

**The Windsor-Essex Parkway Project
Geotechnical Investigation and
Design Report – Tunnel T-5
(Sta. 14+510W to 14+630W)**

Geocres No. 40J6-36

May / 2012

The Windsor-Essex Parkway Project Geotechnical Investigation and Design Report – Tunnel T-5 (Sta. 14+510W to 14+630W)

Geocres No. 40J6-36


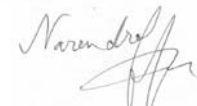

May / 2012

Hatch Mott MacDonald
2800 Speakman Drive
Mississauga, Ontario L5K 2R7
Canada
Tel: 905 855 2010
Fax: 905 855 2607

The Windsor-Essex Parkway Project Geotechnical Investigation and Design Report – Tunnel T-5 (Sta. 14+510W to 14+630W)

Geocres No. 40J6-36

Revision History					
Revision	Date	Status	Prepared By	Checked By	Reviewed By
0	05/25/2012	Issued for Construction	SF	DD	NSV

	Name, Title	Signature	Date
Prepared By	Siavash Farhangi, Ph.D., P.Eng., Senior Geotechnical Engineer		05/25/2012
Reviewed By	Narendra Verma, Ph.D., P.Eng., F.ASCE, D.GE. Principal Geotechnical Engineer (Designated MTO RAQS Contact)		05/25/2012
Approved By	Brian Lapos, M.Sc., P.Eng., Geotechnical Engineer (Project Manager, AMEC)		05/25/2012

This document has been prepared for the titled project or named part thereof and should not be relied upon or used for any other project without an independent check being carried out as to its suitability and prior written authority of HMM being obtained. HMM accepts no responsibility or liability for the consequence of this document being used for a purpose other than the purposes for which it was commissioned. Any person using or relying on the document for such other purpose agrees, and will by such use or reliance be taken to confirm his agreement to indemnify HMM for all loss or damage resulting therefrom. HMM accepts no responsibility or liability for this document to any party other than the person by whom it was commissioned.



Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

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

List of Contents and Appendices

1	Introduction	1
1.1	Preface.....	1
1.2	Report Introduction.....	2
2	Background Information	4
2.1	Geological Setting	4
2.2	Site Seismic Background	5
2.3	Existing Site Conditions and Proposed Tunnel Layout	5
2.4	Frost Depth.....	5
3	Geotechnical Investigations.....	6
3.1	Scope and Procedures of Geotechnical Investigations.....	6
3.2	Fieldwork	6
3.3	Instrumentation.....	8
3.4	Geotechnical and Analytical Laboratory Testing	9
3.5	Data Interpretation	9
4	Subsurface Conditions	12
4.1	Surficial Fills, Topsoil and Upper Granular Deposit	12
4.2	Silty Clay to Clayey Silt Stratum.....	12
4.3	Lower Granular Deposit.....	15
4.4	Bedrock	15
4.5	Groundwater Conditions	16
4.6	Subsurface Gases	17
5	Development of Geotechnical Designs	19
5.1	Tunnel Configuration.....	19
5.2	Geotechnical Design Criteria and Considerations	20
5.3	Design Soil Properties.....	20
5.4	Excavation and Temporary Cut Slopes	21
5.5	Pile Foundations	22
5.5.1	Resistance to Axial Loads	22
5.5.2	ULS and SLS Resistance to Lateral Loads.....	23
5.5.3	Soil Pile Interaction Assessment.....	27
5.5.4	Pile Cap/Abutment Stem Anchoring.....	29

5.6	RSS False Abutment Walls	30
5.6.1	Global Stability.....	31
5.6.2	Stress Deformation Analyses	32
5.6.3	Serviceability Limit States (SLS) Assessment	34
5.6.4	RSS Wall External Stability	35
5.6.5	RGM Foundation – Loads & Design	36
5.7	Wingwalls	37
5.8	Backfilling.....	38
5.9	Permanent Subdrainage System	40
6	Other Geotechnical Recommendations	41
6.1	General Construction Requirements	41
6.2	Construction Dewatering	42
6.3	Instrumentation and Monitoring during Construction	42
6.4	Corrosion Potential	43
6.5	Construction Quality Control	44
7	Limitations of Report.....	45
8	Closure	47
9	References	48

List of Tables

Table 3-1: Test Holes At and Around Tunnel T-5 Site	6
Table 3-2: Overburden Thickness and Instrumentation in Boreholes	8
Table 4-1: Summary of Index Properties of the Clayey Silt to Silty Clay Stratum.....	13
Table 4-2: Summary of Interpreted Compressibility Properties	14
Table 4-3: Summary of Interpreted Elastic Properties of the Soils	15
Table 4-4: Summary of Intact Properties of Rock Core Samples	16
Table 4-5: Summary of Measured Water Levels	17
Table 4-6: Pumping Tests Data.....	17
Table 5-1: Summary of Control Elevations at Abutments.....	19
Table 5-2: Summary of Interpreted Design Clay Strength and Consolidation History	21
Table 5-3: Design Soil Properties	21
Table 5-4: Soil Properties for Pile Interaction Assessment	24
Table 5-5: Fill Properties for Pile Interaction Assessment	24
Table 5-6: Lateral Resistance Reduction Factor for Pile Groups	26
Table 5-7: Lateral Load Capacity Reduction Factor for Pile Groups using p-y Method.....	27
Table 5-8: Assumed Earth Pressures on Pile Cap Straps.....	30
Table 5-9: Assumed Proprietary Product Properties	31
Table 5-10: Assumed Backfill Material Properties	31
Table 5-11: Summary of Abutment Slope Stability Analyses	32
Table 5-12: Summary of Calculated Cumulative Deformations	34
Table 5-13: Subgrade Ultimate Bearing Capacity.....	35
Table 5-14: Soil properties for use in ULS at Sliding	36
Table 5-15: Tentative Abutment Dimensions	36
Table 5-16: Estimated load on 1.5 m thick RGM at the Underside of RSS.....	37
Table 5-17: Calculated Factors of Safety for Wing Wall Global Instability.....	37
Table 5-18: Tentative Wing Wall Dimensions	38
Table 5-19: Soil Parameters for Earth Pressure Calculations	39
Table 6-1: Results of Analytical Testing on Soils.....	43

