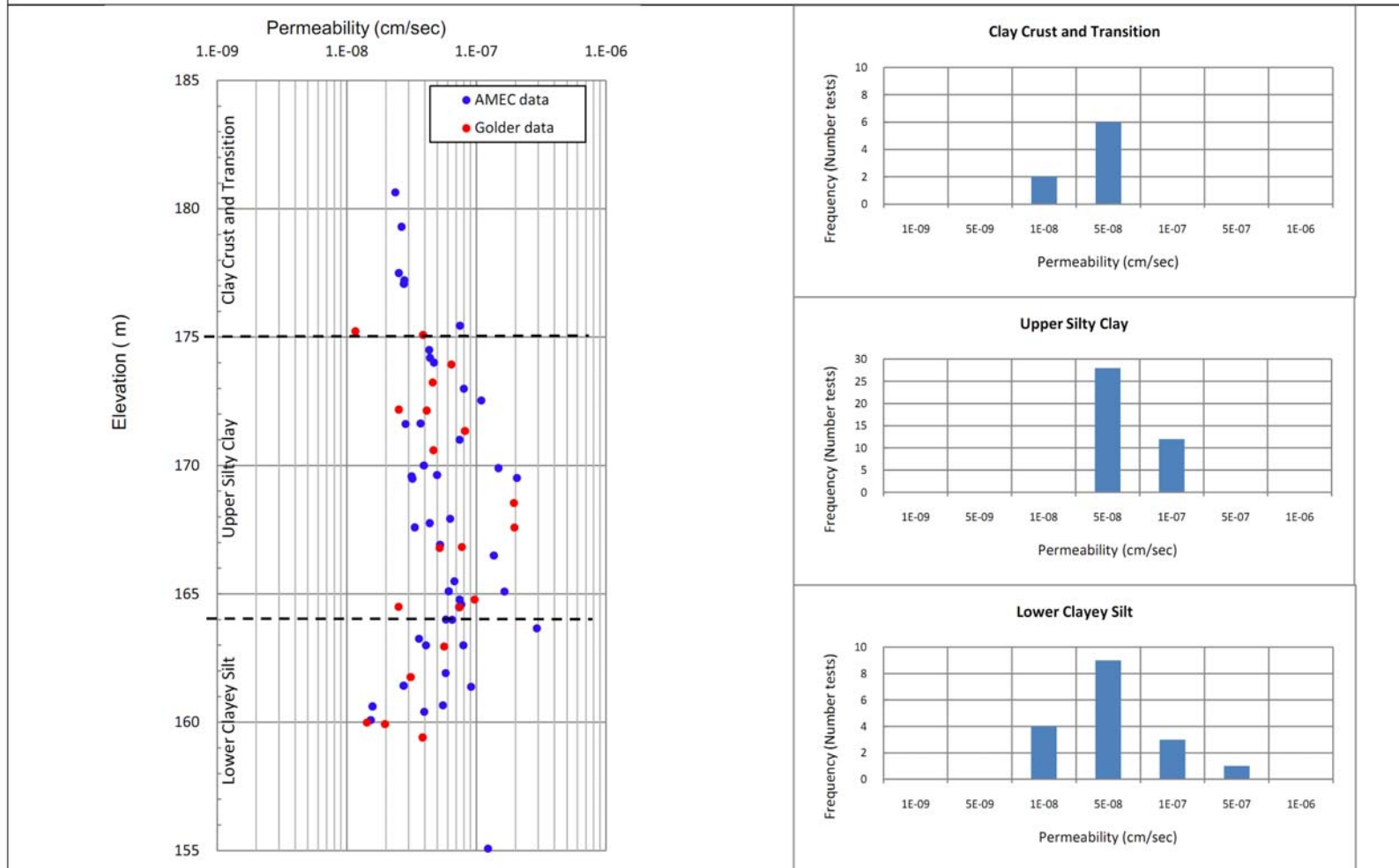


Figure 4-5: Inferred Clay Stratum Permeability Values from CPT Pore Pressure Dissipation Tests



**Project:** Windsor-Essex Parkway  
**Document:** Geotechnical Investigation and Design Report – Tunnel T-7  
(Station 10+450L to Station 10+700L) Geocres No. 40J6-37  
**Doc No.:** 285380-04-119-0028

**Date:** May / 2012  
**Rev:** 0  
**Page No.:** Figures 8 of 8

## **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 strata, 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	
<u>Cohesionless Soils</u>	<u>SPT N-Value*</u>
Very loose	0 to 5
Loose	5 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

Consistency of	
<u>Cohesive Soils</u>	<u>Undrained Shear Strength</u>
	<u>kPa</u>
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.



# MTC SOIL CLASSIFICATION

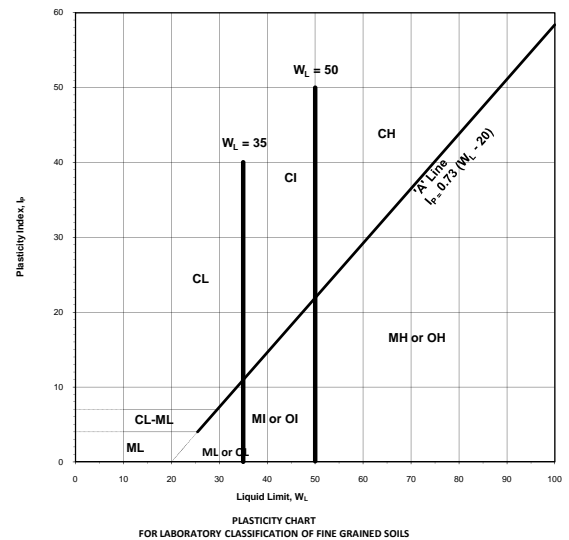
## Based on MTC Soil Classification Manual



MAJOR DIVISION				GROUP SYMBOL	TYPICAL DESCRIPTION	INFORMATION REQUIRED FOR DESCRIBING SOILS	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (LITTLE OR NO FINES)	WIDE RANGE IN GRAIN SIZE & SUBSTANTIAL AMOUNTS OF ALL INTERMEDIATE PARTICLE SIZES	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, & HARDNESS OF THE COARSE GRAINS, LOCAL OR GEOLOGICAL NAME & OTHER PERTINENT DESCRIPTIVE INFORMATION, & SYMBOL IN PARENTHESIS.	$C_u = \frac{D_{60}}{D_{10}} \quad \text{GREATER THAN 4;}$	
			PREDOMINANTLY ONE SIZE OF A RANGE OF SIZES WITH SOME INTERMEDIATE SIZES MISSING	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} \quad \text{BETWEEN 1 AND 3}$	
	SANDS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	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		NOT MEETING ALL GRADATION REQUIREMENTS FOR GW	
			PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES			
		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	ATTERBERG LIMITS BELOW A-LINE OR $I_p$ LESS THAN 4 ABOVE A-LINE WITH $I_p$ BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS	
			PREDOMINANTLY ONE SIZE OR A RANGE OF SIZES WITH SOME INTERMEDIATE SIZE MISSING	SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES			
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	SILT AND CLAYS	LIQUID LIMIT LESS THAN 35 AND 50	DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	GIVE TYPE, NAME, IF NECESSARY, INDICATE DEGREE AND CHARACTER OF PLASTICITY, AMOUNT AND MAXIMUM SIZE OF COARSE GRAINS, COLOUR IN WET CONDITION, ODOUR, IF ANY, LOCAL OR GEOLOGIC NAME & OTHER PERTINENT DESCRIPTIVE INFORMATION & SYMBOL IN PARENTHESIS.	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	
			NONE	QUICK	NONE			
			MEDIUM TO HIGH	NONE TO VERY SLOW	MEDIUM			
			SLIGHT TO MEDIUM	SLOW	SLIGHT			
		LIQUID LIMIT BETWEEN 35 AND 50	NONE TO SLIGHT	SLOW TO QUICK	SLIGHT		ATTERBERG LIMITS ABOVE A- LINE WITH $I_p$ GREATER THAN 7   ATTERBERG LIMITS ABOVE A- LINE WITH $I_p$ GREATER THAN 7	
			HIGH	NONE	MEDIUM TO HIGH			
			SLIGHT TO MEDIUM	VERY SLOW	SLIGHT			
			SLIGHT TO MEDIUM	SLOW TO NONE	MEDIUM			
		LIQUID LIMIT GREATER THAN 50	HIGH TO VERY HIGH	NONE	HIGH			
			MEDIUM TO HIGH	NONE TO VERY SLOW	SLIGHT TO MEDIUM			
			SLIGHT TO MEDIUM	VERY SLOW	SLIGHT			
			SLIGHT TO MEDIUM	SLOW TO NONE	MEDIUM			
	HIGH ORGANIC SOILS	READILY IDENTIFIED BY COLOUR, ODOUR, SPONGY FEEL & FREQUENTLY BY FIBROUS TEXTURE			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	NOT MEETING ALL GRADATION FOR SW   ATTERBERG LIMITS BELOW A- LINE OR $I_p$ LESS THAN 4 ABOVE A-LINE WITH $I_p$ BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS  ATTERBERG LIMITS ABOVE A- LINE WITH $I_p$ GREATER THAN 7	

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

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



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



AMEC Earth & Environmental,  
a Division of AMEC American

[www.amec.com](http://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	SUSCEPTIBILITY TO SURFICIAL EROSION	SUSCEPTIBILITY TO FROST ACTION	DRAINAGE CHARACTERISTICS
WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GW	PERVIOUS	EXCELLENT	NEGLECTIBLE	EXCELLENT	MEDIUM	NEGLECTIBLE	NEGLECTIBLE	EXCELLENT
POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GP	VERY PERVIOUS	GOOD	NEGLECTIBLE	GOOD	MEDIUM	NEGLECTIBLE	NEGLECTIBLE	EXCELLENT
SILTY GRAVELS, POORLY GRADED GRAVEL- SAND-SILT MIXTURES	GM	SEMI-PERVIOUS TO IMPERVIOUS	GOOD	NEGLECTIBLE	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	NEGLECTIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SW	PERVIOUS	EXCELLENT	NEGLECTIBLE	EXCELLENT	LOW TO MEDIUM	SLIGHT	NEGLECTIBLE	EXCELLENT
POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SP	PERVIOUS	GOOD	VERY LOW	FAIR TO GOOD	LOW TO MEDIUM	MODERATE	NEGLECTIBLE 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	NEGLECTIBLE	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 PRACTICALLY
ORGANIC SILTY CLAY OF MEDIUM PLASTICITY	OI	SEMI-PERVIOUS 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 NEGLECTIBLE	NEGLECTIBLE	PRACTICALLY IMPERVIOUS
ORGANIC CLAYS OF HIGH PLASTICITY	OH	IMPERVIOUS	POOR	HIGH	POOR	LOW	MODERATE	NEGLECTIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
PEAT AND OTHER HIGHLY ORGANIC SOILS	Pt	-	-	-	-	LOW	SEVERE	-	FAIR TO GOOD

# RECORD OF BOREHOLE No T7-1

1 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4679413.6N, 332295.2E ORIGINATED BY DG  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 7 Jul 11 - 7 Jul 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub> W      W <sub>L</sub> WATER CONTENT (%)				
								○ UNCONFINED      + FIELD VANE ● POCKET PEN.      × LAB VANE						
181.5	Ground Surface													
180.9	50mm ASPHALT													
0.2	Over 200mm Crushed Limestone Sand and Gravel fill													
	FILL Silty Clay and Topsoil Green and black		1	SS	3									
180.0														
1.5	SAND Poorly Graded (Fine) Trace organics, saturated Green grey to brown		2	SS	3									
179.4														
2.1	CLAYEY SILT Some sand, trace gravel Very soft to very stiff Grey		3	SS	10									
	-Trace medium-coarse gravel Trace fine-medium gravel, pink clay nodules		4	SS	15									
			5	SS	16									
	-Trace fissures		6	SS	14									
	-Trace pink clay nodules		7	SS	6									
	Fine sand nodules Trace fine gravel, pink clay nodules		8	TW	PH								20.6	2 17 49 32
			9	TW	PH									
			10	TW	PH									
				VT										
	-Trace fine-coarse gravel		11	TW	PH									4 22 38 36
			12	TW	PH								20.6	3 22 40 35
				VT										
	-Trace fine-medium gravel		13	SS	2									

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/05/12

## METRIC

Continued Next Page

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

**RECORD OF BOREHOLE No T7-1**

3 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4679413.6N, 332295.2E ORIGINATED BY DG  
DIST            HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
DATUM Geodetic DATE 7 Jul 11 - 7 Jul 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED ● POCKET PEN.	+ FIELD VANE × LAB VANE									
151.3								20	40	60	80	100						
30.2	<b>SAND</b> And weathered <b>LIMESTONE</b> Cobbles and boulders (inferred) Very dense		24	SS	50/ 115mm											-no recovery, spoon bouncing continued to drill to 32m		
			25	SS	50/ 150mm													
148.9																		
32.6	<b>LIMESTONE</b> Medium to coarse grained Porous, vuggy, fractured at location between 33.07m and 33.22m Clacite crystallization is visible Brown		26	RC												RQD = 100%		
146.7																		
34.8	<b>LIMESTONE</b> Laminated, medium to fine grained, porous		27	RC														
146.4																		
35.1	Pitted at location between 34.78m and 35.14m Brown to Grey																	
146.0																		
145.5	<b>LIMESTONE</b> Fine Grained Vuggy, calcite crystals visible Grey																	
35.7	<b>LIMESTONE</b> Fine Grained Laminated, porous and dense Grey																	
	<b>END OF BOREHOLE</b> No groundwater observed prior to starting wash boring below approx. 9.6 m on July 7, 2011																	
	Water Level measured in Piezometer VWP T7-1-P9 at elevation 180.4m on July 24, 2011																	
	Water Level measured in Piezometer VWP T7-1-P9 at elevation 180.5m on August 6, 2011																	
	Water Level measured in Piezometer VWP T7-1-P20 at elevation 180.4m July 24, 2011																	
	Water Level measured in Piezometer VWP T7-1-P20 at elevation 180.4m on August 6, 2011																	

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No T7-2

1 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679331.1, E332388.2 ORIGINATED BY SD  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 5 Jul 11 - 7 Jul 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)			
								○ UNCONFINED	+ FIELD VANE										
								● POCKET PEN.	× LAB VANE										
181.2	Ground Surface																		
0.0	TOPSOIL														-Vibrating Wire Piezometers (VWP) installed in sampled borehole -Spider Magnets (mg) installed in adjacent boring at 4679332.1N, 332390.8E -Nilcon vane advanced to sampled borehole from 5.5m to 25m (EL. 175.7m to EL. 156.2m)				
180.9	Black																		
0.3	CLAYEY SILT																		
	Some sand and gravel		1	SS	11														
	Soft to very stiff																		
	Mottled grey and brown to grey		2	SS	18														
	Moist																		
	Brown with Grey seams																		
	Brown		3	SS	22														
	-Becoming grey		4	SS	18														
	Grey		5	SS	12														
			6	SS	8														
	Moist to wet		7	SS	5														
			8	TW	PH														
	Some sand, trace gravel		9	SS	PH														
	Moist to wet																		
			10	SS	3														
				VT															
			11	TW	PH														
	Wet Silt Seams		12	TW	PH														
				VT															
	Wet		13	TW	PH														

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/05/12



## METRIC

Continued Next Page

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

# RECORD OF BOREHOLE No T7-2

3 OF 3

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679331.1, E332388.2 ORIGINATED BY SD  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 5 Jul 11 - 7 Jul 11 CHECKED BY MSO

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

-NW Casing refusal  
-VWP #T7-2-P32 installed at 32.31m below ground surface (EL. 148.9m)  
-continue with NQ Core, no bedrock  
-only 15" recovery 1 solid piece, the rest possibly lost in BH  
RQD = 25%

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 08/05/12

## METRIC

Continued Next Page

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

## METRIC

[illegible]

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 07/05/12

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

## METRIC

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

# RECORD OF BOREHOLE No CPT T7-1

1 OF 1

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679345.0, E332316.9 ORIGINATED BY TA  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE 11 Jul 11 - 11 Jul 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
181.2	Ground Surface						20	40	60	80	100					
0.0	TOPSOIL															
180.8																
0.4	SAND Poorly graded Trace to some silt Brown		1A, B	SS	8											
180.1																
1.1	SILTY CLAY Some sand, trace gravel Mottled brown and grey Brown -Trace fissures		2	SS	15											
	Grey -Trace oxidation		3	SS	14											
177.7																
3.5	END OF BOREHOLE (continued with CPT to refusal)  Borehole dry upon completion															
						</										

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



# RECORD OF BOREHOLE No CPT T7-2

1 OF 1

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679276.9, E332433.5 ORIGINATED BY TA  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE Jul 23, 11 - Jul 23, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100	○ UNCONFINED	+	FIELD VANE									
								20 40 60 80 100	● POCKET PEN.	×	LAB VANE									
181.2	Ground Surface						181													
0.0	<b>TOPSOIL</b>																			
180.9																				
0.3	<b>SAND</b>																			
180.4	Poorly graded, trace silt, brown																			
0.8	<b>SILTY CLAY</b>		1	SS	11		180													
	Some sand, trace gravel																			
	Mottled brown-grey to brown																			
179.2			2	SS	13															
2.0	<b>END OF SAMPLED BOREHOLE</b>						179													
	Continue with CPT from 2.0 m to refusal at 22.1 m (El. 179.2 m to El. 157.1 m)																			
	No groundwater observed on July 23, 2011						178													
							177													
							176													
							175													
							174													
							173													
							172													
							171													
							170													
							169													
							168													
							167													

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No DMT T7-1

1 OF 1

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679368.7, E332355.7 ORIGINATED BY LC  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE Jul 15, 11 - Jul 15, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								20 40 60 80 100									10 20 30		
181.5	Pavement Surface																		
0.0	25mm Asphalt																		
	152mm Crushed Limestone, Silty																		
	254mm Brown Silty Sand with gravel																		
	to																		
180.7	304mm Weathered Brown Sandy																		
0.8	Clay with Topsoil		1	SS	4														
180.3	FILL																		
1.2	SILTY CLAY																		
	Some sand, trace gravel																		
	Trace organics, weathered brown																		
	END OF SAMPLED BOREHOLE																		
	Continue with DMT from 2.0 m to																		
	refusal at 16.8 m (El. 179.5 m o El.																		
	164.7 m)																		
	No groundwater observed during																		
	drilling on July 15, 2011																		

# RECORD OF CONE PENETRATION TEST CPT T7-1

METRIC

PROJECT Windsor-Essex Parkway

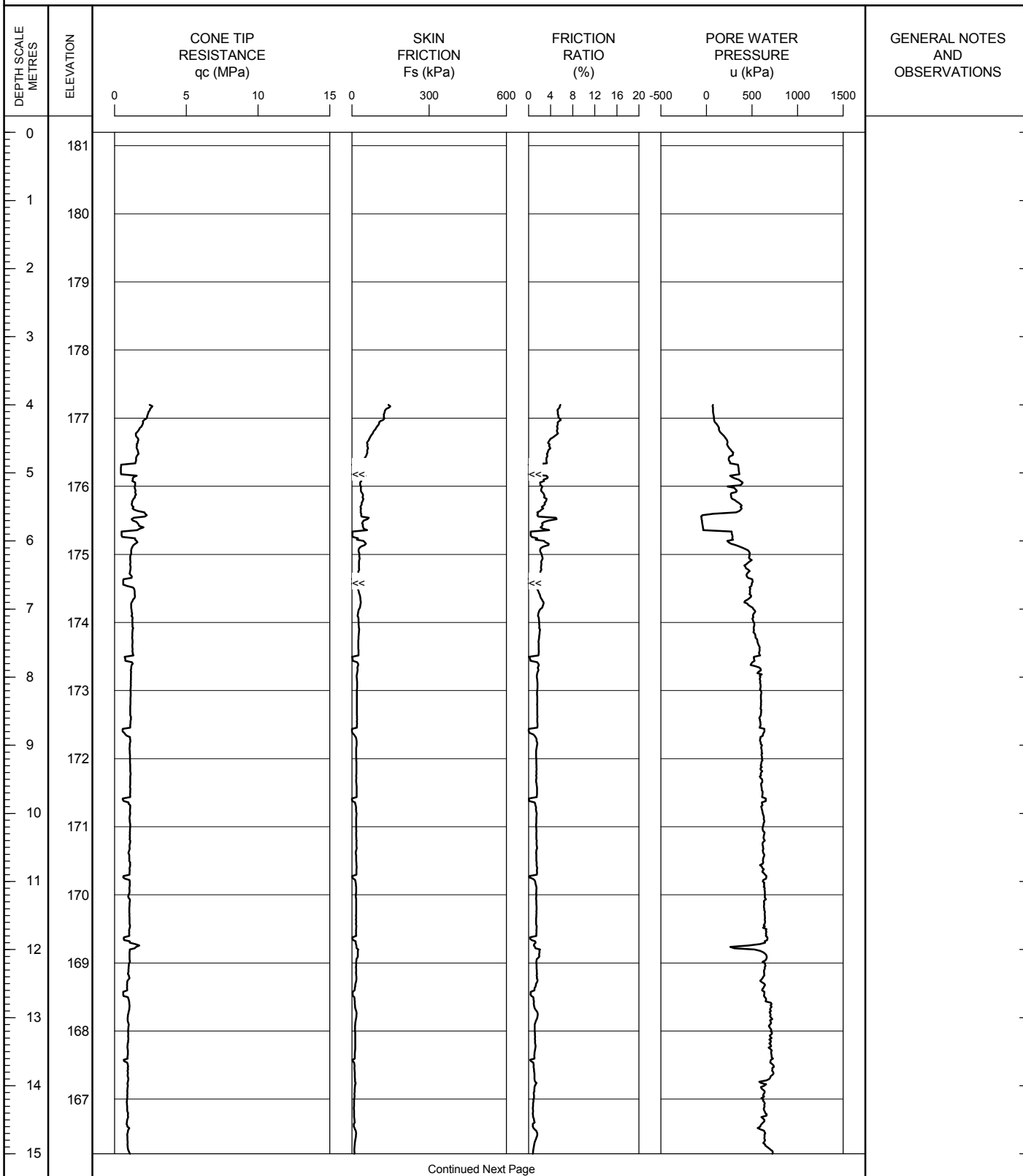
TEST DATE 7/22/2011 - 7/22/2011

SHEET 1 OF 2

LOCATION N4679345.0; E332316.9

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2 PREDRILL DEPTH: 3.37 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT T7-1

METRIC

PROJECT Windsor-Essex Parkway

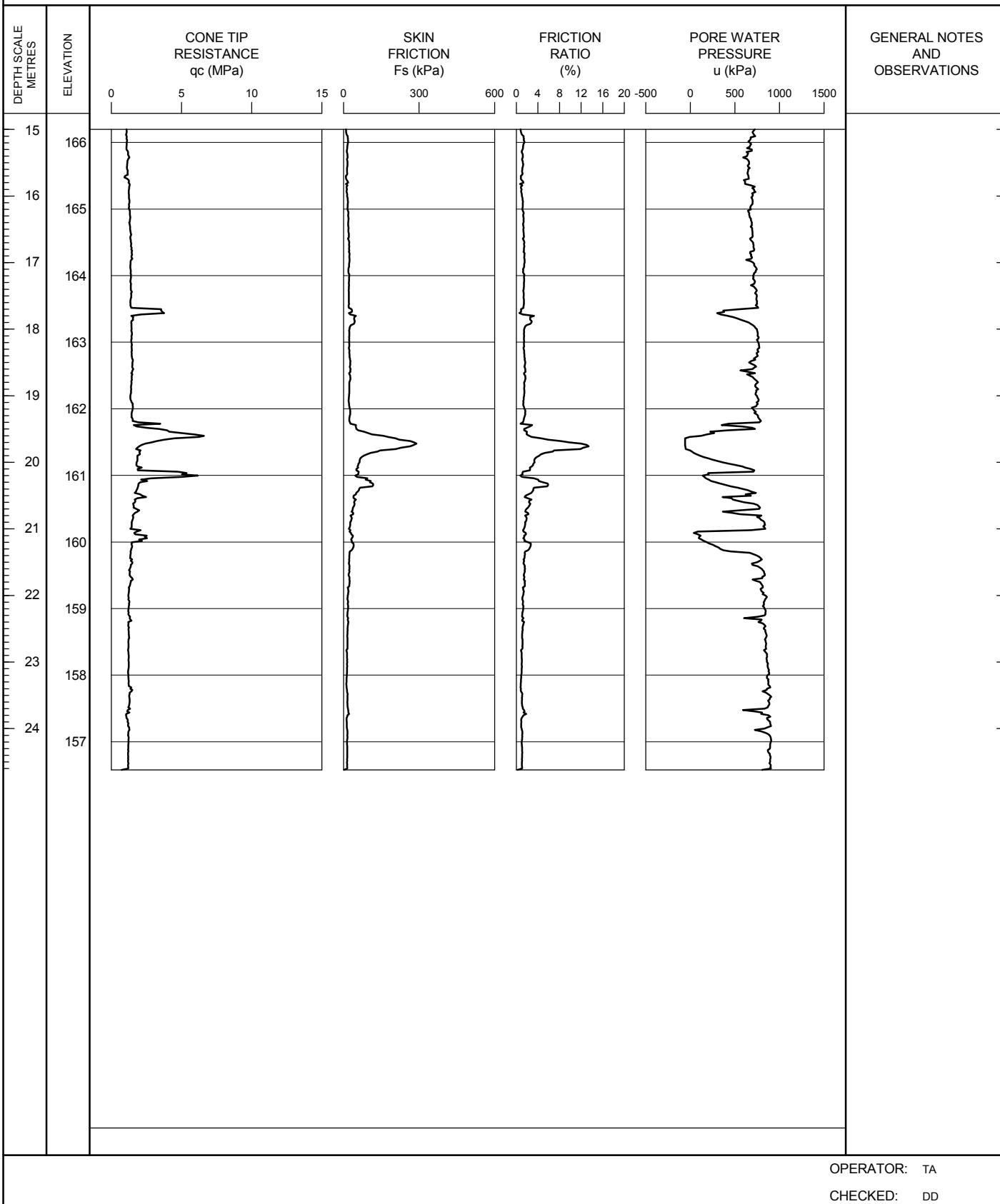
TEST DATE 7/22/2011 - 7/22/2011

SHEET 2 OF 2

LOCATION N4679345.0; E332316.9

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2 PREDRILL DEPTH: 3.37 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEP CPT LOG CPT T7-1.GPJ ONTARIO MOT.GDT 21/12/11

OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT T7-2

**METRIC**

PROJECT Windsor-Essex Parkway

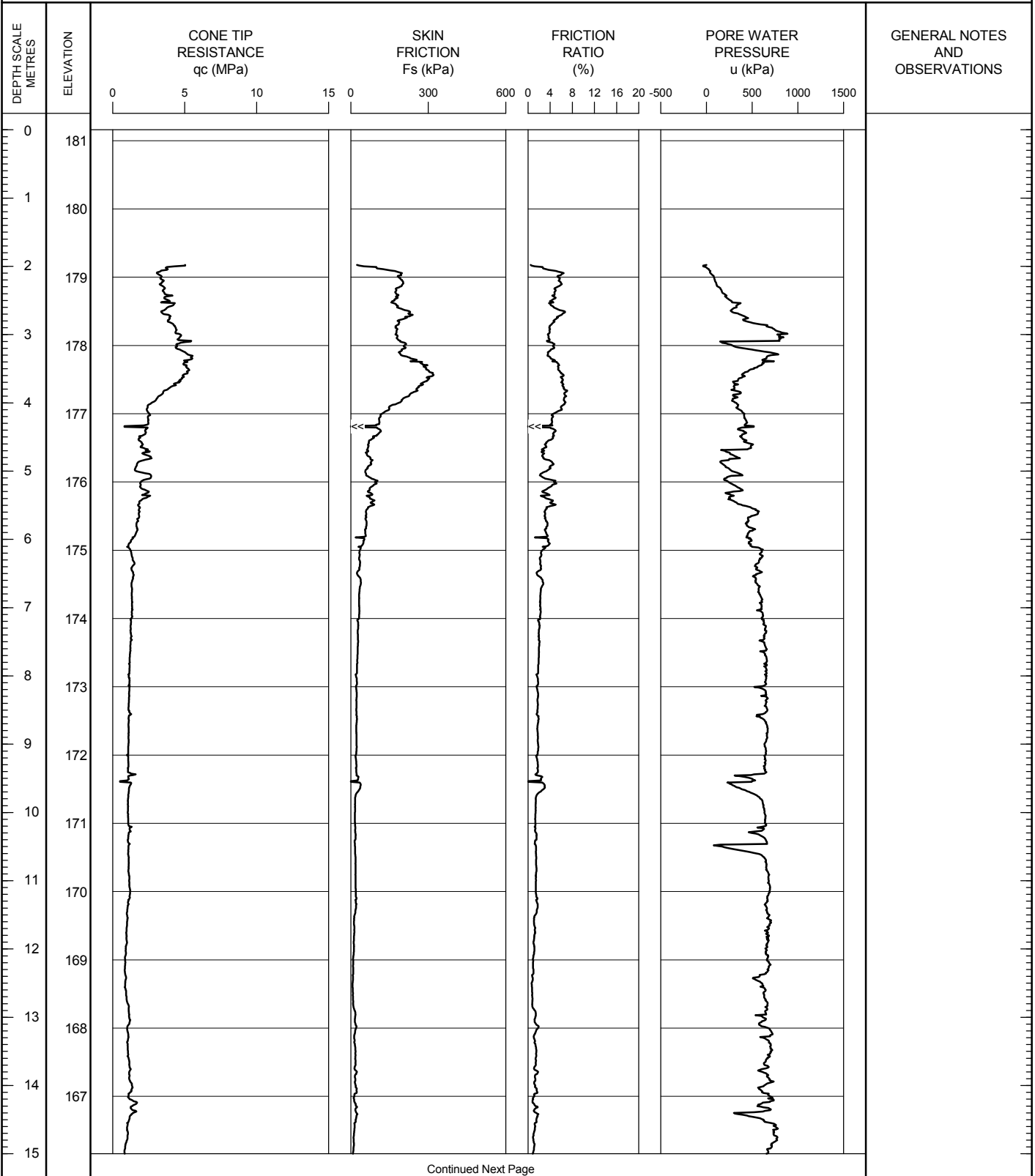
TEST DATE 7/23/2011 - 7/23/2011

SHEET 1 OF 2

LOCATION N4679276.9; E332433.5

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

# RECORD OF CONE PENETRATION TEST CPT T7-2

**METRIC**

PROJECT Windsor-Essex Parkway

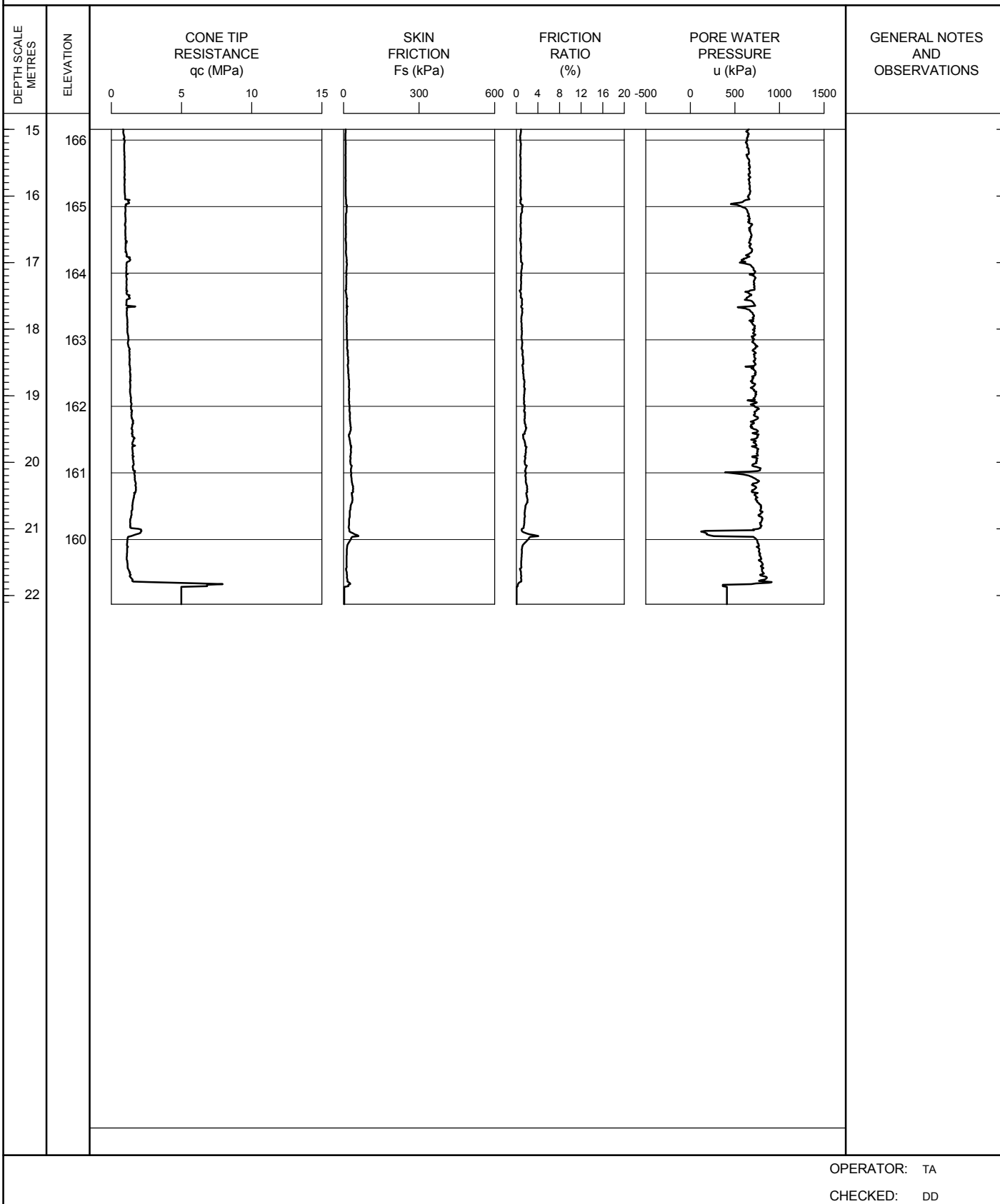
TEST DATE 7/23/2011 - 7/23/2011

SHEET 2 OF 2

LOCATION N4679276.9; E332433.5

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2    PREDRILL DEPTH: 1.98    CORRECTION FACTOR A: 0.8    CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

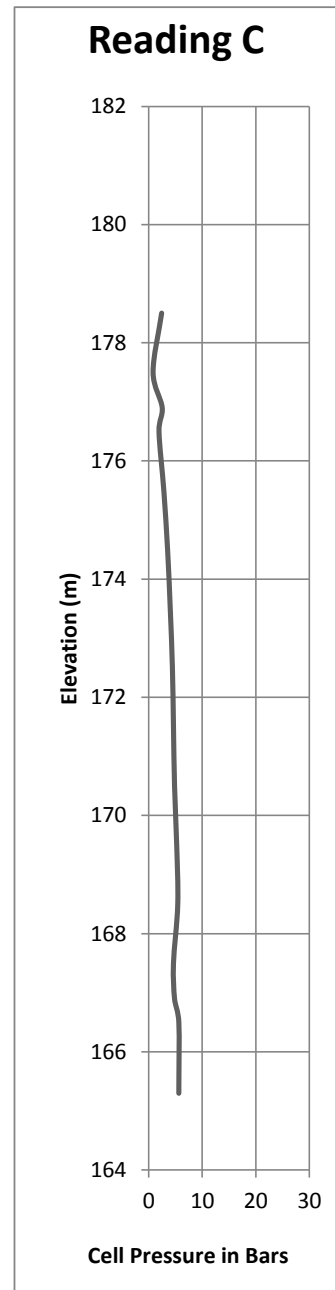
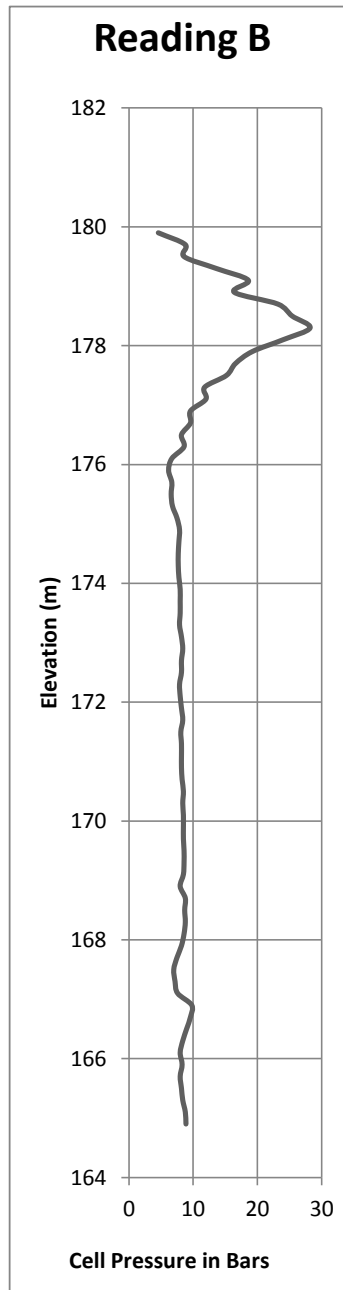
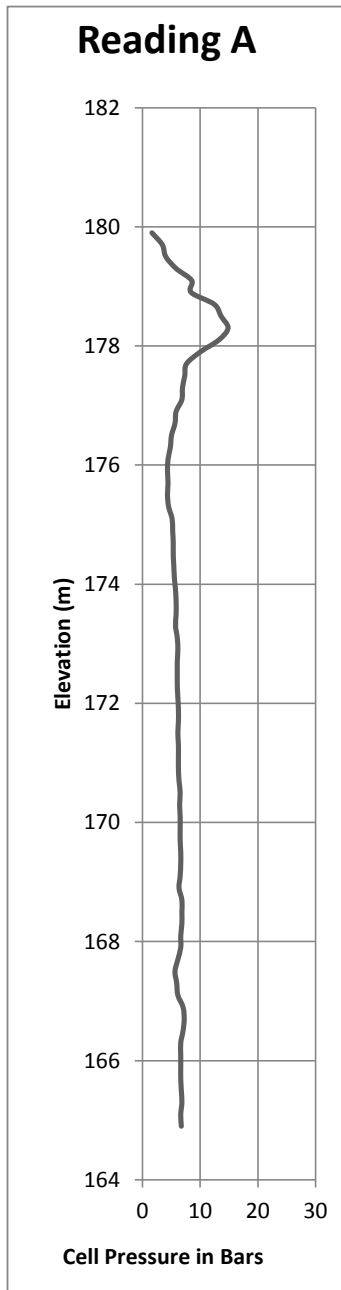


# RECORD OF DILATOMETER TEST DMT T7-1

Project : Windsor-Essex Parkway  
Location: N 4679368.7; E 332355.7  
Ground Surface Elevation : 181.5

Test Date: 7/15/2011  
Predrill Depth : 1.5 m  
Delta A: 0.14 Bar

Sheet 1 of 1  
Datum Geodetic  
Delta B: 0.22 Bar



Operator: LC  
Checked: DD

## RECORD OF NILCON VANE TEST NIL T7-2

Project : Windsor-Essex Parkway

Test Date: 7/8/2011

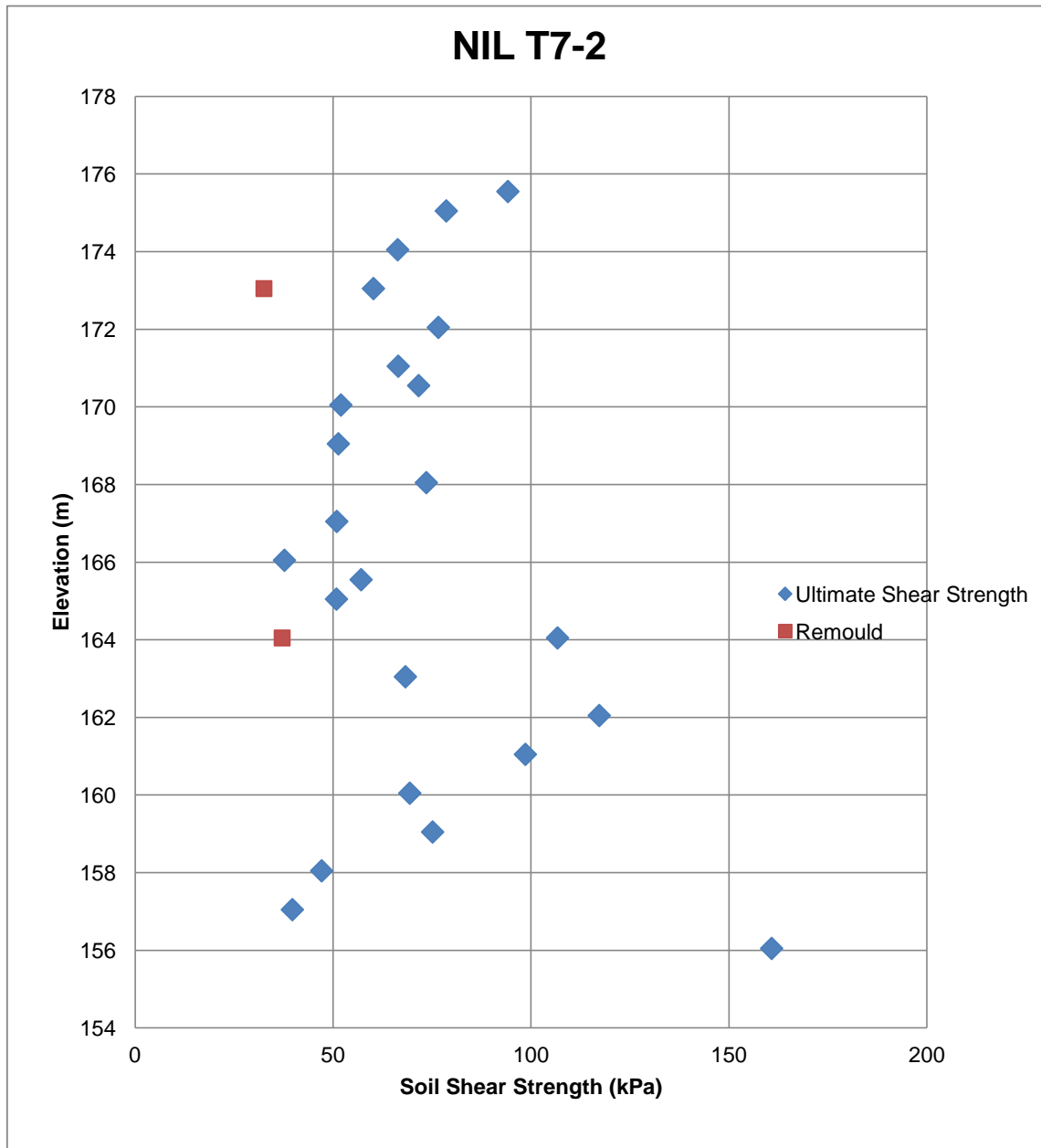
Sheet 1 of 1

Location: N4679332.1; E332390.8

Predrill Depth : 4.6 m

Datum Geodetic

Ground Surface Elevation: 181.0 m



Operator: SD

Checked: DD

# RECORD OF BOREHOLE No TB5-1

1 OF 1

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679286.0, E332362.0 ORIGINATED BY TA  
 DIST                      HWY WEP BOREHOLE TYPE CME 75 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE Jul 6, 11 - Jul 6, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED      + FIELD VANE ● POCKET PEN.      × LAB VANE							
181.0	Pavement Surface														
180.9	ASPHALT														
180.7	Grey FILL														
180.3	Crushed Limestone														
180.2	Greenish Brown FILL		1A, B	SS	8										
179.9	Silty Clay														
179.1	Some sand, trace gravel														
179.0	Trace topsoil														
	TOPSOIL		2	SS	8										
	Mottled Brown-Grey														
	CLAYEY SILT														
	Some sand, trace gravel														
	Stiff		3	SS	10										
			4	SS	18										
	-Trace to some pink nodules														
	Stiff		5	SS	14										
			6	SS	10										
	Firm		7	SS	7										
			8	SS	5										
				VT											
			9	SS	4										
				VT											
			10	SS	5										
				VT											
170.9	END OF BOREHOLE														
10.1	(no refusal)														
	Borehole dry on completion														

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



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# RECORD OF BOREHOLE No TB5-2

1 OF 1

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679261.2, E332400.9 ORIGINATED BY LC  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE Jul 10, 11 - Jul 10, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE							
								20 40 60 80 100									10 20 30	
180.8	Ground Surface																	
0.0	TOPSOIL																	
180.2																		
0.6	Brown-Grey CLAYEY SILT Some sand, trace gravel Heavily weathered topsoil/roots in fissures Firm Brown-Grey -Weathered -Trace roots, hairline sand/silt lenses		1	SS	6													
			2	SS	11													
			3	SS	19													
			4	SS	15													
			5	SS	9													
			6	SS	8													
			7	SS	7													
			8	SS	6													
				VT														
			9	SS	4													
			10	SS	4													
				VT														

-end of drilling  
July 10; continue  
July 11

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 22/09/11

# RECORD OF BOREHOLE No TB5-3

1 OF 1

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679239.6, E332429.4 ORIGINATED BY LC  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE Jul 10, 11 - Jul 10, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE						W <sub>p</sub>	W	W <sub>L</sub>
181.3 0.0 180.9 0.4	Ground Surface  Black <b>TOPSOIL</b> Sandy  Fine Brown/Yellow <b>SAND</b> -Trace to some silt -Trace to some topsoil/roots -To firm heavily weathered Brown/Grey fill -Some sand, trace gravel with rootlets																		
			A	AS			181												
			1	SS	5														
179.8 1.5			2	SS	17		180												
			3	SS	18		179												
			4	SS	19		178												
			5	SS	11		177												
			6	SS	7		176												
			7	SS	6		175												
				VT															
							174												
			9	SS	5		173												
			10	SS	4		172												
				VT															
171.2 10.1	<b>END OF BOREHOLE</b> (no refusal)  Borehole dry on completion						171												
							170												
							169												
							168												
							167												

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 22/09/11

# RECORD OF BOREHOLE No TB5-4

1 OF 1

**METRIC**

W.P. RFP No. 09-54-1007 LOCATION N4679221.9, E332459.0 ORIGINATED BY SD  
 DIST                      HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS  
 DATUM Geodetic DATE Jul 10, 11 - Jul 10, 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
								○ UNCONFINED ● POCKET PEN.	+ FIELD VANE × LAB VANE									
181.7	Ground Surface																	
0.0 181.4 0.3 181.1 0.6	Black TOPSOIL some sand																	
	Brown SAND Some silt		1	SS	6													
	Mottled Brown CLAYEY SILT Some sand, trace gravel Moist Brown		2	SS	15													
	-Fissured with sandy silt hairline lenses		3	SS	19													
	Stiff to very stiff -Fissured		4	SS	15													
			5	SS	15													
	Grey, stiff		6	SS	9													
	Firm		7	SS	7													
			8	SS	5													
				VT														
			9	SS	5													
			10	SS	4													
				VT														
171.6 10.1	END OF BOREHOLE (no refusal)																	
	Borehole dry on completion																	

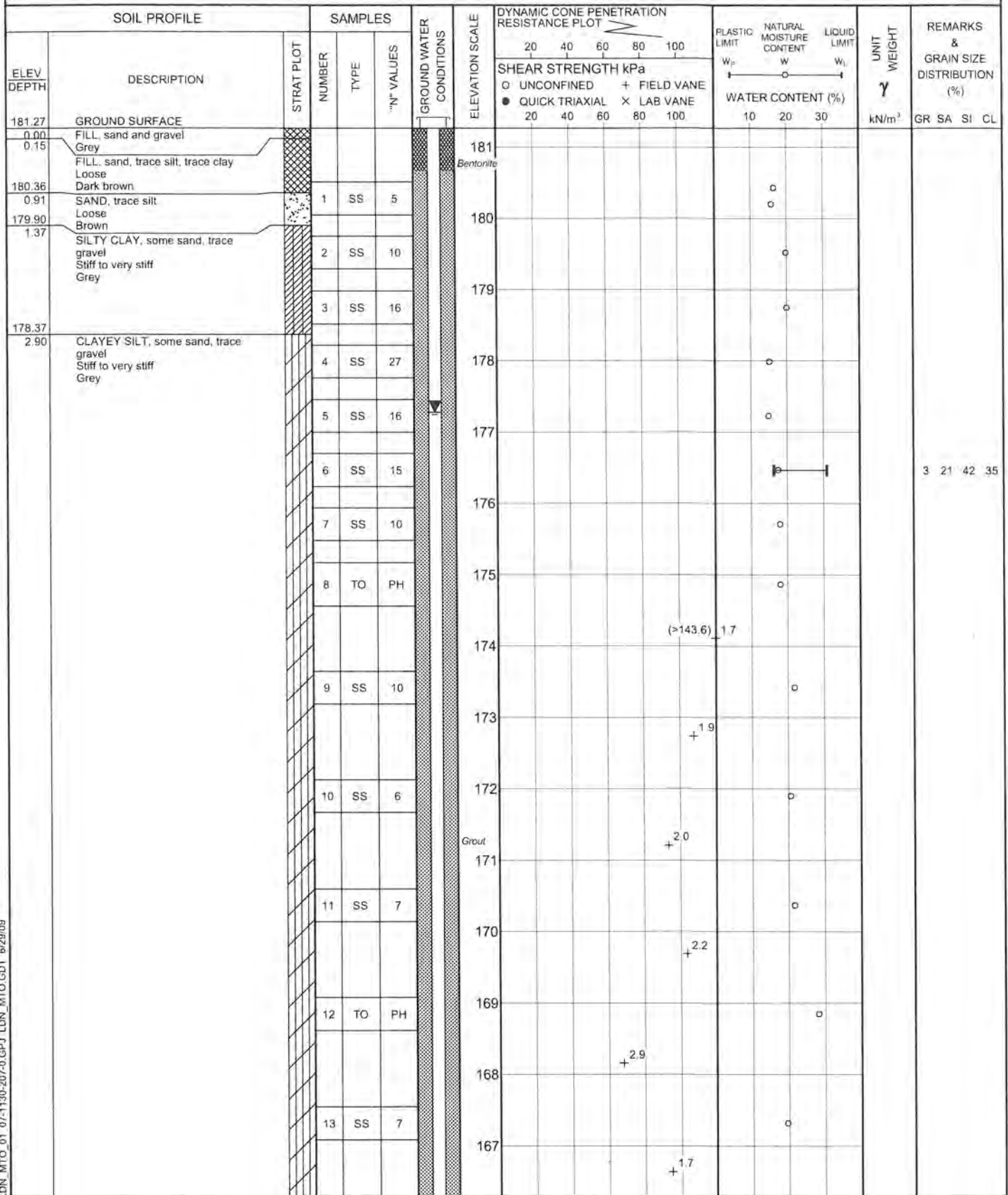
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 22/09/11

## **Appendix B: Borehole and CPT Logs from Previous Investigations**



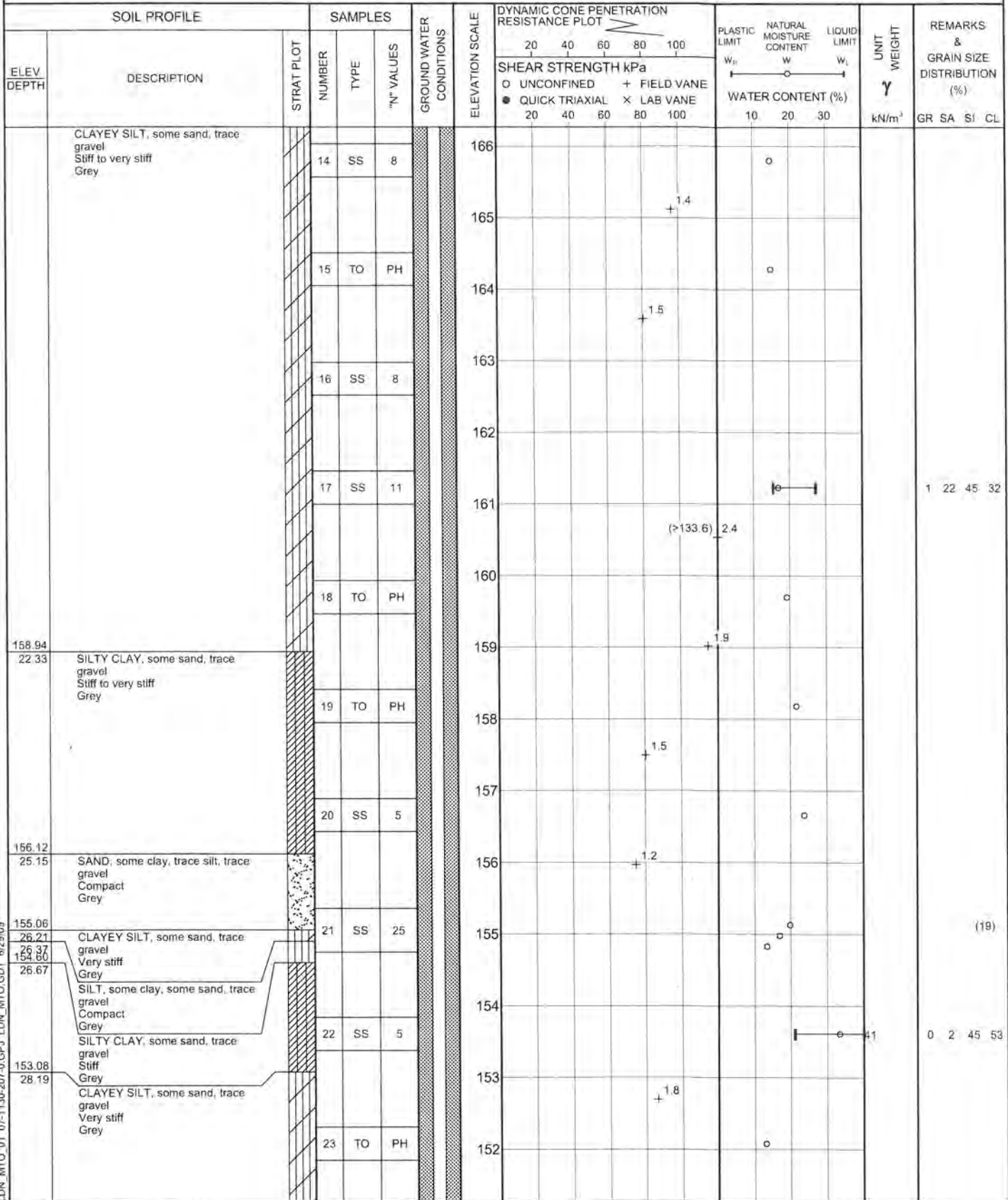
PROJECT 07-1130-207-0 **RECORD OF BOREHOLE No 127** 1 OF 4 **METRIC**  
W.P. LOCATION N 4679370.9, E 332251.6 ORIGINATED BY SM  
DIST WEST HWY 401/3 BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC COMPILED BY BRS  
DATUM GEODETIC DATE March 11, 2008 - March 13, 2008 CHECKED BY *SLF*



Continued Next Page

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

PROJECT 07-1130-207-0		<b>RECORD OF BOREHOLE No 127</b>		2 OF 4	<b>METRIC</b>
W.P.	LOCATION	N 4679370.9, E 332251.6		ORIGINATED BY SM	
DIST WEST HWY 401/3	BOREHOLE TYPE	POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY BRS	
DATUM GEODETIC	DATE	March 11, 2008 - March 13, 2008		CHECKED BY <i>SJB</i>	



# RECORD OF BOREHOLE No 127

3 OF 4

**METRIC**

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4679370.9 E 332251.6

ORIGINATED BY SM

DIST

WEST

HWY 401/3

BOREHOLE TYPE

POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY BRS

DATUM GEODETIC

DATE

March 11, 2008 - March 13, 2008

CHECKED BY *SJB*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
150.54	CLAYEY SILT, some sand, trace gravel Very stiff Grey		24	SS	163		151								(39)
30.73	SANDY SILT, trace clay, trace gravel, with cobbles Very dense Grey						150								
			25	SS	100/50mm		149								
148.47	DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous Brown to grey  (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		26	NQ RC			148								UC
32.80			27	NQ RC			147								
			28	NQ RC			146								
145.16	END OF BOREHOLE														
36.11	Borehole dry during drilling between March 11 and 13, 2008.  Water level measured in deep piezometer at elev. 177.74m on March 20, 2008.  Water level measured in deep piezometer at elev. 178.27m on July 22, 2008.  Water level measured in deep piezometer at elev. 178.12m on August 11, 2008.  Water level measured in deep piezometer at elev. 177.87m on September 19, 2008.  Water level measured in deep piezometer at elev. 177.74m on November 11, 2008.  Water level measured in deep piezometer at elev. 177.28m on January 26, 2009.														