Drawings

285380-03-060-WIP2-2501	Hwy. 401 Oakwood Tunnel T-5 General Arrangement
285380-03-061-WIP2-2506	Hwy. 401 Oakwood Tunnel T-5 Foundation Layout
285380-04-090-WIP2-2503	Location Plan and Interpreted Stratigraphic Profile Sta. 14+000W to Sta. 14+700W
285380-04-090-WIP2-2504	Hwy. 401 Oakwood Tunnel T-5 Borehole Locations and Soil Strata
285380-04-091-WIP2-2505	Hwy. 401 Oakwood Tunnel T-5 Soil Stratigraphy

List of Applicable OPSDs

OPSD 3000.100	Foundation Piles Steel H-Pile Driving Shoe
OPSD 3000.150	Foundation Piles Steel H-Pile Splice
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement

List of Figures

- Figure 3-1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures (Ladd & DeGroot, 2004)
- Figure 3-2: Field Vane Undrained Strength Ratio at OCR=1 vs. Plasticity Index for Homogeneous Clays (Ladd & DeGroot, 2004)
- Figure 3-3: Soil Properties Profiles – Tunnel T-5
- Figure 4-1: Compressibility Parameters at WEP
- Figure 4-2: C_c versus C_α Relationship at WEP
- Figure 4-3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP
- Figure 4-4: Relationship between $\sin \phi'$ and Plasticity Index for Normally Consolidated Soils (Kenney, 1959)
- Figure 4-5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests
- Figure 5-1: Tunnel T-5 – Typical Excavation Profile
- Figure 5-2: Tunnel T-5 – Hwy. 401 Oakwood Tunnel Plan and Sections of Abutment Walls
- Figure 5-3: Tunnel T-5 – Hwy. 401 Oakwood Tunnel Plan and Sections of Wing Walls

List of Appendices

- Appendix A: Borehole, CPT and DMT Logs from Additional 2011 Geotechnical Investigation
- Appendix B: Borehole and CPT Logs from Previous Investigations
- Appendix C: Geotechnical Laboratory Test Results
- Appendix D: Analytical Laboratory Test Results
- Appendix E: Slope Stability Analysis Results
- Appendix F: Stress-Deformation Analysis Results
- Appendix G: Pile Analysis Results
- Appendix H: Seepage Analysis Results
- Appendix I: Selected Site Photographs
- Appendix J: Selected Core Photographs

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 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 (PIC). This team includes Dragados Canada, Inc., Acciona Infrastructure Canada Inc., and Fluor Canada Ltd. This combination brings a wide range of local and international experience to the project.

1.2 Report Introduction

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

This report presents the geotechnical design of Tunnel T-5, located in Windsor sector of the proposed Windsor-Essex Parkway (WEP) project. The proposed 120.8 m long, 4 to 5 span Tunnel T-5 structure will carry a trail over Highway 3 (SR2) and Highway 401, and support parkland landscape between Stations 14+510W and 14+630W. As for all other tunnels at this project, Tunnel T-5 will be a cut-and-cover construction. The proposed structural solution incorporates structural deck on concrete girders supported on semi-integral abutments on piles.

The report includes the results of the additional geotechnical investigation carried out to support the design (i.e., the layout and configuration) available at the time of preparation of this report and addresses review comments from peer reviews and MTO. This report is issued for construction (IFC). The design presented in this report was generally advanced from the preliminary geotechnical design developed for the WEMG proposal in June 2010 (ref. R-43)¹. The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines as well as the Parkway Infrastructure Constructors (PIC).

¹ References are listed in Section 9.

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

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

2 Background Information

2.1 Geological Setting

The WEP project site is located within the Essex Clay Plain (a part of the St. Clair Clay Plain physiographic region described in references R-16, R-18, R-19 and R-26). The Essex Clay Plain was deposited during the retreat of the late Pleistocene Era ice sheets, when a series of glacial lakes inundated the area. The ice sheets generally deposited materials with a glacial till like gradation in the Windsor area. Depending on the locations of the glacial ice sheets and depths of water in the ice-contact glacial lakes, the materials may have been directly deposited at the contact between the ice sheet and bedrock or, as the lake levels rose and the ice sheets retreated and floated, the soil and rock debris within and at the base of ice may have been deposited through the lake water (i.e., lacustrine environment). It is considered that unlike typical till deposits (that have undergone consolidation and densification under the weight of the ice sheet), the majority of the “glacial till” soils in the Windsor and Detroit area were deposited through water and have a soft to firm consistency below a surficial crust layer that has become stiff to hard due to weathering and desiccation. Geologically, the deposit in the project area is considered to be slightly over-consolidated, having experienced no major overburden stresses in excess of the existing stresses.