PROJECT <u>07-1130-207-0</u>		<b>RECORD OF BOREHOLE No 127A</b>		1 OF 1	<b>METRIC</b>
W.P. _____		LOCATION <u>N 4679370.9 :E 332251.6</u>		ORIGINATED BY <u>SM</u>	
DIST <u>WEST</u>	HWY <u>401/3</u>	BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>BRB</u>	
DATUM <u>GEODETIC</u>		DATE <u>March 13, 2008</u>		CHECKED BY <u>SJB</u>	

[illegible]

PROJECT <u>09-1132-0080</u>		<b>RECORD OF BOREHOLE No CPT-322</b>		1 OF 1		<b>METRIC</b>	
W.P. _____		LOCATION <u>N 4679294.0 ; E 332478.2</u>		ORIGINATED BY <u>TA</u>			
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>DMB</u>			
DATUM <u>GEODETIC</u>		DATE <u>January 7, 2010</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED      + FIELD VANE								
						● QUICK TRIAXIAL      × LAB VANE										
181.50	ROAD SURFACE							20 40 60 80 100								
0.05	ASPHALT PAVEMENT															
181.04	FILL, limestone gravel, crushed Grey															
0.46	TOPSOIL, clayey Very stiff Black		1	SS	17											
180.28	CLAYEY SILT, some sand, trace gravel, with occasional fissures, silt partings and seams Hard Brown becoming grey below about elev. 177.5m		2	SS	35											
1.22																
			3	SS	44											
177.84			4	SS	37											
3.66	END OF BOREHOLE															
	Borehole dry during drilling on January 7, 2010.															

LDN\_MTO\_06 09-1132-0080.GPJ LDN\_MTO.GDT 11/03/10

PROJECT: 09-1132-0080

# RECORD OF CONE PENETRATION TEST CPT-322

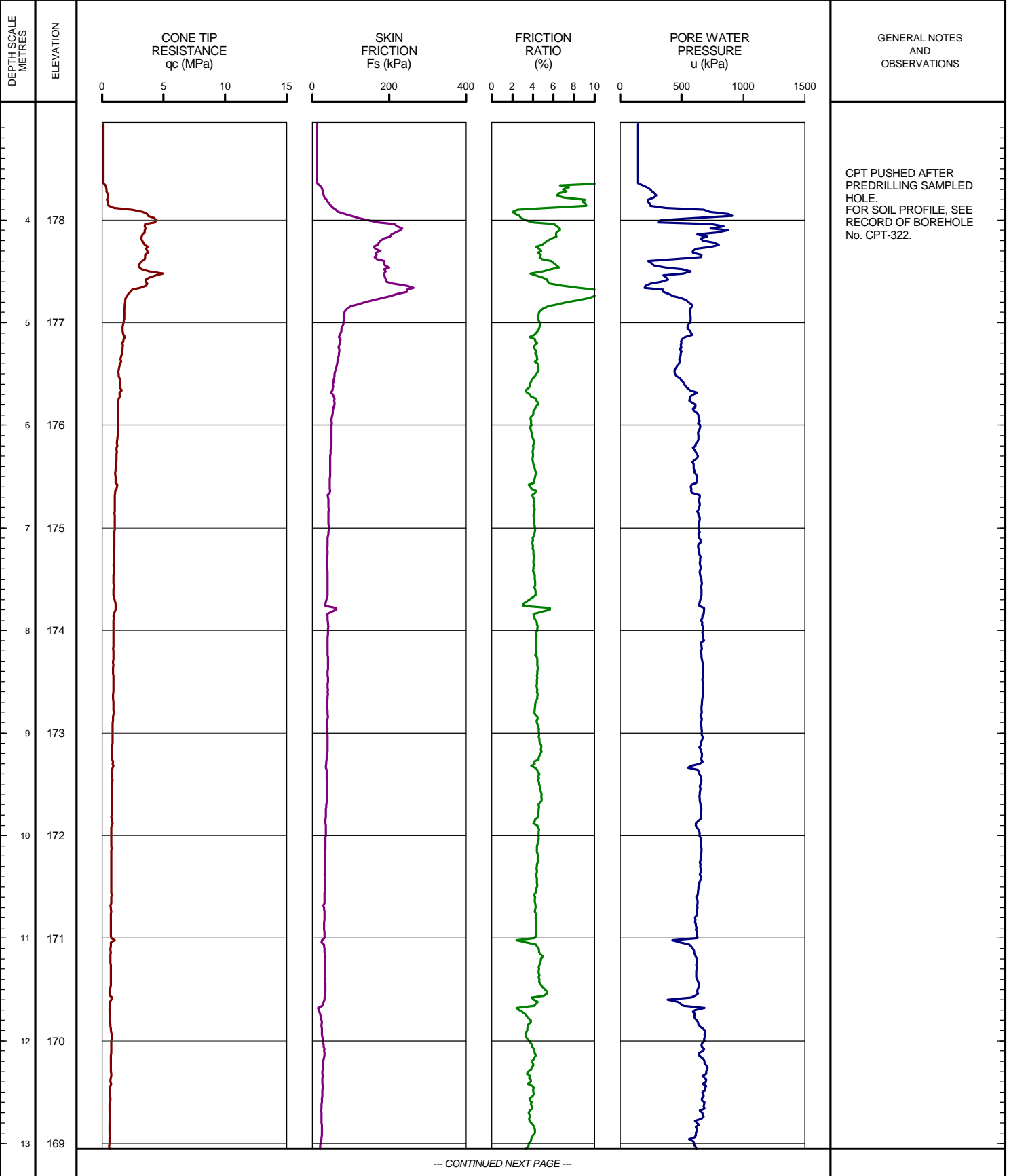
SHEET 1 OF 2

LOCATION: N 4679294.0 ;E 332478.2

TEST DATE: January 8, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.50m PREDRILL DEPTH: 3.05m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LDN\_CPT\_01 09-1132-0080-CPT.GPJ GLDR\_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

# RECORD OF CONE PENETRATION TEST CPT-322

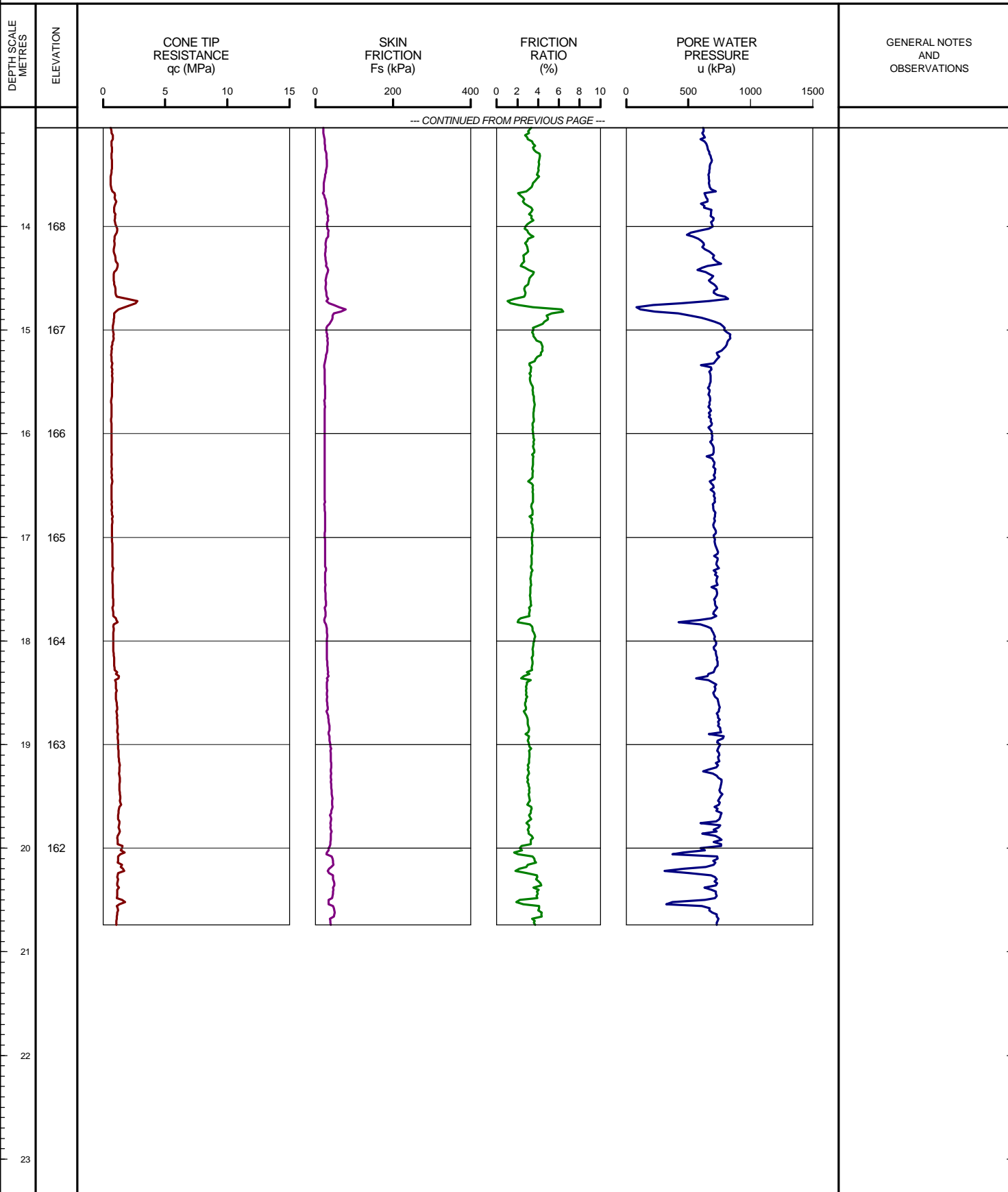
SHEET 2 OF 2

LOCATION: N 4679294.0 ;E 332478.2

TEST DATE: January 8, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.50m PREDRILL DEPTH: 3.05m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LON\_CPT\_01 09-1132-0080-CPT.GPJ GLDR\_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



OPERATOR: TA

CHECKED:



# RECORD OF BOREHOLE No 122

1 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4679265.4 :E 332537.9

ORIGINATED BY SM

DIST WEST HWY 401/3

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

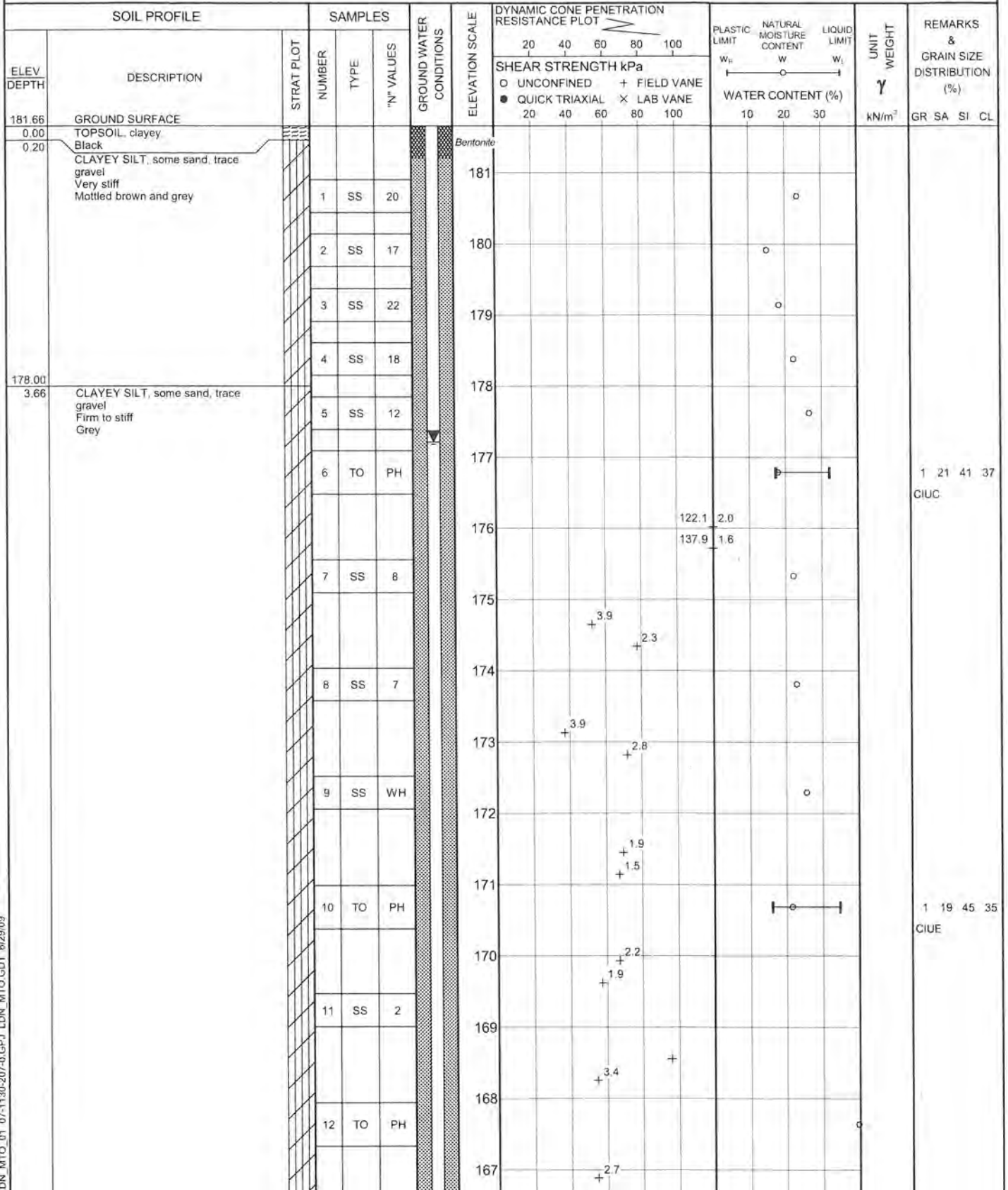
COMPILED BY BRS

DATUM GEODETIC

DATE

January 24, 2008 - January 29, 2008

CHECKED BY *SS*



Continued Next Page

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

LDN MTO\_01 07-1130-207-0.GPJ LDN MTO GDT 8/29/09



PROJECT 07-1130-207-0		<b>RECORD OF BOREHOLE No 122</b>		3 OF 4	<b>METRIC</b>
W.P. _____		LOCATION N 4679265.4 : E 332537.9		ORIGINATED BY SM	
DIST WEST HWY 401/3		BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY BRS	
DATUM GEODETIC		DATE January 24, 2008 - January 29, 2008		CHECKED BY <i>SJS</i>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT $w_p$	NATURAL MOISTURE CONTENT $w$	LIQUID LIMIT $w_l$	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE	WATER CONTENT (%)					
150.42	SILTY SAND, fine to medium, trace clay Compact Grey		23	SS	13									(29)
31.24	SANDY SILT, trace clay, with clayey silt intrusions Very dense Grey		24	SS	84									
146.61	LIMESTONE, fresh, medium strong, thinly laminated to laminated, very fine to fine grained, faintly to strongly porous Brown to grey  (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		25	SS	100/ 2.5mm									
35.05			26	SS	50/ 3.8mm									
			27	NQ RC										
			28	NQ RC										
			29	NQ RC										
			30	NQ RC										
141.33	END OF BOREHOLE													
40.33	<p>Borehole dry during drilling between January 24 and 29, 2008.</p> <p>Water level measured in deep piezometer at elev. 178.01m on July 22, 2008</p> <p>Water level measured in deep piezometer at elev. 178.26m on August 11, 2008.</p> <p>Water level measured in deep piezometer at elev. 178.26m on September 19, 2008.</p> <p>Water level measured in deep piezometer at elev. 177.54m on November 11, 2008.</p> <p>Water level measured in deep piezometer at elev. 177.21m on January 28, 2009.</p>													

LDN MTO\_01 07-1130-207-0.GPJ LDN MTO GDT 8/29/09

SHEET 4 OF 4

DATUM: GEODETIC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

[illegible]

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



**RECORD OF BOREHOLE No 126**

1 OF 1

**METRIC**

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4679237.2 :E 332335.5

ORIGINATED BY DM

DIST WEST HWY 401/3

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY BRS

DATUM GEODETIC

DATE

March 26, 2008

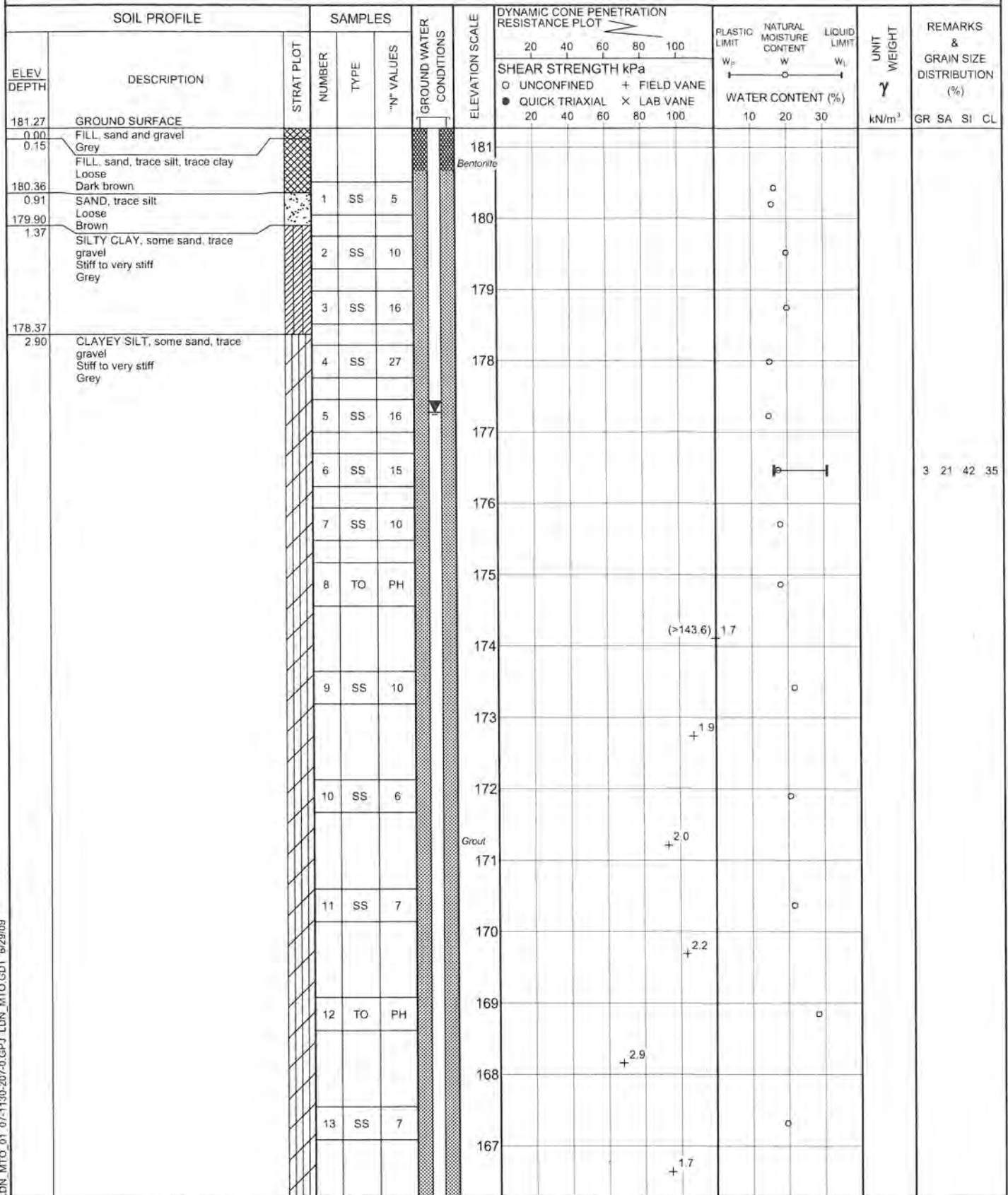
CHECKED BY *SSB*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE							
								20 40 60 80 100							
								● QUICK TRIAXIAL × LAB VANE							
								20 40 60 80 100							
180.61	GROUND SURFACE														
0.00	FILL, sand and gravel, trace silt Compact Brown		1	SS	24		180								
179.09															
1.52	CLAYEY SILT, some sand, trace gravel Stiff to hard Brown, becoming grey at about elev. 177.0m		2	SS	8		179								
			3	SS	20		178								
			4	SS	33		177								
			5	SS	26		176								
			6	SS	17		175								
			7	SS	11		174								
			8	SS	10		173								
			9	SS	10										
			10	SS	8										
172.38	END OF BOREHOLE														
8.23	Borehole dry during drilling on March 26, 2008.														

DN\_MTO\_01 07-11-30-207-0.GPJ LUN\_MTO.GDT 6/23/09

LDN\_MTO\_01 07-1130-207-0.GPJ LDN\_MTO.GDT 6/29/09

PROJECT 07-1130-207-0 **RECORD OF BOREHOLE No 127** 1 OF 4 **METRIC**  
W.P. LOCATION N 4679370.9 , E 332251.6 ORIGINATED BY SM  
DIST WEST HWY 401/3 BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC COMPILED BY BRS  
DATUM GEODETIC DATE March 11, 2008 - March 13, 2008 CHECKED BY *SLF*



Continued Next Page

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



# RECORD OF BOREHOLE No 127

2 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4679370.9, E 332251.6

ORIGINATED BY SM

DIST WEST HWY 401/3

BOREHOLE TYPE

POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

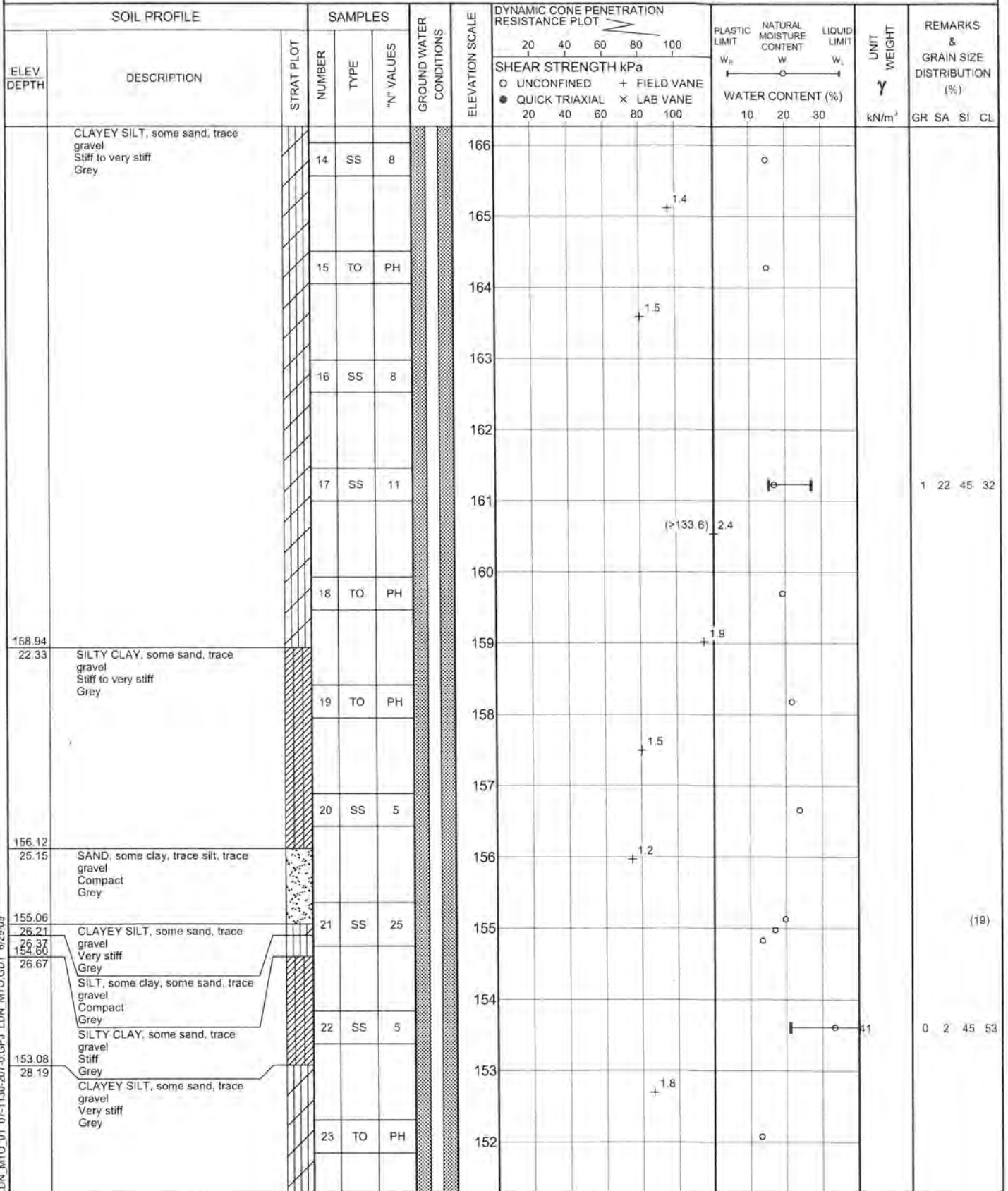
COMPILED BY BRS

DATUM GEODETIC

DATE

March 11, 2008 - March 13, 2008

CHECKED BY SJB



Continued Next Page

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

LDN\_MTO\_01\_07-1130-207-0.GPJ LDN\_MTO.GDT 6/29/09

PROJECT 07-1130-207-0		RECORD OF BOREHOLE No 127		3 OF 4		METRIC							
W.P. _____		LOCATION N 4679370.9 E 332251.6		ORIGINATED BY SM									
DIST WEST HWY 401/3		BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY BRS									
DATUM GEODETIC		DATE March 11, 2008 - March 13, 2008		CHECKED BY <i>SJB</i>									
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>l</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
							20 40 60 80 100						
150.54	CLAYEY SILT, some sand, trace gravel Very stiff Grey		24	SS	163								(39)
30.73	SANDY SILT, trace clay, trace gravel, with cobbles Very dense Grey												
			25	SS	100/50mm								
148.47													
32.80	DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous Brown to grey  (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		26	NQ RC									UC
			27	NQ RC									
			28	NQ RC									
145.16													
36.11	END OF BOREHOLE												
	<p>Borehole dry during drilling between March 11 and 13, 2008.</p> <p>Water level measured in deep piezometer at elev. 177.74m on March 20, 2008.</p> <p>Water level measured in deep piezometer at elev. 178.27m on July 22, 2008.</p> <p>Water level measured in deep piezometer at elev. 178.12m on August 11, 2008.</p> <p>Water level measured in deep piezometer at elev. 177.87m on September 19, 2008.</p> <p>Water level measured in deep piezometer at elev. 177.74m on November 11, 2008.</p> <p>Water level measured in deep piezometer at elev. 177.28m on January 26, 2009.</p>												

LDN\_MTO\_01\_07-1130-207-0.GPJ LDN\_MTO.GDT 6/29/09

PROJECT: 07-1130-207-0

## RECORD OF DRILLHOLE: 127

SHEET 4 OF 4

LOCATION: N 4679370.9 ; E 332251.6

DRILLING DATE: March 11, 2008 - March 13, 2008

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	ELEVATION	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth Ro - Rough Br - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols										DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
				DEPTH (m)						RECOVERY		R Q D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec	DIP w/11 CORE AXIS	TYPE AND SURFACE DESCRIPTION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
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LDN-ROCK-03 07-1130-207-0-ROCK-GPJ GLDR LDN-GDT 6/29/09 DATA INPUT: WDF

DEPTH SCALE

1:75



LOGGED: SG

CHECKED: SJB



# RECORD OF BOREHOLE No 129

1 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4679625.1 E 332109.7

ORIGINATED BY LZ/CC/MA/SM

DIST WEST HWY 401/3

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

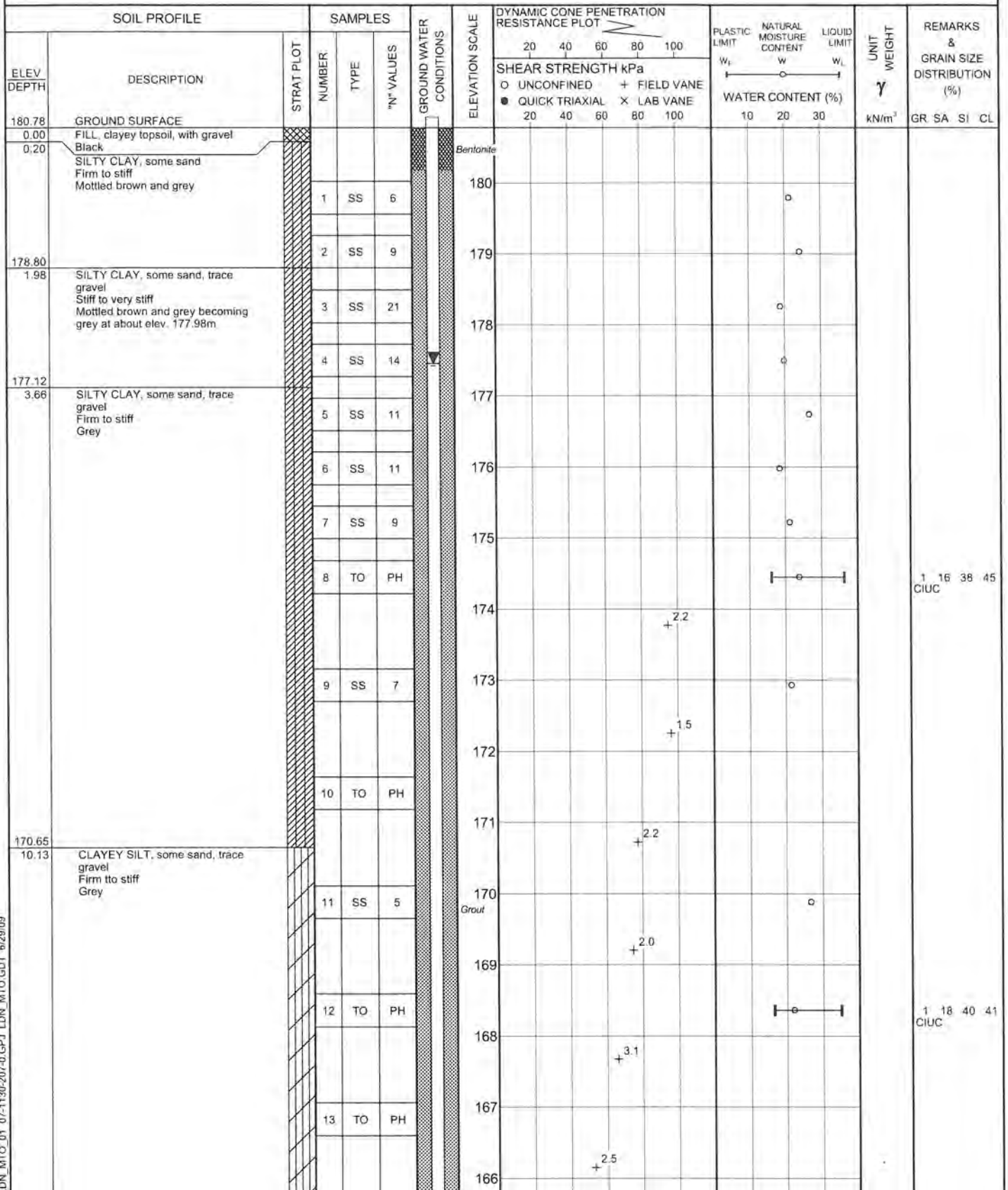
COMPILED BY BRS

DATUM GEODETIC

DATE

March 4, 2008 - March 10, 2008

CHECKED BY SJB



Continued Next Page

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

LDN MTO\_01 07-1130-207-0.GPJ LDN MTO GDT 6/29/09



# RECORD OF BOREHOLE No 129

2 OF 4

METRIC

PROJECT 07-1130-207-0

W.P. \_\_\_\_\_

LOCATION N 4679625.1, E 332109.7

ORIGINATED BY LZ/CC/MA/SM

DIST WEST HWY 401/3

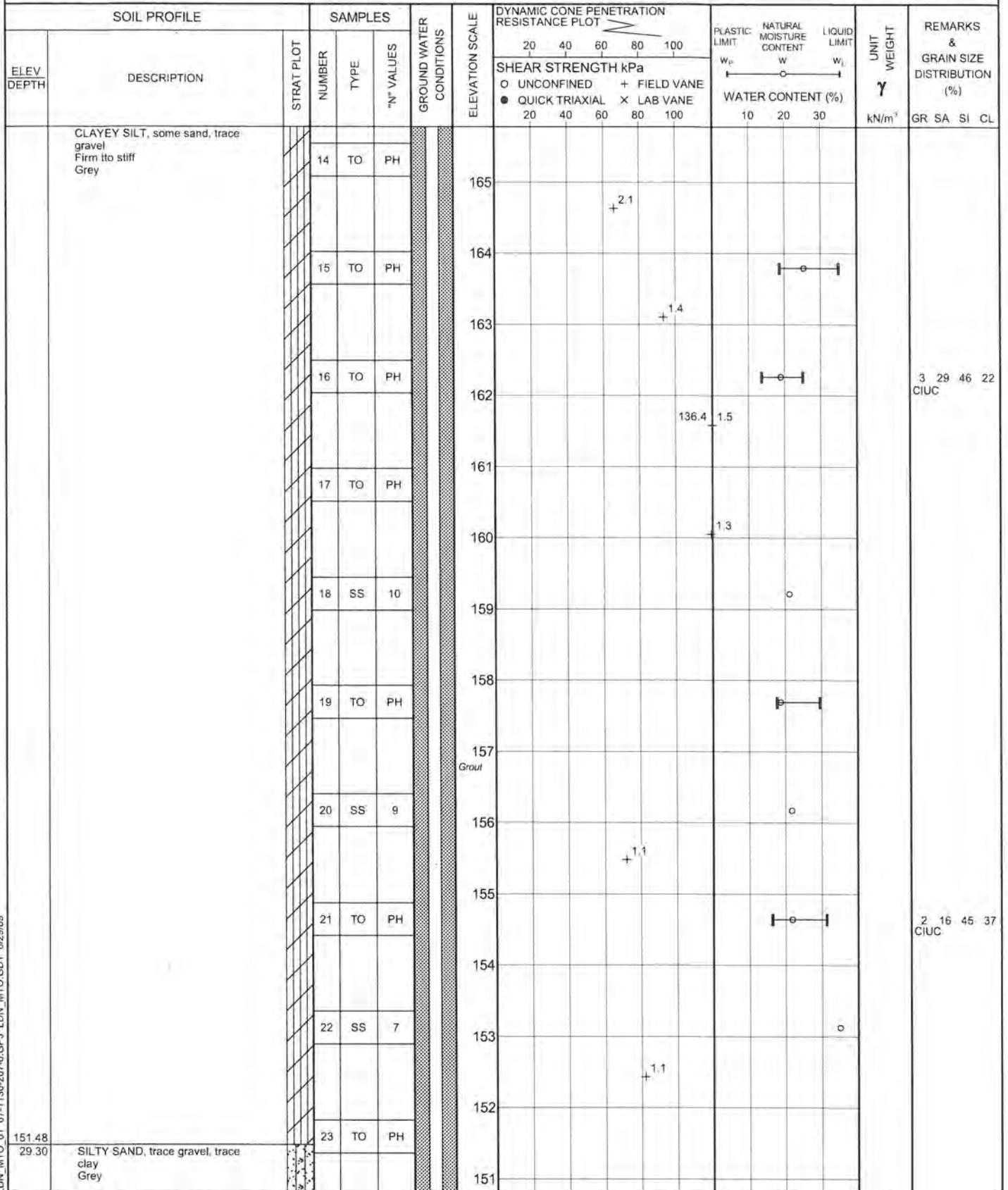
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY BRS

DATUM GEODETIC

DATE March 4, 2008 - March 10, 2008

CHECKED BY *SSB*



Continued Next Page

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

LDN MTO\_01 07-1130-207-0.GPJ LDN\_MTO.GDT 6/23/09

# RECORD OF BOREHOLE No 129

3 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4679625.1 E 332109.7

ORIGINATED BY LZ/CC/MA/SM

DIST WEST HWY 401/3

BOREHOLE TYPE

POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY BRS

DATUM GEODETIC

DATE

March 4, 2008 - March 10, 2008

CHECKED BY **SJB**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
150.55														
30.23	SAND AND GRAVEL, medium to coarse, trace silt Compact to very dense Grey		24	SS	21		150							
						Bentonite	149							
			25	SS	125	Screen	148							
147.88	DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous Grey to brown					Bedrock soil interface	147							
32.90	(FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		26	SS	100/120mm		146							
			27	NQ RC			145	0						
			28	NQ RC			144	94						
			29	NQ RC			143	33						
			30	NQ RC			142	174						
143.78	END OF BOREHOLE						141							
37.00	Borehole dry during drilling between March 4 and 10, 2008.  Water level measured in deep piezometer at elev. 178.50m on July 22, 2008.  Water level measured in deep piezometer at elev. 177.88m on August 11, 2008.  Water level measured in deep piezometer at elev. 177.48m on September 19, 2008.  Water level measured in deep piezometer at elev. 177.57m on November 11, 2008.  Water level measured in deep piezometer at elev. 177.46m on January 28, 2009.													

LDN\_MTO\_01 07-1130-207-0.GPJ LDN\_MTO.GDT 6/29/09



PROJECT: 07-1130-207-0

## RECORD OF DRILLHOLE: 129

SHEET 4 OF 4

LOCATION: N 4679625.1 E 332109.7




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

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (m/min)	COLOUR FLUSH	ELEVATION											DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION	
				DEPTH (m)	RETURN					RECOVERY		R Q D %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec						
										TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 <sup>-1</sup>	10 <sup>-2</sup>	10 <sup>-3</sup>				
																			0 5 10 15 20			0 5 10 15 20
33		ROCK SURFACE		147.88																		
	MUD ROTARY NO ROCK CORE	DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous, interbedded light and dark grey		32.90																		
34					1				147													
35		DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous, interbedded medium and dark brown, stylolites at 35.64 m		146.21 34.57											JN, PL, SM	CI						
					2					146												
36					3					145												
		DOLOSTONE/LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly porous, stylolitic, interbedded light and dark grey		144.56 36.22																		
37		END OF DRILLHOLE		143.78 37.00						144					JN, IR, Ro	CI						
38																						
39																						
40																						
41																						
42																						
43																						
44																						
45																						
46																						
47																						

DEPTH SCALE

1:75



LOGGED: SG

CHECKED: *SG*

**RECORD OF BOREHOLE No 129A**

1 OF 1

**METRIC**

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4679625.1 , E 332109.7

ORIGINATED BY SM

DIST WEST

HWY 401/3

BOREHOLE TYPE

POWER AUGER, HOLLOW STEM

COMPILED BY BRS

DATUM GEODETIC

DATE

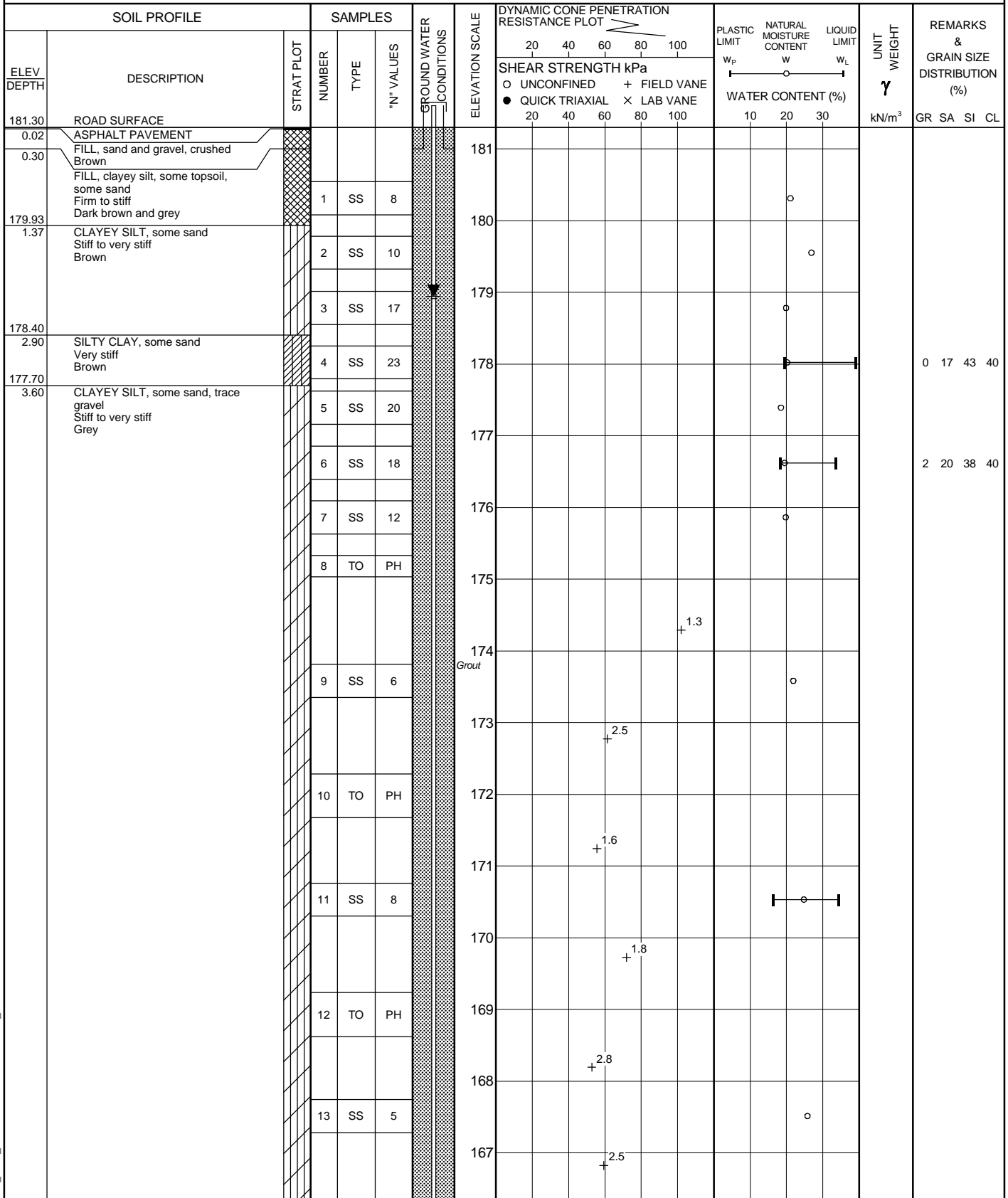
March 4, 2008

CHECKED BY **SSB**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
180.78	SOIL CONDITIONS INFERRED FROM BOREHOLE No. 129 GROUND SURFACE							20 40 60 80 100						
0.00	FILL, clayey topsoil, with gravel							0 UNCONFINED + FIELD VANE						
0.20	Black							● QUICK TRIAXIAL × LAB VANE						
	SILTY CLAY, some sand							20 40 60 80 100						
	Firm to stiff													
	Mottled brown and grey													
178.50	SILTY CLAY, some sand, trace gravel													
1.98	Stiff to very stiff													
	Mottled brown and grey to grey at about elev. 177.98m													
177.12	SILTY CLAY, some sand, trace gravel													
3.66	Firm to stiff													
	Grey													
171.18	END OF BOREHOLE													
9.60	Water level measured in shallow piezometer at elev. 178.95m on July 22, 2008.3													
	Water level measured in shallow piezometer at elev. 178.93m on August 11, 2008.													
	Water level measured in shallow piezometer at elev. 178.95m on September 19, 2008.													
	Water level measured in shallow piezometer at elev. 178.84m on January 28, 2009.													

LDN MTO\_01 07-1130-207-0.GPJ LDN MTO GDT 6/29/09

PROJECT <u>09-1132-0080</u>		<b>RECORD OF BOREHOLE No 323</b>		1 OF 4	<b>METRIC</b>
W.P. _____		LOCATION <u>N 4679521.4 ; E 332167.6</u>		ORIGINATED BY <u>MK/MR</u>	
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>LMK/DMB</u>	
DATUM <u>GEODETIC</u>		DATE <u>December 15, 2009 - December 17, 2009</u>		CHECKED BY _____	

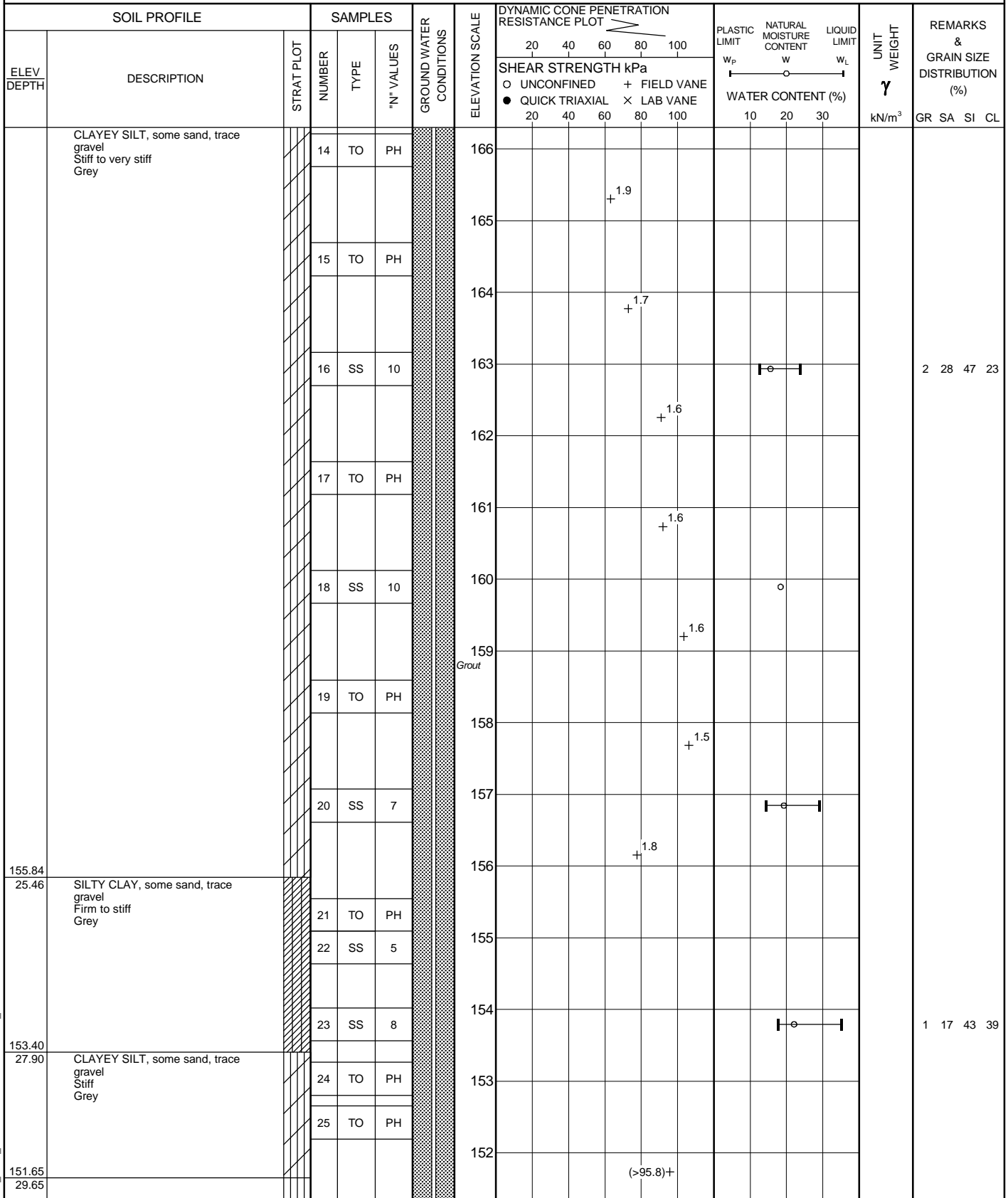


LDN\_MTO\_06 09-1132-0080.GPJ LDN\_MTO.GDT 11/03/10

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT <u>09-1132-0080</u>		<b>RECORD OF BOREHOLE No 323</b>		2 OF 4		<b>METRIC</b>	
W.P. _____		LOCATION <u>N 4679521.4 ; E 332167.6</u>		ORIGINATED BY <u>MK/MR</u>			
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>LMK/DMB</u>			
DATUM <u>GEODETIC</u>		DATE <u>December 15, 2009 - December 17, 2009</u>		CHECKED BY _____			



LDN\_MTO\_06 09-1132-0080.GPJ LDN\_MTO.GDT 11/03/10

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT <u>09-1132-0080</u>		<b>RECORD OF BOREHOLE No 323</b>		3 OF 4	<b>METRIC</b>
W.P. _____		LOCATION <u>N 4679521.4 ; E 332167.6</u>		ORIGINATED BY <u>MK/MR</u>	
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>LMK/DMB</u>	
DATUM <u>GEODETIC</u>		DATE <u>December 15, 2009 - December 17, 2009</u>		CHECKED BY _____	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
												20	40	60	80					
										</										

LDN\_MTO\_06 09-1132-0080.GPJ LDN\_MTO.GDT 11/03/10

INCLINATION: -90°      AZIMUTH: ---

DRILLING CONTRACTOR: LANTECH

DATUM: GEODETIC



**Golder  
Associates**

CHECKED:

1 : 75

DN\_ROCK\_03 09-1132-0080-ROCK.GPJ GLDR\_LDN.GDT 11/03/10 DATA INPUT: LMK

PROJECT: 07-1130-207-0

**RECORD OF CONE PENETRATION TEST CPT-10**

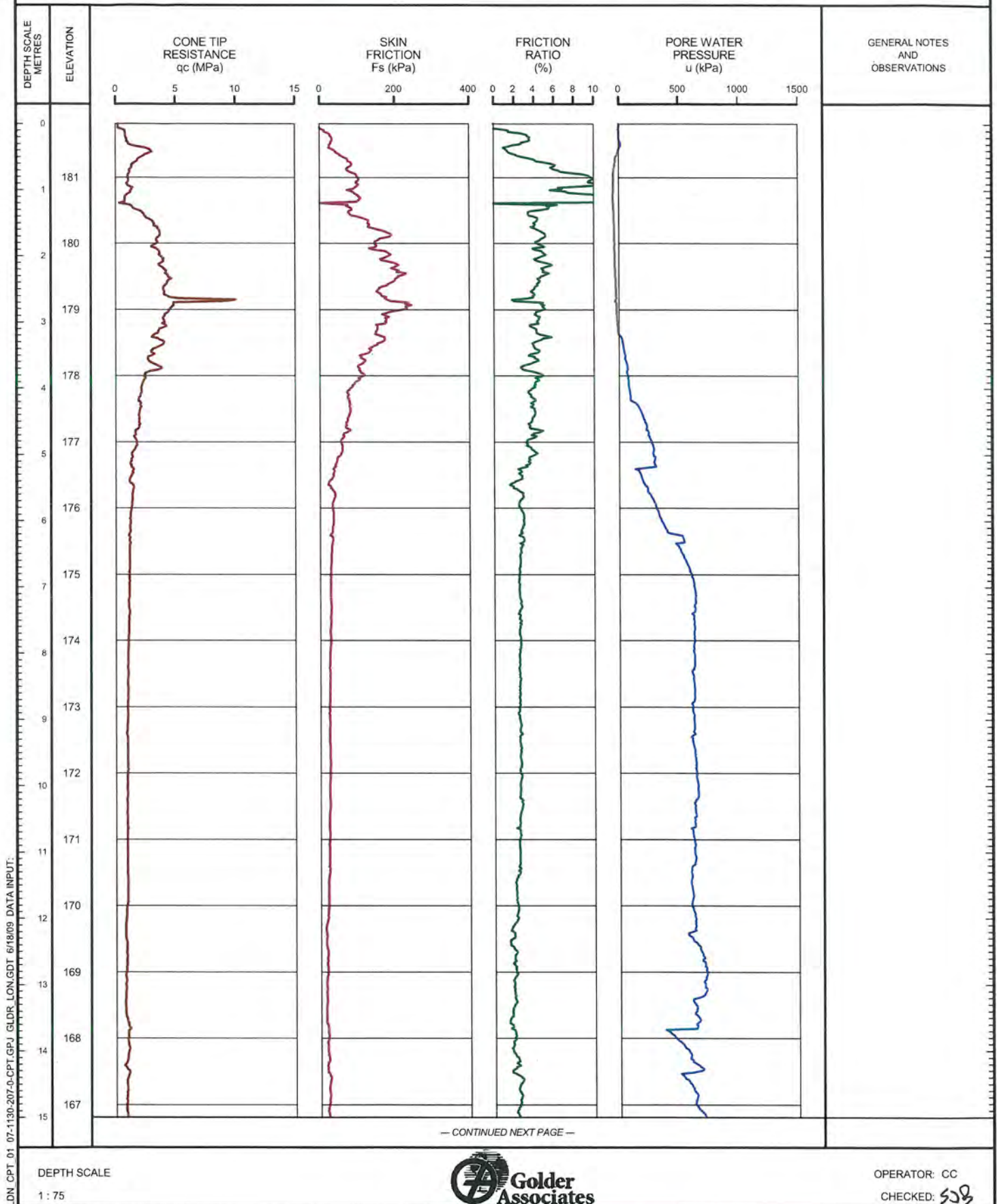
SHEET 1 OF 2

LOCATION: N 4679264.0 ; E 332533.0

TEST DATE: November 10, 2006

DATUM: GEODETIC

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





PROJECT: 07-1130-207-0

**RECORD OF CONE PENETRATION TEST CPT-10**

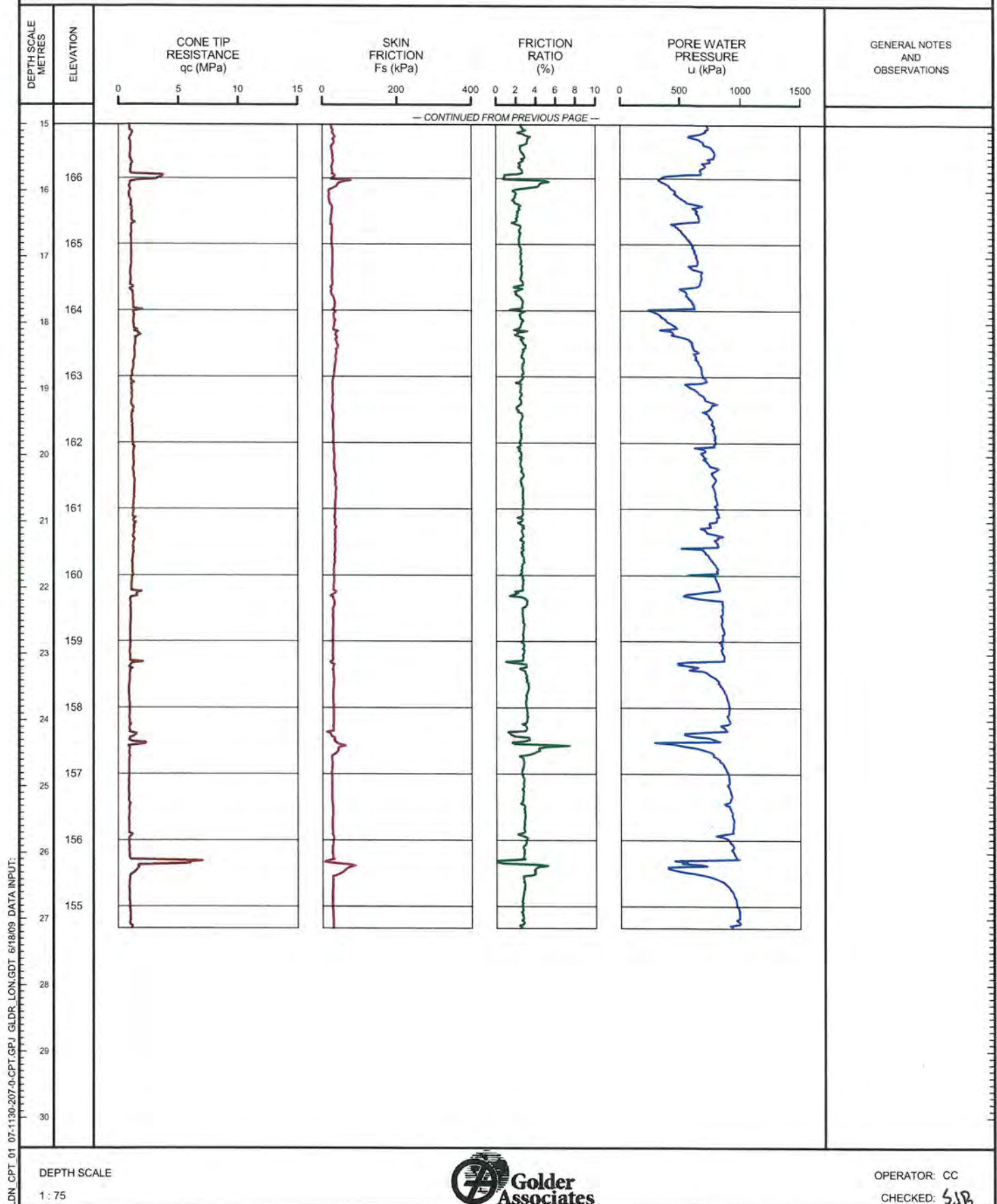
SHEET 2 OF 2

LOCATION: N 4679264.0 :E 332533.0

TEST DATE: November 10, 2006

DATUM: GEODETIC

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



PROJECT: 07-1130-207-0

**RECORD OF CONE PENETRATION TEST CPT-11**

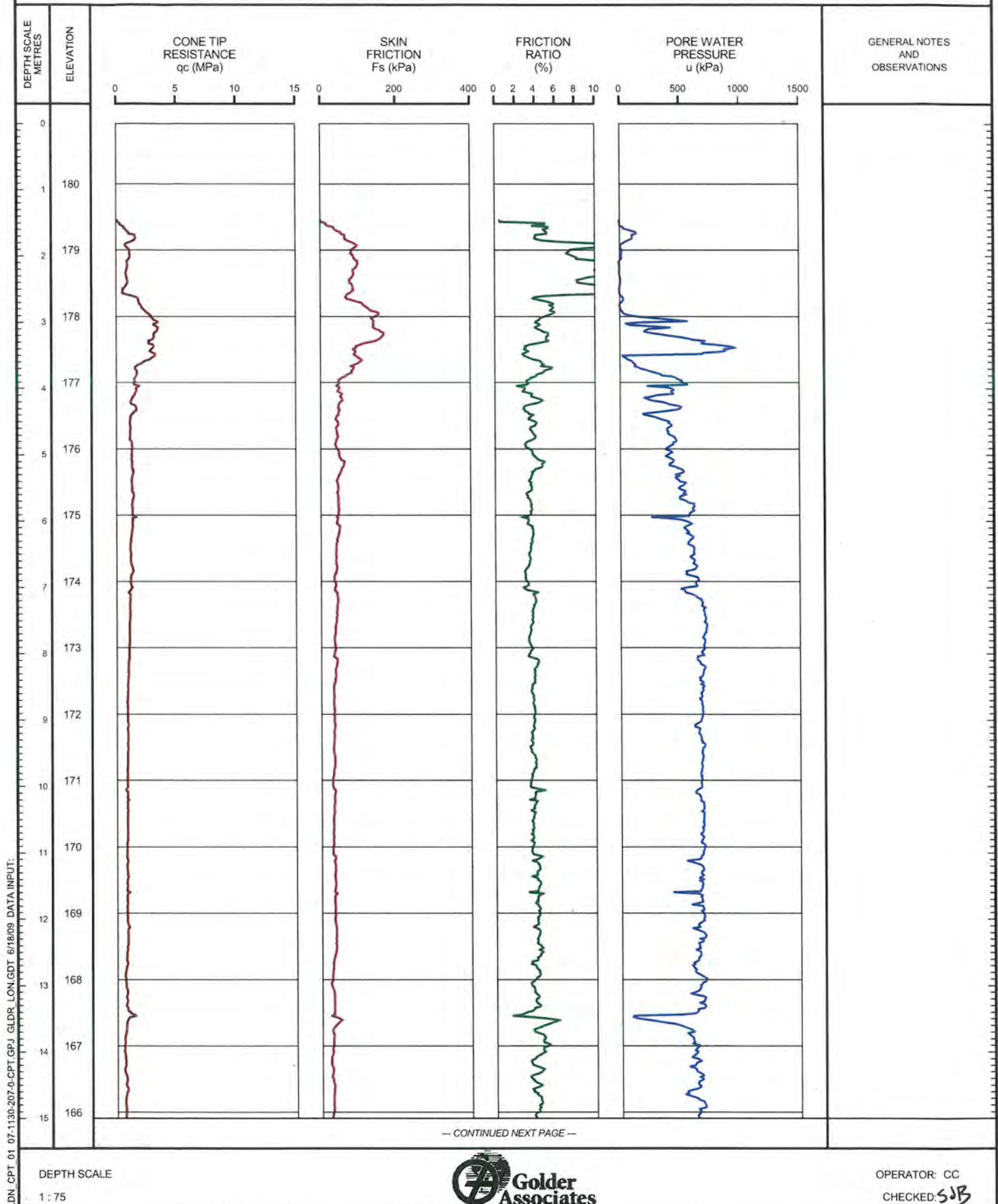
SHEET 1 OF 2

LOCATION: N 4679634.0 ; E 332110.0

TEST DATE: November 10, 2006

DATUM: GEODETIC

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



PROJECT: 07-1130-207-0

# RECORD OF CONE PENETRATION TEST CPT-11

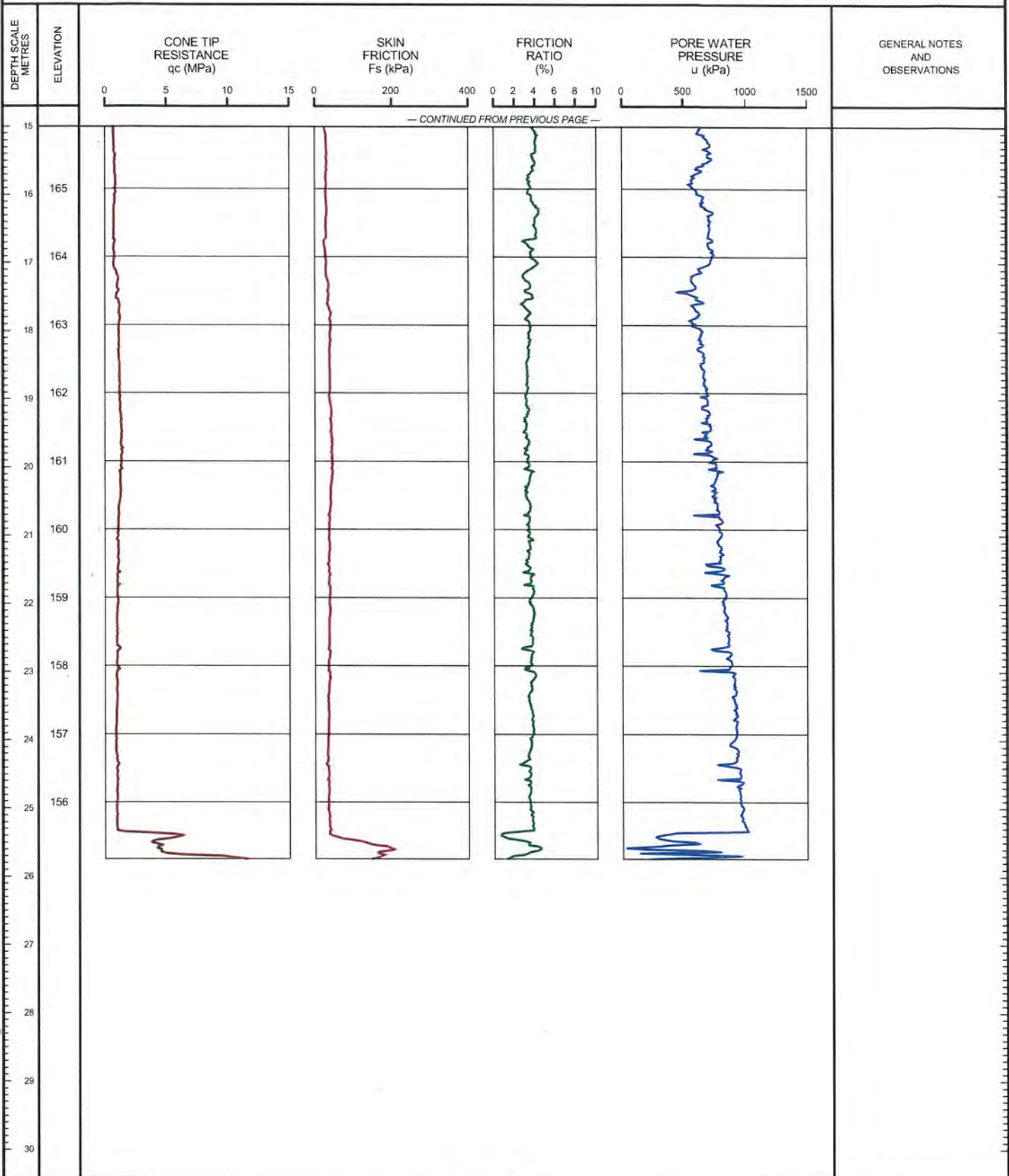
SHEET 2 OF 2

LOCATION: N 4679634.0 :E 332110.0

TEST DATE: November 10, 2006

DATUM: GEODETIC

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



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

DEPTH SCALE  
1 : 75



OPERATOR: CC  
CHECKED: *SSB*



PROJECT <u>07-1130-207-0</u>		<b>RECORD OF BOREHOLE No CPT-123</b>		1 OF 1 <b>METRIC</b>	
W.P. _____		LOCATION <u>N 4679309.7 :E 332536.3</u>		ORIGINATED BY <u>CC</u>	
DIST <u>WEST</u> HWY <u>401/3</u>		BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>SJL</u>	
DATUM <u>GEODETIC</u>		DATE <u>September 10, 2008</u>		CHECKED BY <u>SJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
181.60	GROUND SURFACE												
0.00	FILL, crushed gravel and recycled aggregate		1	AS									
0.28	Grey and black TOPSOIL, clayey												
180.69	Black												
0.91	CLAYEY SILT, trace sand, trace gravel Firm to stiff Mottled brown and grey		2	SS	7								
			3	SS	14								
179.47	END OF BOREHOLE												
2.13	Borehole dry during drilling on September 10, 2008.												

LDN MTO\_01 07-1130-207-0.GPJ LDN MTO.GDT 6/29/09

PROJECT: 07-1130-207-0

## RECORD OF CONE PENETRATION TEST CPT-123

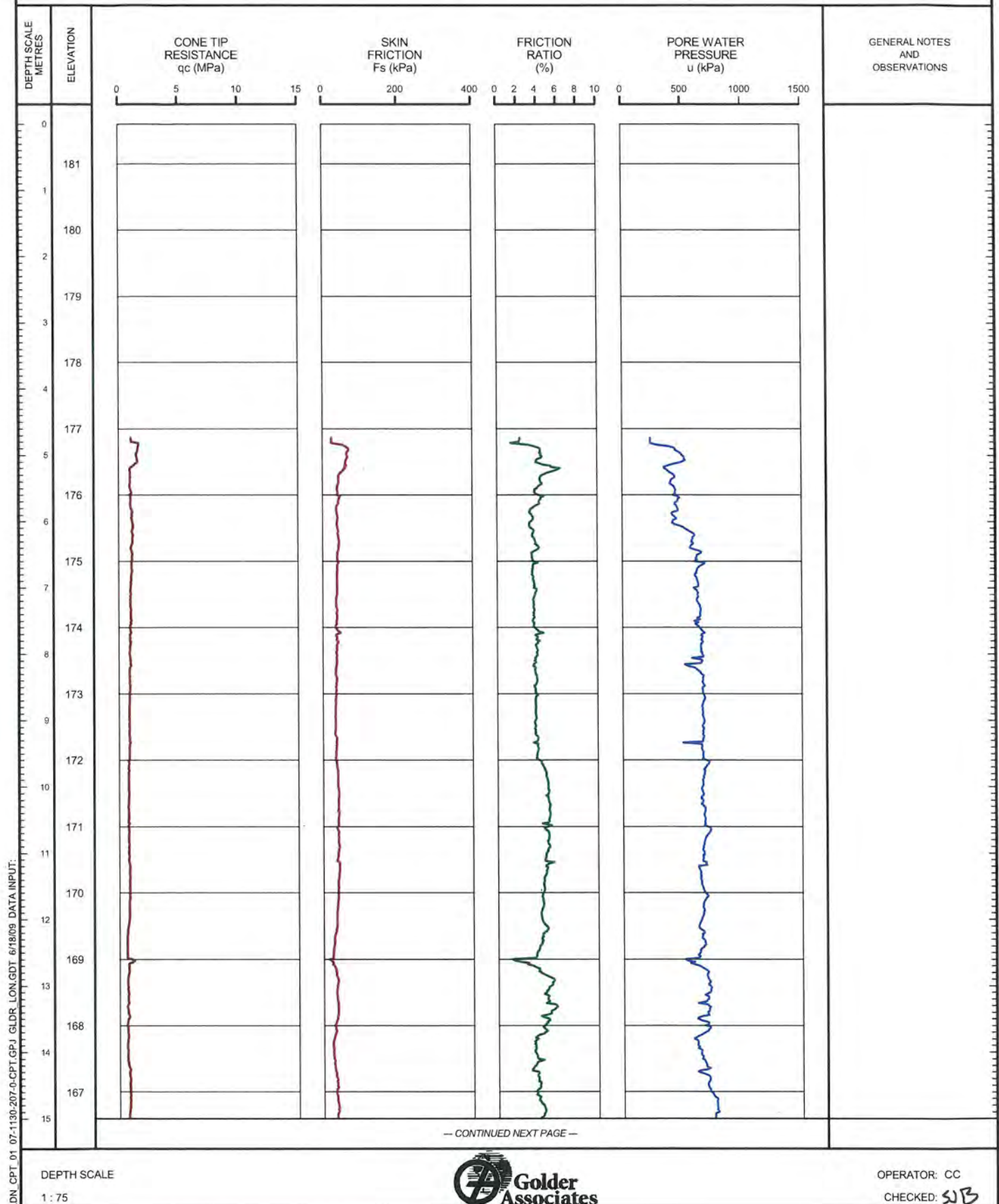
SHEET 1 OF 2

LOCATION: N 4679309.7 ; E 332536.3

TEST DATE: September 29, 2008

DATUM: GEODETIC

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



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

PROJECT: 07-1130-207-0

**RECORD OF CONE PENETRATION TEST CPT-123**

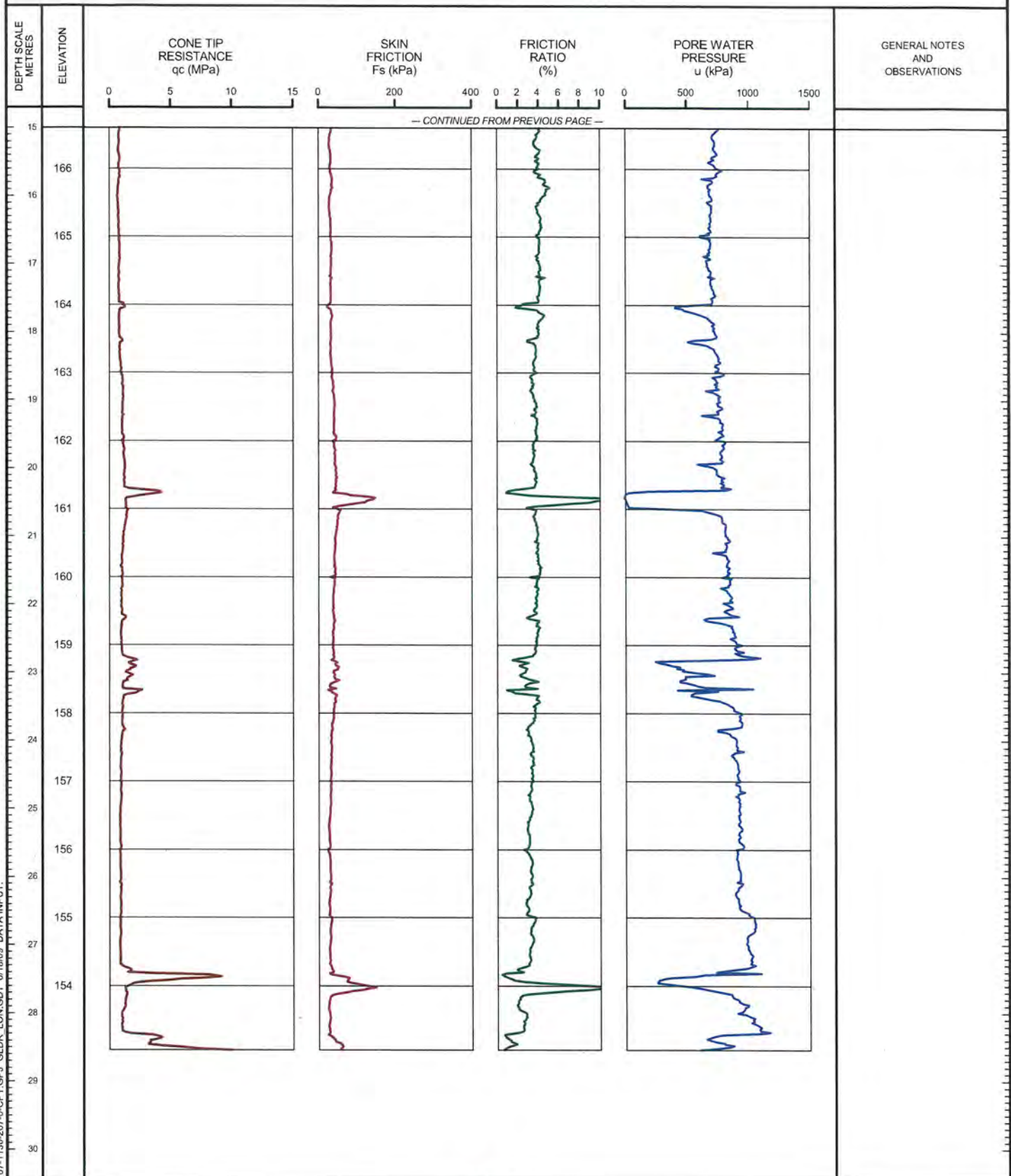
SHEET 2 OF 2

LOCATION: N 4679309.7 ; E 332536.3

TEST DATE: September 29, 2008

DATUM: GEODETIC

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



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

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: *SSB*

PROJECT <u>07-1130-207-0</u>		<b>RECORD OF BOREHOLE No CPT-124</b>		1 OF 1	<b>METRIC</b>
W.P. _____		LOCATION <u>N 4679354.6 :E 332455.0</u>		ORIGINATED BY <u>CC</u>	
DIST <u>WEST</u> HWY <u>401/3</u>		BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>LMK</u>	
DATUM <u>GEODETIC</u>		DATE <u>September 11, 2008</u>		CHECKED BY <u>SJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w <sub>p</sub> — w — w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
181.51	GROUND SURFACE										
0.00	TOPSOIL, silty, some sand, trace clay, trace organics, trace gravel		1	SS	14		181				
180.90	Compact Brown										
0.61	SAND, fine to medium, some silt		2	SS	4						
0.91	Loose Brown										
	CLAYEY SILT, trace sand, trace gravel		3	SS	19		180				
179.68	Firm to very stiff										
1.83	Mottled brown and grey										
	END OF BOREHOLE										
	Water level in borehole at about elev. 180.5m during drilling on September 11, 2008.										

LDN\_MTO\_01 07-1130-207-0.GPJ LDN\_MTO.GDT 6/29/09



PROJECT: 07-1130-207-0

**RECORD OF CONE PENETRATION TEST CPT-124**

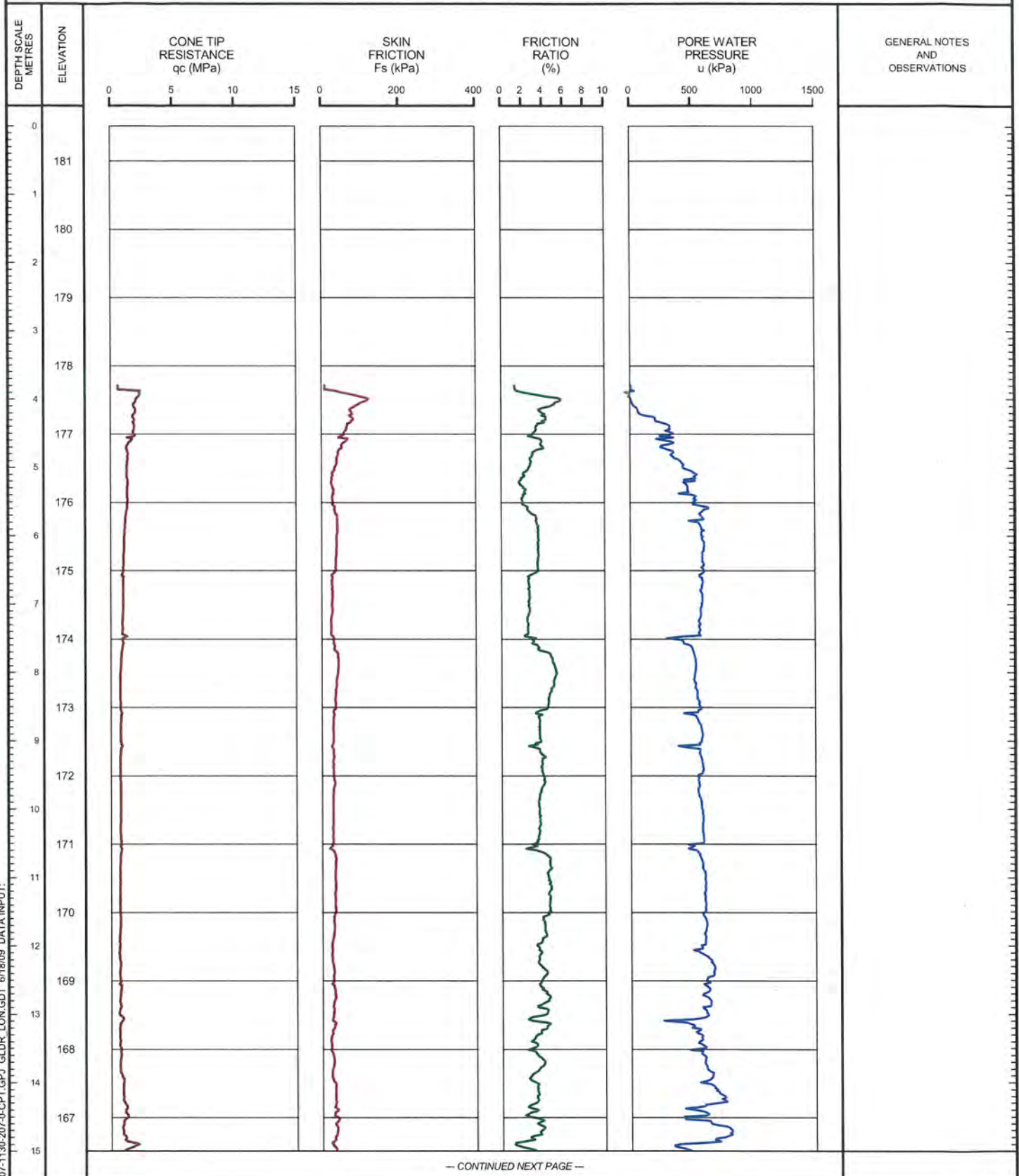
SHEET 1 OF 2

LOCATION: N 4679354.6 :E 332455.0

TEST DATE: September 29, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION:    PREDRILL DEPTH: 3.81m    CORRECTION FACTOR A: 0.584    CORRECTION FACTOR B: 0.012



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

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: *SB*

PROJECT: 07-1130-207-0

**RECORD OF CONE PENETRATION TEST CPT-124**

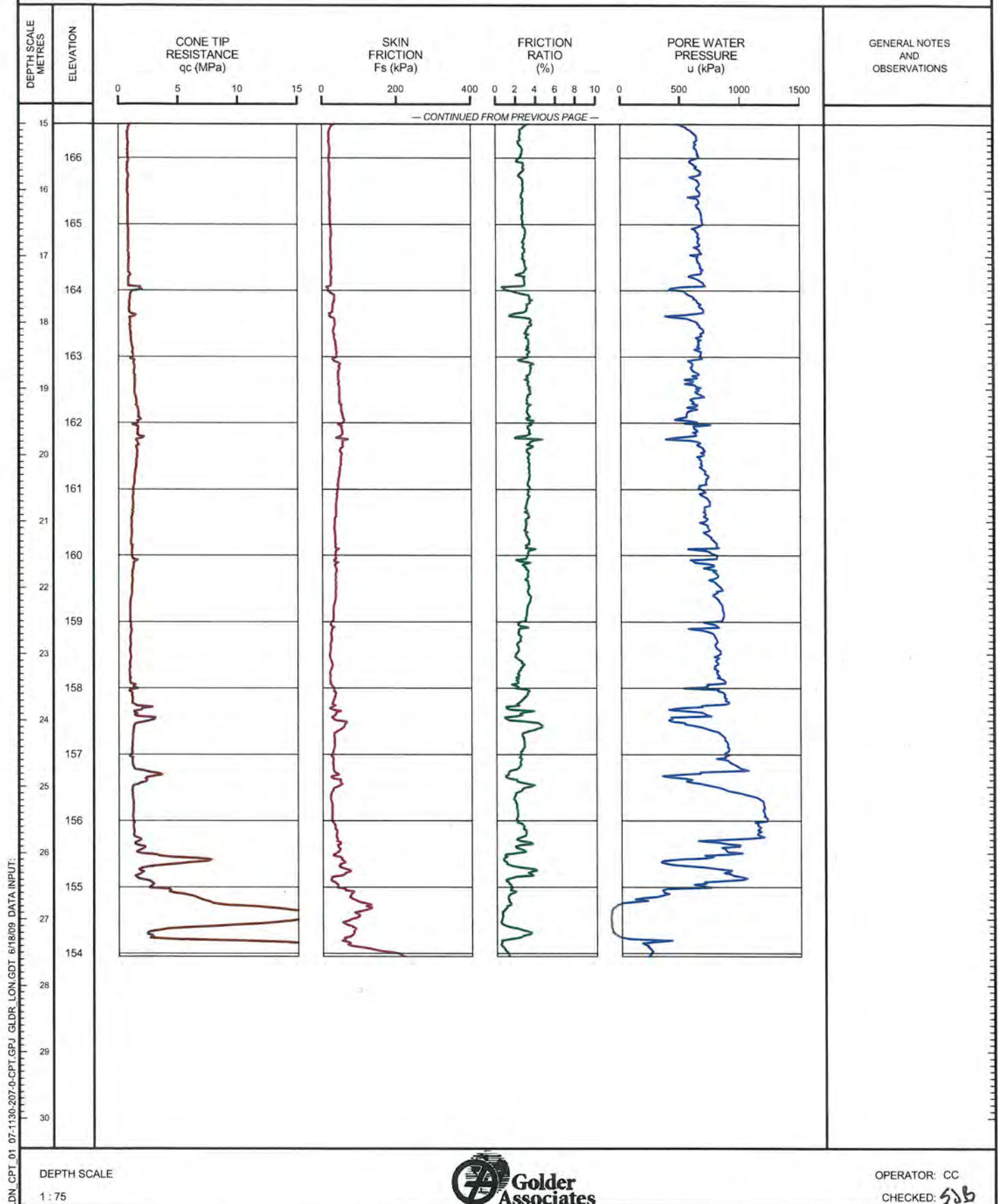
SHEET 2 OF 2

LOCATION: N 4679354.6 :E 332455.0

TEST DATE: September 29, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION:    PREDRILL DEPTH: 3.81m    CORRECTION FACTOR A: 0.584    CORRECTION FACTOR B: 0.012



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

PROJECT <u>07-1130-207-0</u>		<b>RECORD OF BOREHOLE No CPT-128</b>		1 OF 1	<b>METRIC</b>
W.P. _____		LOCATION <u>N 4679490 6 :E 332200.8</u>		ORIGINATED BY <u>CC</u>	
DIST <u>WEST</u> HWY <u>401/3</u>		BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>SJL</u>	
DATUM <u>GEODETIC</u>		DATE <u>September 5, 2008</u>		CHECKED BY <u>SJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
180.87	GROUND SURFACE												
0.00	FILL, silty sand, trace gravel, trace organics with clayey silt Compact Brown		1	SS	10								
180.26	CLAYEY SILT, trace sand, trace gravel Firm to very stiff Mottled brown and grey		2	SS	7								
0.61			3	SS	17								
179.04	END OF BOREHOLE												
1.83	Water level in borehole at about elev. 179.7m during drilling on September 5, 2008.												

LDN MTO\_01 07-1130-207-0.GPJ LDN MTO.GDT 8/29/09



PROJECT: 07-1130-207-0

**RECORD OF CONE PENETRATION TEST CPT-128**

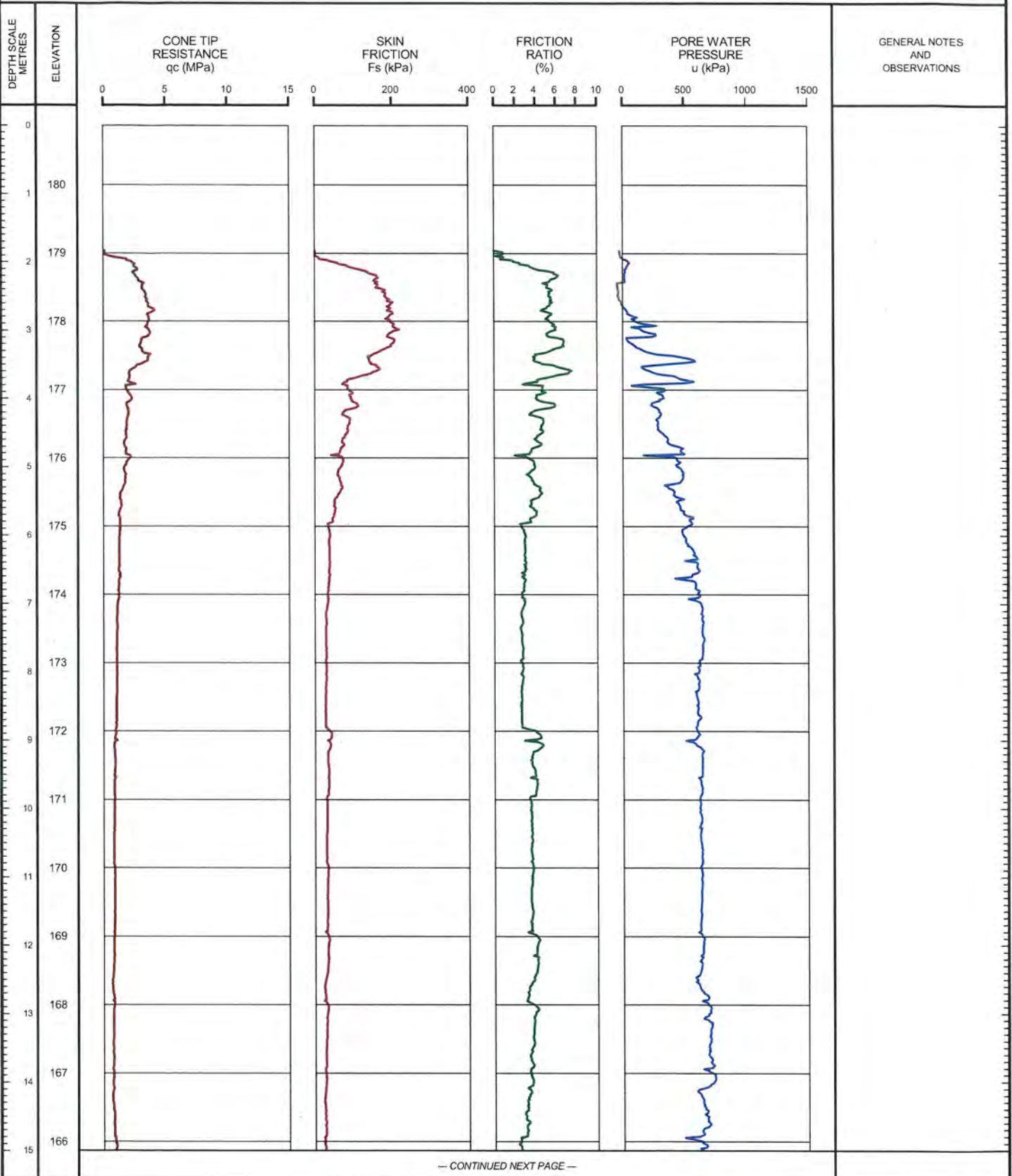
SHEET 1 OF 3

LOCATION: N 4679490.6 E 332200.8

TEST DATE: September 5, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION:    PREDRILL DEPTH: 1.83m    CORRECTION FACTOR A: 0.584    CORRECTION FACTOR B: 0.012



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

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: SJB

PROJECT: 07-1130-207-0

**RECORD OF CONE PENETRATION TEST CPT-128**

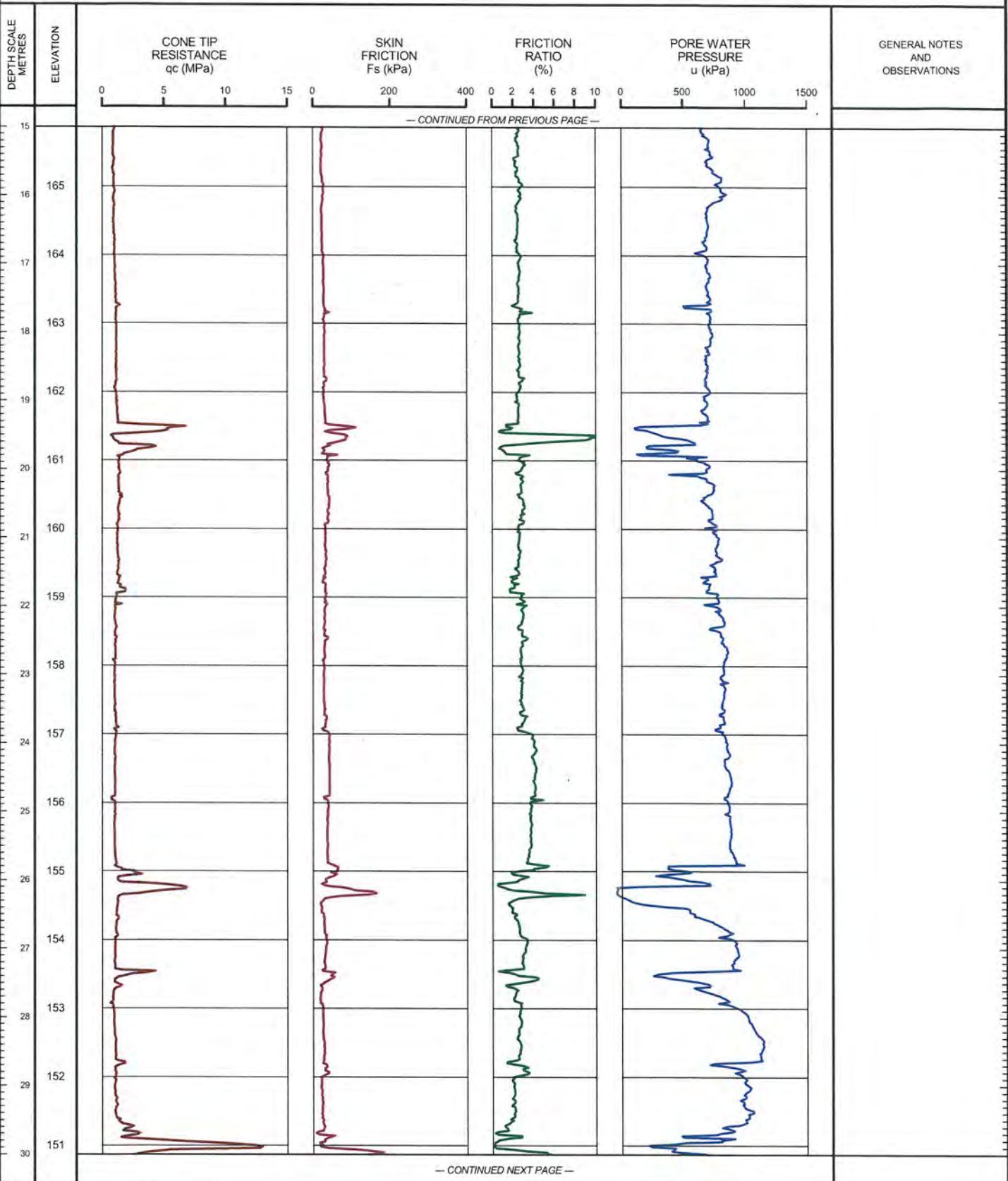
SHEET 2 OF 3

LOCATION: N 4679490.6 ; E 332200.8

TEST DATE: September 5, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION:    PREDRILL DEPTH: 1.83m    CORRECTION FACTOR A: 0.584    CORRECTION FACTOR B: 0.012



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

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: *SB*

PROJECT: 07-1130-207-0

## RECORD OF CONE PENETRATION TEST CPT-128

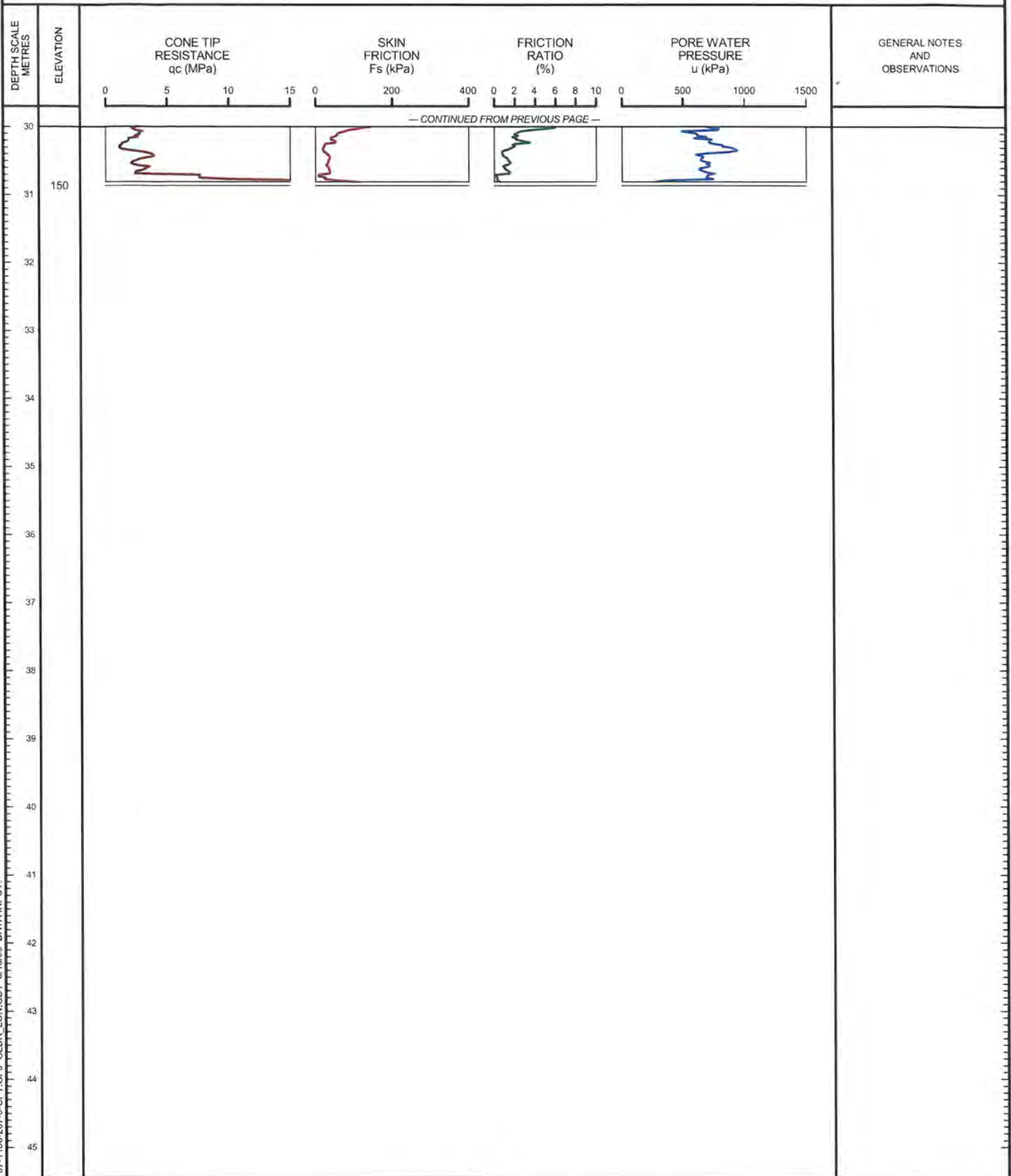
SHEET 3 OF 3

LOCATION: N 4679490.6 E 332200.8

TEST DATE: September 5, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 1.83m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LDN CPT\_01 07-1130-207-0-CPT.GPJ GLDR LON.GDT 6/18/09 DATA INPUT:

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: SSB

PROJECT <u>09-1132-0080</u>		<b>RECORD OF BOREHOLE No CPT-322</b>		1 OF 1		<b>METRIC</b>	
W.P. _____		LOCATION <u>N 4679294.0 ; E 332478.2</u>		ORIGINATED BY <u>TA</u>			
DIST <u>WEST</u> HWY <u>401 / 3</u>		BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>DMB</u>			
DATUM <u>GEODETIC</u>		DATE <u>January 7, 2010</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								20 40 60 80 100	20 40 60 80 100							
181.50	ROAD SURFACE															
0.05	ASPHALT PAVEMENT															
181.04	FILL, limestone gravel, crushed Grey															
0.46	TOPSOIL, clayey Very stiff Black		1	SS	17											
180.28	CLAYEY SILT, some sand, trace gravel, with occasional fissures, silt partings and seams Hard Brown becoming grey below about elev. 177.5m		2	SS	35											
1.22																
			3	SS	44											
			4	SS	37											
177.84	END OF BOREHOLE															
3.66	Borehole dry during drilling on January 7, 2010.															