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

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

The Windsor area, referred to as the Essex Domain (with respect to bedrock geology), is located in the Grenville Front Tectonic Zone (GFTZ) (ref. R-26). The bedrock geology within the Essex Domain was formed as part of the midcontinent rift south-eastern extension. The latter 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

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
 Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

Date: May / 2012
Rev: 0
Page No.: 4

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

2.2 Site Seismic Background

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

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

2.3 Existing Site Conditions and Proposed Tunnel Layout

Tunnel T-5 site is situated near the east end of the Windsor segment of the Parkway. The topography of the lands immediately adjacent to Tunnel T-5 is generally flat with elevations ranging from approximately 181.5² in the area of north abutment to 182 at the south abutment. Adjacent land use is typically residential (see Appendix I for selected site photos).

The tunnel structure will be constructed under WEP Phase II development and will be used to carry trail traffic and parkland over Highway 3 and Highway 401. Highway 3 in the vicinity of Tunnel T-5 will be realigned to the north side of the proposed depressed Highway 401. Highway 401 and Highway 3 at this location will be constructed within permanent cut. The finished grades along the tunnel walls will be raised by about 1 m, i.e., to about 4 m above the existing grades (Drawing 285380-03-060-WIP2-2501).

2.4 Frost Depth

In accordance with MTO-SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-38) and OPSD 3090.101, the frost depth below the ground surface in Windsor area is estimated at 1.0 m³. This estimate is considered applicable to natural soils and/or conventional pavement materials where the ground surface is usually cleaned from the snow cover.

The insulation effects of riprap and other coarse rockfill cover materials are considered to be one half of the insulation offered by soil deposits/cover, and the depth of frost penetration will have to be increased accordingly.

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

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

3 Geotechnical Investigations

3.1 Scope and Procedures of Geotechnical Investigations

Geotechnical investigations involving a number of boreholes, cone penetration tests (CPT) and Nilcon vane tests had been carried out in 2007-09 by Golder Associates (ref. R-16 to R-23) to develop the conceptual design and serve as background information for development of the WEP proposal designs. Additional geotechnical investigation was completed to supplement the available subsurface soil data, as required to support the detailed design development of the WEP embankment and structures. The initial additional investigation program at and around the proposed location of Tunnel T-5 comprised a total of 4 borehole, 2 Nilcon vanes, 4 cone penetration tests and 1 DMT (flat blade dilatometer probe). Based on the findings of the initial additional investigation, a supplementary program consisting of 5 extra CPT and a Nilcon vane was carried out to determine the extent of a weak soil area encountered at the west end of south abutment. Table 3-1 lists the test holes advanced at or in close proximity of the tunnel site during both the previous and the current geotechnical investigations.

Table 3-1: Test Holes At and Around Tunnel T-5 Site

Reference	Boreholes	Nilcon Vane Tests	CPT	DMT
This Investigation (2011)	BH T5-1	NIL T5-1	CPT T5-1	DMT T5-1
	BH T5-2	NIL T5-E1	CPT T5-2	
	BH T5-3	NIL T5-E2	CPT 33-RW	
	BH T5-E1		CPT 34-RW	
			CPT T5-E1	
			CPT T5-E2	
			CPT T5-E3	
			CPT T5-E4	
			CPT T5-E5	
Previous Studies (2007-09)	BH-131	BH-132	CPT-12	
	BH-132		BH/CPT-130	
	BH-326		BH/CPT-328	
			BH/CPT-329	

Drawing 285380-04-090-WIP2-2503 shows the locations of the test holes and an interpreted soil stratigraphic profile along the WEP centreline for the general area at and around Tunnel T-5. The test hole locations and stratigraphic sections at the tunnel location are illustrated on Drawings 285380-04-090-WIP2-2504 and 285380-04-091-WIP2-2505.

3.2 Fieldwork

The boreholes were advanced using track-mounted CME55 auger rigs owned and operated by Marathon Drilling Co. Ltd. under contract to AMICO and under technical supervision by AMEC engineers and technicians. Boreholes were generally advanced using 215 mm OD hollow stem augers, followed by

wash boring with NW (OD=88.9 mm) casing. The depth at which the drilling methods transition occurred is noted on the borehole logs.

Soil sampling was generally carried out using a 50 mm diameter split spoon sampler. Thin-walled Shelby tube (70 mm diameter by 600 mm long) samples were also recovered in the cohesive soil deposits below the upper crust layer. Soil sampling was carried out generally at 0.75 m depth interval in the top 7 to 8 m and at 1.5 m depth intervals thereafter. All samples were identified and placed in airtight containers and transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing⁴. Rock coring of the bedrock was carried out using a 1.5 m long NQ (OD=75.7 mm) or HQ (OD=96.0 mm) sized core barrels.