LDN\_MTO\_06 09-1132-0080.GPJ LDN\_MTO.GDT 11/03/10



PROJECT: 09-1132-0080

# RECORD OF CONE PENETRATION TEST CPT-322

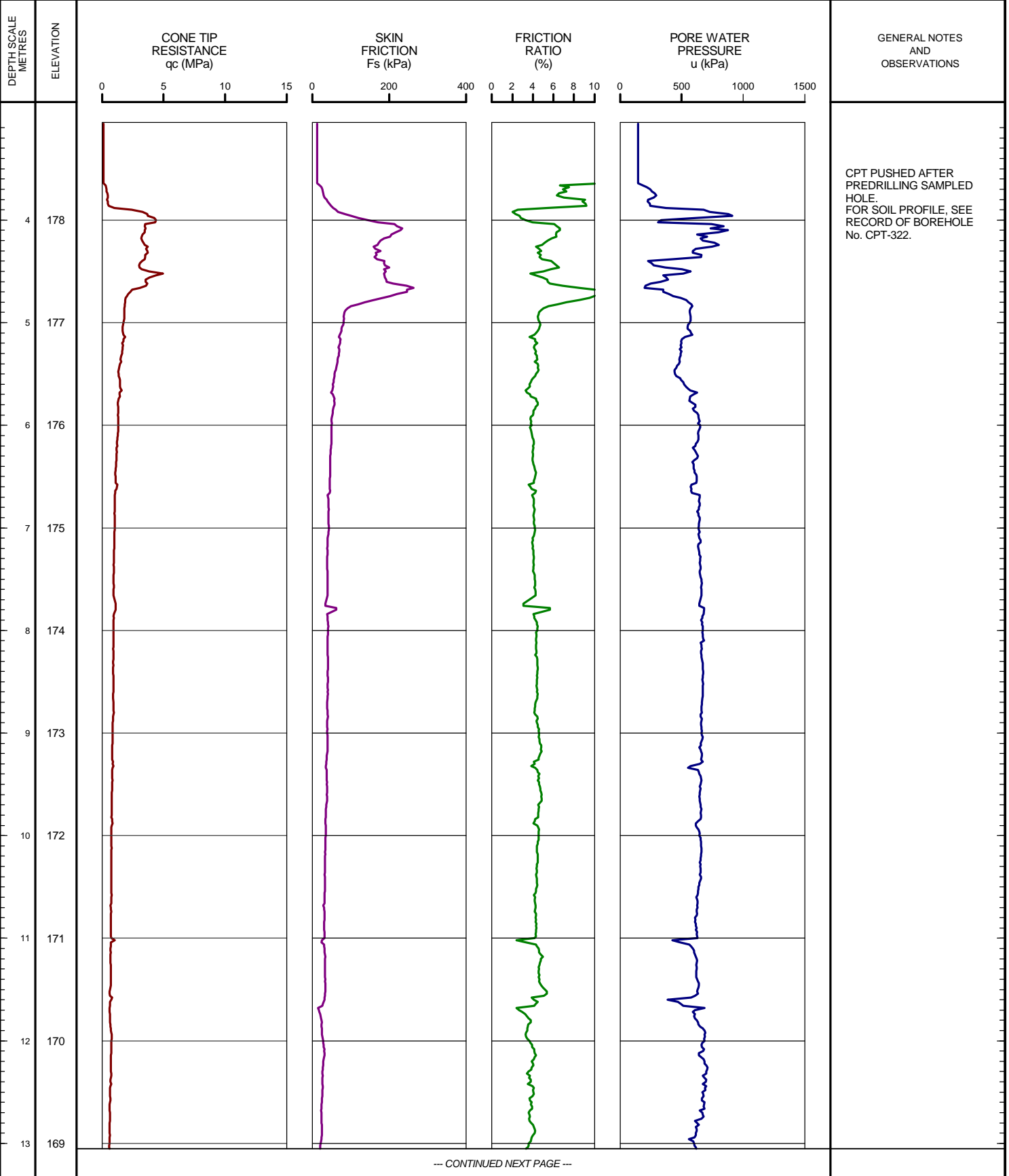
SHEET 1 OF 2

LOCATION: N 4679294.0 ;E 332478.2

TEST DATE: January 8, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.50m PREDRILL DEPTH: 3.05m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LDN\_CPT\_01 09-1132-0080-CPT.GPJ GLDR\_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

# RECORD OF CONE PENETRATION TEST CPT-322

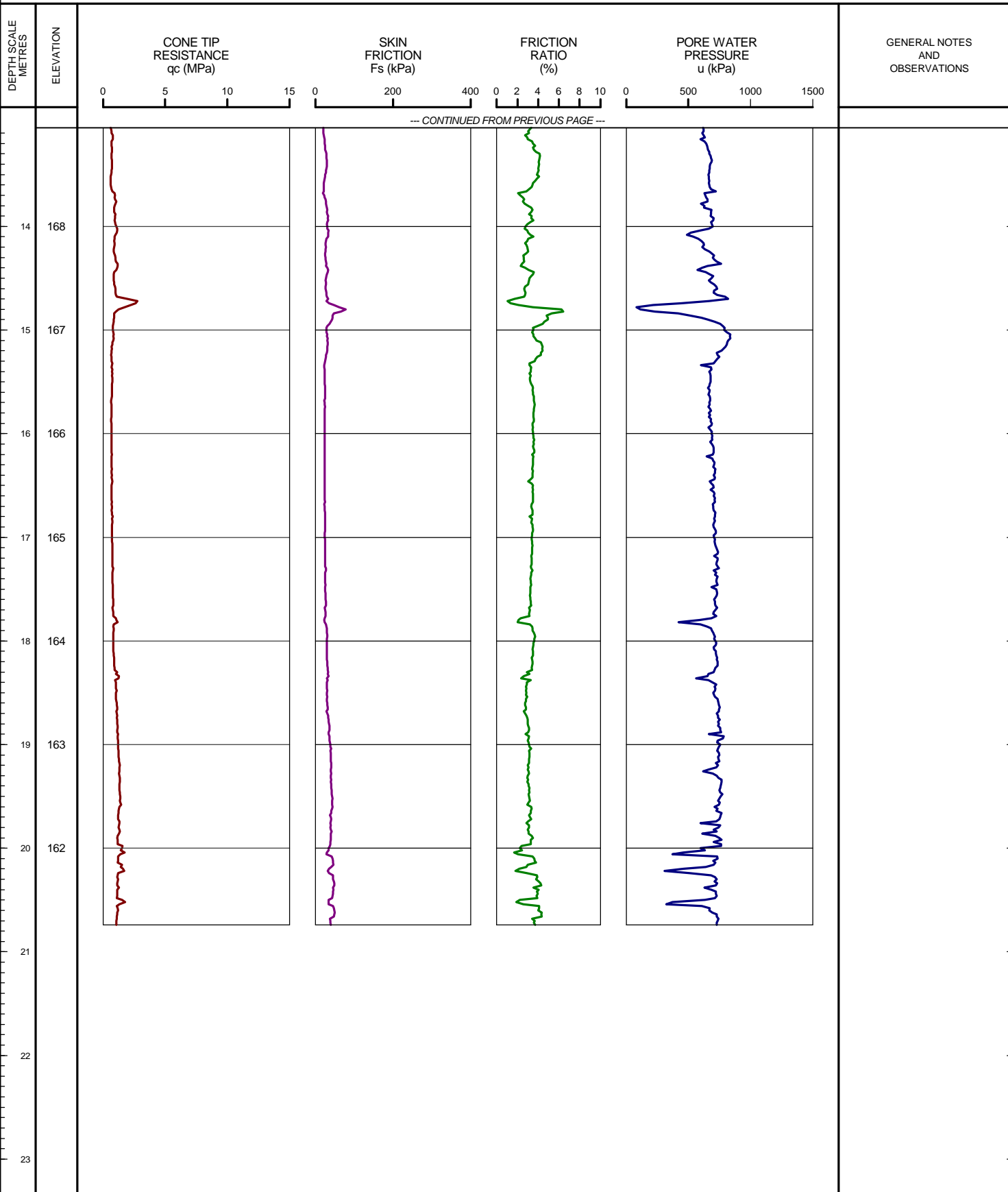
SHEET 2 OF 2

LOCATION: N 4679294.0 ;E 332478.2

TEST DATE: January 8, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.50m PREDRILL DEPTH: 3.05m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LON\_CPT\_01 09-1132-0080-CPT.GPJ GLDR\_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



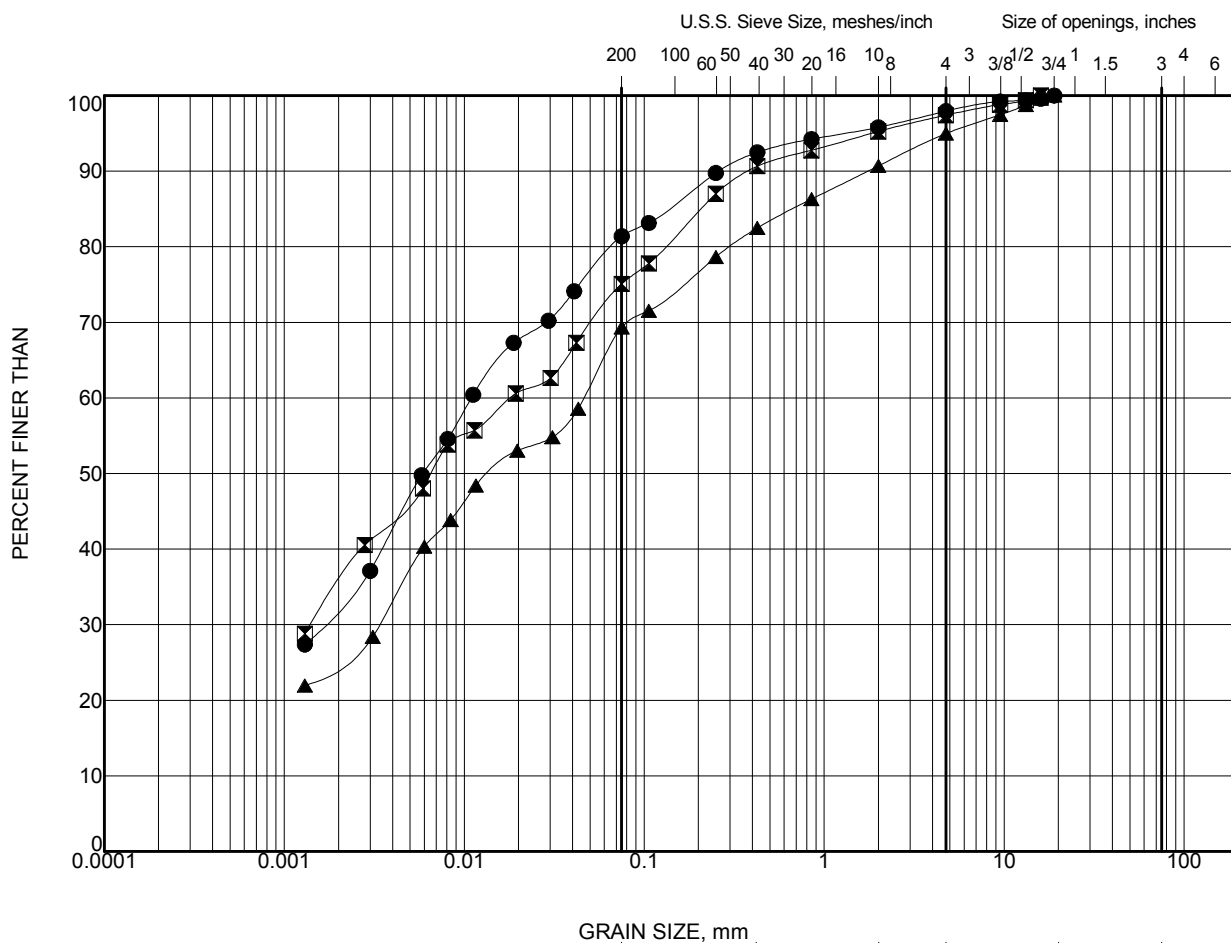
OPERATOR: TA

CHECKED:

## **Appendix C:    Laboratory Test Results**

Project: Windsor-Essex Parkway  
Document: Geotechnical Investigation and Design Report– Tunnel T-7  
(Hwy. 401 Sta. 10+450L to Sta. 10+700L, LaSalle) Geocres No. 40J6-37  
Doc No.: 285380-04-119-0028

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

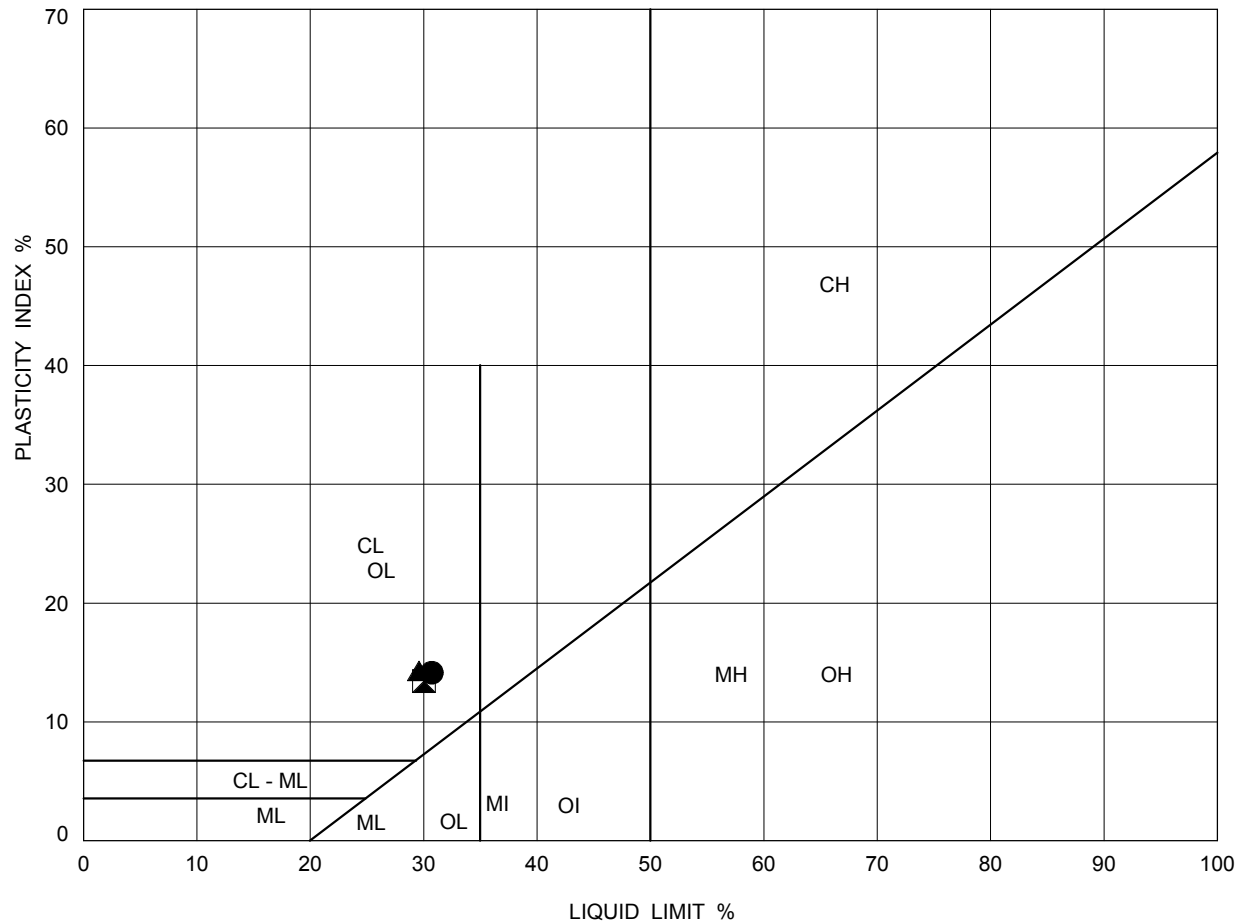


CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T7-1	8	6.1
⊠	T7-1	12	12.2
▲	T7-1	18	21.3

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		<b>GRAIN SIZE DISTRIBUTION CLAY AND SILT</b>	
	PROJECT No. SW8801.1004.101	FILE No.	
	DRAWN SS	SCALE	REV.
	CHECK MSO	Appendix C	



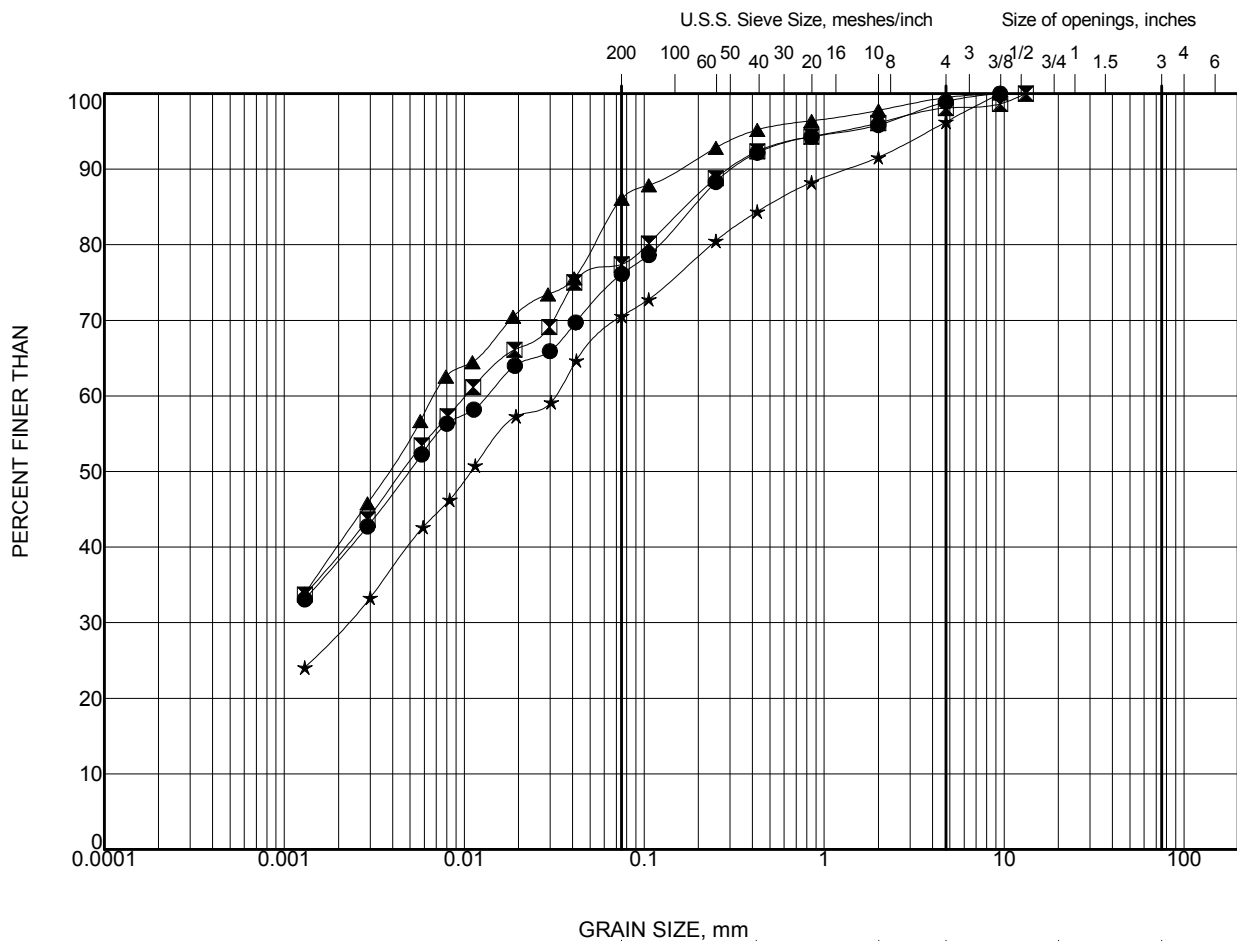
**SOIL TYPE**  
 C = Clay  
 M = Silt  
 O = Organic

**PLASTICITY**  
 L = Low  
 I = Intermediate  
 H = High

**LEGEND:**

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T7-1	8	6.1	31	17	14
⊠	T7-1	12	12.2	30	17	13
▲	T7-1	18	21.3	30	15	15

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART CLAY AND SILT	
PROJECT No. SW8801.1004.101		FILE No.	
DRAWN	SS	SCALE	REV.
CHECK	MSO	Appendix C	



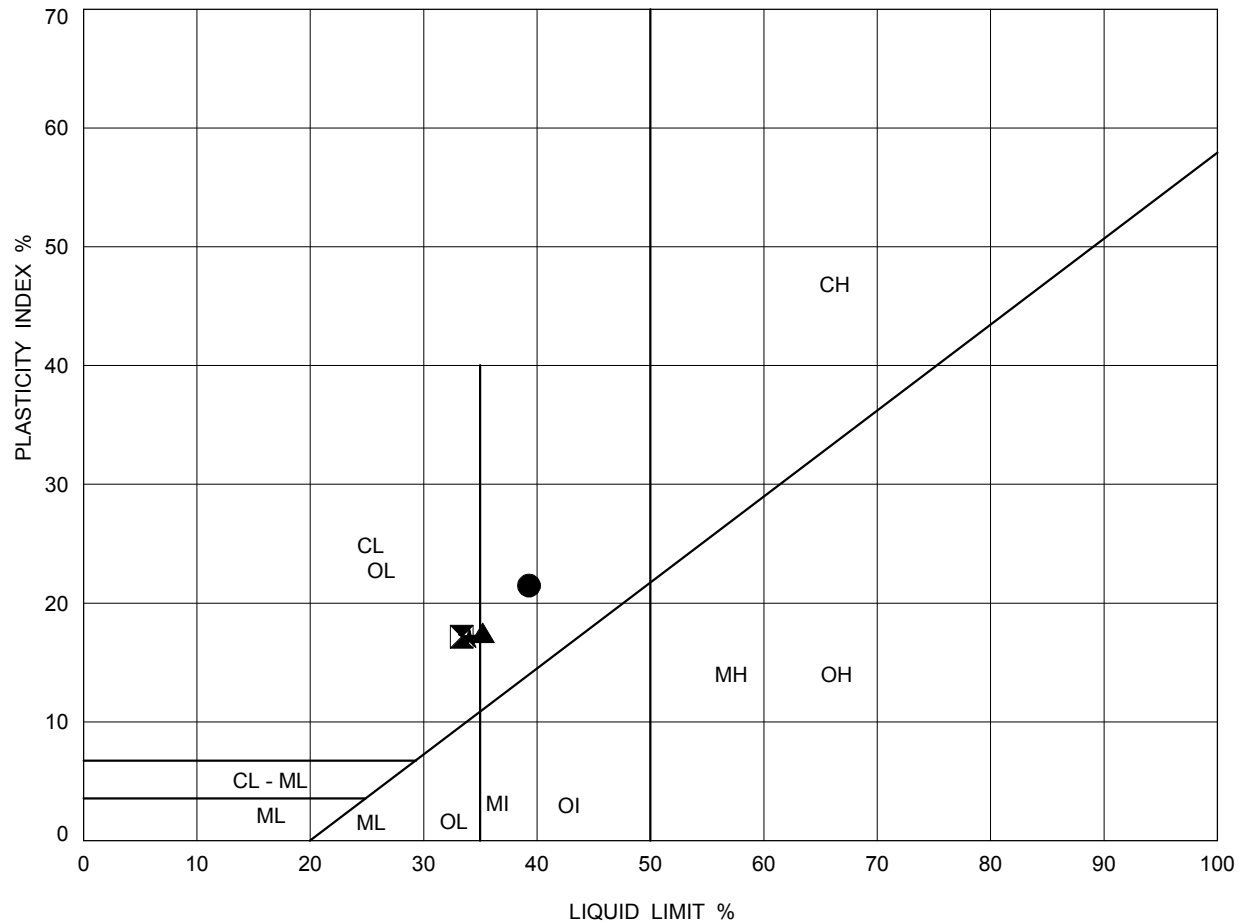
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T7-2	5	3.8
◻	T7-2	10	9.1
▲	T7-2	12	12.2
★	T7-2	18	21.3

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		<b>GRAIN SIZE DISTRIBUTION CLAY AND SILT</b>	
	PROJECT No. SW8801.1004.101	FILE No.	
	DRAWN SS	SCALE	REV.
	CHECK MSO	Appendix C	

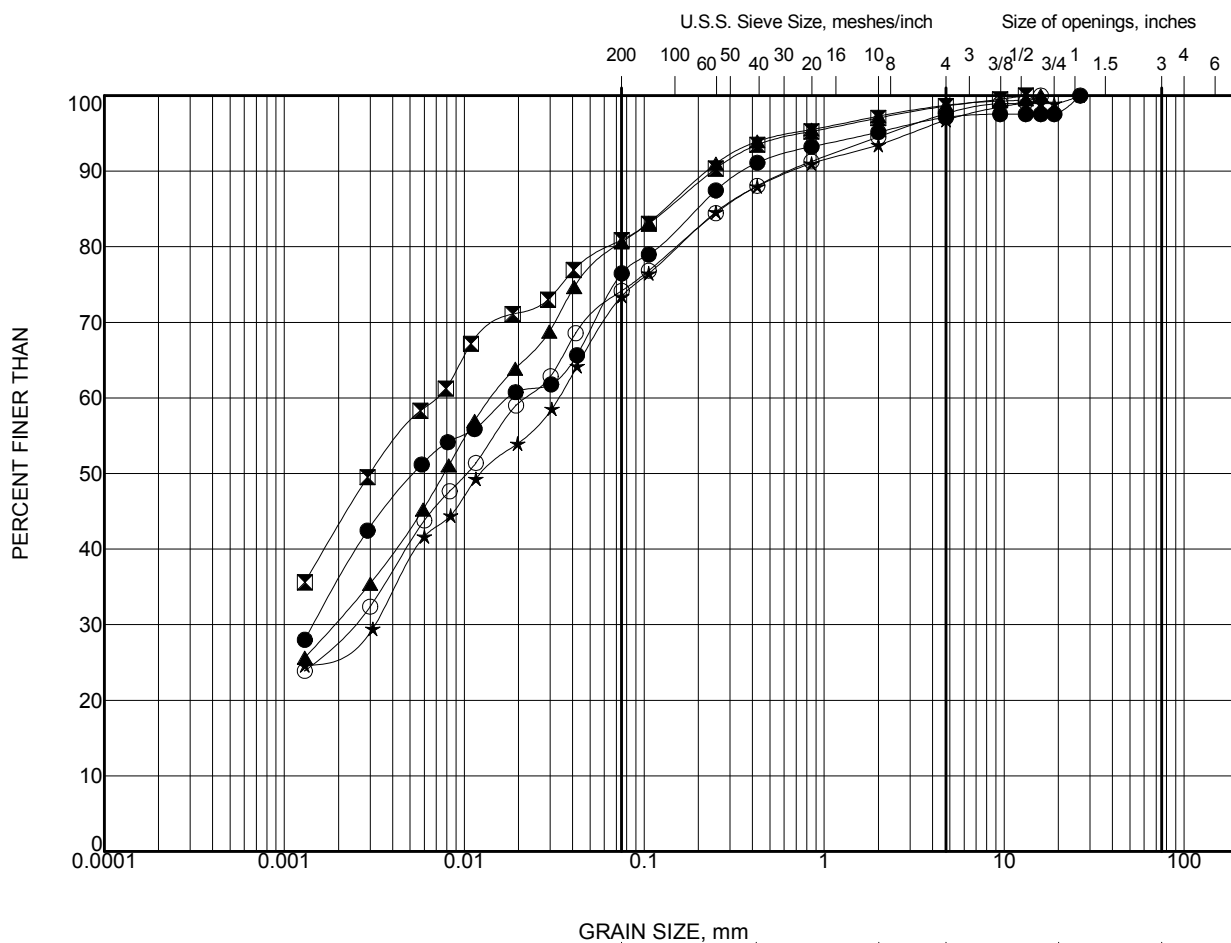




#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T7-2	4	3	39	18	21
⊠	T7-2	9	7.6	33	16	17
▲	T7-2	12	12.2	35	18	17
★	T7-2	19	22.9	34	17	17

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART CLAY AND SILT	
PROJECT No. SW8801.1004.101		FILE No.	
DRAWN	SS	SCALE	REV.
CHECK	MSO	Appendix C	

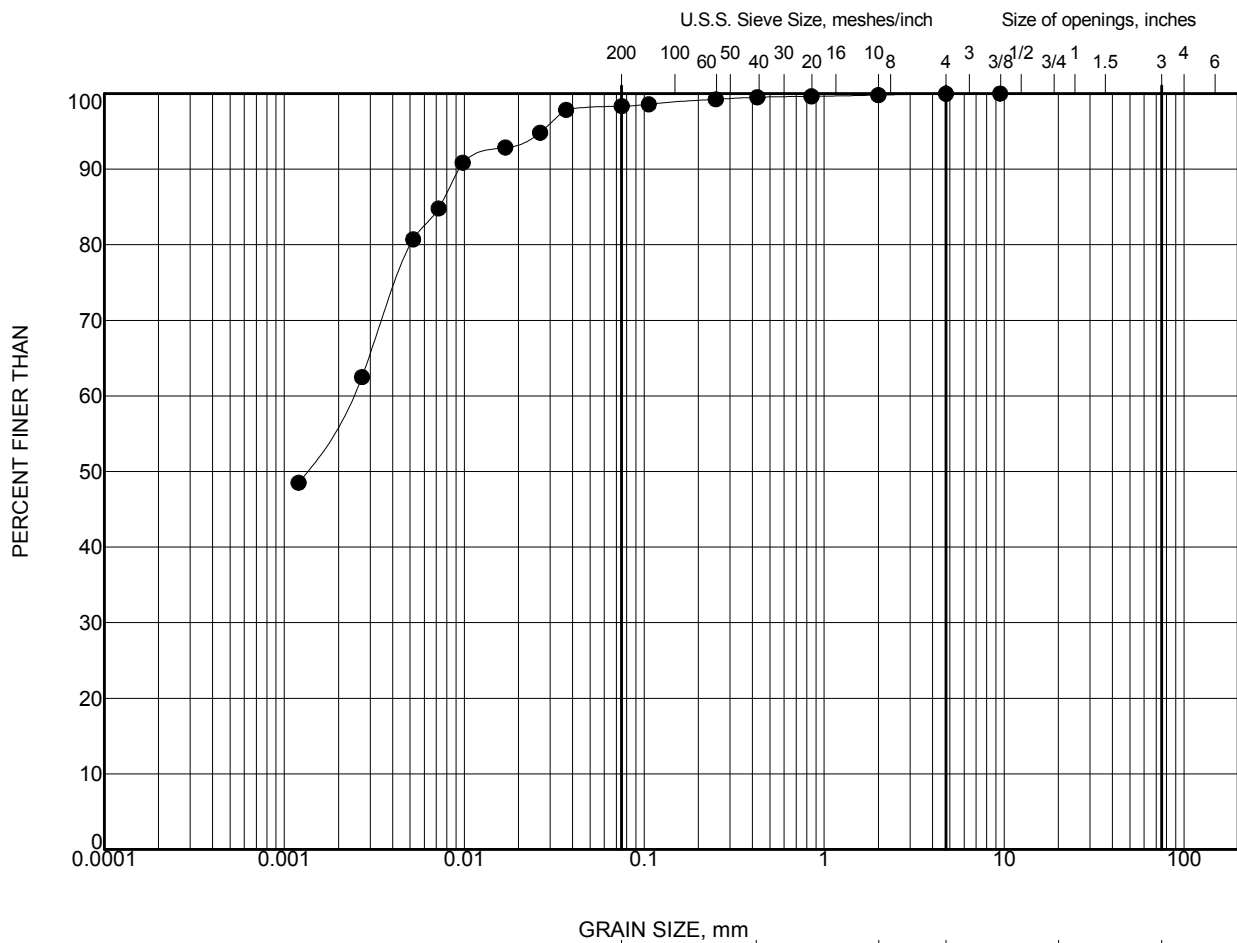


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T7-3	8	6.1
⊠	T7-3	10	9.1
▲	T7-3	13	13.7
★	T7-3	17	19.8
○	T7-3	19	22.9

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		<b>GRAIN SIZE DISTRIBUTION CLAY AND SILT</b>	
	PROJECT No.	SW8801.1004.101	FILE No.
	DRAWN	SS	SCALE
	CHECK	MSO	REV.
<b>Appendix C</b>			

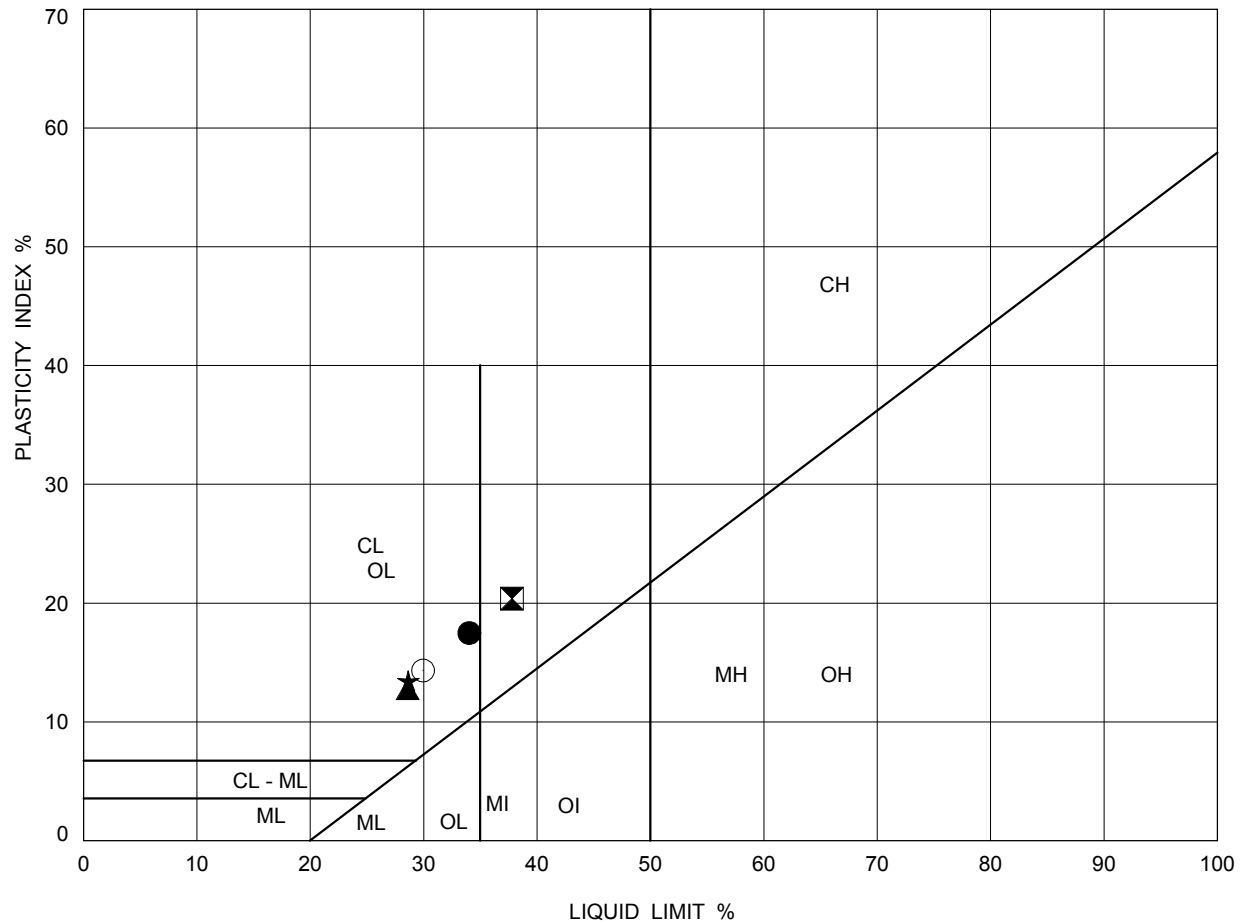


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T7-3	22	27.4

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		<b>GRAIN SIZE DISTRIBUTION CLAY AND SILT</b>	
	PROJECT No. SW8801.1004.101	FILE No.	
	DRAWN SS	SCALE	REV.
	CHECK MSO		
<b>Appendix C</b>			



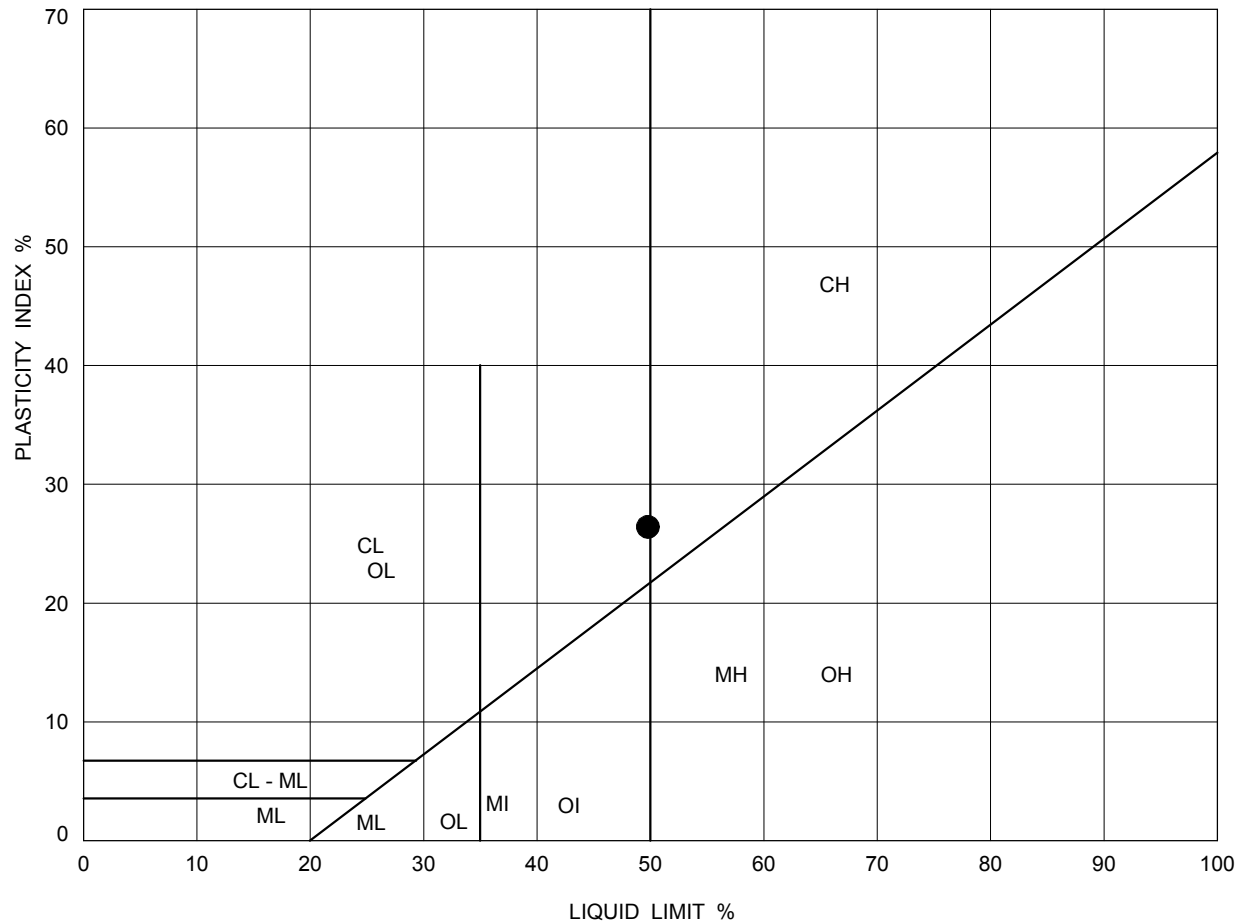
**SOIL TYPE**  
 C = Clay  
 M = Silt  
 O = Organic

**PLASTICITY**  
 L = Low  
 I = Intermediate  
 H = High

**LEGEND:**



SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T7-3	8	6.1	34	17	17
⊠	T7-3	10	9.1	38	17	21
▲	T7-3	13	13.7	29	16	13
★	T7-3	17	19.8	29	15	14
○	T7-3	19	22.9	30	16	14

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART CLAY AND SILT	
PROJECT No. SW8801.1004.101		FILE No.	
DRAWN	SS	SCALE	REV.
CHECK	MSO	Appendix C	



# **LEGEND:**

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T7-3	22	27.4	50	23	27

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART CLAY AND SILT	
 		PROJECT No. SW8801.1004.101	FILE No.
DRAWN	SS	SCALE	REV.
CHECK	MSO	Appendix C	

# ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP**  
 Client: **Hatch Mott MacDonald Limited**  
 Date: **23-Dec-11**

Job No.: **SW8801.1004.101**  
 Sample ID: **T7-1\_TW11**

Depth(m):

## Test Data

Ring # :	B	Ring Height (in) =	0.752	Wt of dry filter paper (g)	0.77
Wet soil + Ring Wt (g)			201.84	Wt of ring (g)	76.53
Wet soil + Wet Paper + Ring (g)			200.93	Wet Paper (g)	2.00
Dry Soil + Dry Paper + Ring (g)			179.68	Ring Dia (in)	2.498
Initial moisture Content (%)			22.40	Final moisture Content (%)	19.55
Area of Ring (in <sup>2</sup> )			4.90	Initial Volume (in <sup>3</sup> )	3.6855
Initial Bulk Density (kg/m <sup>3</sup> )			2075	Initial Dry Density (kg/m <sup>3</sup> )	1695
Specific Gravity of Soil			2.75	Equiv. Thick. of solids (mm)	11.770
Final Bulk Density (kg/m <sup>3</sup> )			2139	Final Dry Density (kg/m <sup>3</sup> )	1789
Initial gauge reading for Load 1			0.2520	Gauge reading for last Loading	0.2126
Initial Voids Ratio			0.623	Final Void Ratio	0.538
Initial Degree of Saturation (%)			99	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	6.0	9.0	13.0	20.0	30.0	45.0	65.0
Load (tsf)	0.0624	0.0936	0.135	0.208	0.312	0.468	0.676
Gauge Reading (in)	0.2520	0.2515	0.2512	0.2500	0.2481	0.24573	0.2430
(H-Hs) mm	7.331	7.320	7.311	7.281	7.232	7.173	7.102
Voids ratio	0.623	0.622	0.621	0.619	0.614	0.609	0.603
t <sub>90</sub> (min)			2.25	3.06	3.24	3.61	3.61
C <sub>v</sub> (m <sup>2</sup> /day)			0.049	0.036	0.034	0.030	0.030
k' (MPa)			8.842	4.382	3.886	4.837	5.385
M <sub>v</sub> (mm <sup>2</sup> / N)			0.1131	0.2282	0.2573	0.2067	0.1857

Trial #	8	9	10	11	12	13	14
Load (kPa)	100	150.0	100.0	65.0	45.0	65.0	100.0
Load (tsf)	1.04	1.560	1.040	0.676	0.468	0.676	1.040
Gauge Reading (in)	0.23865	0.2336	0.2344	0.2358	0.2372	0.2366	0.2351
(H-Hs) mm	6.993	6.865	6.886	6.920	6.956	6.939	6.901
Voids ratio	0.594	0.583	0.585	0.588	0.591	0.590	0.586
t <sub>90</sub> (min)	4.00	3.24					
C <sub>v</sub> (m <sup>2</sup> /day)	0.027	0.033					
k' (MPa)	6.034	7.314					
M <sub>v</sub> (mm <sup>2</sup> / N)	0.1657	0.1367					

Trial #	15	16	17	18	19	20	21
Load (kPa)	150.0	225.0	335.0	505.0	760.0	1140.0	1710.0
Load (tsf)	1.56	2.340	3.484	5.252	7.904	11.856	17.784
Gauge Reading (in)	0.2324	0.2268	0.2183	0.2069	0.1933	0.1789	0.1637
(H-Hs) mm	6.834	6.692	6.476	6.185	5.840	5.476	5.089
Voids ratio	0.581	0.569	0.550	0.525	0.496	0.465	0.432
t <sub>90</sub> (min)		3.24	4.00	4.00	7.02	5.76	3.61
C <sub>v</sub> (m <sup>2</sup> /day)		0.032	0.026	0.025	0.014	0.016	0.025
k' (MPa)		9.810	9.406	10.665	13.254	18.385	25.428
M <sub>v</sub> (mm <sup>2</sup> / N)		0.1019	0.1063	0.0938	0.0754	0.0544	0.0393

# ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

**Project:** WEP **Job No.:** SW8801.1004.101  
**Client:** Hatch Mott MacDonald Limited  
**Date:** 23-Dec-11 **Sample ID:** T7-1\_TW11 **Depth(m):**

Trial #	22	23	24	25	26	27	28
Load (kPa)	855.0	425.0	215	105.0	55.0	25.0	13.5
Load (tsf)	8.892	4.420	2.236	1.092	0.572	0.260	0.140
Gauge Reading (in)	0.16621	0.1697	0.17481	0.1815	0.1885	0.1968	0.2049
(H-Hs) mm	5.153	5.242	5.371	5.540	5.719	5.930	6.134
Voids ratio	0.438	0.445	0.456	0.471	0.486	0.504	0.521
t <sub>90</sub> (min)							
C <sub>v</sub> (m <sup>2</sup> /day)							
k' (MPa)							
M <sub>v</sub> (mm <sup>2</sup> / N)							

Trial #	29
Load (kPa)	6.5
Load (tsf)	0.068
Gauge Reading (in)	0.2126
(H-Hs) mm	6.331
Voids ratio	0.538
t <sub>90</sub> (min)	
C <sub>v</sub> (m <sup>2</sup> /day)	
k' (MPa)	
M <sub>v</sub> (mm <sup>2</sup> / N)	



# ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP**  
 Client: **Hatch Mott MacDonald Limited**  
 Date: **23-Dec-11**

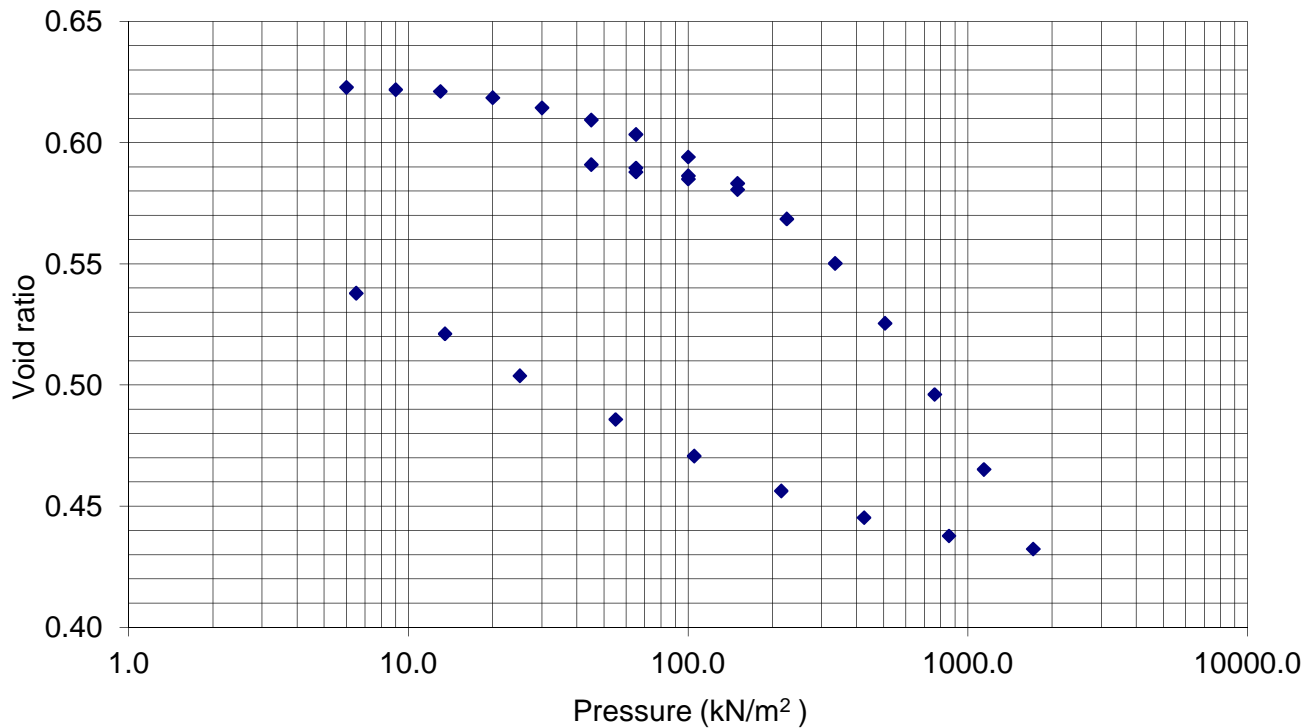
Sample ID: **T7-1\_TW11**

Job No.: **SW8801.1004.101**

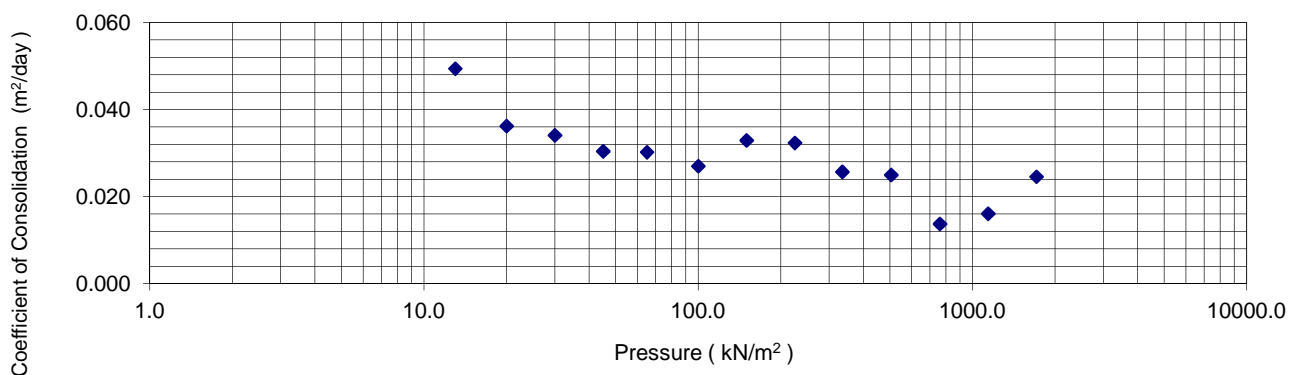
Depth(m):

$\sigma'_v$  versus  $e$  and  $c_v$

## Void Ratio Vs Pressure



## Coefficient of Consolidation Vs Pressure



# ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

**Project:** WEP **Job No.:** SW8801.1004.101  
**Client:** Hatch Mott MacDonald Limited  
**Date:** 23-Dec-11 **Sample ID:** T7-1\_TW11 **Depth(m):**

## Strain Energy Data

Presssure (kN/m <sup>2</sup> )	c <sub>v</sub> (m <sup>2</sup> /day)	Void ratio
6.0		0.623
9.0		0.622
13.0	0.049	0.621
20.0	0.036	0.619
30.0	0.034	0.614
45.0	0.030	0.609
65.0	0.030	0.603
100.0	0.027	0.594
150.0	0.033	0.583
100.0		0.585
65.0		0.588
45.0		0.591
65.0		0.590
100.0		0.586
150.0		0.581
225.0	0.032	0.569
335.0	0.026	0.550
505.0	0.025	0.525
760.0	0.014	0.496
1140.0	0.016	0.465
1710.0	0.025	0.432
855.0		0.438
425.0		0.445
215.0		0.456
105.0		0.471
55.0		0.486
25.0		0.504
13.5		0.521
6.5		0.538

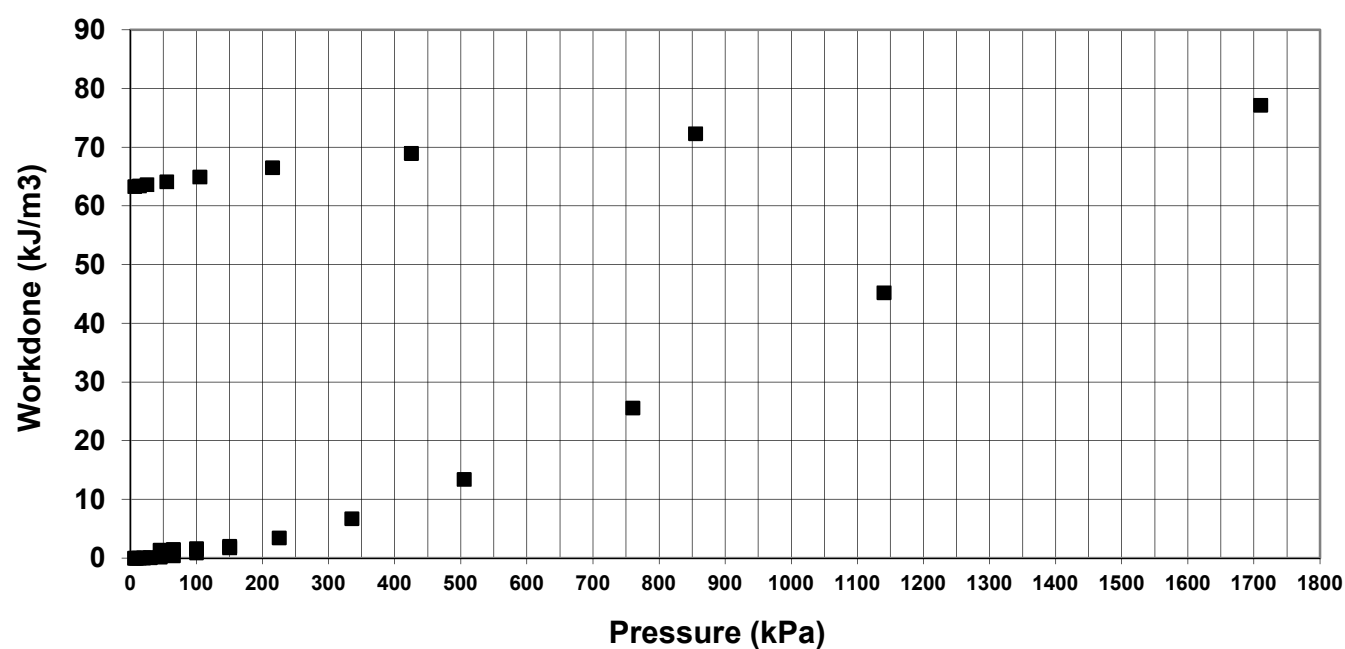
Presssure (KN/m <sup>2</sup> )	Height mm	Total Work (KJ/m <sup>3</sup> )
6.0	19.101	0.000
9.0	19.089	0.004
13.0	19.081	0.009
20.0	19.050	0.036
30.0	19.001	0.100
45.0	18.942	0.216
65.0	18.872	0.421
100.0	18.762	0.899
150.0	18.634	1.754
100.0	18.655	1.612
65.0	18.690	1.460
45.0	18.725	1.355
65.0	18.709	1.403
100.0	18.671	1.571
150.0	18.604	2.021
225.0	18.461	3.455
335.0	18.246	6.730
505.0	17.955	13.424
760.0	17.609	25.593
1140.0	17.245	45.230
1710.0	16.859	77.174
855.0	16.922	72.324
425.0	17.011	68.962
215.0	17.141	66.525
105.0	17.310	64.946
55.0	17.488	64.123
25.0	17.700	63.638
13.5	17.904	63.417
6.5	18.101	63.307

# ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP**  
 Client: **Hatch Mott MacDonald Limited**  
 Date: **23-Dec-11** Sample ID: **T7-1\_TW11**

Job No.: **SW8801.1004.101**  
 Depth(m):

## Strain Energy Method for Preconsolidation Pressure



## **Appendix D: Analytical Laboratory Results**



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

Date Received: 13-JUL-11  
Report Date: 19-JUL-11 13:52 (MT)  
Version: FINAL

Client Phone: 519-735-2499

## Certificate of Analysis

<b>Lab Work Order #:</b>	<b>L1030740</b>
Project P.O. #:	NOT SUBMITTED
Job Reference:	SW8801.1004.101
Legal Site Desc:	
C of C Numbers:	092959

Gayle Braun  
Senior Account Manager

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

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

# ALS ENVIRONMENTAL ANALYTICAL REPORT

		Sample ID Description Sampled Date Sampled Time Client ID	L1030740-1 SOIL 07-JUL-11 T7-1 SA#9@27'	L1030740-2 SOIL 09-JUL-11 T7-3 SA#12@42'	L1030740-3 SOIL 06-JUL-11 T7-2 SA#10@30'		
Grouping	Analyte						
<b>SOIL</b>							
<b>Physical Tests</b>	% Moisture (%)		15.8	18.2	18.5		
	pH (pH units)		7.98	7.94	8.03		
	Redox Potential (mV)		130	120	131		
	Resistivity (ohm cm)		2960	2500	2440		
<b>Leachable Anions &amp; Nutrients</b>	Sulphide (mg/kg)		<0.20	<0.20	<0.20		
<b>Anions and Nutrients</b>	Sulphate (mg/kg)		200	307	360		

## Reference Information

### Test Method References:

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

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

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

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

### Chain of Custody Numbers:

092959

### GLOSSARY OF REPORT TERMS

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

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

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

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

*mg/L - milligrams per litre.*

*< - Less than.*

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

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

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

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

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



## Quality Control Report

Workorder: L1030740

Report Date: 19-JUL-11

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Client: AMEC EARTH & ENVIRONMENTAL

11865 County Road 42

TECUMSEH ON N8N 2M1

Contact: SHANE MACLEOD

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
<b>MOISTURE-WT</b>	<b>Soil</b>							
Batch	R2218263							
<b>WG1311478-2</b>	<b>LCS</b>							
% Moisture			106		%		70-130	13-JUL-11
<b>WG1311478-1</b>	<b>MB</b>							
% Moisture			<0.10		%		0.1	13-JUL-11
<b>PH-WT</b>	<b>Soil</b>							
Batch	R2220797							
<b>WG1315023-1</b>	<b>CVS</b>							
pH			99		%		80-120	19-JUL-11
<b>RESISTIVITY-WT</b>	<b>Soil</b>							
Batch	R2220855							
<b>WG1315028-1</b>	<b>CVS</b>							
Resistivity			99		%		70-130	19-JUL-11
<b>SO4-WT</b>	<b>Soil</b>							
Batch	R2219765							
<b>WG1312668-3</b>	<b>LCS</b>							
Sulphate			103		%		60-140	15-JUL-11
<b>WG1312668-1</b>	<b>MB</b>							
Sulphate			<20		mg/kg		20	15-JUL-11
<b>SULPHIDE-WT</b>	<b>Soil</b>							
Batch	R2218729							
<b>WG1312664-1</b>	<b>CVS</b>							
Sulphide			106		%		50-120	14-JUL-11
<b>WG1312662-1</b>	<b>MB</b>							
Sulphide			<0.20		mg/kg		0.2	14-JUL-11
Batch	R2219387							
<b>WG1313378-1</b>	<b>CVS</b>							
Sulphide			106		%		50-120	15-JUL-11
<b>WG1313370-2</b>	<b>DUP</b>	<b>L1030740-3</b>						
Sulphide		<0.20	<0.20	RPD-NA	mg/kg	N/A	20	15-JUL-11
<b>WG1313370-1</b>	<b>MB</b>							
Sulphide			<0.20		mg/kg		0.2	15-JUL-11

# Quality Control Report

Workorder: L1030740

Report Date: 19-JUL-11

Page 2 of 3

## Legend:

---

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

## Sample Parameter Qualifier Definitions:

---

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

---

# Quality Control Report

Workorder: L1030740

Report Date: 19-JUL-11

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## Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
<b>Physical Tests</b>							
Redox Potential	1	07-JUL-11	19-JUL-11 14:14	24	290	hours	EHTR
	2	09-JUL-11	19-JUL-11 14:15	24	242	hours	EHTR
	3	06-JUL-11	19-JUL-11 14:16	24	314	hours	EHTR
Resistivity	1	07-JUL-11	19-JUL-11 14:34	7	12	days	EHT
	2	09-JUL-11	19-JUL-11 14:35	7	10	days	EHT
	3	06-JUL-11	19-JUL-11 14:36	7	13	days	EHTL
<b>Leachable Anions &amp; Nutrients</b>							
Sulphide	3	06-JUL-11	15-JUL-11 15:25	7	9	days	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 L1030740 were received on 13-JUL-11 10:30.

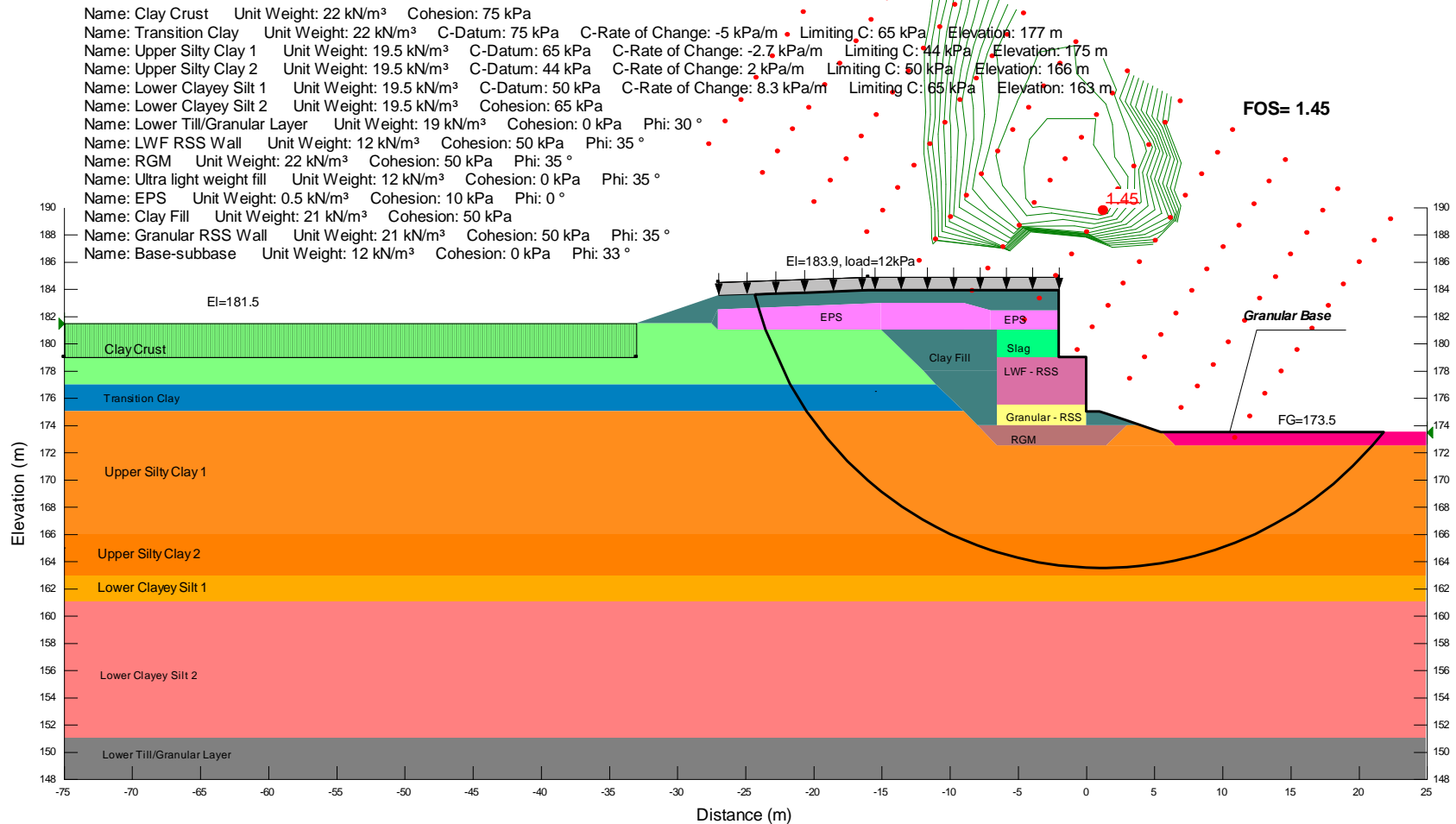
ALS recommended hold times may vary by province. They are assigned to meet known provincial and/or federal government requirements. In the absence of regulatory hold times, ALS establishes recommendations based on guidelines published by the US EPA, APHA Standard Methods, or Environment Canada (where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

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

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

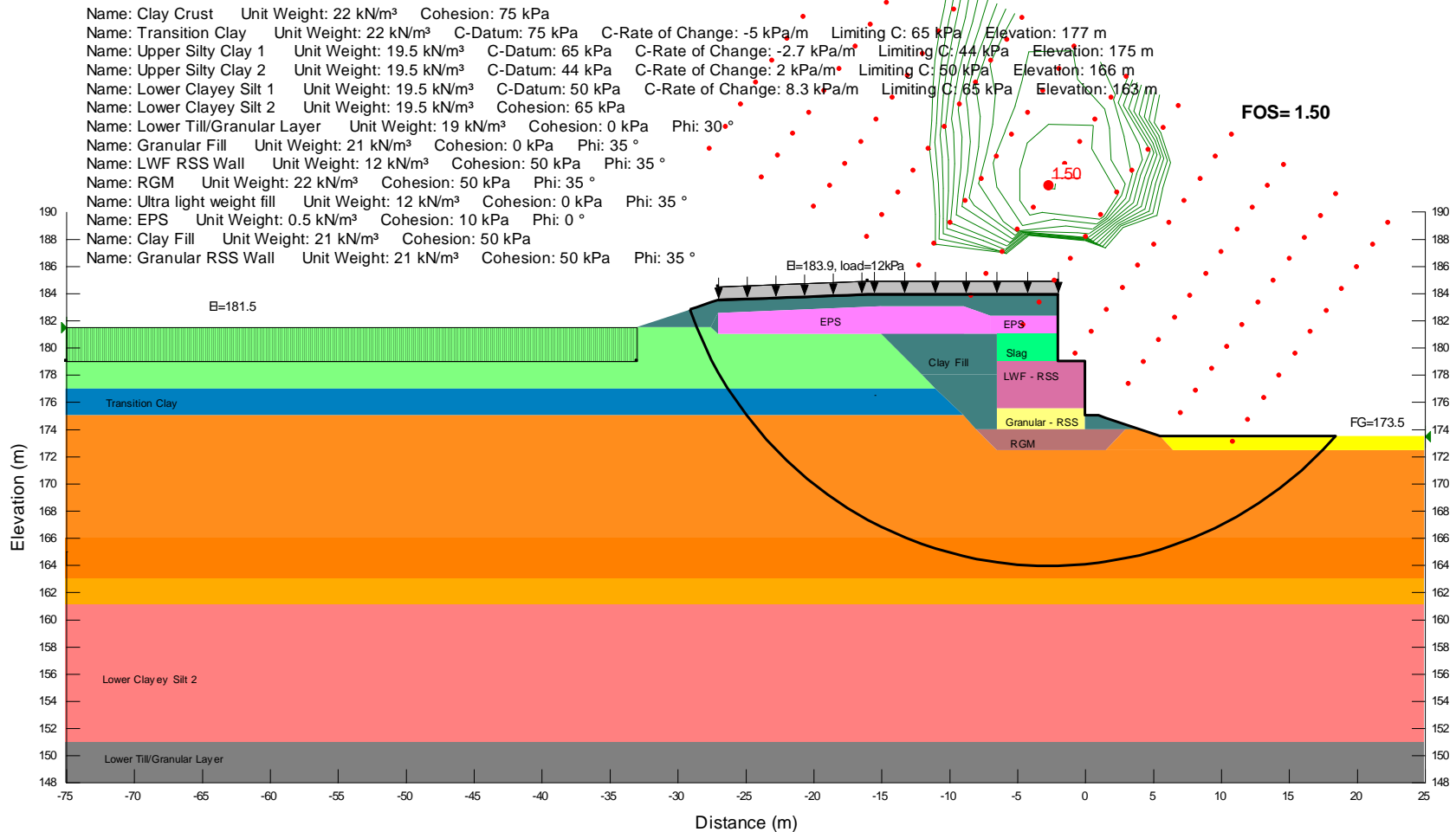
## **Appendix E: Slope Stability Analyses**

File Name: T7N\_Sta10+450L\_Drainage Blanket.gsz  
Analysis Name: Short-term  
Method: Morgenstern-Price



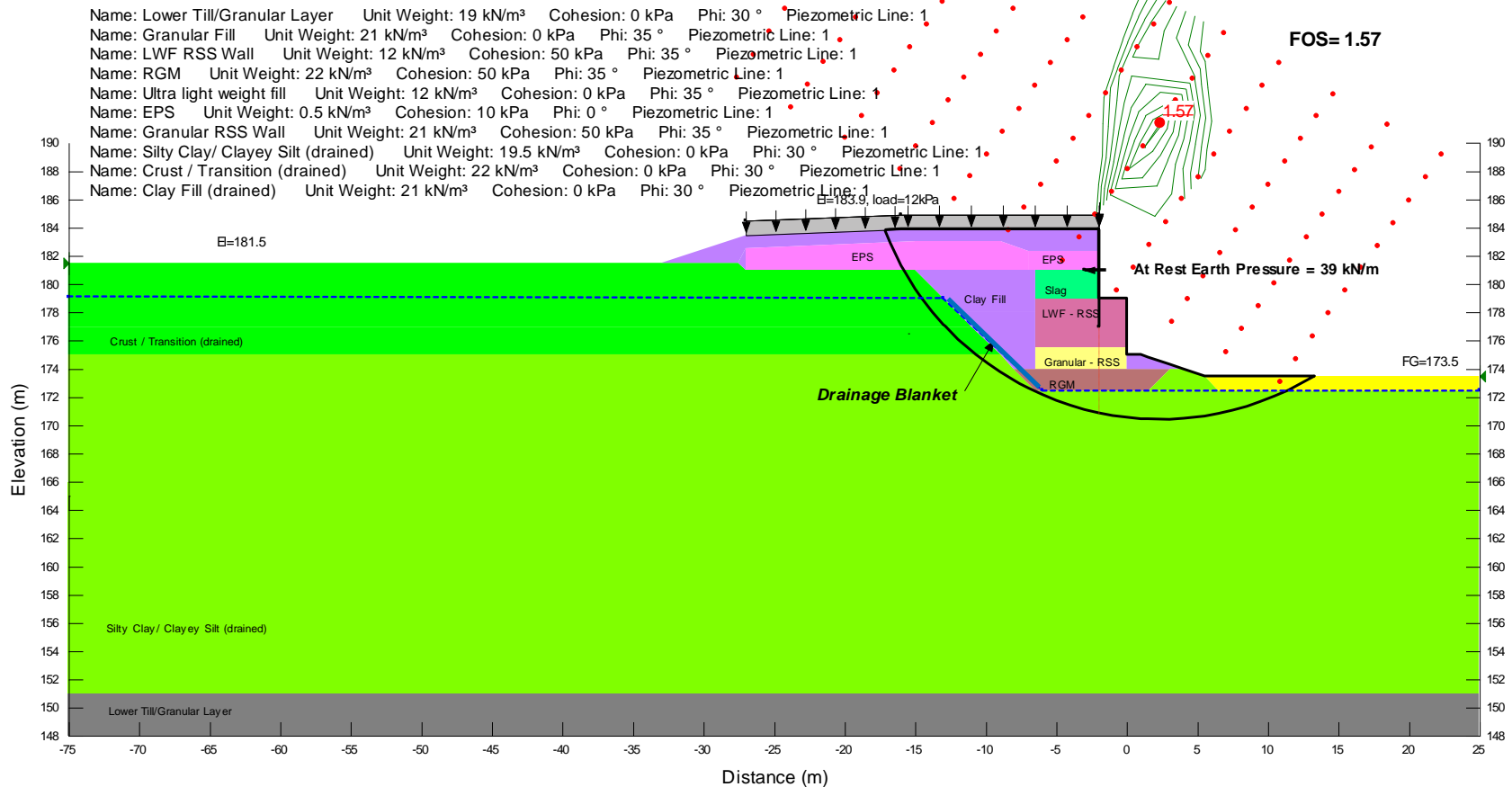
**Figure E.1: Slope Stability – Short Term – North Abutment - Station 10+450L Hwy 401**

File Name: T7N\_Sta10+450L\_Drainage Blanket.gsz  
Analysis Name: End of Construction  
Method: Morgenstern-Price



**Figure E.2: Slope Stability – End of Construction North Abutment - Station 10+450L Hwy 401**

File Name: T7N\_Sta10+450L\_Drainage Blanket.gsz  
Analysis Name: Long-term  
Method: Morgenstern-Price



**Figure E.3: Slope Stability – Long Term - North Abutment - Station 10+450L Hwy 401**



File Name: T7S\_Slope\_Sta10+575L\_20120313-tr-dd.gsz  
Analysis Name: Short-term  
Method: Morgenstern-Price

Name: Clay Crust Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 75 kPa  
Name: Transition Clay Unit Weight: 22 kN/m<sup>3</sup> C-Datum: 75 kPa C-Rate of Change: -5 kPa/m Limiting C: 65 kPa Elevation: 177 m  
Name: Upper Silty Clay 1 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 65 kPa C-Rate of Change: -2.7 kPa/m Limiting C: 44 kPa Elevation: 175 m  
Name: Upper Silty Clay 2 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 44 kPa C-Rate of Change: 2 kPa/m Limiting C: 50 kPa Elevation: 166 m  
Name: Lower Clayey Silt 1 Unit Weight: 19.5 kN/m<sup>3</sup> C-Datum: 50 kPa C-Rate of Change: 8.3 kPa/m Limiting C: 65 kPa Elevation: 163 m  
Name: Lower Clayey Silt 2 Unit Weight: 19.5 kN/m<sup>3</sup> Cohesion: 65 kPa  
Name: Lower Till/Granular Layer Unit Weight: 19 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 30°  
Name: RGM Unit Weight: 22 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35°  
Name: EPS Unit Weight: 0.5 kN/m<sup>3</sup> Cohesion: 10 kPa  
Name: Clay Fill Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa  
Name: Granular RSS Wall Unit Weight: 21 kN/m<sup>3</sup> Cohesion: 50 kPa Phi: 35°  
Name: Base-Subbase Unit Weight: 12 kN/m<sup>3</sup> Cohesion: 0 kPa Phi: 35°

FOS: 1.44

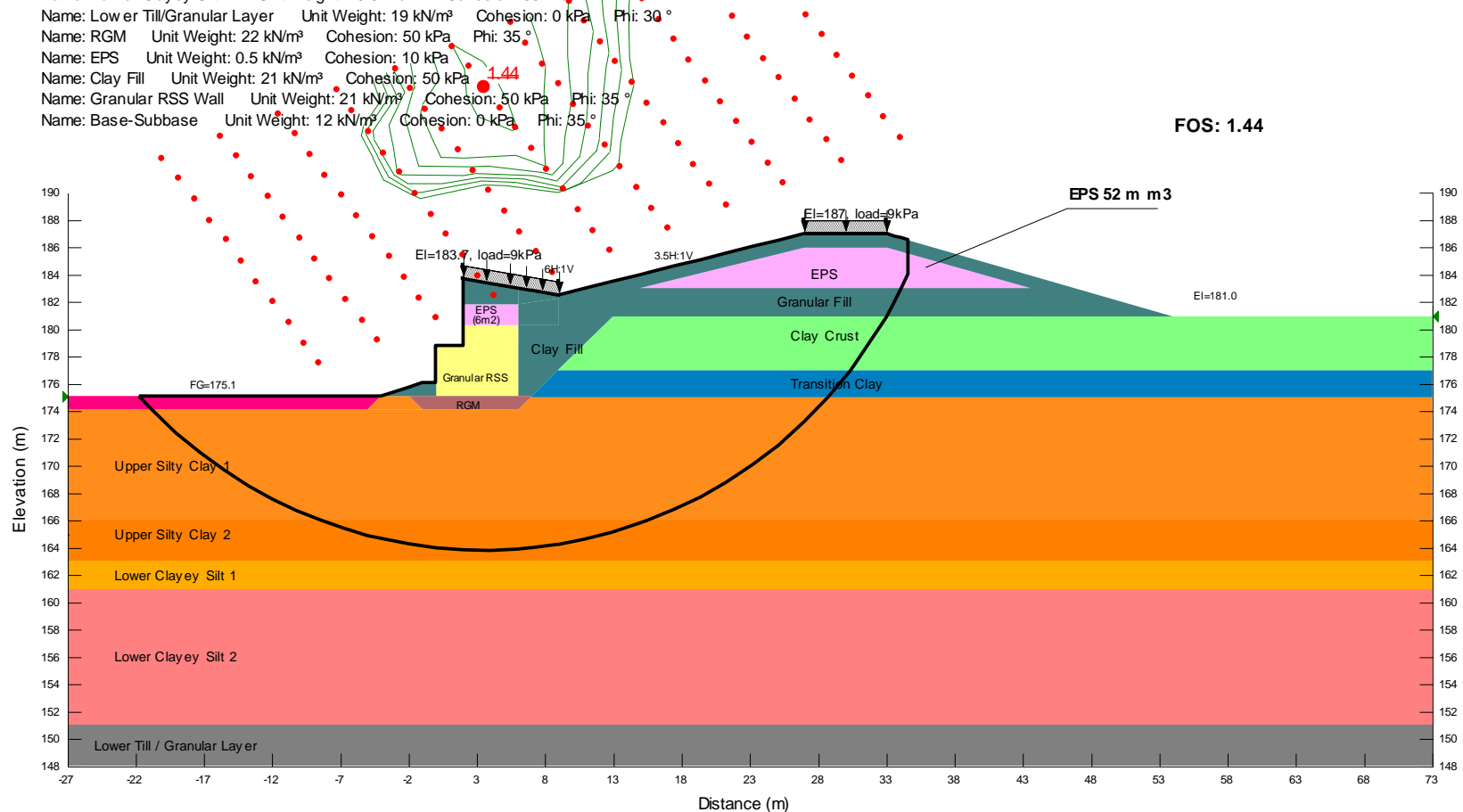
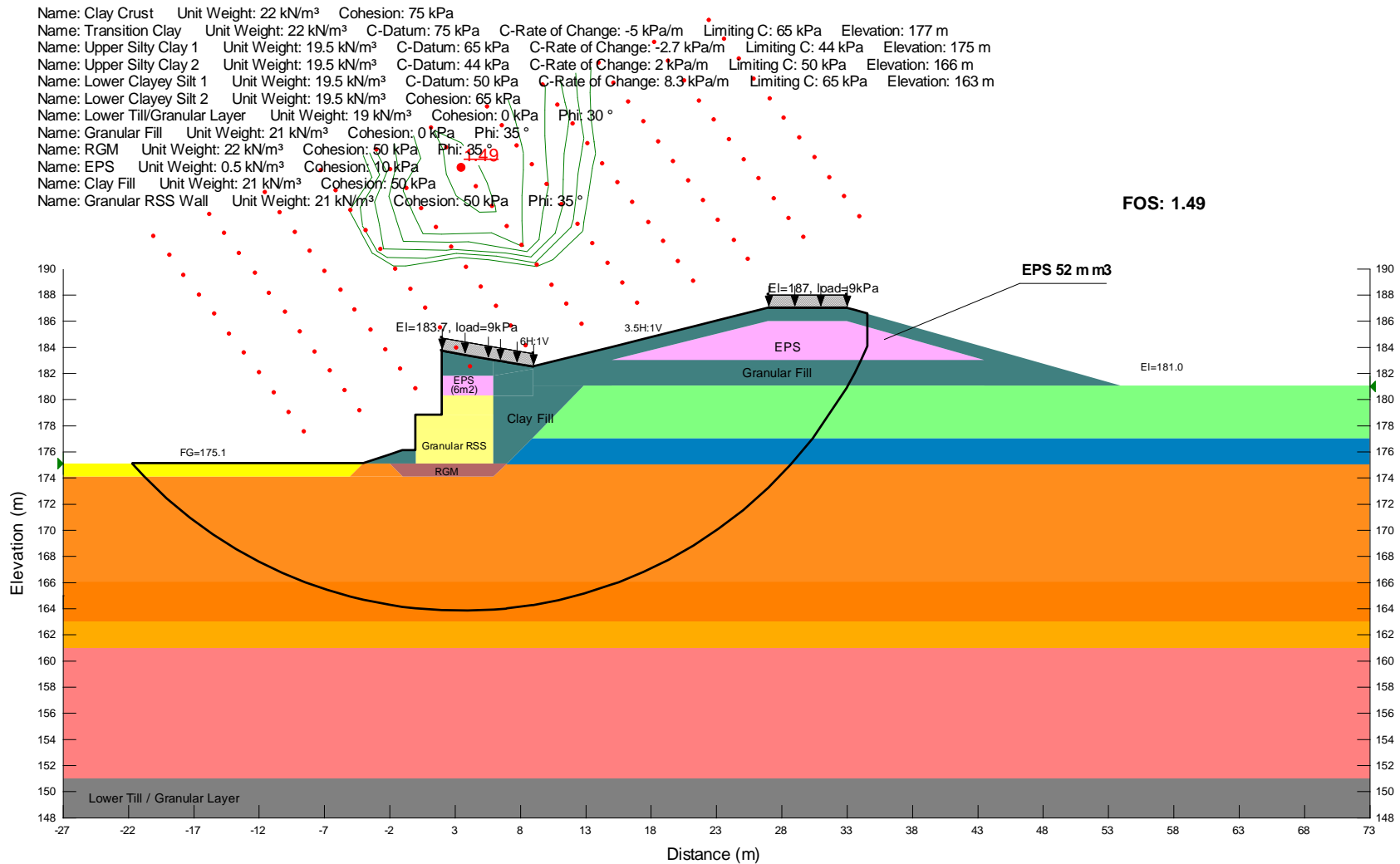


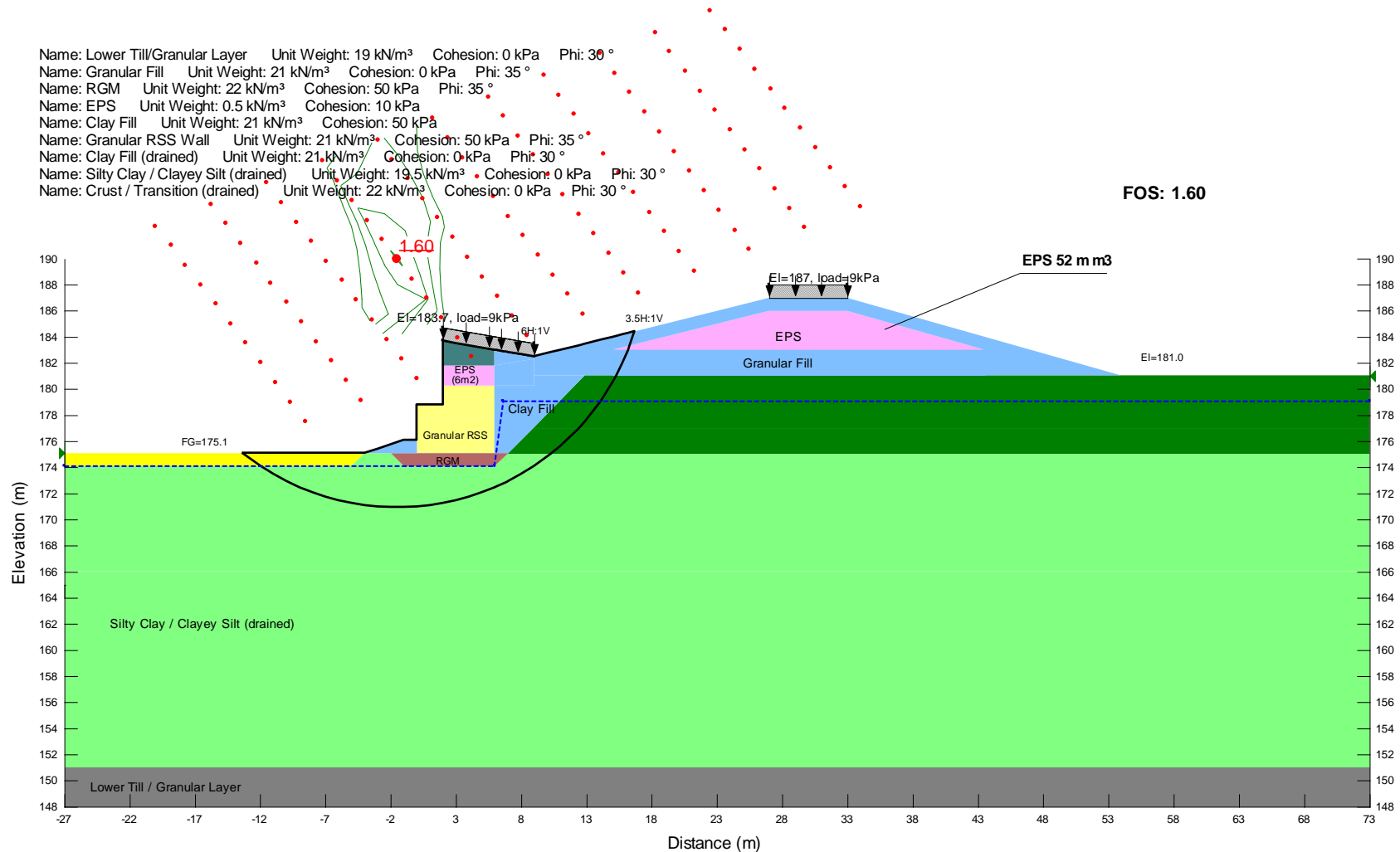
Figure E.4: Slope Stability - Short Term - South Abutment with Trail - Station 10+575L Hwy 401

File Name: T7S\_Slope\_Sta10+575L\_20120313-tr.gsz  
Analysis Name: End of Construction  
Method: Morgenstern-Price



**Figure E.5: Slope Stability – End of Construction - South Abutment with Trail - Station 10+575L Hwy 401**

File Name: T7S\_Slope\_Sta10+575L\_20120313-tr.gsz  
Analysis Name: Long-term  
Method: Morgenstern-Price



**Figure E.6: Slope Stability - Long Term - South Abutment with Trail - Station 10+575L Hwy 401**

File Name: T7S\_Sta10+450L\_20120314-tr.gsz  
Analysis Name: Short-Term  
Method: Morgenstern-Price

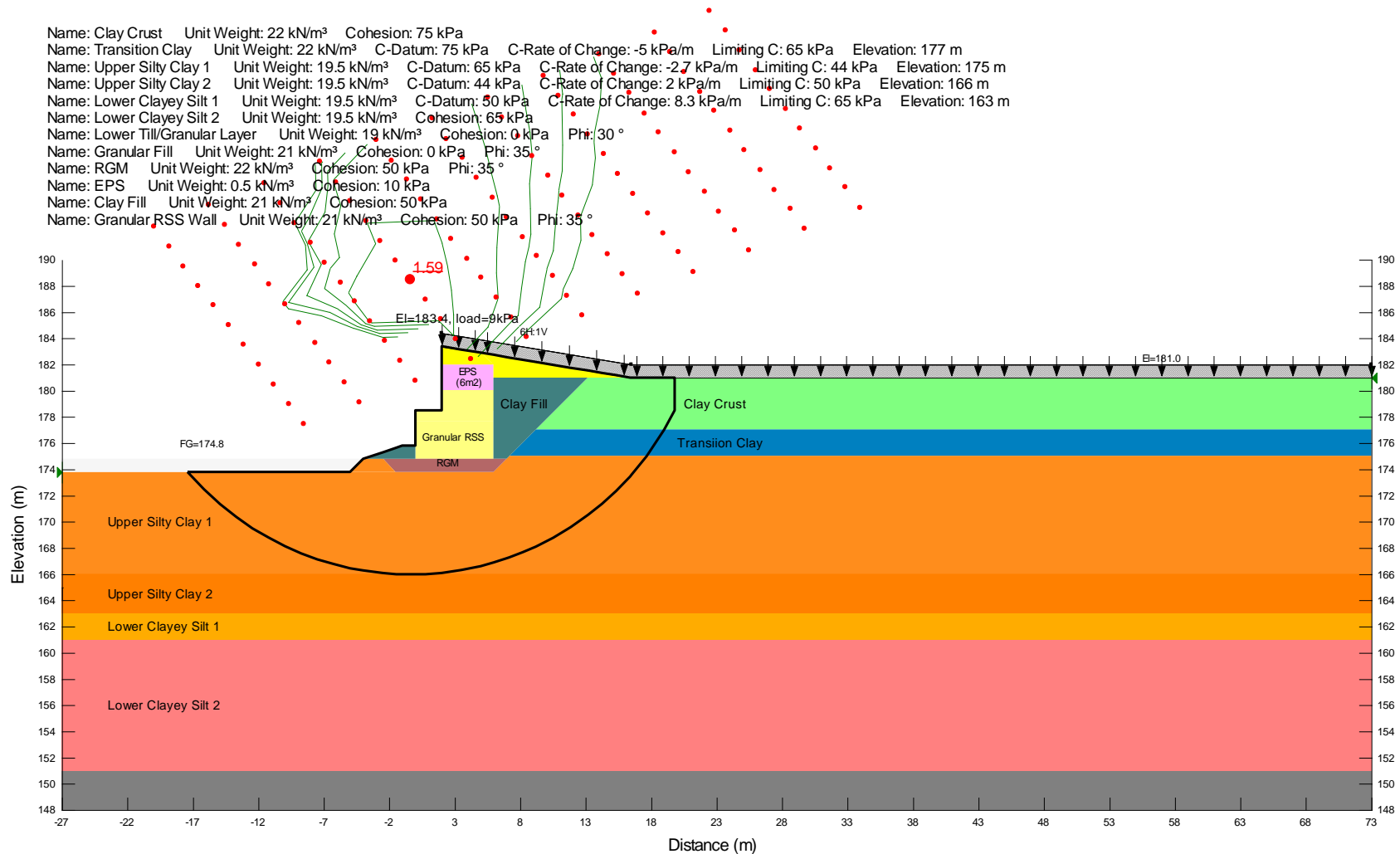


Figure E.7: Slope Stability - Short Term - South Abutment - Station 10+450L Hwy 401

File Name: T7S\_Sta10+450L\_20120314-tr.gsz  
Analysis Name: End of Construction  
Method: Morgenstern-Price

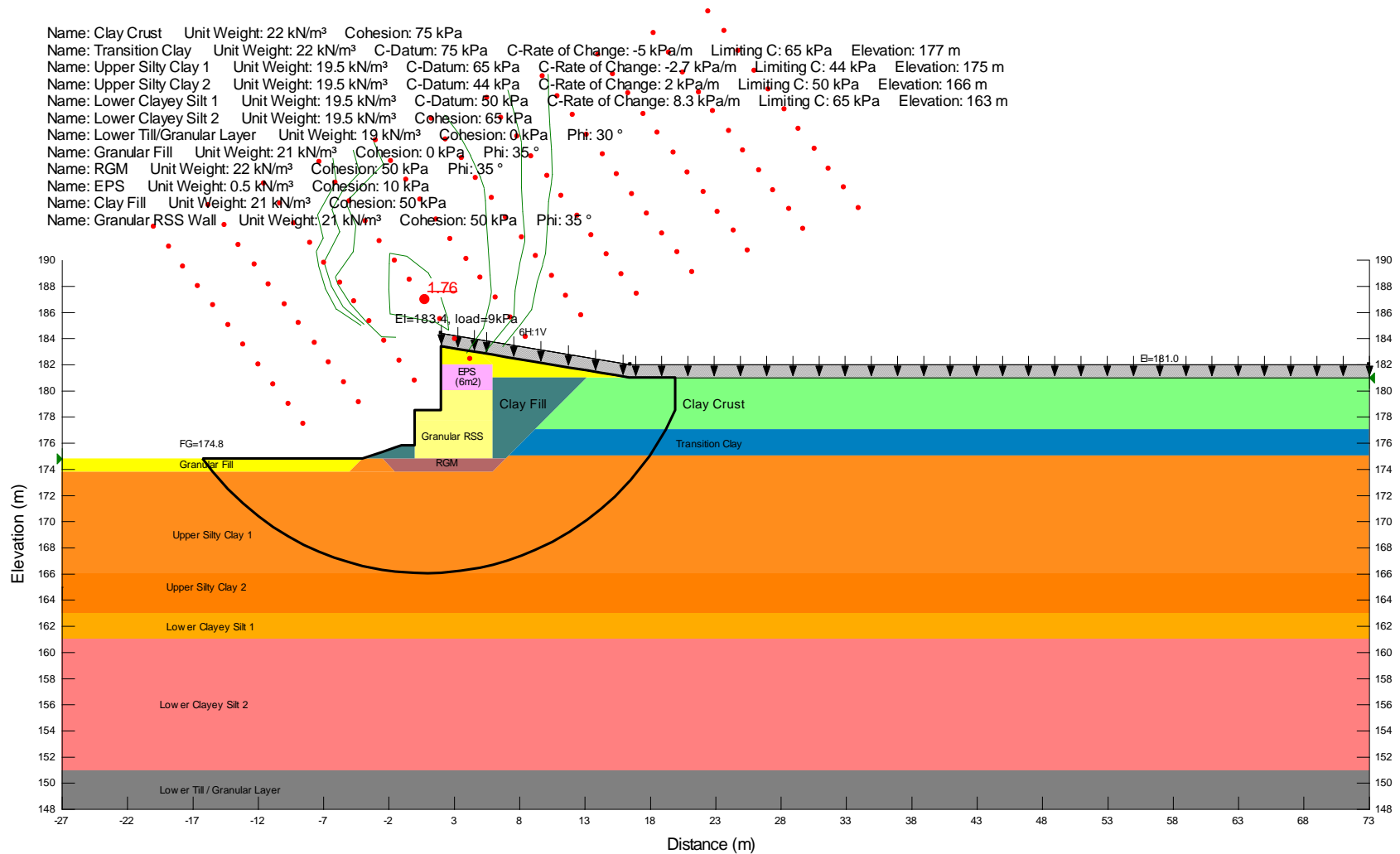


Figure E.8: Slope Stability – End of Construction - South Abutment - Station 10+450L Hwy 401

File Name: T7S\_Sta10+450L\_20120314-tr.gsz  
Analysis Name: Long-term  
Method: Morgenstern-Price