Standard Penetration Tests (SPT, ASTM D1586⁵) were carried out in conjunction with split spoon sampling. Field vane tests (using conventional vanes) were carried out in between sampling at selected depths. Table 3-2 summarizes the depths of overburden penetration and rock coring as well as the list of instruments and the accompanying Nilcon vane tests. The Nilcon vane tests were carried typically adjacent the boreholes.

Rock cores were examined in the field and transported to AMEC's Tecumseh (Windsor) laboratories for further examination. For each core run, rock core recovery and rock quality designation (RQD) were determined. The recovery and RQD values are given on the borehole logs. The rock cores were photographed in the laboratory. Compression strength tests were carried out on rock core samples selected from across the WEP length.

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

The Nilcon vane tests and CPT were carried out in cohesive soil strata after augering through the stiffer/denser surficial materials. The Nilcon tests were carried out at 0.5 to 1.0 m depth intervals at an appropriate rate of rotational strain (ASTM D2573). The CPT cone was pushed at a constant rate into the ground using hydraulic ram system of the drill rig (ASTM D5778). All CPT were advanced to refusal. Pore pressure dissipation tests were carried out at selected depths.

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

The locations of test holes and inferred soil profile along the WEP alignment between Sta. 14+000W and 14+700W (i.e., in the general area of the tunnel) are shown on Drawing 285380-04-090-WIP2-2503. The test hole location and soil stratigraphic sections at the tunnel location are shown on Drawings 285380-04-090-WIP2-2504 and 285380-04-091-WIP2-2505.

⁴ Advanced lab tests (one dimensional consolidation, and consolidated undrained triaxial tests) were carried out in AMEC's Scarborough Laboratory.

⁵ American Society for Testing and Materials

Borehole, DMT, Nilcon and CPT logs from the additional 2011 investigation are included in Appendix A. Relevant borehole logs from the previous investigation are included in Appendix B. Borehole logs illustrate the interpreted soil conditions, field test results and laboratory index test results.

Table 3-2: Overburden Thickness and Instrumentation in Boreholes

Borehole	Location ⁶	Overburden Thickness, m	Test Name & Elevation				
			Rock Coring	Nilcon Vane	S-Piez.	VWP	MSG
BH T5-1 (AMEC)	N 4,680,035 E 331,973	32.0	148.5 to 145.1	177.3 to 153.7			
BH T5-2 (AMEC)	N 4,679,949 E 331,947	32.3	149.0 to 147.2			170.6 & 160.6	170.6 & 160.6
BH T5-3 (AMEC)	N 4,680,037 E 331,884	32.6	148.9 to 146.8		148.6 to 147.1	170.8 & 160.8	171.3 & 160.9
BH T5-E1 (AMEC)	N 4,680,013 E 331,864	> 32.5		174.6 to 158.6			
BH-131 (Golder)	N 4,679,945 E 331,856	32.7	148.1 to 144.7		146.4 to 145.2		
BH-131A (Golder)	N 4,679,945 E 331,856					171.8	
BH-132 (Golder)	N 4,680,071 E 331,910	33.4	148.0 to 143.7	176.7 to 160.7	148.8 to 147.3		
BH-132A (Golder)	N 4,680,071 E 331,910					172.5	
BH-326 (Golder)	N 4,679,918 E 331, 985	32.6	149.1 to 144.1			144.1	

Legend: S-Piez. Screen elevations for Standpipe Piezometers
VWP Sensor elevation for Vibrating Wire Piezometer
MSG Spider Magnet Heave/Settlement Gauge

3.3 Instrumentation

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

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

⁶ Location coordinates are in UTM-NAD 83 (Zone 17).

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

Spider Magnet Heave/Settlement Gauges (MSG): Magnetic targets are anchored to the ground around a PVC pipe. The anchors are not coupled to the access pipe, and are free to move with the soil. An estimate of ground heave/settlement can be made by measurement of ring elevations. Ring/Gauge elevations are provided in Table 3-2 and applicable borehole logs.

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

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

3.4 Geotechnical and Analytical Laboratory Testing

All recovered soil samples and rock cores were examined in the field and the laboratory. Natural moisture content tests were carried out on most of the recovered samples; grain size distribution and Atterberg limit tests were carried out on selected representative samples. Following these soil classification tests, representative soil samples were selected for advanced tests (1 Isotropically Consolidated Undrained Compression - CIUC triaxial test and 2 one-dimensional consolidation tests). The index test results are shown on borehole logs included in Appendix A. The results of consolidation and shear tests are included in Appendix C.

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

3.5 Data Interpretation

Field Vane Test Data Correction: The chart in Figure 3-1⁷ developed initially by Bjerrum (1972) and updated subsequently by Ladd et al (1977) based on circular arc failure analyses of embankment failures suggest correction by multiplying the field vane data by 1.05 to 1.10 for soils with plasticity index of about 15 (ref. R-5 and R-31). However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), the Canadian Foundations Manual suggests that the vane test data for clays with PI<20 should not be corrected (ref. R-1 and R-43, Figure 3-2). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI.

⁷ All figures are included at the end of the report text.