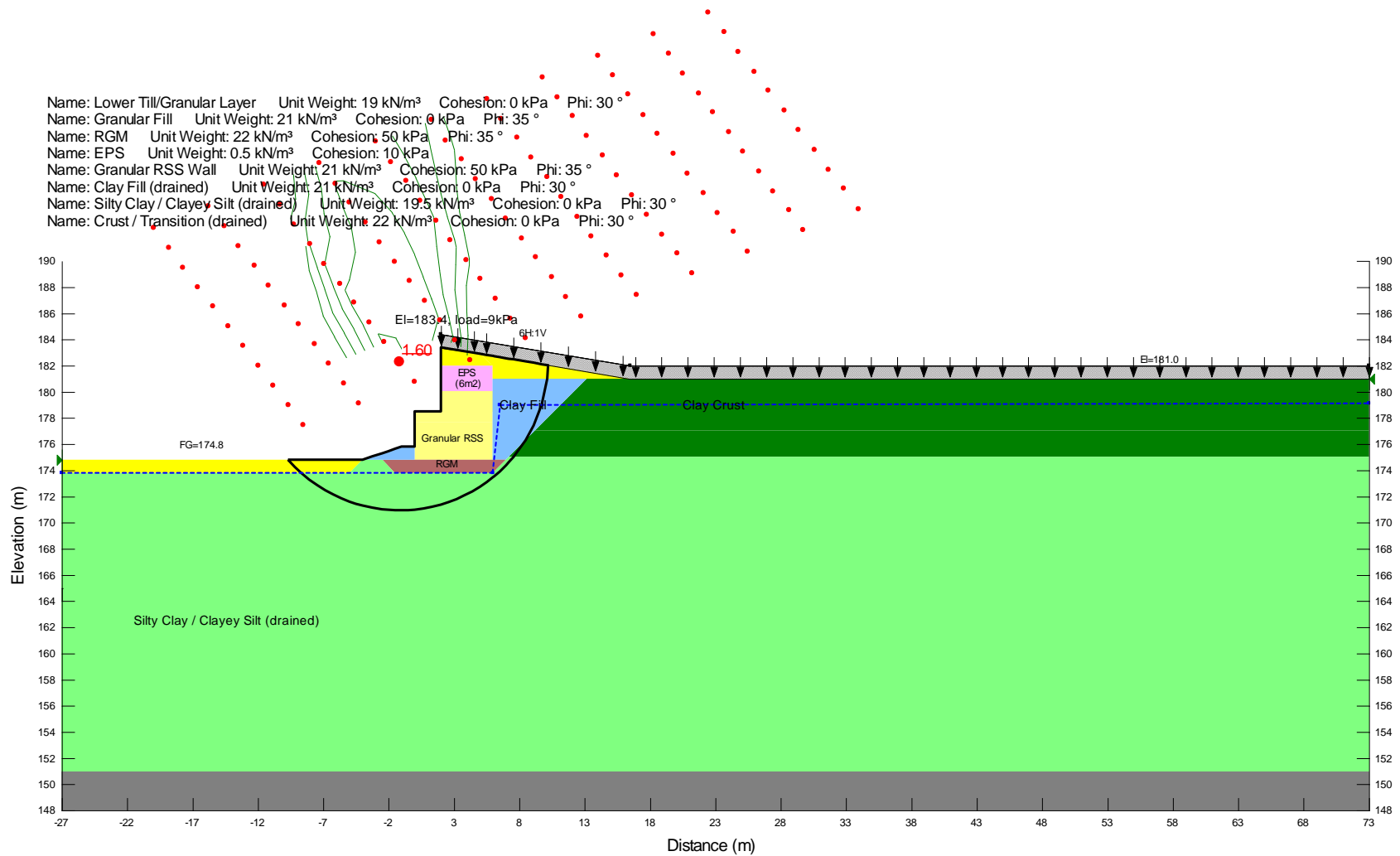
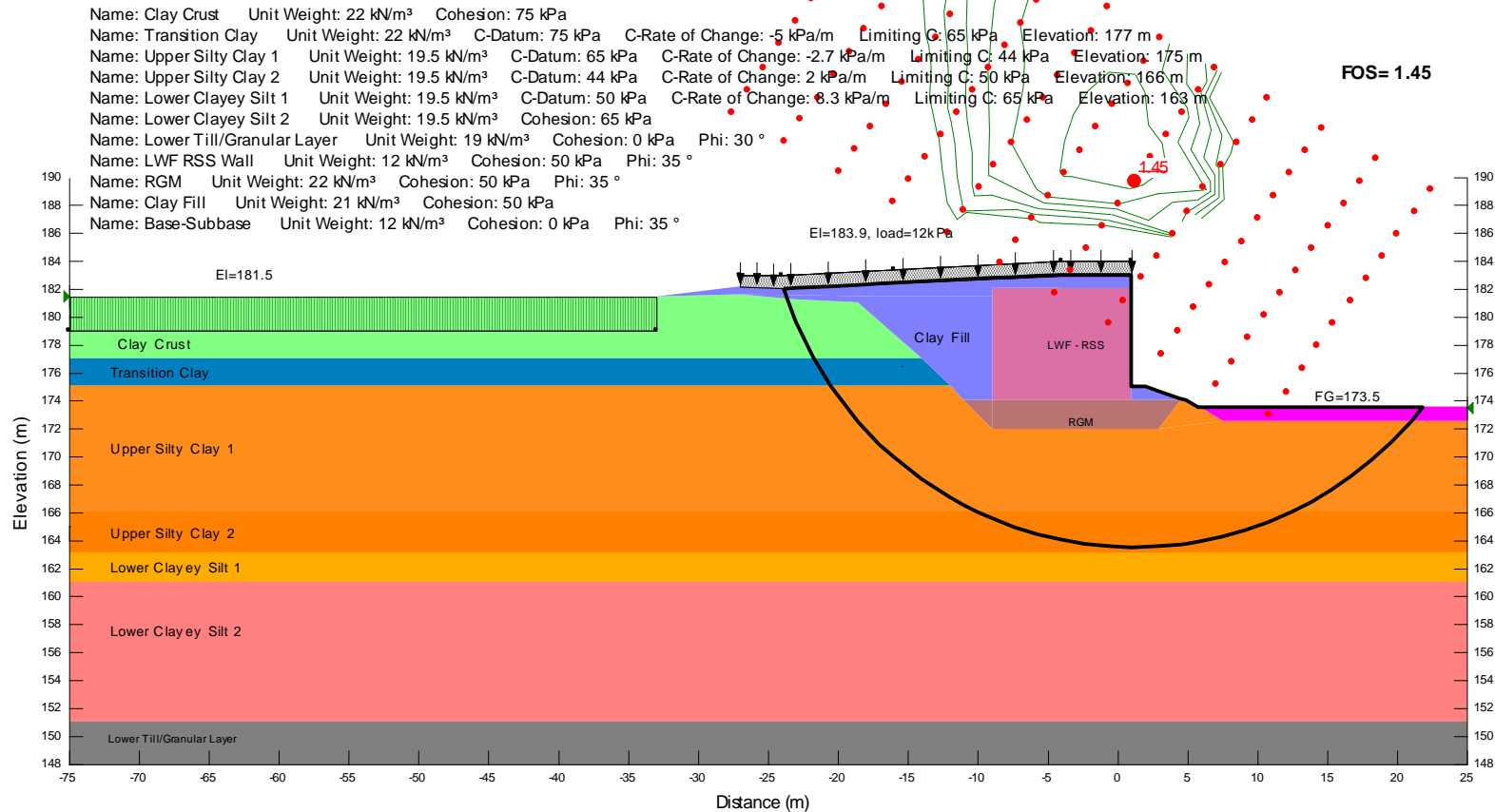


Figure E.9: Slope Stability - Long Term - South Abutment - Station 10+450L Hwy 401

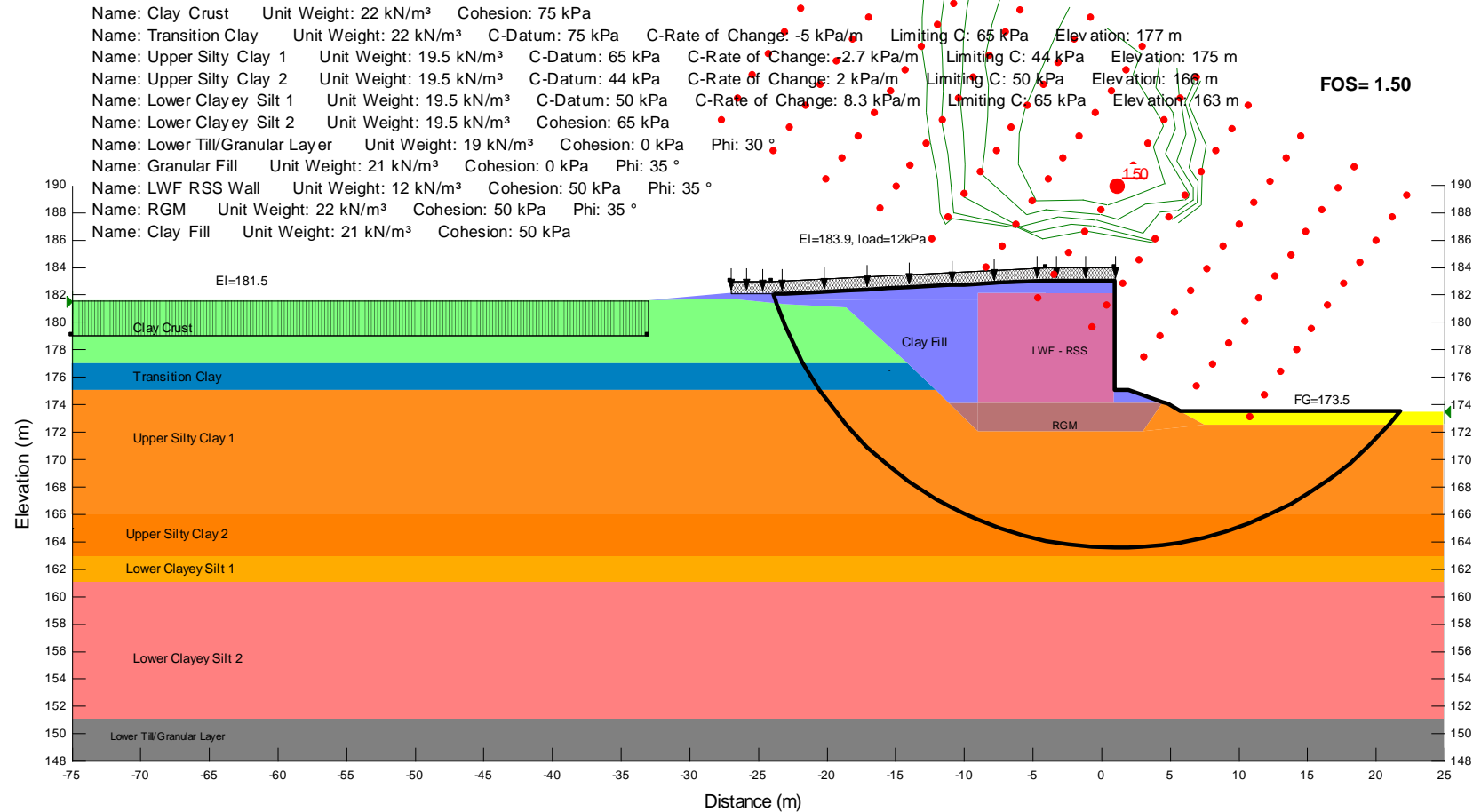
File Name: T7N\_Sta10+450L\_wing wall\_20120321-tr-dd.gsz  
Analysis Name: Short-term  
Method: Morgenstern-Price



**Figure E.10: Slope Stability – Short Term – North Abutment Wing Wall - Station 10+450L Hwy 401**



File Name: T7N\_Sta10+450L\_wing wall\_20120321-tr.gsz  
Analysis Name: End of Construction  
Method: Morgenstern-Price



**Figure E.11: Slope Stability – End of Construction – North Abutment Wing Wall - Station 10+450L Hwy 401**

File Name: T7N\_Sta10+450L\_wing wall\_20120321-tr.gsz  
Analysis Name: Long-term  
Method: Morgenstern-Price

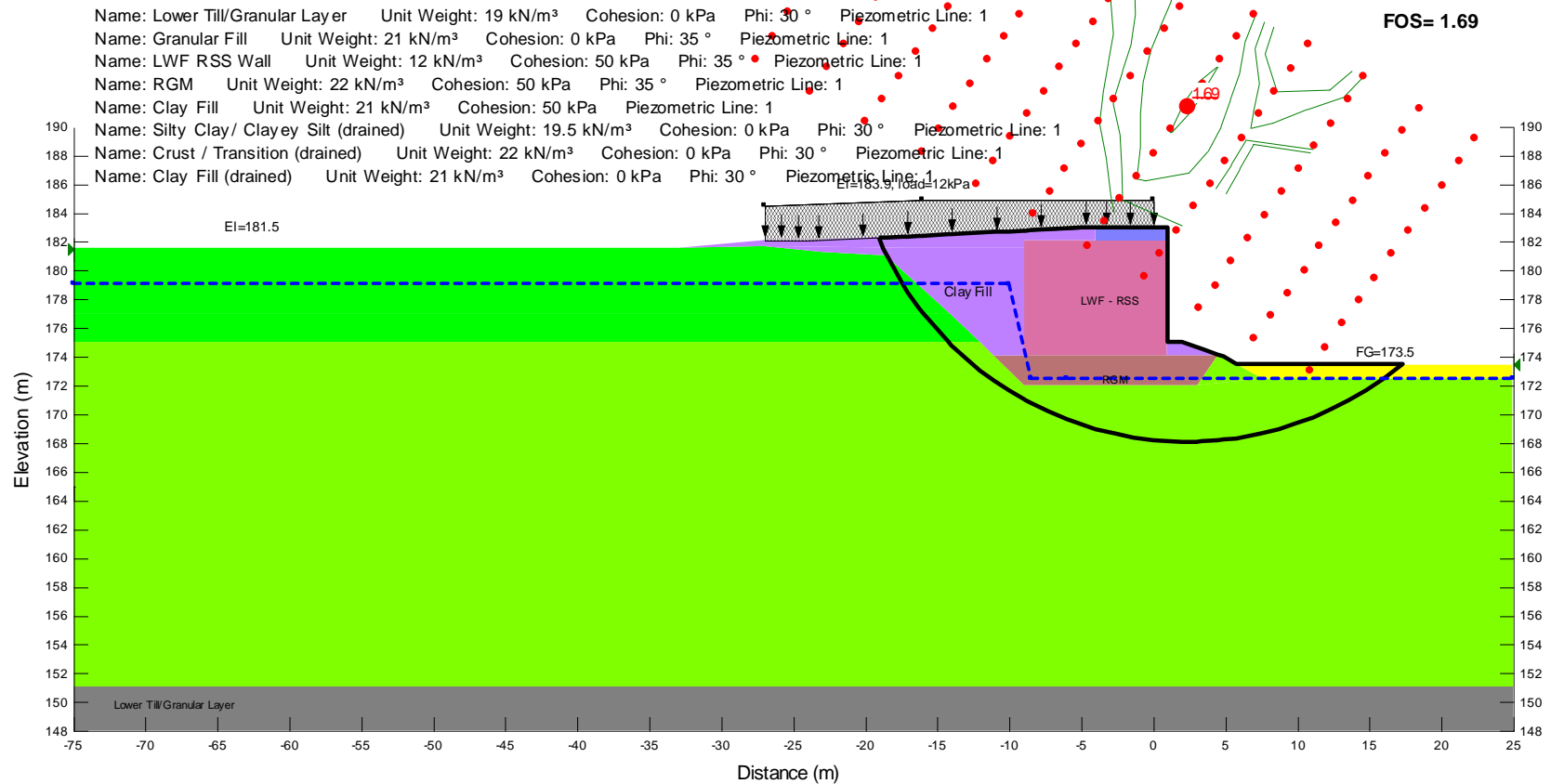


Figure E.12: Slope Stability – Long Term – North Abutment Wing Wall - Station 10+450L Hwy 401

File Name: T7S\_Sta10+450L\_wing wall 20120314-tr.gsz  
Analysis Name: Short-Term  
Method: Morgenstern-Price

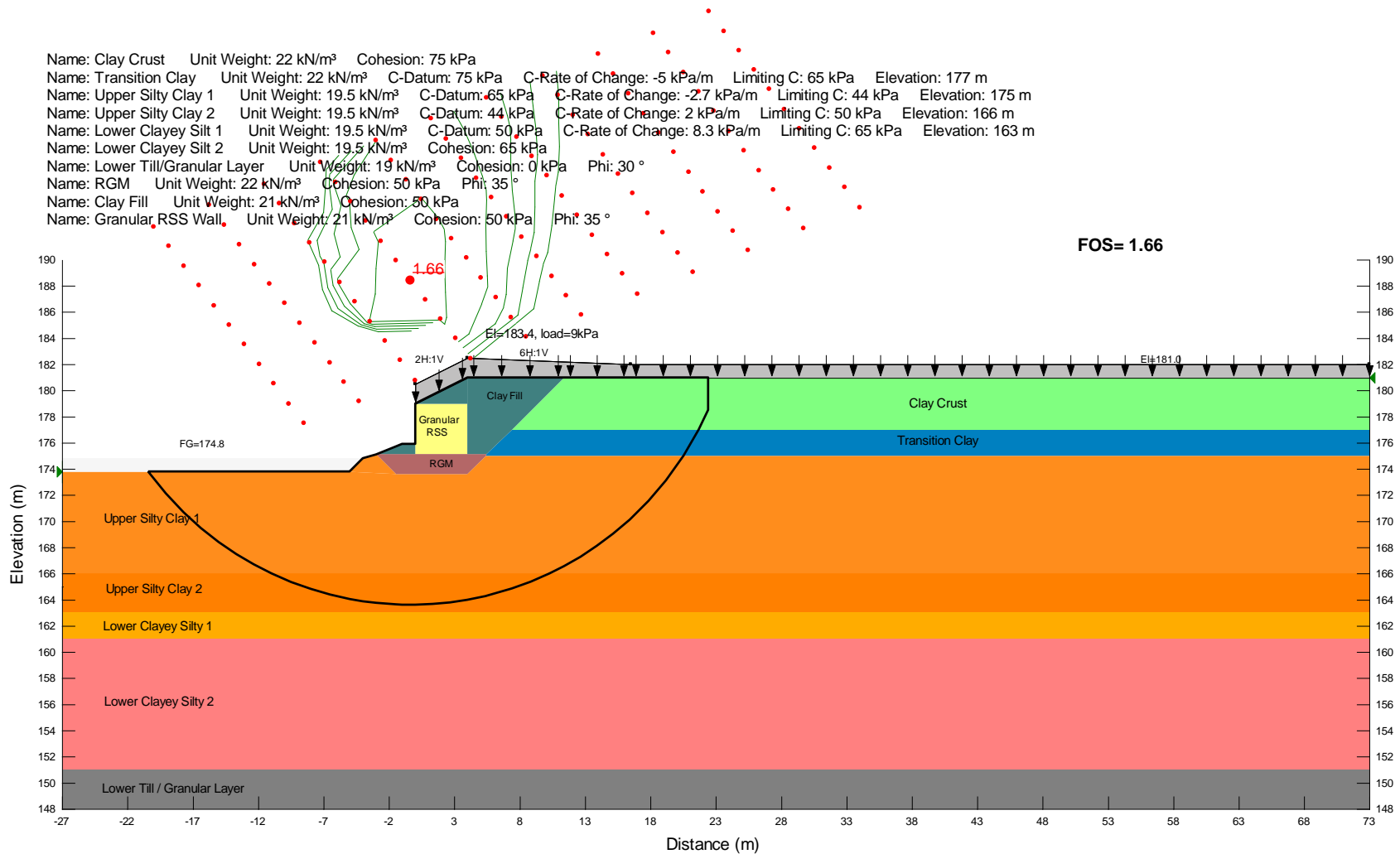


Figure E.13: Slope Stability –Short Term – South Abutment Wing Wall - Station 10+450L Hwy 401

File Name: T7S\_Sta10+450L\_wing wall 20120314-tr.gsz  
Analysis Name: End of Construction  
Method: Morgenstern-Price

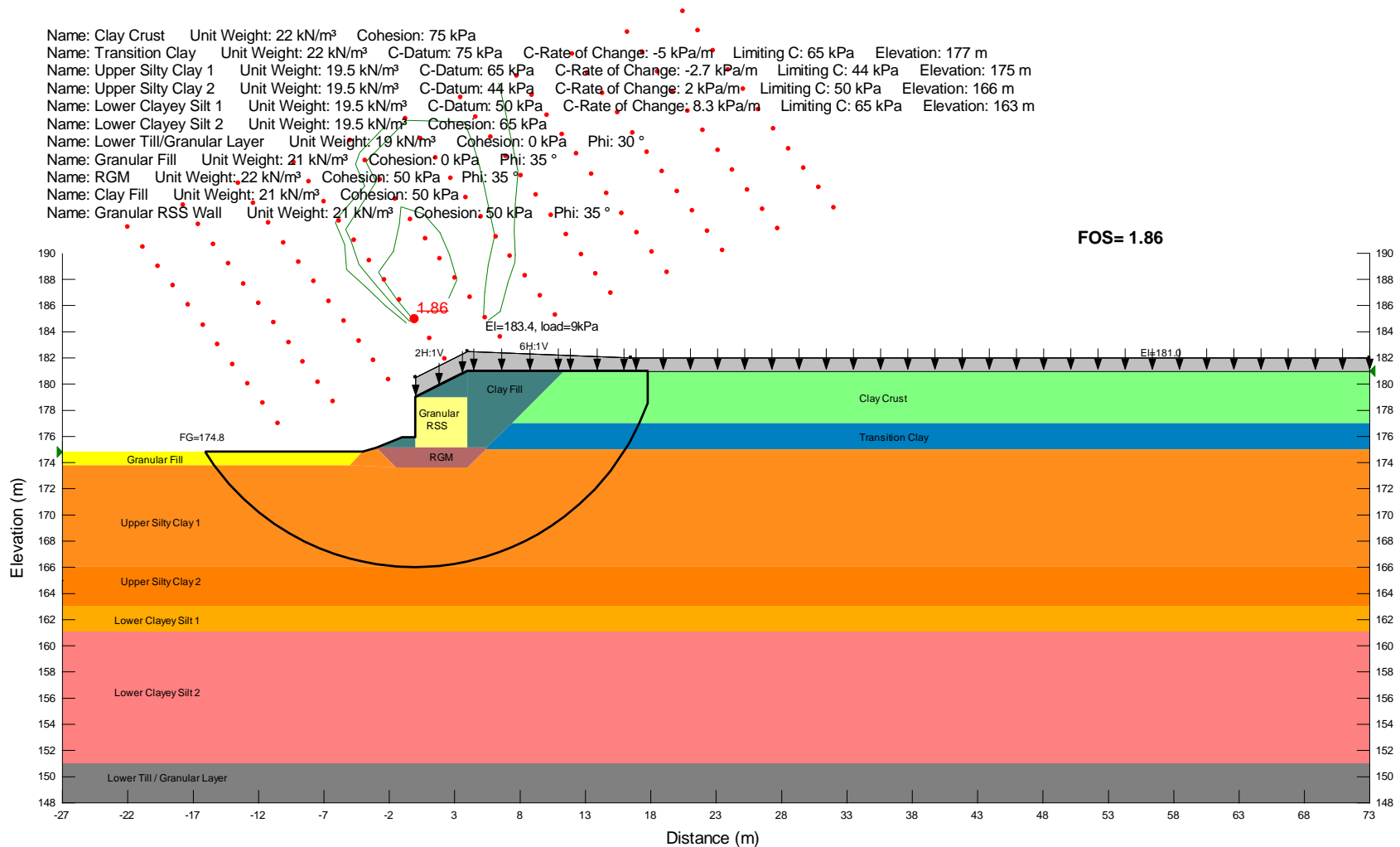
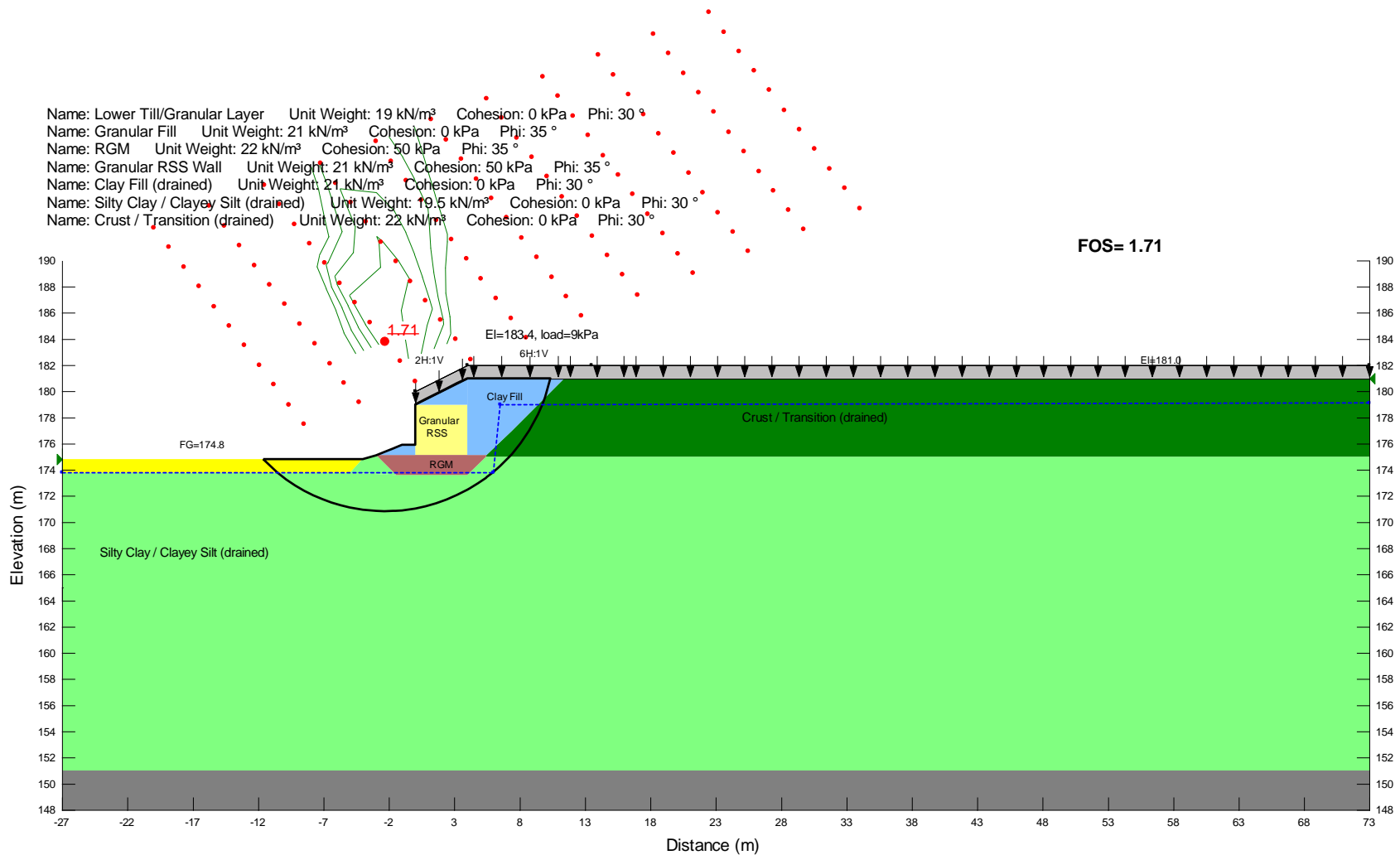


Figure E.14: Slope Stability – End of Construction – South Abutment Wing Wall - Station 10+450L Hwy 401

File Name: T7S\_Sta10+450L\_wing wall 20120314-tr.gsz  
Analysis Name: Long-term  
Method: Morgenstern-Price



**Figure E.15: Slope Stability – Long Term – South Abutment Wing Wall - Station 10+450L Hwy 401**

File Name: T7S\_Sta10+450L\_return wall\_20120320-tr.gsz  
Analysis Name: Short-term  
Method: Morgenstern-Price

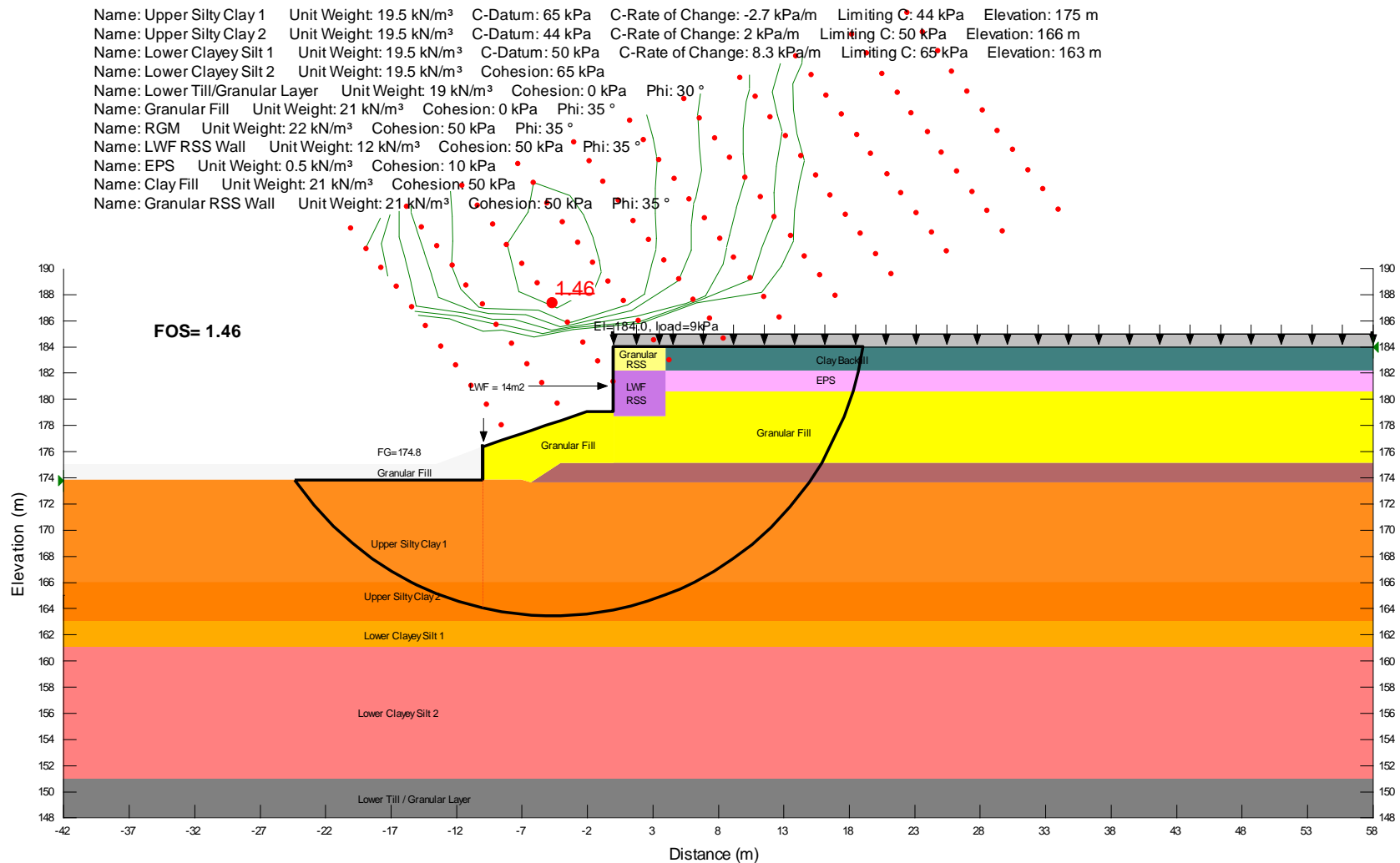


Figure E.16: Slope Stability – Short Term – South Abutment – Return Wall – Station 10+450L Hwy 401

File Name: T7S\_Sta10+450L\_return wall\_20120320-tr.gsz  
Analysis Name: End of Construction  
Method: Morgenstern-Price

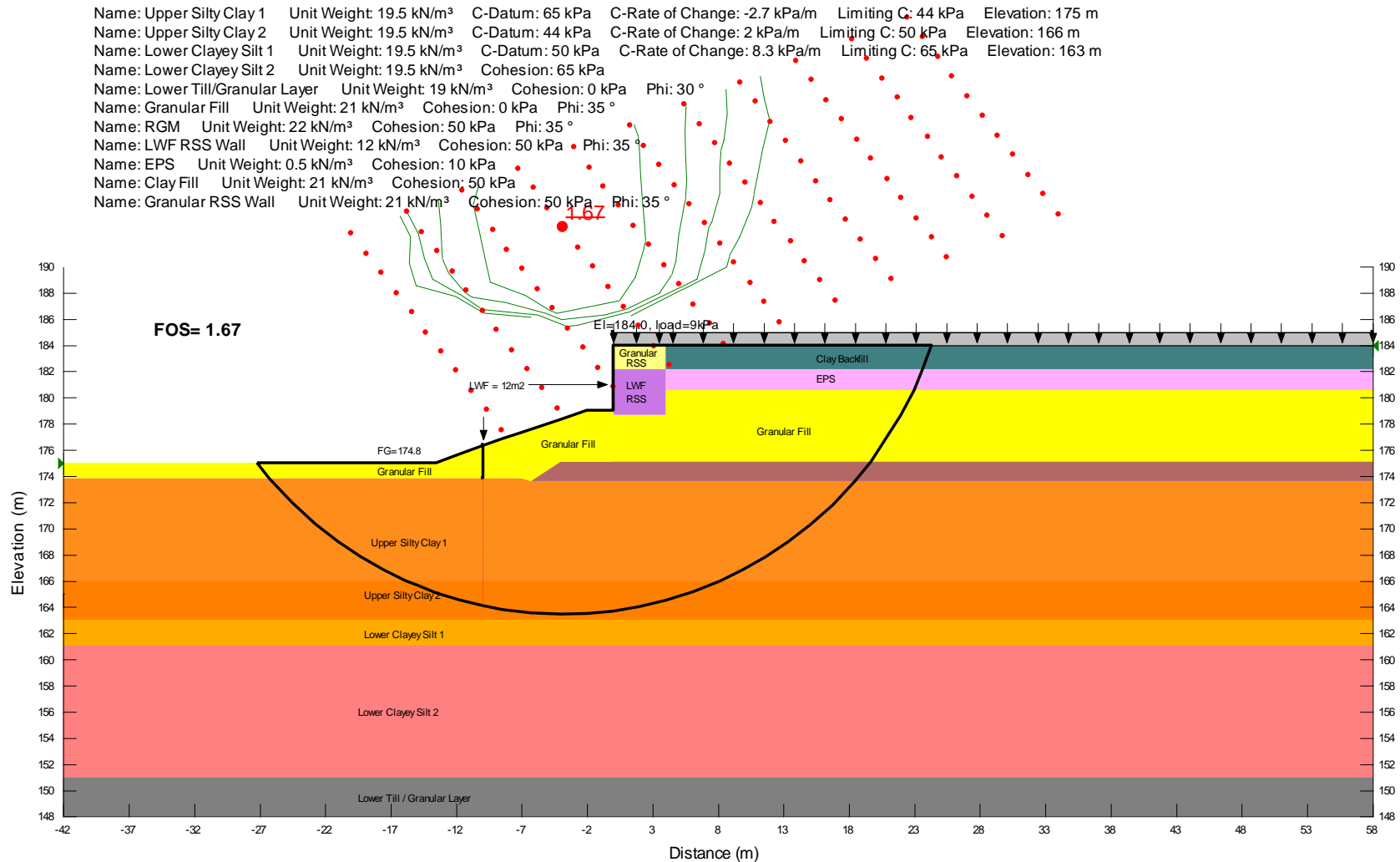
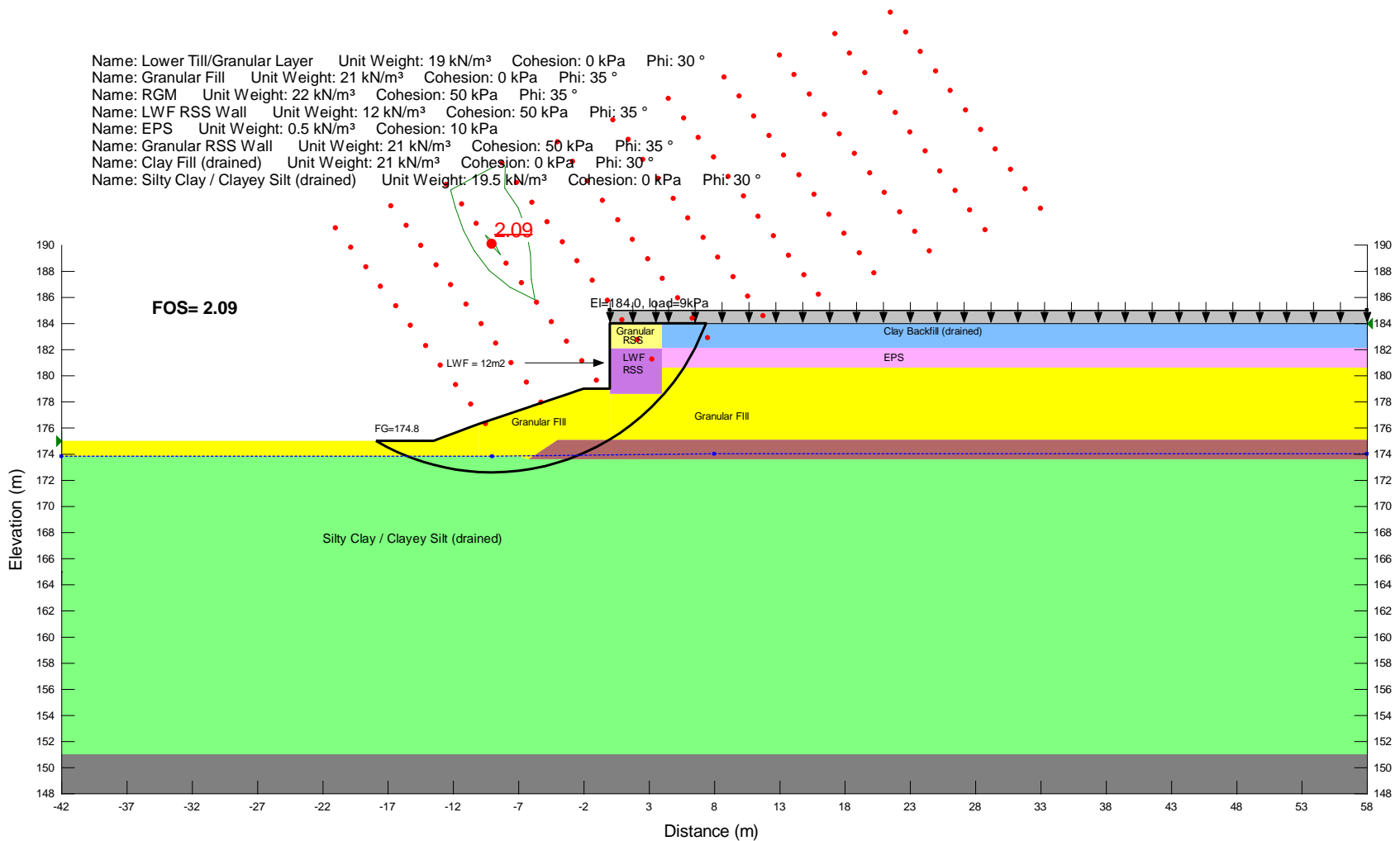


Figure E.17: Slope Stability – End of Construction – South Abutment – Return Wall – Station 10+450L Hwy 401



File Name: T7S\_Sta10+450L\_return wall\_20120320-tr.gsz  
Analysis Name: Long-term  
Method: Morgenstern-Price

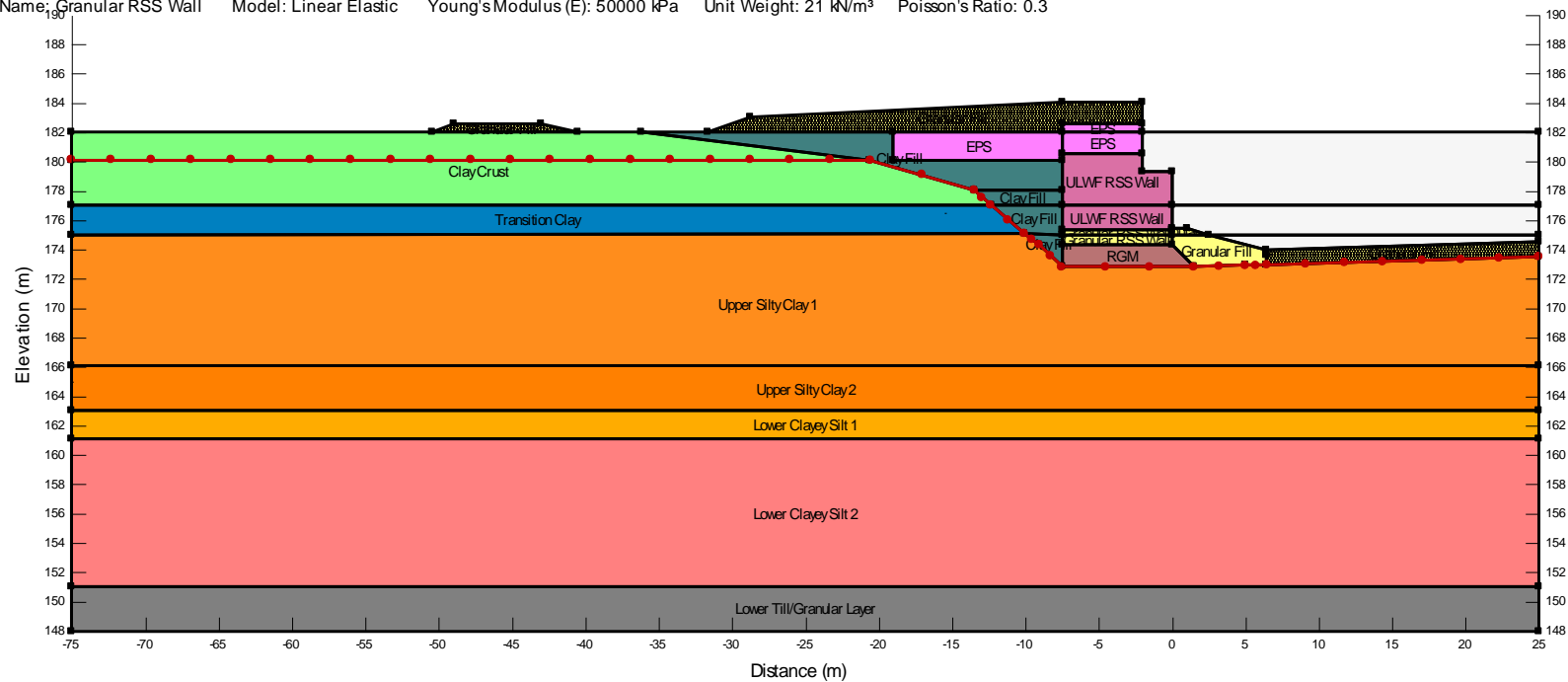


**Figure E.18: Slope Stability – Long Term – South Abutment – Return Wall – Station 10+450L Hwy 401**

## **Appendix F: Stress-Deformation Analyses**

Title: Tunnel 7 - Station 10+575L North Wall (Transverse)  
Name: Approach Fill and Hwy 401 Fill  
Method: Coupled Stress/PWP

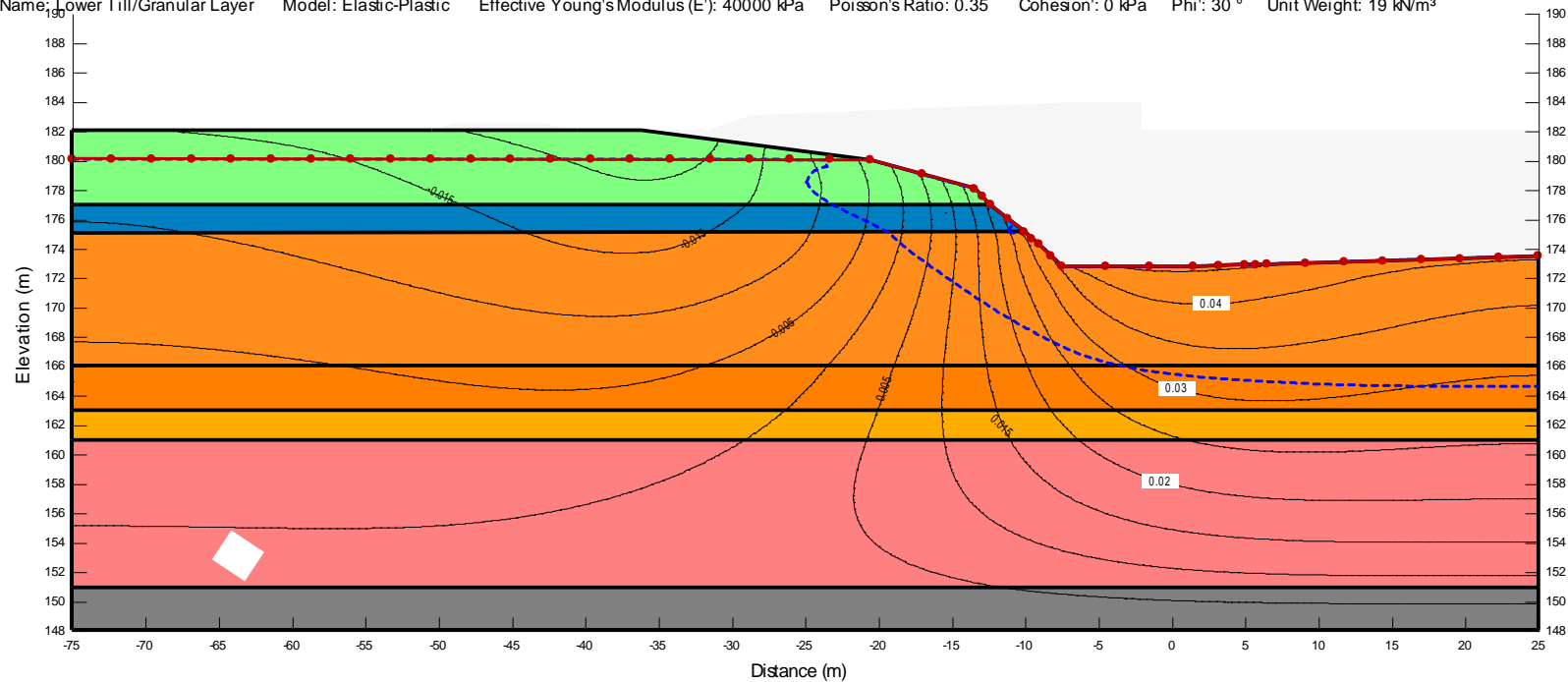
Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: Transition Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 2.1 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.62 Unit Weight: 19.5 kN/m<sup>3</sup> Phi': 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.15 Poisson's Ratio: 0.35 Lambda: 0.052 Kappa: 0.0057 Initial Void Ratio: 0.4 Unit Weight: 19.5 kN/m<sup>3</sup> Phi': 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup> Phi': 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.33 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup> Phi': 26 °  
Name: Lower Till/Granular Layer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 19 kN/m<sup>3</sup>  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: ULWF RSS Wall Model: Linear Elastic Young's Modulus (E): 30000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: RGM Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m<sup>3</sup> Poisson's Ratio: 0.3  
Name: EPS Model: Linear Elastic Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m<sup>3</sup> Poisson's Ratio: 0.2  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular RSS Wall Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.3



**Figure F1. Finite Element Modeling – North Wall Configuration – Station 10+575L Hwy 401**

Title: Tunnel 7 - Station 10+575L North Wall (Transverse)  
Name: Excavation  
Method: Coupled Stress/PWP

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: Transition Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 2.1 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.62 Unit Weight: 19.5 kN/m³ Phi': 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.15 Poisson's Ratio: 0.35 Lambda: 0.052 Kappa: 0.0057 Initial Void Ratio: 0.4 Unit Weight: 19.5 kN/m³ Phi': 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m³ Phi': 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.33 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m³ Phi': 26 °  
Name: Lower Till/Granular Layer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 19 kN/m³



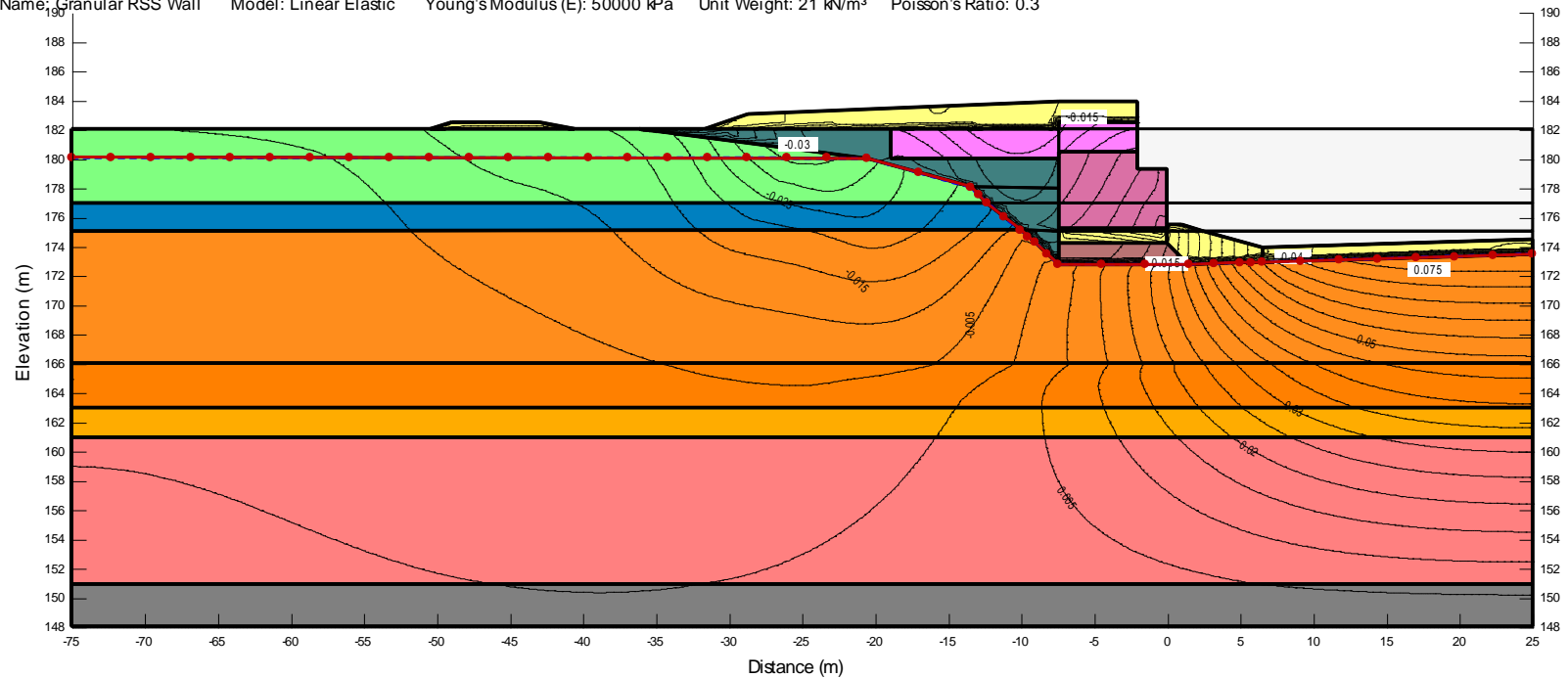
**Figure F1-1. Finite Element Modeling – North Wall Configuration – Drained Analysis – End of Excavation - Station 10+575L Hwy 401**