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

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

Where:

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

Q_t is the corrected total cone tip resistance;

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

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

The CPT based S_u profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles. In this regard, the N_{kt} factor values used to calibrate the CPT strength profiles varied slightly for different segments of the WEP and the soil strata. Thus, an N_{kt} factor of 14 was used to estimate the undrained shear strength of the clay crust and transition layers. The N_{kt} factors used for the underlying grey silty clay to clayey silt stratum and the lower clayey silt stratum were 116 and 12, respectively. In CPTs indicating pore pressures higher than cone tip resistance (e.g., soft clay stratum in CPT-328 and CPT T5-2), 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-31). The following relationship was used to compute the pre-consolidation pressures:

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

Where:

S_u is the undrained shear strength;

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

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

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

OCR is the overconsolidation ratio; and

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

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

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

Flat Blade Dilatometer (DMT) Test Data: DMT tests were conducted following the ASTM D6635-01 (2007) method. The soil properties from the results of these tests were developed in general using the guidelines layout in ISSMGE, 2001 (ref. R-27), except that the undrained shear strength values for the clay deposits were estimated using the relationship $S_u = S \sigma'_{vo} (0.5 K_d)^{1.25}$, where $S = 0.18$ and K_d is the horizontal stress index represented by:

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

Where:

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

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

The constant 0.18 for S_u/σ'_{vo} for OCR=1 is based on average plasticity index of the silty clay to clayey silt stratum and the Chandler 1988 relationship (ref. R-11). Interestingly, the undrained shear strength (S_u) profiles inferred from the DMTs show lesser scatter than the S_u values obtained from the conventional field vane tests in boreholes and the Nilcon vane test values.

The undrained shear strength (S_u), pre-consolidation pressure (σ'_p), natural water content (w_N) and compression index (C_c) profiles based on field and laboratory testing from boreholes, CPTs and DMT carried out in the vicinity of Tunnel T-5 are presented in Figure 3-3. Also included on these figures are $0.18 \times \sigma'_{vo}$ curve (representing undrained strength for OCR=1 condition) and simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units.

4 Subsurface Conditions

The general soil stratigraphy at the borehole locations consists of the following successive strata: surficial layers of occasional fills, topsoil, and upper granular deposit; an extensive clayey silt to silty clay deposit below about elevation 181, and a lower granular deposit below about elevation 150, overlying limestone and dolostone bedrock below about elevation 149. The thickness of the Clayey Silt to Silty Clay deposit varies between about 29.0 m and 31.0 m. The lower granular deposit (sandy silt / silty sand / sand and gravel) varied in thickness between 0 to 2.8 m. The bedrock was encountered at depths ranging from about 32.0 m to 33.4 m below the ground surface.

4.1 Surficial Fills, Topsoil and Upper Granular Deposit

All boreholes, except for DMT T5-1 and Borehole BH T5-1 encountered an up to 0.9 m thick layer of brown to black topsoil. The thickness of the topsoil is expected to vary through the project area.

Boreholes DMT T5-1 and BH T5-1 were drilled on existing pavement and encountered 300 mm thick concrete overlying crushed limestone sand and gravel fill which extended to 0.9 m below existing grades. A 1 m thick surficial silty sand fill layer was also encountered in Boreholes BH/CPT-328 and BH/CPT-329.

A non-cohesive fine silty sand was encountered in all boreholes, except Boreholes CPT/BH-328, BH T5-2 and BH T5-E1 below fill and/ topsoil. The thickness of this unit varies between 0.1 to 1.2 m.

4.2 Silty Clay to Clayey Silt Stratum

The cohesive silty clay stratum was encountered directly underlying the surficial topsoil or fill/granular deposit. The encountered depth below existing ground surface was from 0.3 to 3.1 m. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into 4 layers as follows: brown desiccated stiff to very stiff clay crust, transition zone, upper grey silty clay to clayey silt deposit (referred to hereafter as upper silty clay), and then a (generally coarser) lower grey clayey silt deposit (referred to as lower clayey silt). The natural water content, Atterberg limits and bulk unit weights determined for the samples recovered during the additional (2011) geotechnical investigation of the clay sub-strata are summarized in Table 4-1. The plasticity charts (Appendix C) suggest the silty clay deposit to be a low to medium plasticity material.

The CPT and Nilcon test results indicated presence of distinctly weaker silty clay deposit at the west segment of the south abutment relative to the rest of the tunnel area. Specifically, the undrained strength below the transition zone was about 40 kPa at the southwest corner compared with about 50 kPa elsewhere (Table 5-2 and Figure 3-3).

The stress-strain properties and the effective shear strength properties of the silty clay to clayey silt soils were based on published correlations (Kulhawy and Mayne, 1990, ref. R-28, Leroueil et al, 2001, ref. R-33 and Terzaghi et al. ref. R-42) as well as on the tests reported in Golder's Subsurface Condition Interpretation Report (ref. R-17) and the tests performed during the additional geotechnical investigation carried out as part of the detailed design development for the entire WEP length.