Method: Coupled Stress/PWP

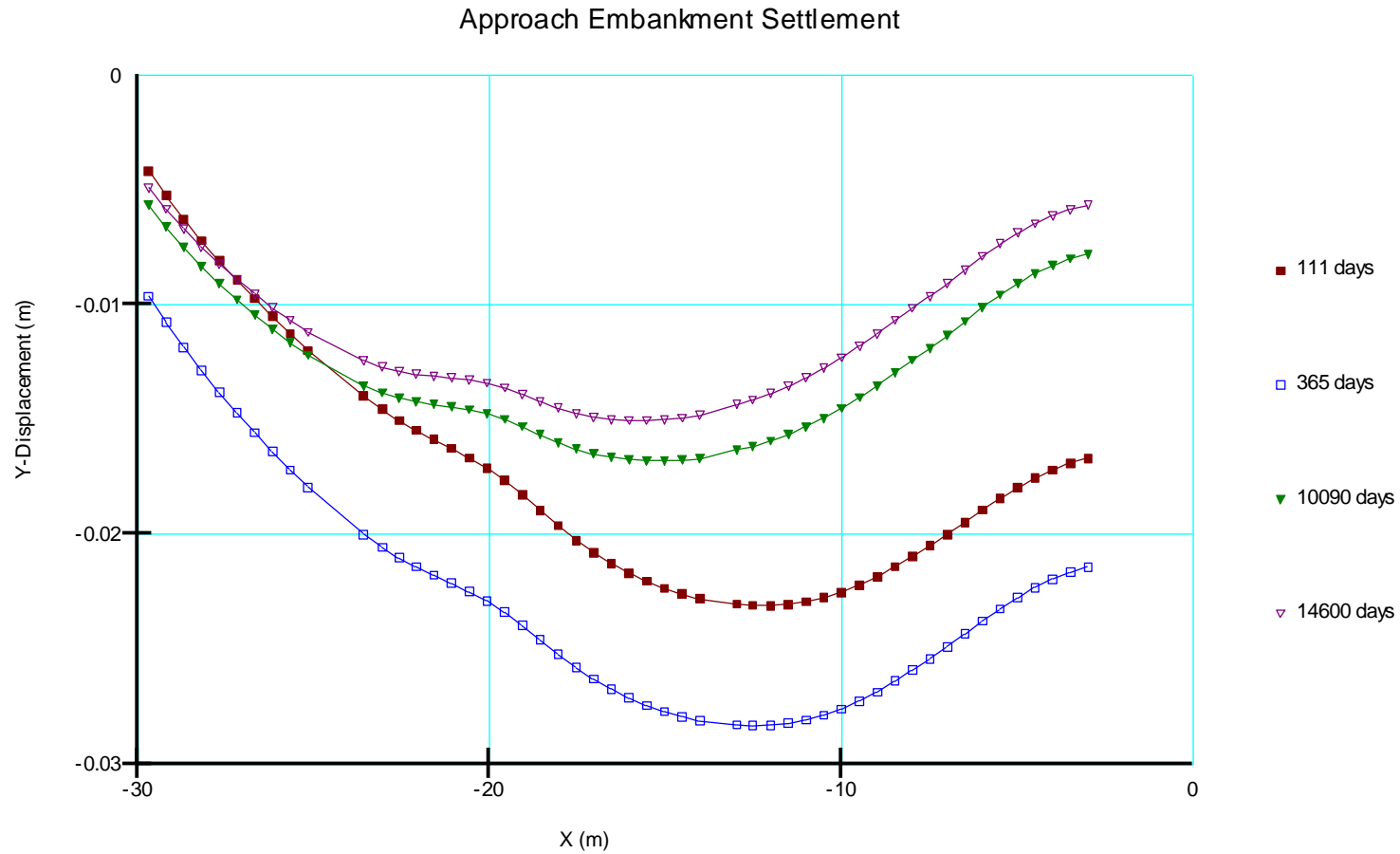
**Figure F1-2. Finite Element Modeling – North Wall Configuration – End of Construction - Drained Analysis – Station 10+575L Hwy 401**

Title: Tunnel 7 - Station 10+575L North Wall (Transverse)  
Name: Approach Fill and Hwy 401 Fill  
Method: Coupled Stress/PWP

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: Transition Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 2.1 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.62 Unit Weight: 19.5 kN/m<sup>3</sup> Phi': 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.15 Poisson's Ratio: 0.35 Lambda: 0.052 Kappa: 0.0057 Initial Void Ratio: 0.4 Unit Weight: 19.5 kN/m<sup>3</sup> Phi': 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup> Phi': 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.33 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup> Phi': 26 °  
Name: Lower Till/Granular Layer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 19 kN/m<sup>3</sup>  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: ULWF RSS Wall Model: Linear Elastic Young's Modulus (E): 30000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: RGM Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m<sup>3</sup> Poisson's Ratio: 0.3  
Name: EPS Model: Linear Elastic Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m<sup>3</sup> Poisson's Ratio: 0.2  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular RSS Wall Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.3



**Figure F1-3. Finite Element Modeling – North Wall Configuration – Long Term - Drained Analysis – Station 10+575L Hwy 401**

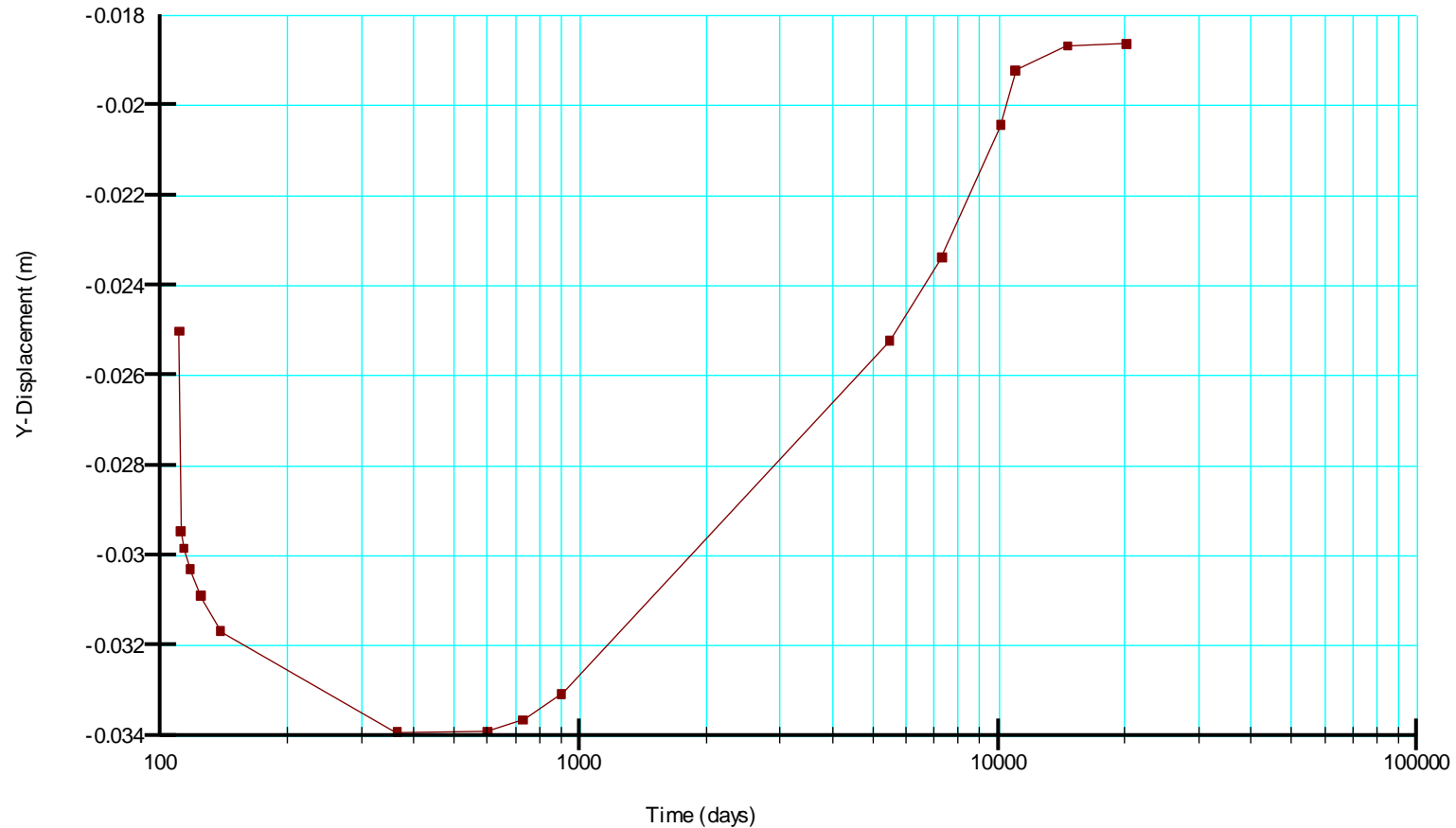


**Legend:**  
 111 days = Placement of Pavement  
 14600 days = Long Term

**Figure F1-4. Finite Element Modeling Graph – North Wall Configuration – Cumulative Settlement Approach Embankment - Drained Analysis – Station 10+575L Hwy. 401**



### Settlement at the top of RSS Facing

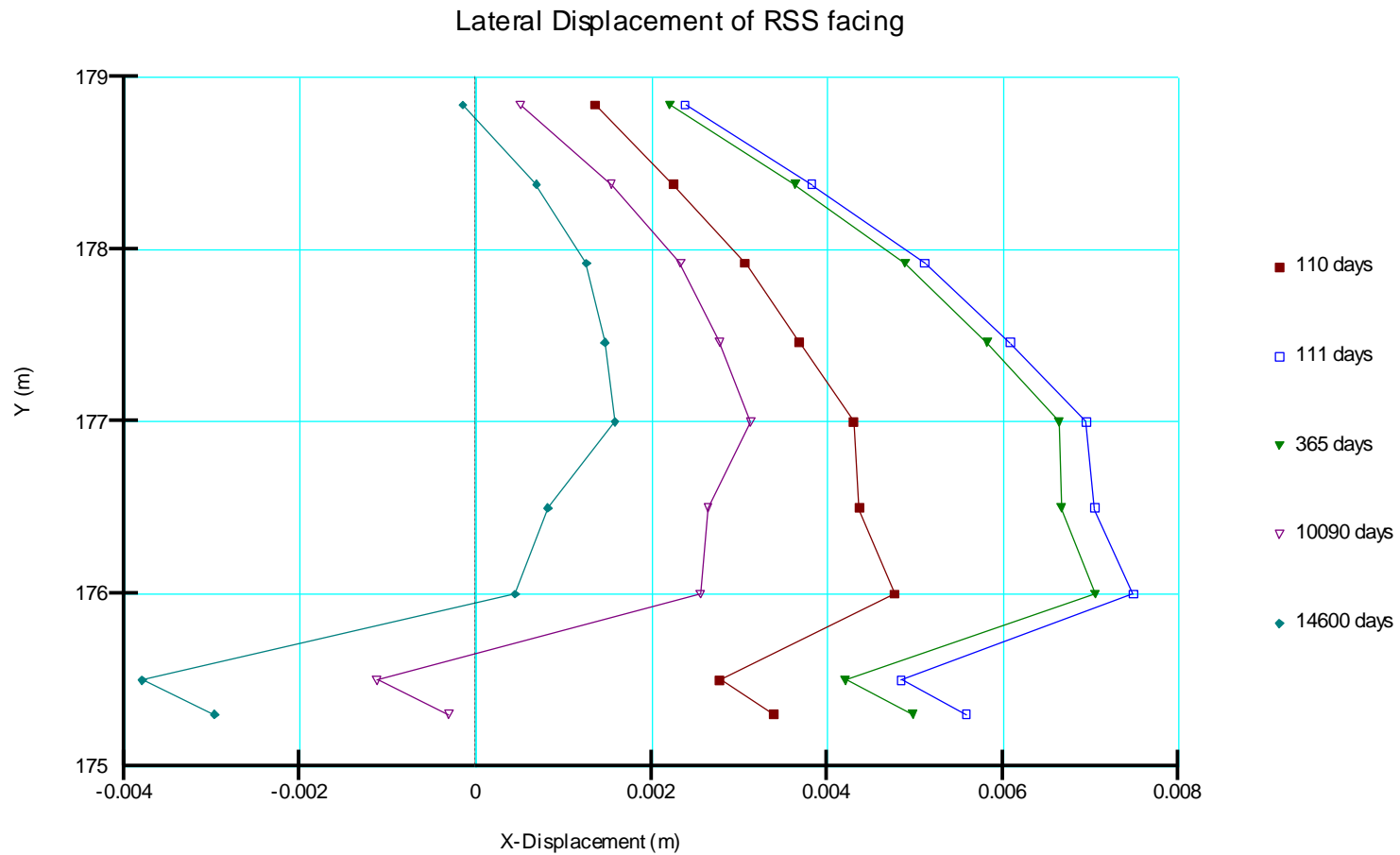


**Legend:**

111 days = Pavement Placement

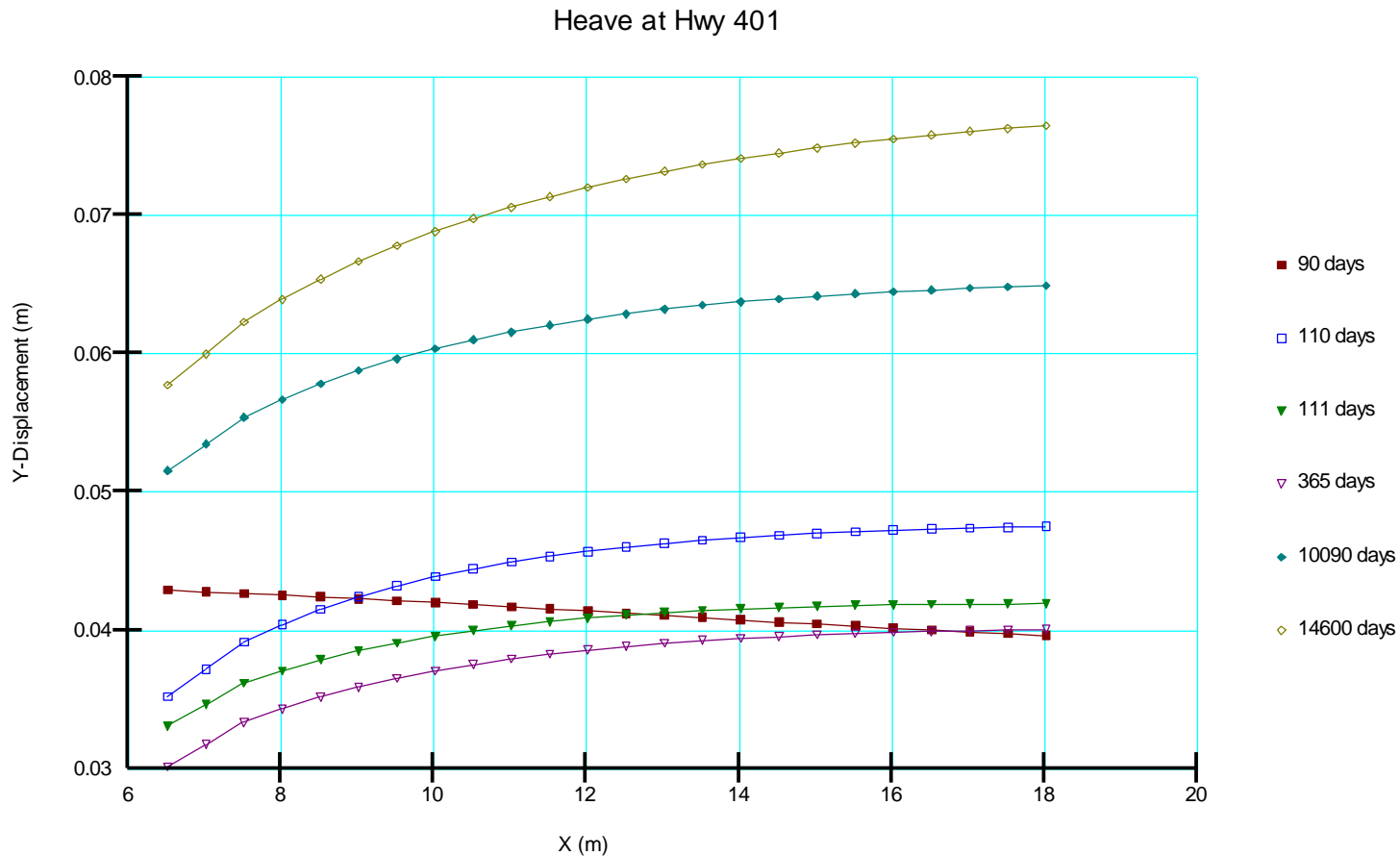
14600 days = Long Term

**Figure F1-5. Finite Element Modeling Graph – North Wall Configuration – Cumulative Settlement Top of RSS Wall Facing - Drained Analysis – Station 10+575L Hwy. 401**



**Legend:**  
 110 days = RSS completion  
 111 days = Pavement Placement (EOC)  
 14600 days = Long Term

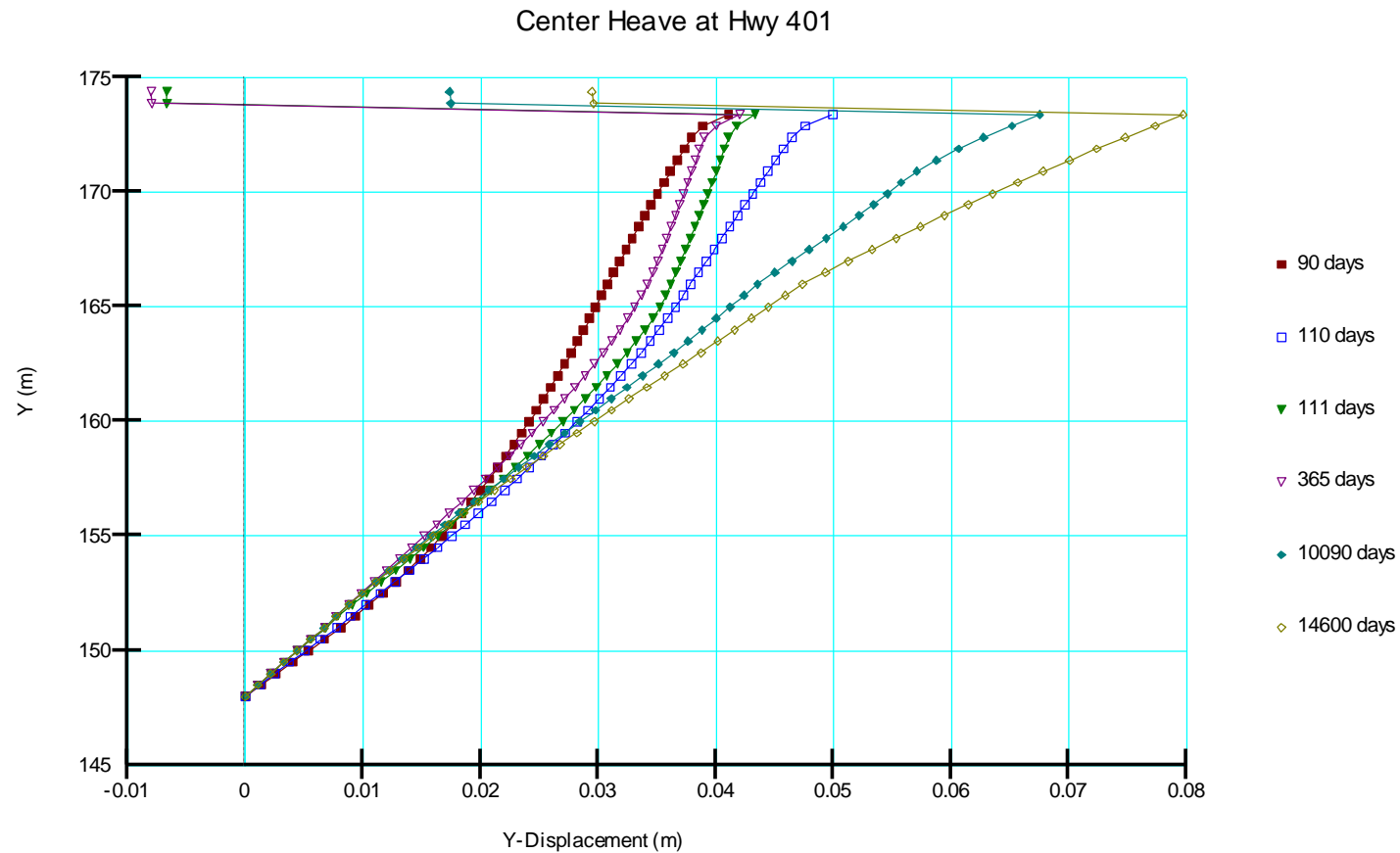
**Figure F1-6. Finite Element Modeling Graph – North Wall Configuration – Cumulative Lateral Displacement RSS Wall Facing - Drained Analysis – Station 10+575L Hwy. 401**



**Legend:**

90 days = End of Excavation  
 110 day = RSS Completion  
 111 days = Pavement Placement (EOC)  
 14600 days = Long Term

**Figure F1-7. Finite Element Modeling Graph – North Wall Configuration – Cumulative Heave Highway 401 - Drained Analysis – Station 10+575L Hwy. 401**

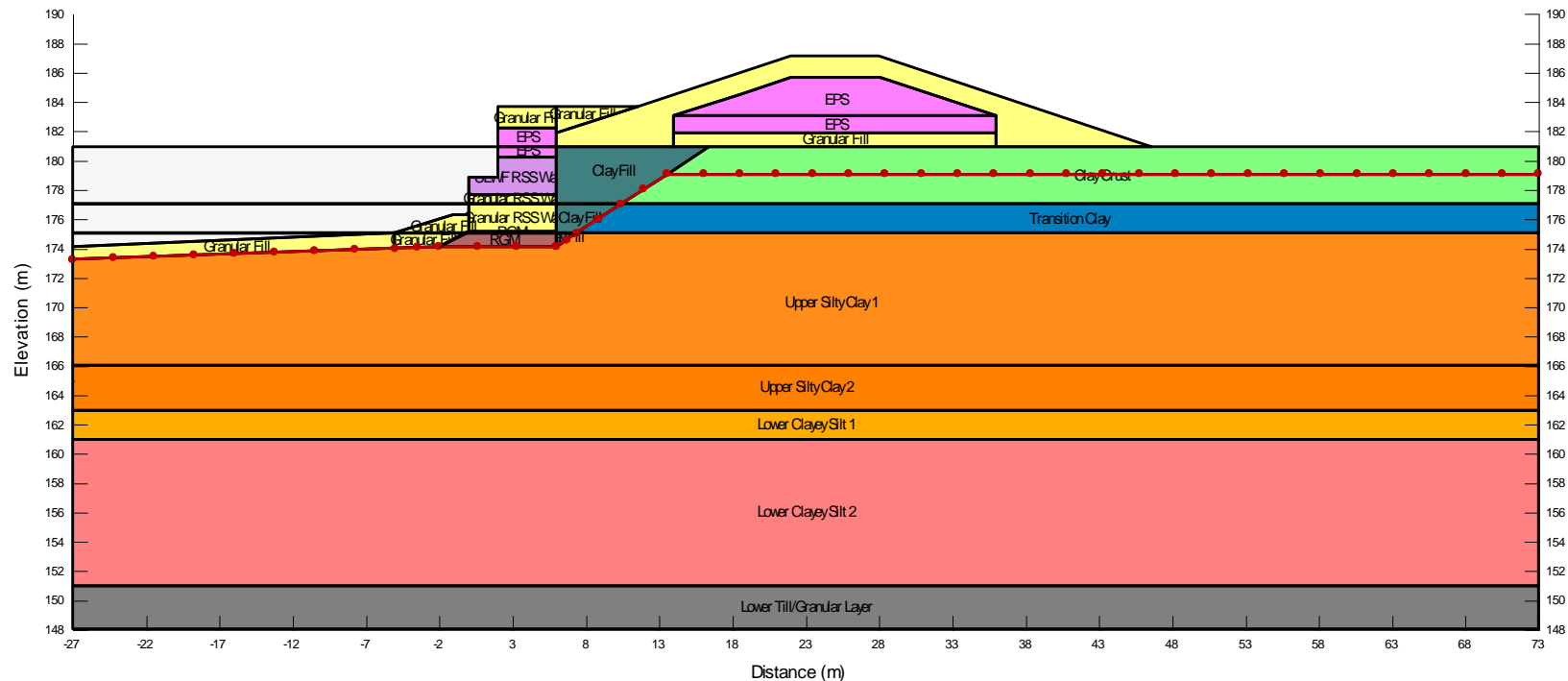


**Legend:**

90 days = End of Excavation  
 110 day = RSS Completion  
 111 days = Pavement Placement (EOC)  
 14600 days = Long Term

**Figure F1-8. Finite Element Modeling Graph – North Wall Configuration – Cumulative Heave Highway 401 - Drained Analysis – Station 10+575L Hwy. 401**

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E): 20300 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
 Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: Transition Poisson's Ratio: 0.35 Cohesion: 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
 Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 2.1 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.62 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
 Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.15 Poisson's Ratio: 0.35 Lambda: 0.052 Kappa: 0.0057 Initial Void Ratio: 0.4 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
 Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
 Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.33 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
 Name: Lower Till/Granular Layer Model: Elastic-Plastic Effective Young's Modulus (E): 40000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa  $\Phi'$ : 30 ° Unit Weight: 19 kN/m<sup>3</sup>  
 Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
 Name: RGM Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m<sup>3</sup> Poisson's Ratio: 0.49  
 Name: ULWF RSS Wall Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.3  
 Name: EPS Model: Linear Elastic Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m<sup>3</sup> Poisson's Ratio: 0.2  
 Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 25000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
 Name: Granular RSS Wall Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.3

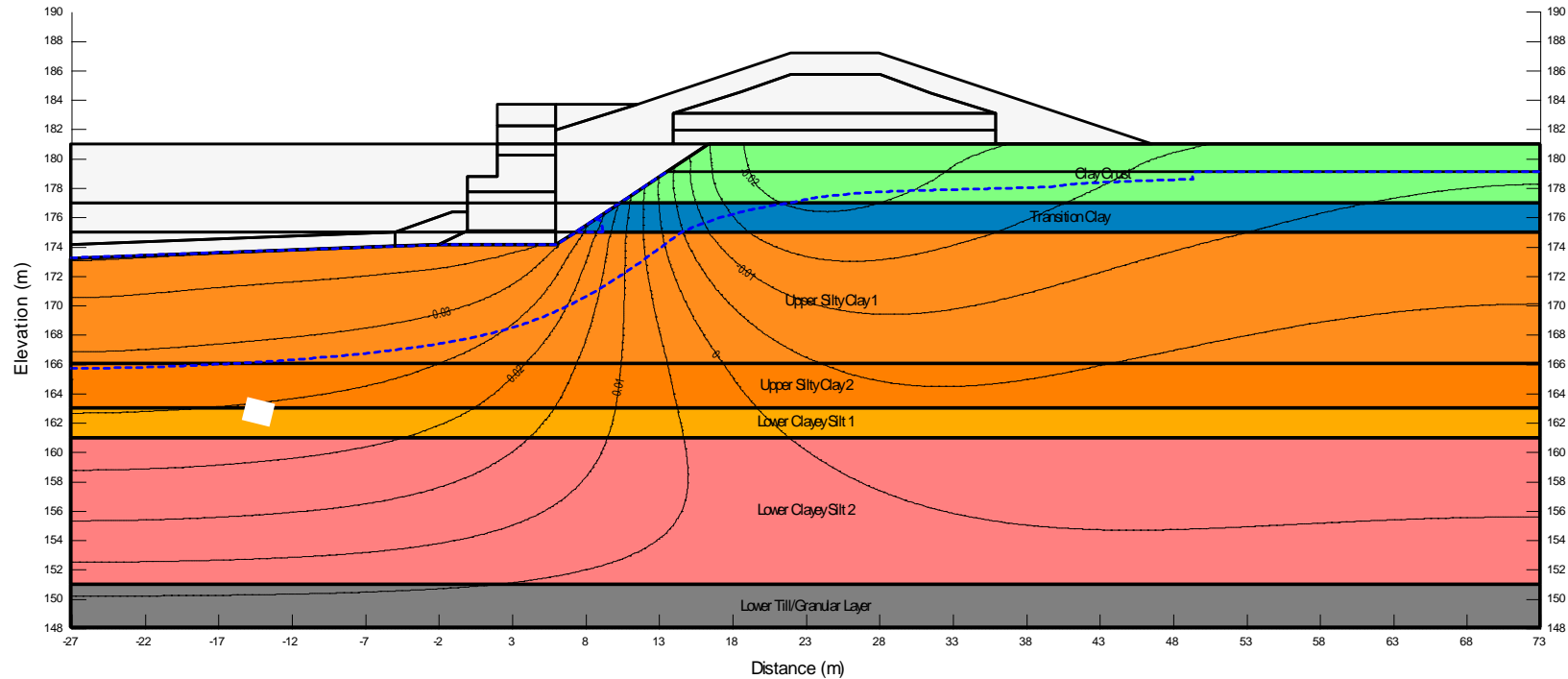


**Figure F2. Finite Element Modeling –South Wall Configuration– Station 10+575L Hwy 401**

Title: Tunnel 7 - Station 10+575L South Wall (Transverse)  
Name: Excavation  
Method: Coupled Stress/PWP

End of Excavation - Vertical Displacement

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E): 20300 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa  $\Phi$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: Transition Poisson's Ratio: 0.35 Cohesion: 0 kPa  $\Phi$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 2.1 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.62 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.15 Poisson's Ratio: 0.35 Lambda: 0.052 Kappa: 0.0057 Initial Void Ratio: 0.4 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.33 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi$ : 26 °  
Name: Lower Till/Granular Layer Model: Elastic-Plastic Effective Young's Modulus (E): 40000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa  $\Phi$ : 30 ° Unit Weight: 19 kN/m<sup>3</sup>

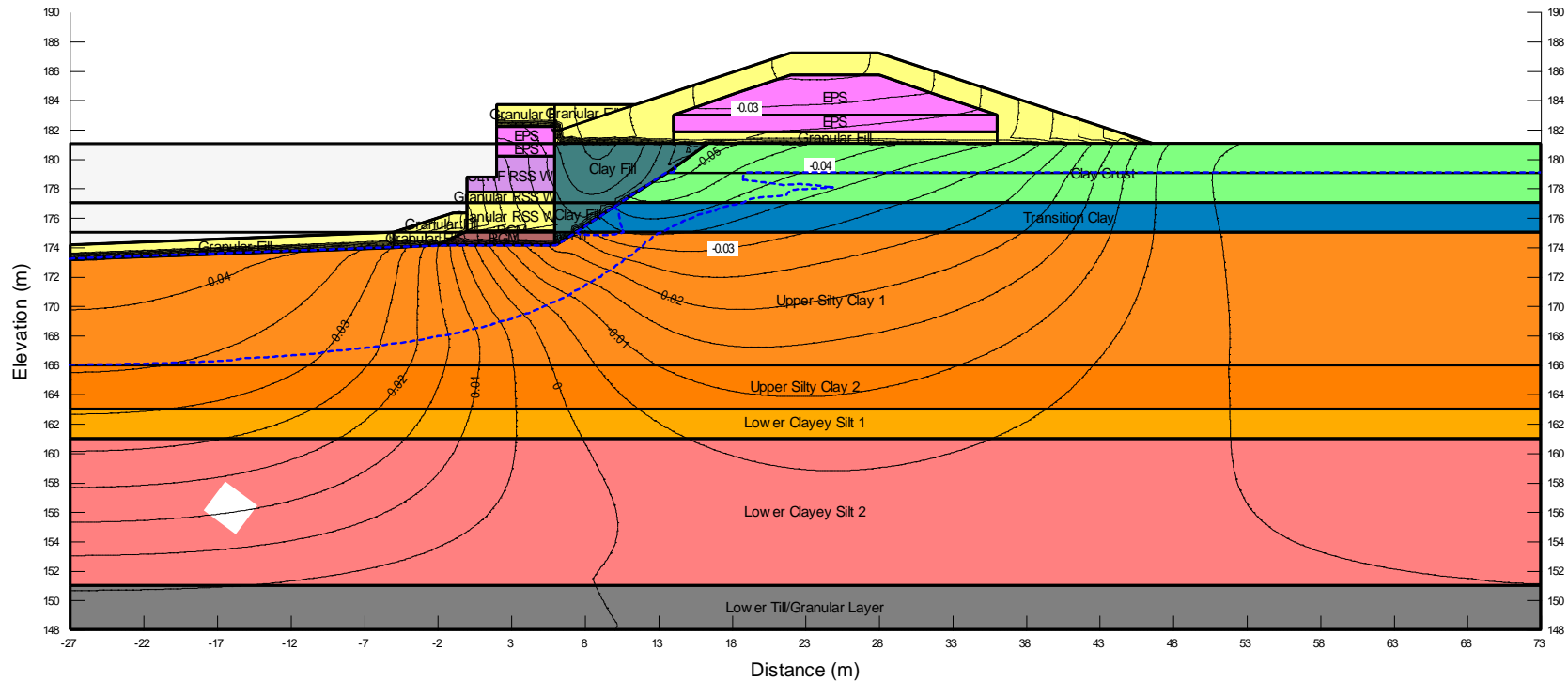


**Figure F2-1. Finite Element Modeling – South Wall Configuration – End of Excavation - Drained Analysis – Station 10+575L Hwy 401**

Title: Tunnel 7 - Station 10+575L South Wall (Transverse)  
Name: Trail Embankment and Hwy 401  
Method: Coupled Stress/PWP

End of Construction - Vertical Displacement

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E'): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: Transition Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 2.1 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.62 Unit Weight: 19.5 kN/m³ Ph  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.15 Poisson's Ratio: 0.35 Lambda: 0.052 Kappa: 0.0057 Initial Void Ratio: 0.4 Unit Weight: 19.5 kN/m³ P  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m³  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.33 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m³  
Name: Lower Till/Granular Layer Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 19 kN/m³  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35  
Name: RGM Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.49  
Name: ULWF RSS Wall Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m³ Poisson's Ratio: 0.3  
Name: EPS Model: Linear Elastic Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m³ Poisson's Ratio: 0.2  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 25000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35  
Name: Granular RSS Wall Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.3



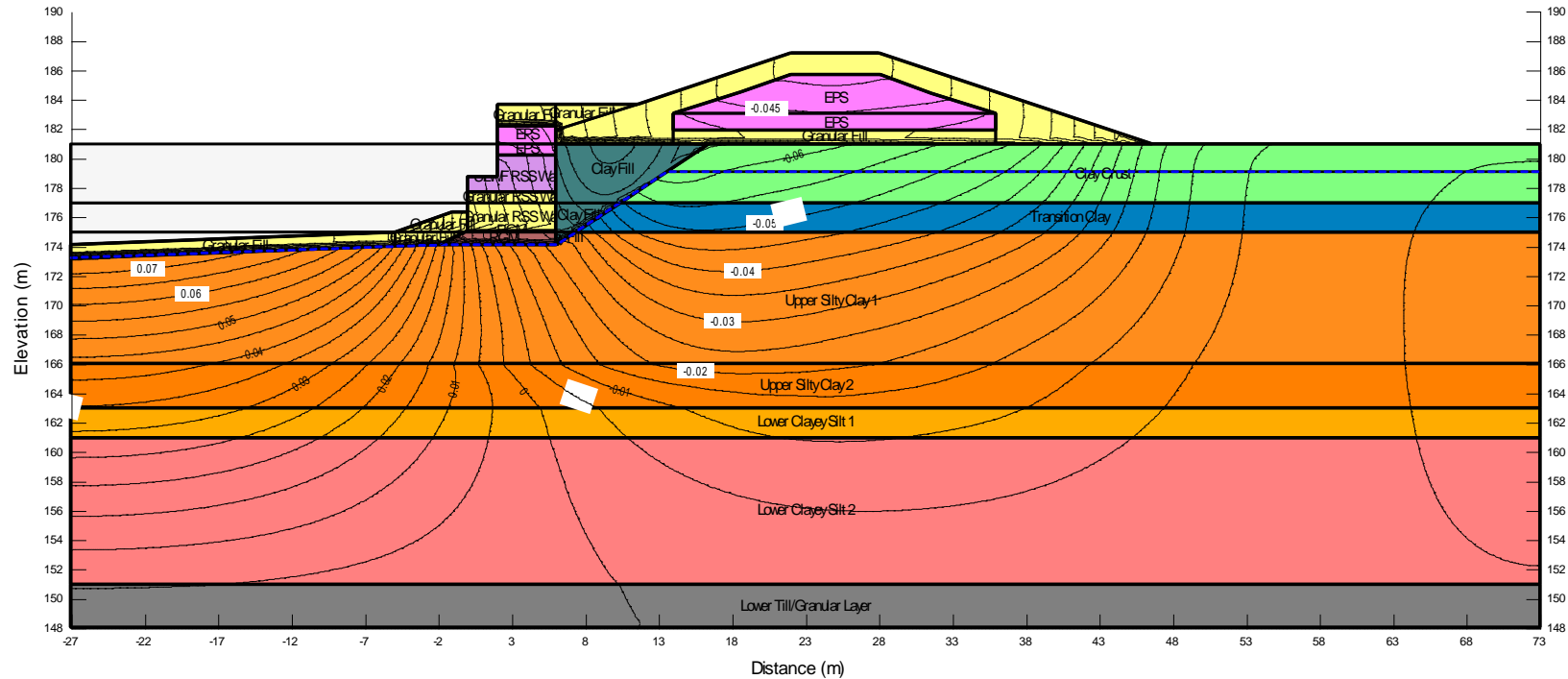
**Figure F2-2. Finite Element Modeling – South Wall Configuration – End of Construction - Drained Analysis – Station 10+575L Hwy 401**



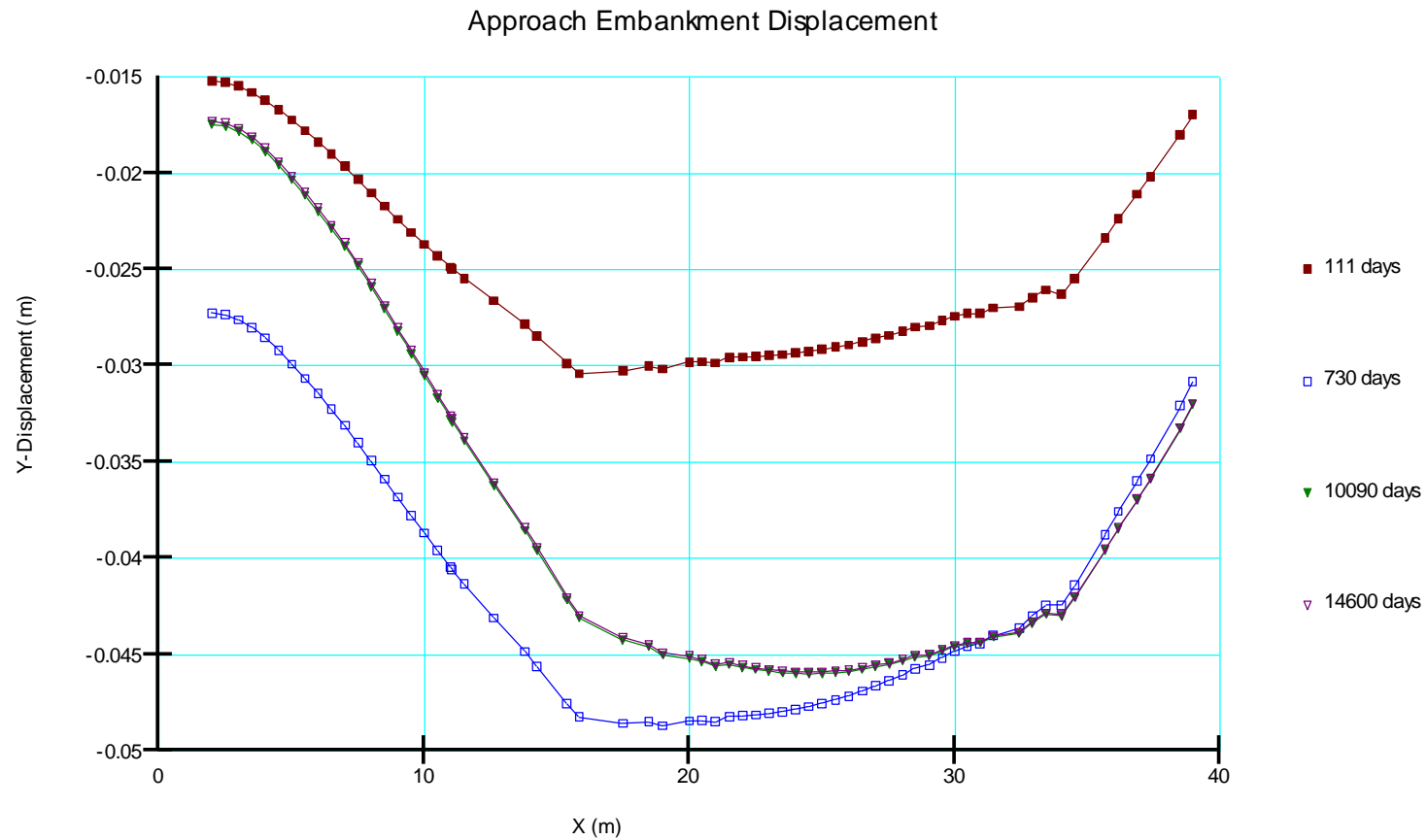
Title: Tunnel 7 - Station 10+575L South Wall (Transverse)  
Name: Trail Embankment and Hwy 401  
Method: Coupled Stress/PWP

Long Term - Vertical Displacement

Name: Clay Crust Model: Elastic-Plastic Effective Young's Modulus (E): 20300 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Transition Clay Model: Elastic-Plastic Effective E-Modulus Function: Transition Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 22 kN/m<sup>3</sup>  
Name: Upper Silty Clay 1 Model: Soft Clay (MCC) O.C. Ratio: 2.1 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.62 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Upper Silty Clay 2 Model: Soft Clay (MCC) O.C. Ratio: 1.15 Poisson's Ratio: 0.35 Lambda: 0.052 Kappa: 0.0057 Initial Void Ratio: 0.4 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 1 Model: Soft Clay (MCC) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Clayey Silt 2 Model: Soft Clay (MCC) O.C. Ratio: 1.33 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.0069 Initial Void Ratio: 0.48 Unit Weight: 19.5 kN/m<sup>3</sup>  $\Phi'$ : 26 °  
Name: Lower Till/Granular Layer Model: Elastic-Plastic Effective Young's Modulus (E): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa  $\Phi'$ : 30 ° Unit Weight: 19 kN/m<sup>3</sup>  
Name: Granular Fill Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: RGM Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m<sup>3</sup> Poisson's Ratio: 0.49  
Name: ULWF RSS Wall Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 12 kN/m<sup>3</sup> Poisson's Ratio: 0.3  
Name: EPS Model: Linear Elastic Young's Modulus (E): 10000 kPa Unit Weight: 0.5 kN/m<sup>3</sup> Poisson's Ratio: 0.2  
Name: Clay Fill Model: Linear Elastic Young's Modulus (E): 25000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.35  
Name: Granular RSS Wall Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 21 kN/m<sup>3</sup> Poisson's Ratio: 0.3

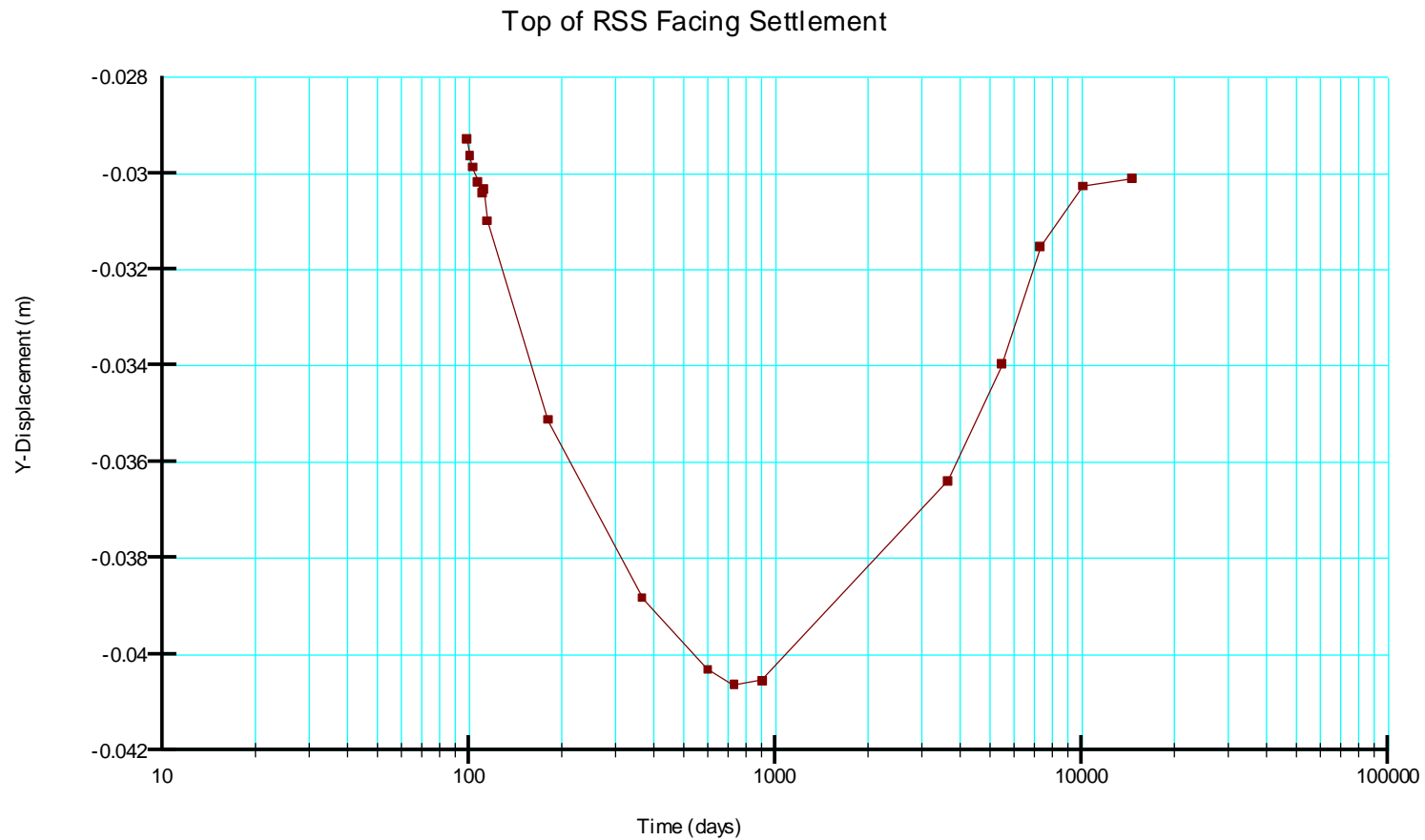


**Figure F2-3. Finite Element Modeling – South Wall Configuration – Long Term - Drained Analysis – Station 10+575L Hwy 401**



**Legend:**  
111 days = Pavement Placement (EOC)  
14600 days = Long Term

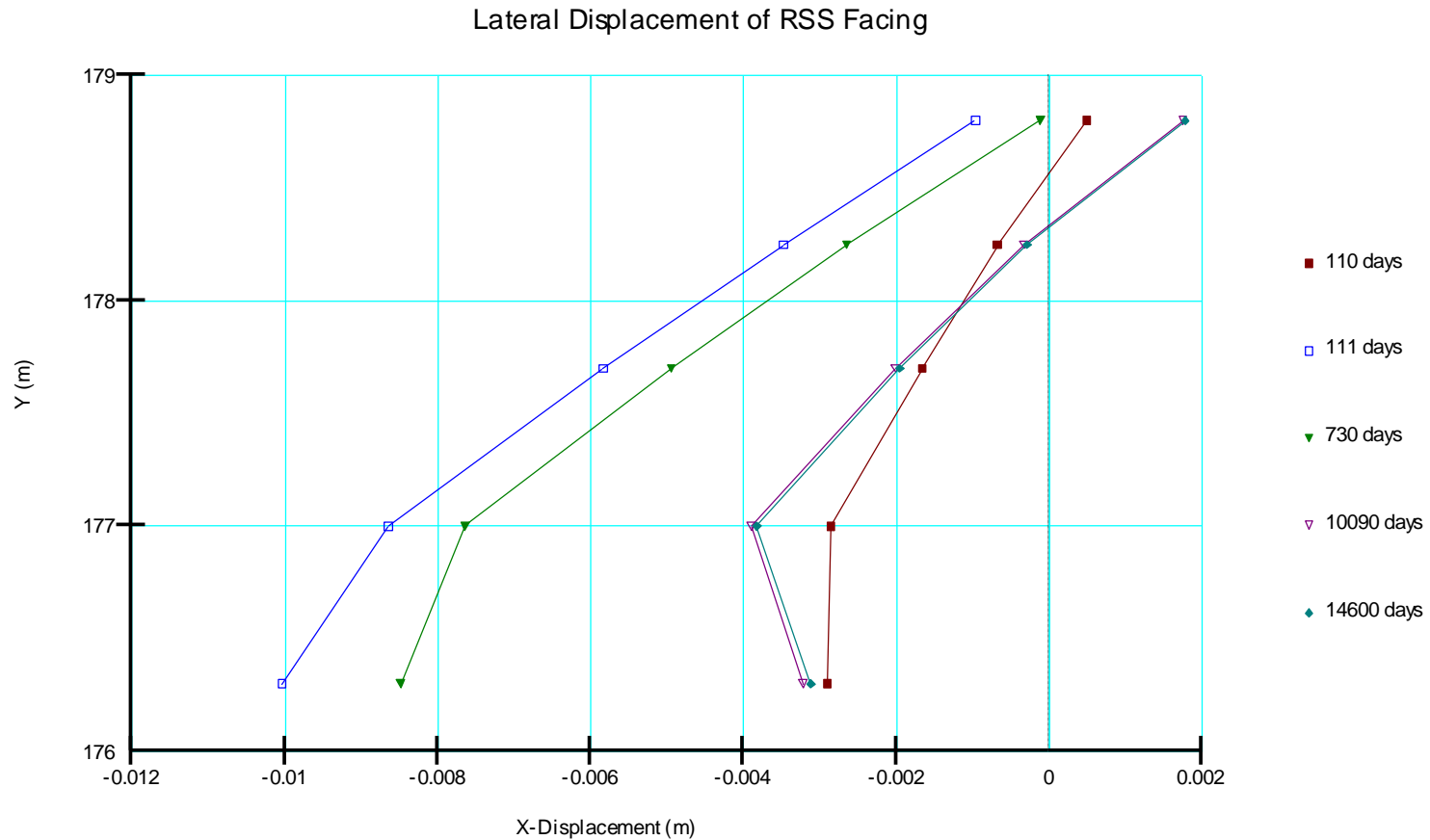
**Figure F2-4. Finite Element Modeling Graph – South Wall Configuration – Cumulative Approach Embankment Settlement - Drained Analysis – Station 10+575L Hwy. 401**



**Legend:**

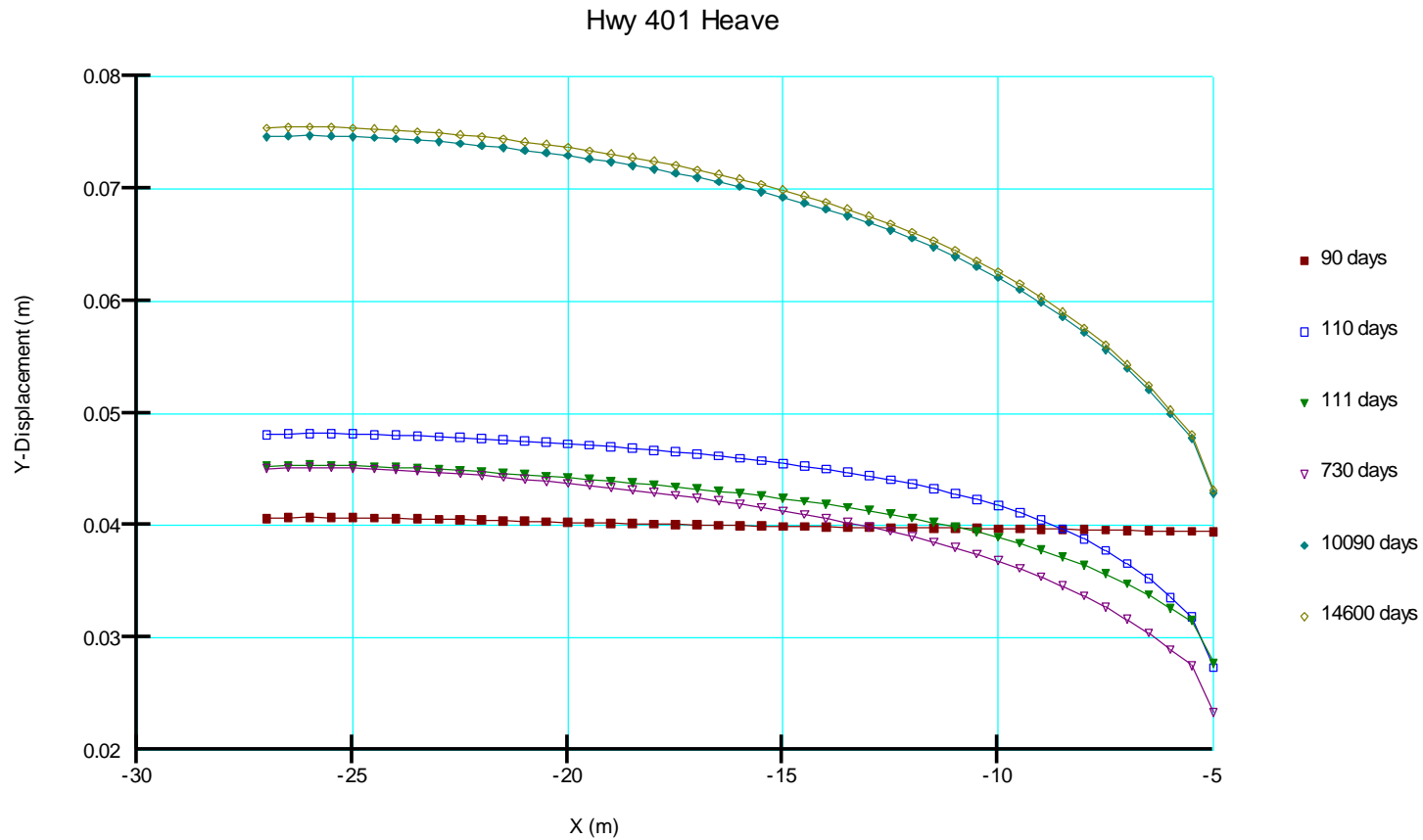
110 days = RSS completion  
111 days = Pavement Placement (EOC)  
14600 days = Long Term

**Figure F2-5. Finite Element Modeling Graph – South Wall Configuration – Cumulative Settlement Top of RSS Wall Facing - Drained Analysis – Station 10+575L Hwy. 401**



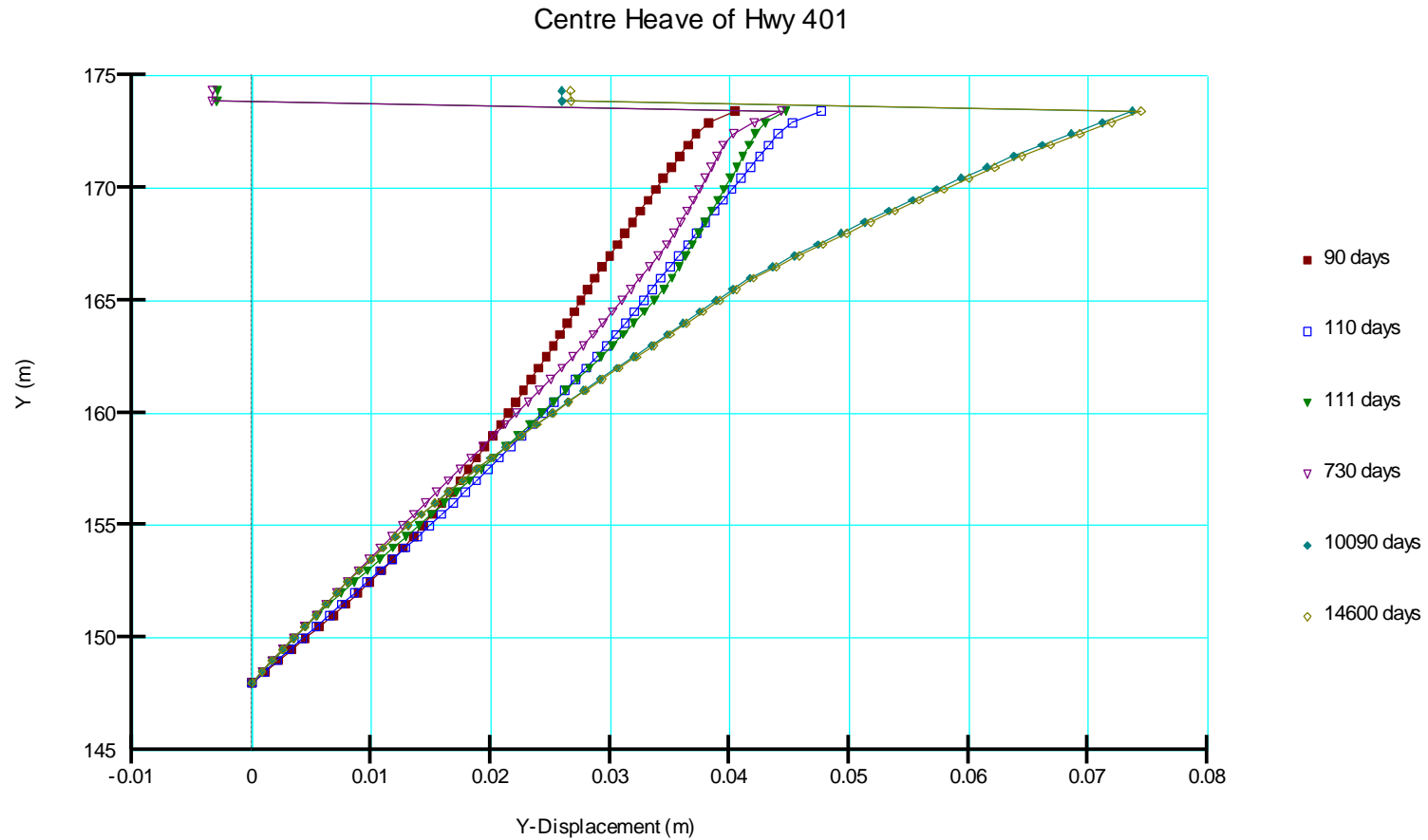
**Legend:**  
 110 days = RSS completion  
 111 days = Pavement Placement (EOC)  
 14600 days = Long Term

**Figure F2-6. Finite Element Modeling Graph – South Wall Configuration – Cumulative Lateral Displacement RSS Wall Facing - Drained Analysis – Station 10+575L Hwy. 401**



**Legend:**  
 90 days = End of Excavation  
 110 day = RSS Completion  
 111 days = Pavement Placement (EOC)  
 14600 days = Long Term

**Figure F2-7. Finite Element Modeling Graph – South Wall Configuration – Cumulative Heave Highway 401 - Drained Analysis – Station 10+575L Hwy. 401**

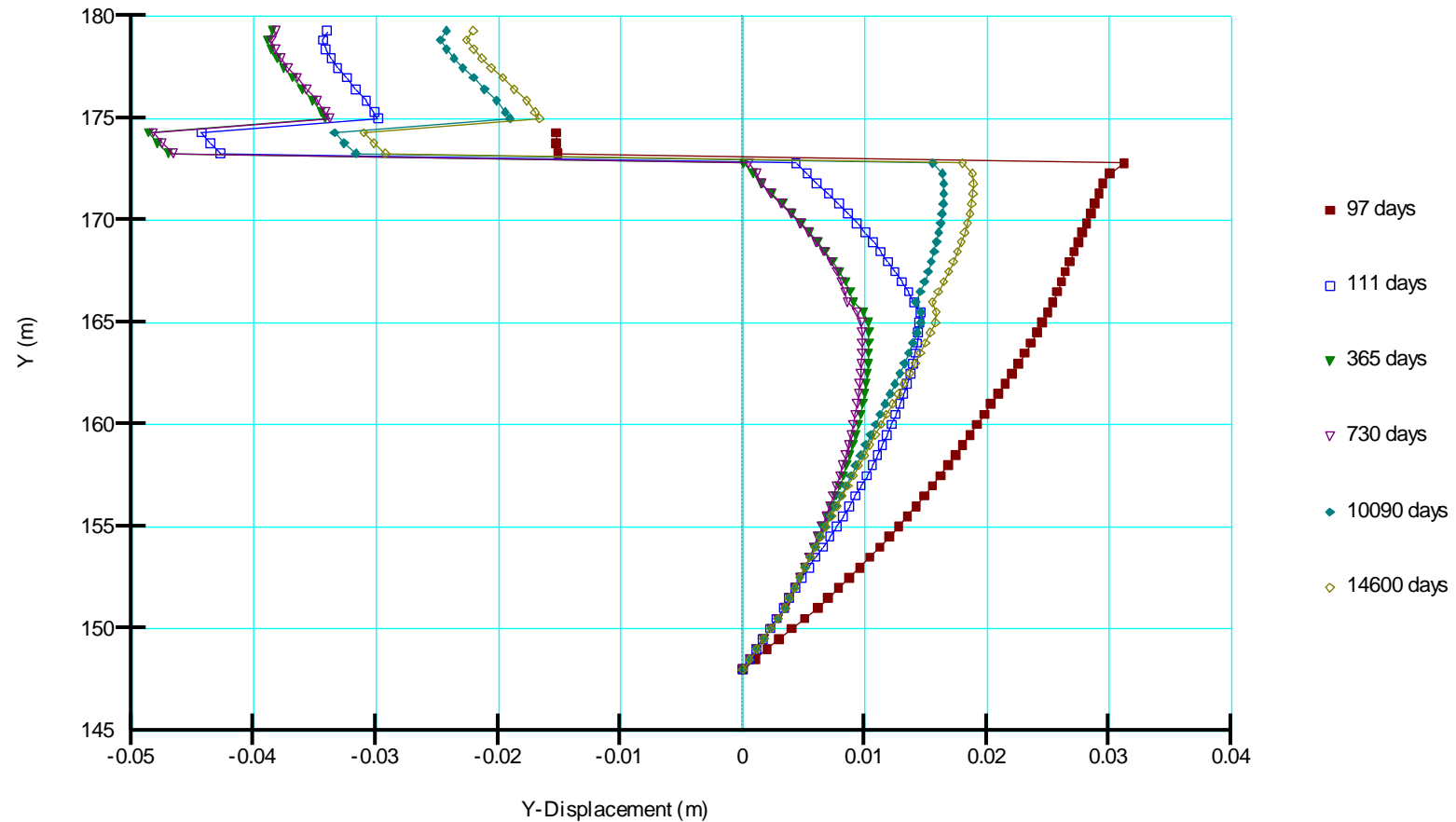


**Legend:**

90 days = End of Excavation  
 110 day = RSS Completion  
 111 days = Pavement Placement (EOC)  
 14600 days = Long Term

**Figure F2-8. Finite Element Modeling Graph – South Wall Configuration – Cumulative Heave Highway 401 - Drained Analysis – Station 10+575L Hwy. 401**

### Verticle Soil Displacement at Pile Location North RSS wall

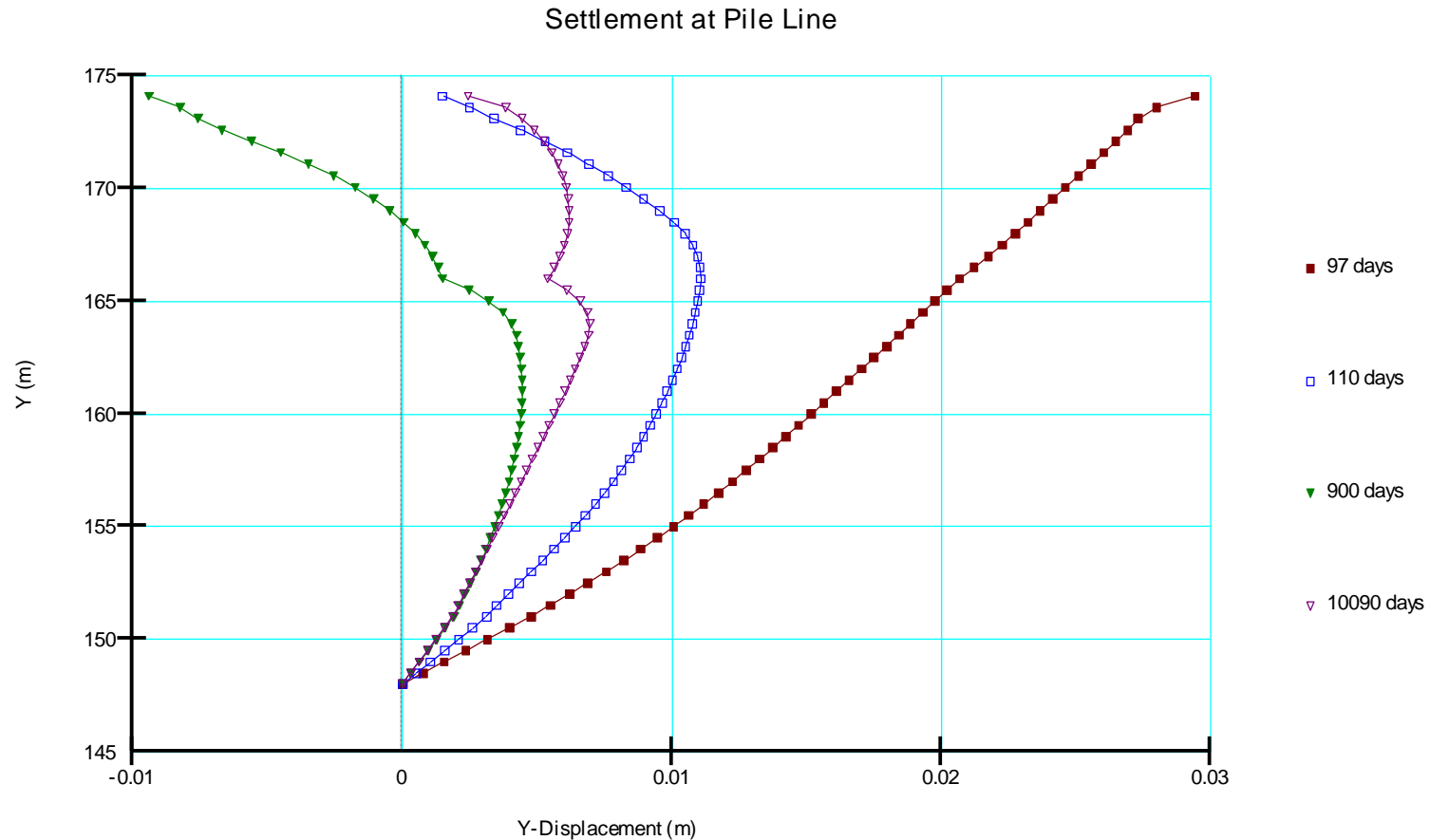


**Legend:**

97 days = RGM Completion / Start of pile Installation  
 111 days = Pavement Placement (EOC)  
 14600 days = Long Term

**Figure F3-1. Finite Element Modeling Graph – North Wall Configuration – Cumulative Displacement at Pile Location - Drained Analysis – Station 10+575L Hwy. 401**

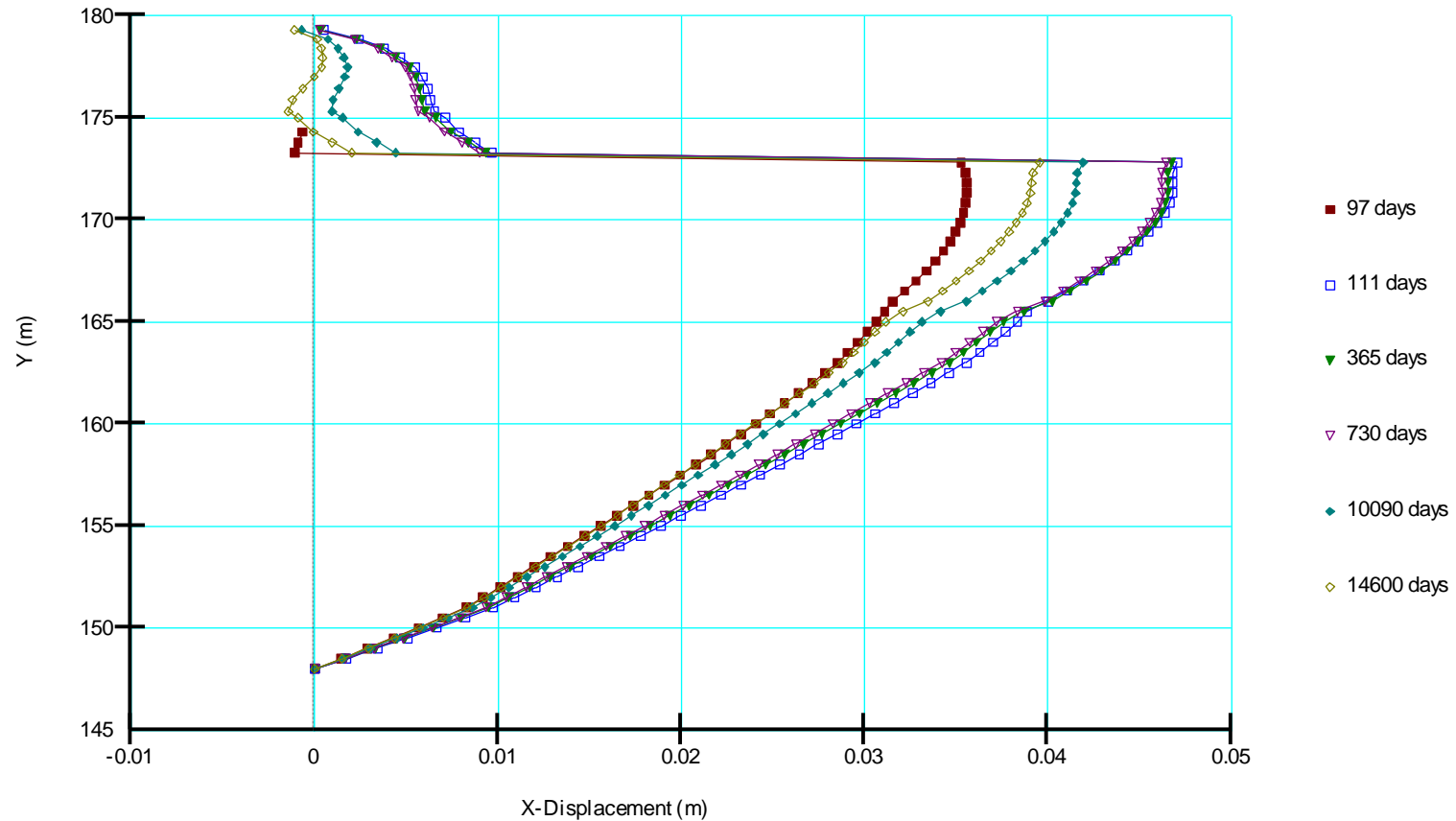




**Legend:**  
97 days = RGM Completion / Start of pile Installation  
111 days = Pavement Placement (EOC)  
14600 days = Long Term

**Figure F3-2. Finite Element Modeling Graph – South Wall Configuration – Cumulative Settlement at Pile Location - Drained Analysis – Station 10+575L Hwy. 401**

### Lateral Soil Displacement at Pile Location North RSS wall



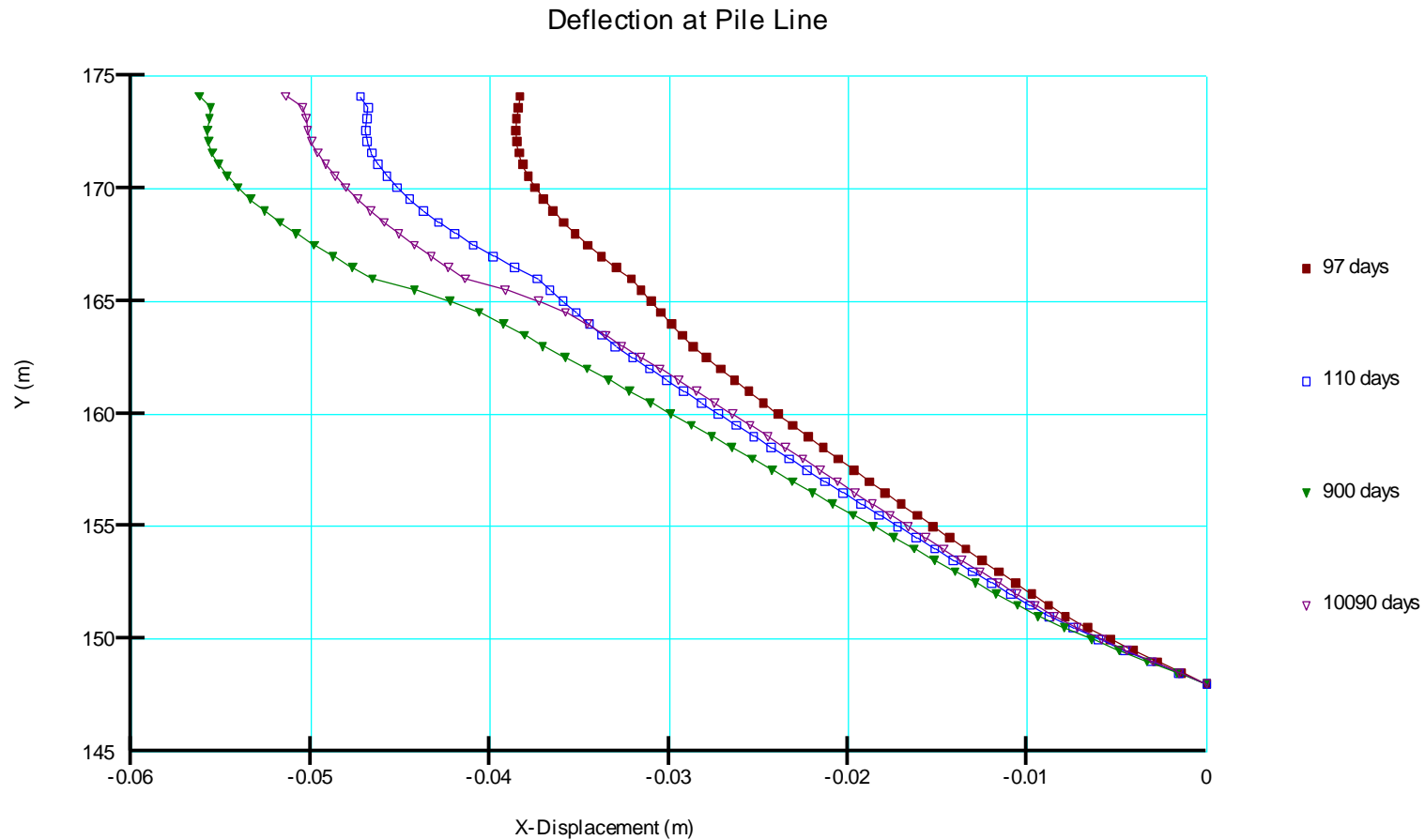
**Legend:**

97 days = RGM Completion / Start of pile Installation

111 days = Pavement Placement (EOC)

14600 days = Long Term

**Figure F4-1. Finite Element Modeling Graph – North Wall Configuration – Cumulative Deflection at Pile Location - Drained Analysis – Station 10+575L Hwy. 401**



**Legend:**

97 days = RGM Completion / Start of pile Installation

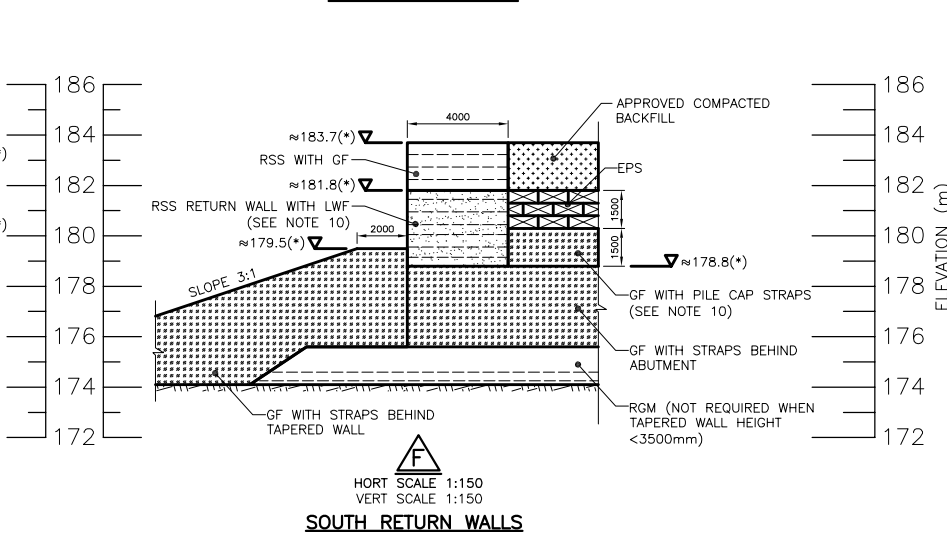
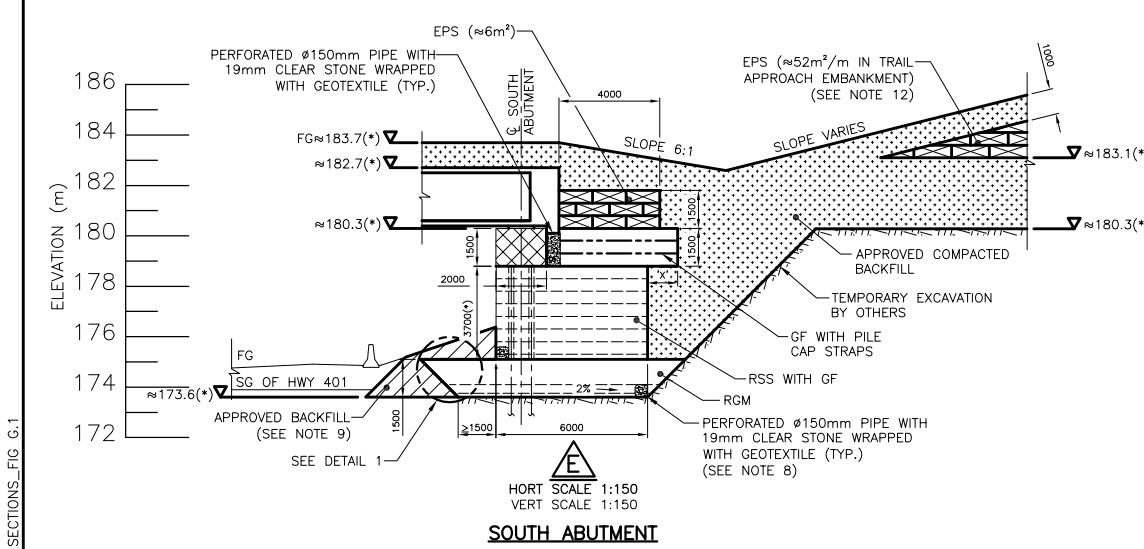
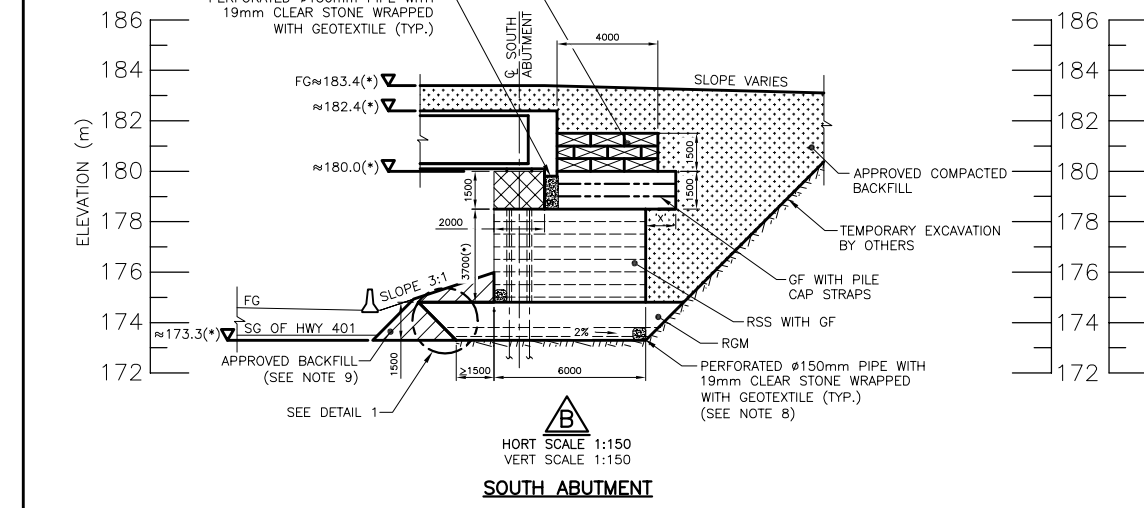
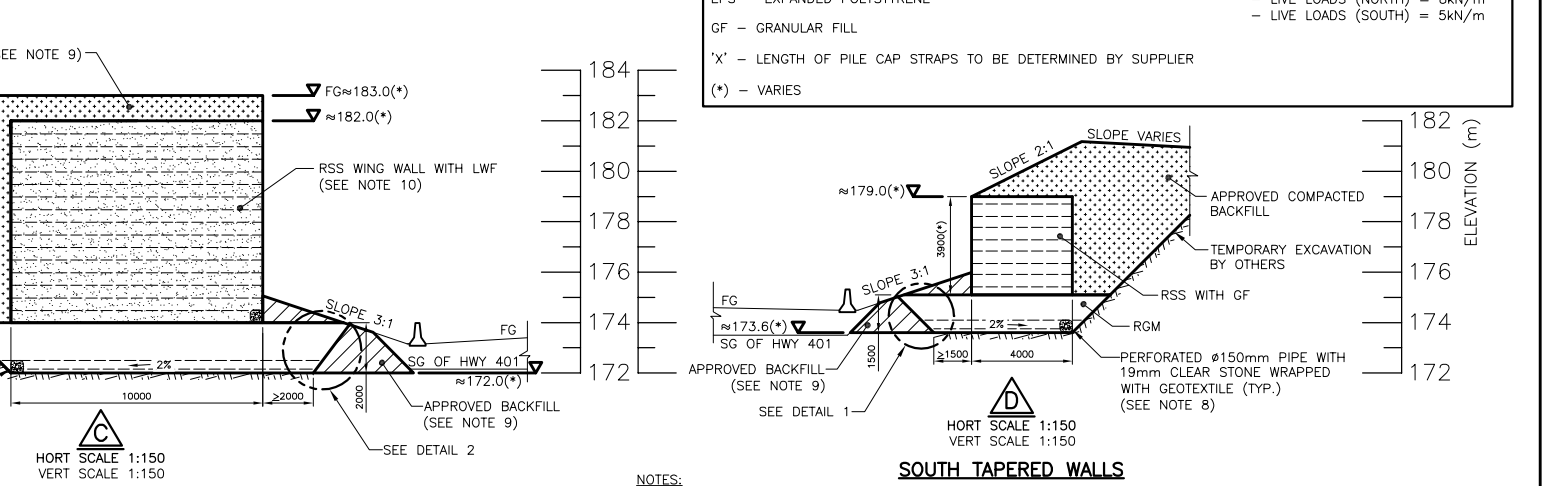
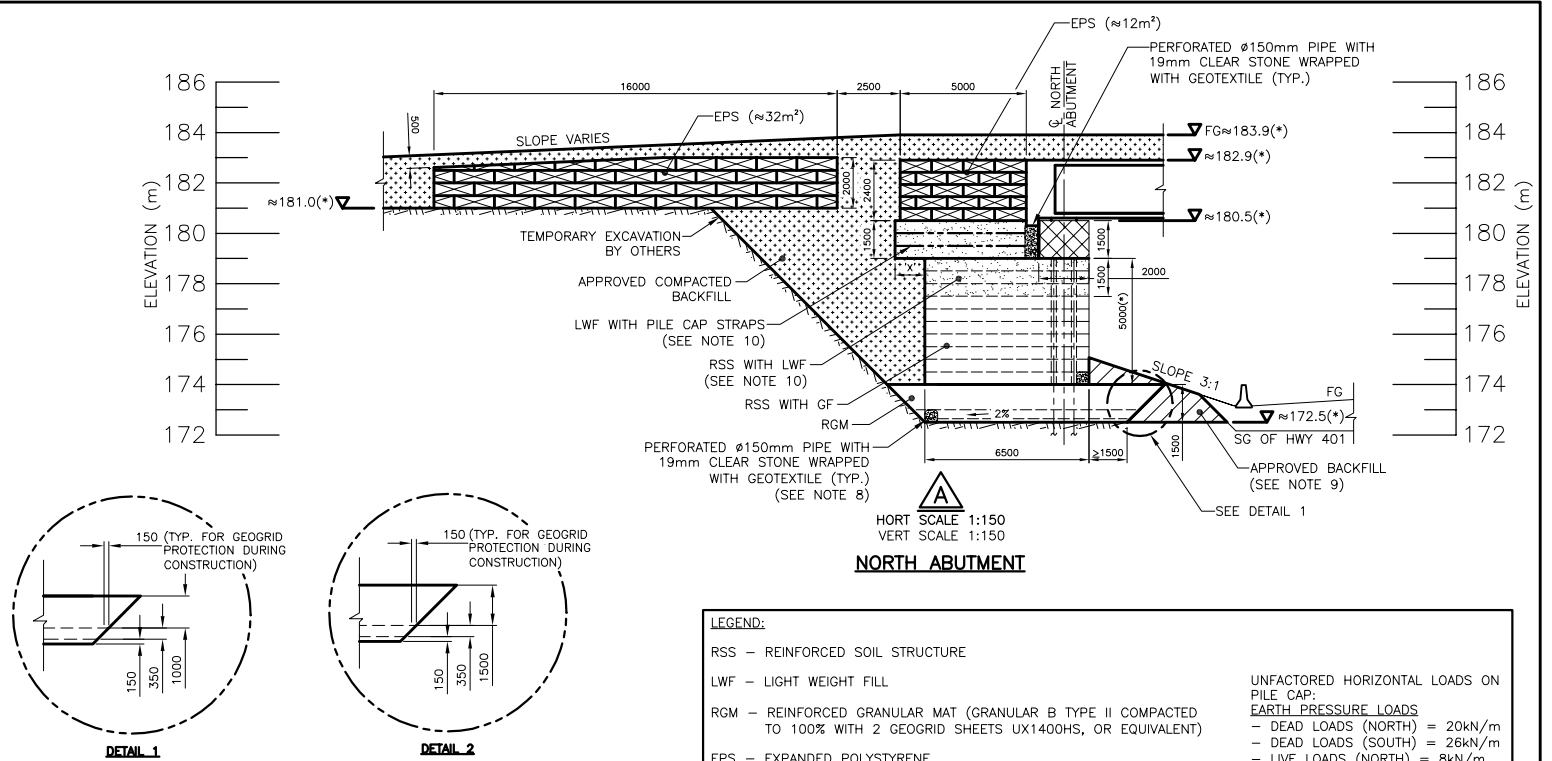
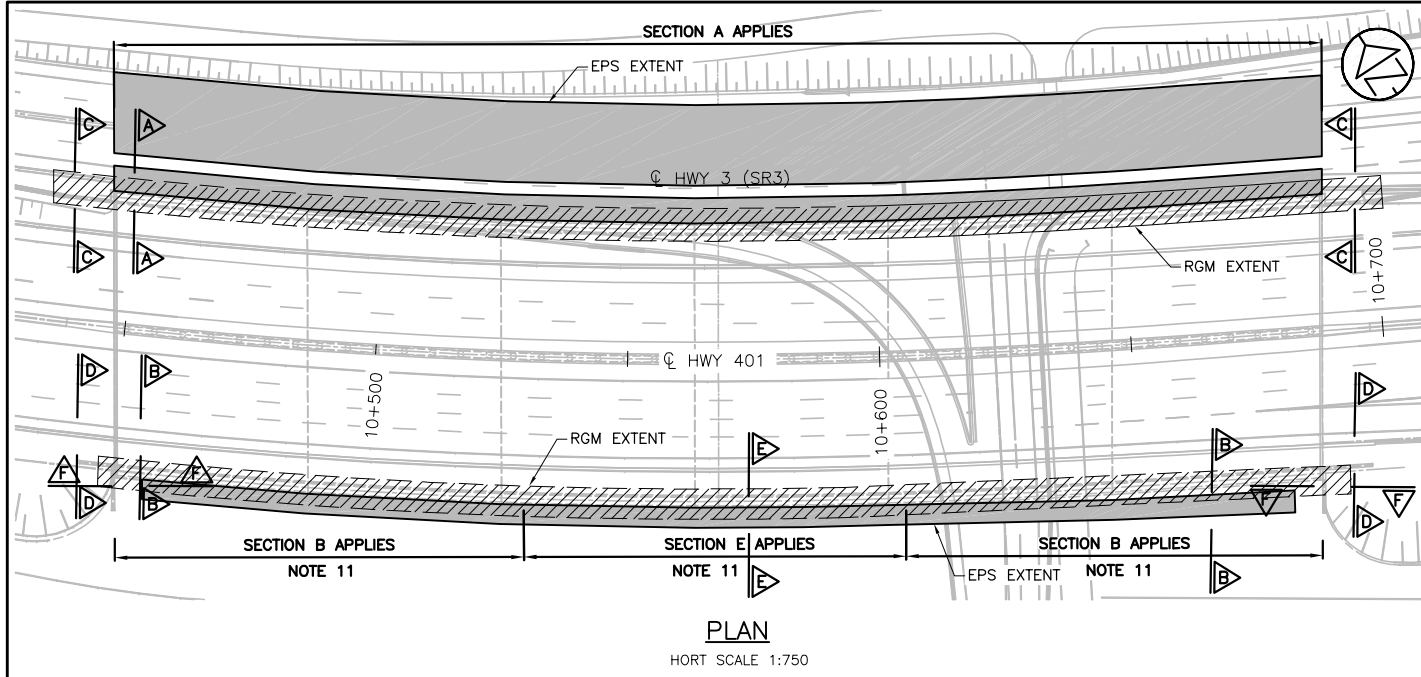
111 days = Pavement Placement (EOC)

14600 days = Long Term

**Figure F4-2. Finite Element Modeling Graph – South Wall Configuration – Cumulative Deflection at Pile Location - Drained Analysis – Station 10+575L Hwy. 401**

## **Appendix G: Conceptual Drawings**

DOC: T-7 PLAN W ABMT SECTIONS-FIG G.1



- LEGEND:**
- RSS - REINFORCED SOIL STRUCTURE
  - LWF - LIGHT WEIGHT FILL
  - RGM - REINFORCED GRANULAR MAT (GRANULAR B TYPE II COMPACTED TO 100% WITH 2 GEOGRID SHEETS UX1400HS, OR EQUIVALENT)
  - EPS - EXPANDED POLYSTYRENE
  - GF - GRANULAR FILL
  - 'X' - LENGTH OF PILE CAP STRAPS TO BE DETERMINED BY SUPPLIER
  - (\*) - VARIES
- UNFACTORED HORIZONTAL LOADS ON PILE CAP:  
**EARTH PRESSURE LOADS**  
- DEAD LOADS (NORTH) = 20kN/m  
- DEAD LOADS (SOUTH) = 26kN/m  
- LIVE LOADS (NORTH) = 8kN/m  
- LIVE LOADS (SOUTH) = 5kN/m

- NOTES:**
- THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
  - THIS DRAWING ILLUSTRATES THE GENERAL ARRANGEMENTS AT SELECTED REPRESENTATIVE LOCATIONS OF THE NORTH AND SOUTH ABUTMENTS OF TUNNEL T-7 BASED ON GEOTECHNICAL DESIGN ANALYSES.
  - THE ILLUSTRATED RSS WALL WIDTH AND RGM DIMENSIONS REPRESENT THE MINIMUM DIMENSIONS BASED ON GLOBAL STABILITY REQUIREMENTS. THE DESIGN OF THE REINFORCED FILL STRUCTURES BY OTHERS.
  - TUNNEL ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGN WERE INTERPRETED FROM LIMITED INFORMATION INDICATED ON STRUCTURAL DRAWINGS AVAILABLE IN DECEMBER 2011. ABUTMENT ELEVATIONS VARY ALONG TUNNEL T-7.
  - 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.
  - 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.
  - FOR DETAILS REGARDING FILL CONSTRUCTION, SEE APPLICABLE CONSTRUCTION NOTES.
  - HIGHEST SUBDRAIN ELEVATION WITHIN RGM SHALL BE AT LEAST 1m BELOW THE TOP OF RGM.
  - ABUTMENT BACKFILL ABOVE SEAT ELEVATION SHALL BE PLACED ONLY AFTER THE PLACEMENT OF THE GRANULAR BASE OVER HWY 401 AND SUBSTANTIAL COMPLETION OF THE RSS TOE BERM.
  - ALL LWF SHALL BE ENCAPSULATED IN FILTER FABRIC (TERRAFIX 270R, OR EQUIVALENT.)
  - EXTENTS OF ZONES FOR SECTION 'B' AND 'E' TO BE COORDINATED WITH ACTUAL LOCATION AND EXTENT OF WEST APPROACH EMBANKMENT AT TB-5.
  - FOR DETAILS ON EPS IN TRAIL EMBANKMENT SEE CORRESPONDING TRAIL DRAWINGS.

## **Appendix H: Rock Core Photographs**



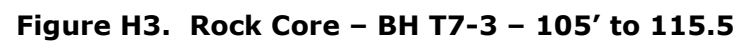


**Figure H1. Rock Core – BH T7-1 – 107.8' to 117.8'**





**Figure H2. Rock Core – BH T7-2 – 106' to 115'**



## **Appendix I: SEEP Model Results**

Title: Tunnel 7 - Station 10+575L South Wall (Transverse)  
Name: Steady-State Seepage  
Method: Steady-State

Name: Clay Crust Model: Saturated Only K-Sat: 0.00029 m/days Volumetric Water Content: 0.35 m<sup>3</sup>/m<sup>3</sup> Mv: 0 kPa  
Name: Transition Clay Model: Saturated Only K-Sat: 0.00029 m/days Volumetric Water Content: 0.35 m<sup>3</sup>/m<sup>3</sup> Mv: 0 kPa  
Name: Upper Silty Clay 1 Model: Saturated Only K-Sat: 9.2e-005 m/days Volumetric Water Content: 0.38 m<sup>3</sup>/m<sup>3</sup> Mv: 0 kPa  
Name: Upper Silty Clay 2 Model: Saturated Only K-Sat: 9.2e-005 m/days Volumetric Water Content: 0.29 m<sup>3</sup>/m<sup>3</sup> Mv: 0 kPa  
Name: Lower Clayey Silt 1 Model: Saturated Only K-Sat: 9.2e-005 m/days Volumetric Water Content: 0.33 m<sup>3</sup>/m<sup>3</sup> Mv: 0 kPa  
Name: Lower Clayey Silt 2 Model: Saturated Only K-Sat: 9.2e-005 m/days Volumetric Water Content: 0.33 m<sup>3</sup>/m<sup>3</sup> Mv: 0 kPa  
Name: Lower Till/Granular Layer Model: Saturated Only K-Sat: 1 m/days Volumetric Water Content: 0.41 m<sup>3</sup>/m<sup>3</sup> Mv: 0 kPa

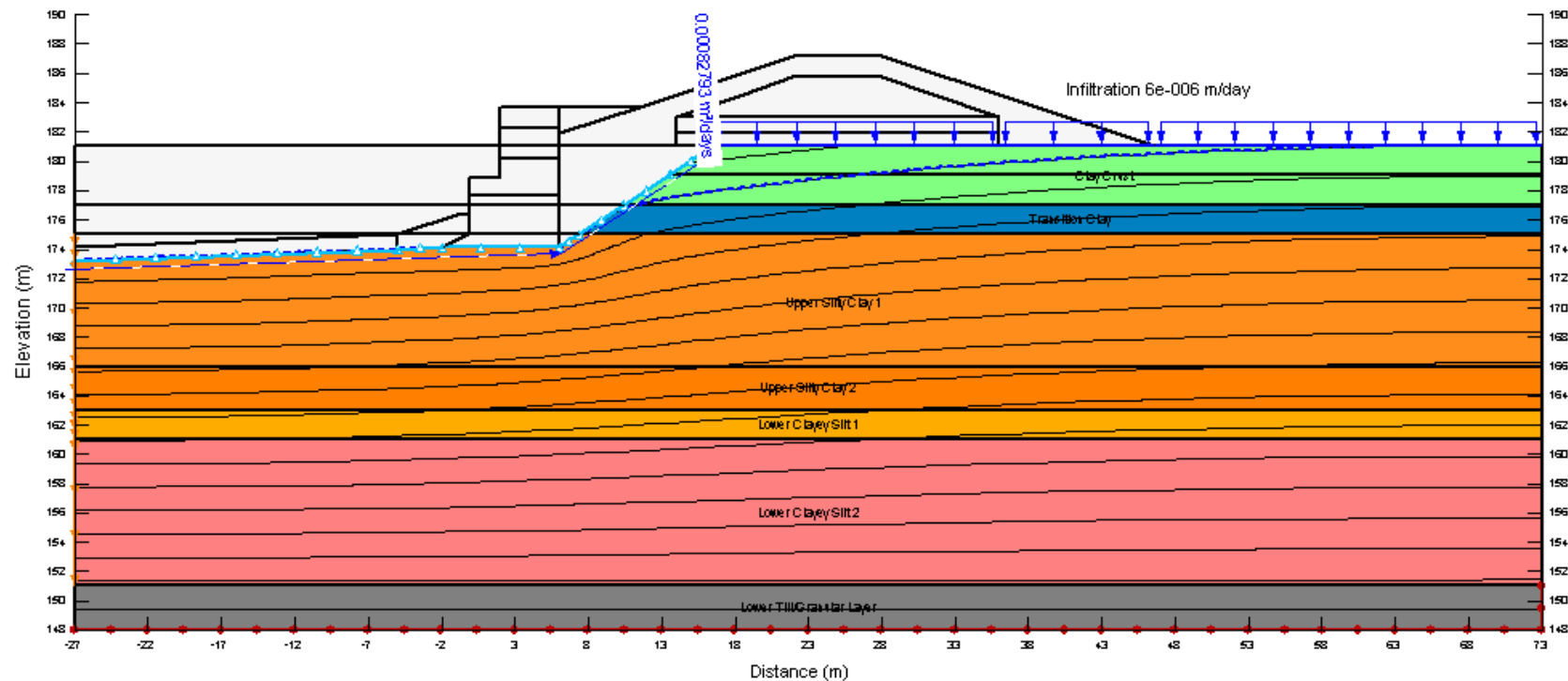


Figure I-1. Seepage Analyses – Station 10+575L Hwy 401