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

Property	Clay Crust	Transition	Grey Silty Clay	Clayey Silt
Elevation Range, m	182(*) to 177	177 to 175	175 to 160	160 to 152
Natural Water Content, w_N , %	4 to 37	17 to 32	11 to 41	7 to 40
Liquid Limit, w_L , %	30 to 42	34 to 41	24 to 42	27 to 38
Plastic Limit, w_P , %	13 to 20	15 to 18	10 to 20	12 to 18
Plasticity Index, PI, %	15 to 22	18 to 25	10 to 23	12 to 20
Liquidity Index, LI	0.00 to 0.56	0.34 to 0.78	0.13 to 0.57	0.24 to 0.42
Unit Weight, γ , kN/m ³	-	-	19.1 to 22.2	20.7 to 22.9

(*) Elevation varies

As illustrated on Figure 3-3, the undrained shear strength of the silty clay stratum varied with depth generally as follows:

Soil Layer	East & Centre of Tunnel	Southwest Part of Tunnel
Crust layer	> 80±20 kPa	> 80±20 kPa
Transition layer	80±20 kPa to 50±10 kPa	80±20 kPa to 40±10 kPa
Upper silty clay	50±10 kPa to 45±10 kPa	40±10 kPa to 45±10 kPa
Lower clayey silt	>75±15 kPa	>75±15 kPa

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

$$C_c = 0.0086w_N - 0.0086$$

$$C_r = 0.11C_c$$

$$C_s = 0.25C_c$$

$$C_\alpha = 0.028C_c$$

The interpreted average values used for the clay substrata for the Tunnel T-5 site are summarized in Table 4-2.

Oedometer tests carried out on upper Grey Silty Clay samples taken at 15 m depth below ground surface from Boreholes BH T5-2 and T5-3 indicated the following compressibility indexes: $C_c = 0.12$ and 0.27 , $C_r = 0.036$ and 0.040 , $C_s = 0.020$ and 0.060 . Results reported from Borehole BH-132 (ref. R-17) on one sample taken at 12 m depth were $w_N = 32\%$, $C_c = 0.31$, $C_r = 0.060$, $C_s = 0.092$. Compared to interpreted compressibility characteristics (Table 4-2), the compression indexes measured at these particular samples suggest relatively large scatter, which seems to be associated with the higher variations of the moisture content observed around elevation 165.

Table 4-2: Summary of Interpreted Compressibility Properties

Property	Clay Crust	Transition	Grey Silty Clay	Clayey Silt
Average Natural Water Content, w_N , %	20	25	23	21
Virgin Compression Index, C_c	0.17	0.20	0.19	0.17
Recompression Index, C_r	0.018	0.022	0.021	0.018
Swelling Index, C_s	0.041	0.050	0.048	0.041
Secondary Compression Index, C_α	0.0046	0.0055	0.0053	0.0046

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

	0 kPa
Angle of internal friction, ϕ	30°
Friction angle at critical state ⁸ , Φ_c	25 – 26°

CIUC triaxial tests carried out on a silty clay sample from Borehole BH T5-3 (obtained from 15.5 m depth below ground surface) indicated an effective friction angle of 29 degrees. CIUC tests carried out on three samples from Borehole BH-132 had also shown a peak effective friction angle of 29 to 32 degrees.

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

$$E_u = 300 S_u$$

$$E' = 0.9E_u$$

For the unweathered portion of the silty clay stratum the empirical relationship were used based on average shear strength profiles for the material, as shown in Table 4-3.

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.

⁸ Based on triaxial tests (ref. R-18)

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

Soils Stratigraphy	Undrained Elastic Modulus, MPa	Undrained Poisson's Ratio (*)	Drained Elastic Modulus, MPa	Drained Poisson's Ratio (*)
Clay Crust	35	0.49	32	0.35
Transition	28		25	
Grey Silty Clay	18		16	
Clayey Silt	20		18	

(*) Assumed values

4.3 Lower Granular Deposit

Underlying the silty clay to clayey silt stratum and overlying the bedrock, a discontinuous and heterogeneous non-cohesive material deposit (varying from silty sand, sand and gravel, and clayey silts with sand) was encountered. Based on SPT N-values ranging generally from 28 to greater than 100, this material is considered to be in a compact to very dense state of compactness. This layer was approximately 0 to 2.8 m thick and varies significantly throughout the tunnel site.

4.4 Bedrock

Where rock coring was undertaken, a grey, limestone bedrock was encountered. The bedrock was generally fresh, medium strong, thinly laminated, fine grained, faintly to moderately porous and highly fractured. Bedrock was encountered at elevations ranging from 148.0 to 149.3 in the vicinity of Tunnel T-5. The Rock Quality Designation (RQD) of the recovered rock cores varied between 0 to 97 per cent, indicating a very poor to excellent quality. With the exception of Boreholes BH T5-1 and BH T5-2, rock quality generally increases with depth.

Based on this core logging the rock mass classification was estimated to range from 2.8 to 5 for the Q-System (Barton et. al., 1974, ref. R-3) and 53 to 58 for the Rock Mass Rating (RMR) based on Bieniawski (1976, ref. R-5) and indicates that the rock mass can be considered as a Fair quality rock mass based on the later system. Boreholes cores show that rock quality generally improves with depth. Photographs of rock cores recovered from the additional investigation are provided in Appendix J.

It was found during the preliminary investigations (ref. R-19) that little variation in the strength of the rock mass conditions was identified from site to site. For this reason in order to obtain a reasonable statistical sample, the density, unit weight and uniaxial compressive strength of the samples from all of the key sites have been grouped and are summarised in (Table 4-4). The average strength of the limestone is determined to be 85.5 MPa and is 'strong rock' based on the ISRM (1978, ref. R-28). Additionally, based on the coefficient of variation, enough tests have been performed to characterise the compressive strength.

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

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

Parameters	Density	Unit Weight	UCS
	(kg/m ³)	(kN/m ³)	(MPa)
Average	2502	24.54	85.5
Standard Deviation	96	0.94	25.4
Minimum Value	2340	22.95	35.5
Maximum Value	2660	26.09	135.3
Number of Samples, N	12	12	16

4.5 Groundwater Conditions

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

The highest piezometric levels within the overburden and the bedrock were recorded at elevations 179.9 and 179.6, respectively (Table 4-5). These observations suggest an essentially hydrostatic condition or a slight downward gradient between the overburden and the bedrock. In consideration of experience at other locations along the project alignment localised occurrences of artesian condition in bedrock cannot be ruled out.

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

Table 4-5: Summary of Measured Water Levels

Borehole	Surface El, m	Piezo. Type	Screen / Sensor El, m	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	El, m
BH T5-2	181.3	VWP	170.6	Silty Clay	July 23, 2011	179.5
		VWP	160.6	Silty Clay	July 23, 2011	179.1
BH T5-3	181.5	S-Piez.	148.6 – 147.1	Limestone	July 2, 2011	179.1
		VWP	170.8	Silty Clay	July 2, 2011	179.4
		VWP	160.8	Silty Clay	July 2, 2011	179.2
BH-131	180.8	S-Piez	146.3 – 144.8	Limestone	July 10, 2011	179.2
BH-131A	180.8	S-Piez	171.6 – 172.0	Silty Clay	Jan. 28, 2009	179.7
BH-132	181.5	S-Piez	147.3 – 148.8	Limestone	July 23, 2011	179.3
BH-132A	181.5	S-Piez	172.3 – 172.6	Silty Clay	Jan. 28, 2009	180.2
BH-326	181.8	S-Piez	144.1 – 144.4	Limestone	Feb. 24, 2010	179.6

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

4.6 Subsurface Gases

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

The H_2S gas can frequently be detected by odour at concentrations in the order of 0.5 mg/L and can be corrosive at concentrations of about 2 mg/L to 3 mg/L in the groundwater. The gas odour was not detected during the drilling at the Tunnel T-5 site.

Although the presence of the H_2S and CH_4 gases was not observed during the 2011 geotechnical investigation at Tunnel T-5 site, their presence cannot be ruled out. Pumping tests were conducted at three locations across the proposed parkway to determine concentration levels of hydrogen sulphide gas in the groundwater of the area. A summary of the results of these tests is provided in Table 4-6.

Table 4-6: Pumping Tests Data

Test #	Approximate Location	H_2S Gas Concentration, mg/L
TOW-1	Bridge B-11	< 0.02
TOW-2	North of Tunnel T-7	20.0
TOW-3	South of Tunnel T-4	7.9

Air quality and subgrade pore pressure monitoring should be carried out during construction. In general, it is recommended that equipment operating in confined spaces be selected to safely operate in a potentially gaseous environment.

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 to total stress changes can be very low. This phenomena leads to reduction in effective stress and hence shear strength (ref. R-25 and R-41). In this regard, it is recommended that the design and construction should address the potential presence of these gases (ref. R-14).

5 Development of Geotechnical Designs

5.1 Tunnel Configuration

Tunnel T-5 (Oakwood) will be constructed along the below-grade section of the WEP between Stations 14+510W and 14+630W, and will accommodate the below-grade traffic of Highways 401 and Highway 3 (Drawing 285380-03-060-WIP2-2501). The proposed Tunnel T-5 is 120.5 m long and its width varies from 118.6 m at the west end to 120.5 m at the east end.

The proposed Tunnel T-5 is a multi-span deck-on-girder structure incorporating semi-integral abutments and centre piers. Based on design information available, The abutments consist of 1.7 m wide \times 1.5 high pile cap founded on deep end-bearing HP 310 \times 110 steel piles (Drawing 285380-03-060-WIP2-2506). The center piers would include 3.2 m wide by 1.25 m high pile caps supported on vertical and batter piles.

False abutments will be constructed using Reinforced Soil System (RSS) walls founded on geogrid reinforced granular fill referred to hereunder as Reinforced Granular Mats (RGM). The retained backfill will be completed with a combination of approved conventional soil fills, lightweight fills (LWF), and extruded polystyrene (EPS).

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

Table 5-1: Summary of Control Elevations at Abutments

Abutment	Section	Station	Existing Ground Surface El, m	Top of Finished Grade El, m	Top of Deck El, m	Top of Pile Cap El, m	Pavement Subgrade El, m
North	1	14+600W	181.5	185.5	184.5	182.1	173.9
	2	14+540W					173.1
	3	14+513W					172.7
South	4	14+617W	182.0	183.2	182.2	179.8	173.8
	5	14+567W					173.0
	6	14+540W					172.0

The retaining walls at the portals will comprise RSS tapered wing walls along the abutments and return RSS walls flared typically at 90 degrees.

Based on the available information, it is considered that Tunnel T-5 construction will involve the following sequence of earthwork, structural elements and loading stages:

- Temporary excavation to about 8 and 9 m depths below grade;
- Installation of a 1.5 m thick Reinforced Granular Mats (RGM) foundation at north and south abutments, respectively. Void forms are anticipated to be incorporated within the RGM to accommodate later pile installation through RGM;

- Temporary trenches along the piers;
- Installation of end bearing piles (HP310×110) for all tunnel supports;
- Completion of the pier footings and immediate backfilling;
- Installation of 500 mm diameter Corrugated Steel Pipes (CSP) around the abutment pile stickups;
- Construction of the false abutments comprising RSS structures and associated permanent subdrainage works, and approved backfill behind the RSS structure; alternatively, to reduce the transient downdrag loads in the abutment piles, the RSS construction may be scheduled before piling;
- Placement of concrete fill within the CSP;
- Construction of the structural abutment (pile cap) and tunnel deck;
- Completion of final stage of backfill behind the semi-integral abutments; and
- Completion of the final topsoil placement and trail materials.

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

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

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

5.3 Design Soil Properties

As indicated in Section 3.5, the design soil properties for the silty clay to clayey silt deposit were interpreted from the CPT and Nilcon vane test profiles and the laboratory test results. Selected typical design values obtained from the profiles are summarized in Table 5-2. Effective cohesion for the upper clay crust and transition zone layers has been neglected due to long term weathering, moisture ingress and fissuring effects.

Table 5-2 differentiates between the conditions observed at the west segment of the south abutment from the remainder of the Tunnel T-5 and surrounding area. As indicated in this table and illustrated in Figure 3-3, the undrained shear strengths estimated for the upper portion of the Grey Silty Clay layer (from CPT-328, CPT T5-2, CPT T5-E4 and Nilcon T5-E1) for the west segment of the south abutment were lower than the general trend observed for the Tunnel T-5 site, and as a result a different shear strength profile was adopted for designing this segment. It should be noted that pockets or zones of the weaker

silty clay encountered at the southwest sector of the tunnel could exist elsewhere at and around the tunnel site.

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

Clay Substratum	Elevation, m	Undrained Shear Strength (S_u), kPa		Effective Stress Parameters	Pre-consolidation Pressure (σ_p'), kPa	OCR
		T-5 Site & Surrounding Area (Except SW Segment)	West Segment of South Abutment			
Clay Crust	> 177	75 (*)		Peak Friction Angle, $\phi_{max} = 30^\circ$ = 0	550	> 5
Transition	177 - 175	75 to 50	75 to 40		550 to 275	4
Grey Silty Clay	175 - 160	50 to 45 to 65	40 to 45 to 65		275 to 205 to 350	1.2
Clayey Silt	< 160	65	65		350	1.2

(*) Applicable for global stability verifications

Note: The undrained shear strength and pre-consolidation pressure values vary with depth as illustrated in Figure 3-3. The effective shear strength parameters are based on the relationship presented in Section 4.2

The design values of the coefficient of horizontal permeability (k_h) and the hydraulic conductivity anisotropy ratio ($A = k_h/k_v$) used for the analysis of stress and deformation response of the soils are provided in Table 5-3. These values are slightly (2 to 5 times) higher than the values interpreted from the field test results (Figure 4-5) and are considered to be within range of precision of the measurements.

Table 5-3: Design Soil Properties

Clay Substratum	Horizontal Permeability, cm/sec	Anisotropy Ratio, k_h/k_v	Initial Void Ratio, e_0
Clay Crust	6.8×10^{-7}	1	0.6
Transition	3.9×10^{-7}	2	0.7
Grey Silty Clay	1.1×10^{-7}		0.6
Clayey Silt	1.1×10^{-7}		0.6
Lower Granular	1.1×10^{-6}	1	0.6

For design purposes the initial groundwater level in the overburden was considered at elevation 179.

5.4 Excavation and Temporary Cut Slopes

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

Excavations are expected to encounter surficial fills, topsoil and water bearing granular soils and will be extended 8.0 and 9.0 m below existing grade (elevation 181.5 and 182) to about elevation 173.5 and 173.0 into the native firm silty clay for the north and south abutments, respectively. The approximate excavation profile for this structure is shown in Figure 5-1, which was developed on the basis of the roadway cross section at Sta. 14+525W.

Basal hydrostatic uplift at the abutments was calculated based on the highest measured water level in the bedrock (179.6), anticipated deepest excavation depth (RGM base at elevation 172.5), and a silt-clay layer thickness of 18.6 m (Borehole BH T5-1) below the deepest excavation. The factor of safety (FS = overburden weight / hydrostatic pressure on top of bedrock) against hydrostatic uplift was 1.5. The FS at deepest pier location (Pier#3, with excavation invert at El=171.5) is approximately 1.4.

As described in Section 4.6, gassy soils near bedrock surface could potentially be encountered and that could impact the pore pressure and undrained shear strength condition of the lower part of the silty clay deposit. While no indications of gassy soils were recorded at this site during the background and additional investigations, it is recommended that in the case of excavations deeper than 5 m careful monitoring of basal heave and pore water pressures below of the bottom of the excavations be carried out during construction. Adequate number of heave gauges and low-displacement type piezometers shall be installed prior to initiation of the major excavations. Warranted by the monitoring of the excavation progress, the excavation rates will have to be adjusted to allow sufficient time to dissipate the pore pressures to safe levels. The excavation guidelines can be revised based on on-site experience.

5.5 Pile Foundations

5.5.1 Resistance to Axial Loads

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

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

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

Based on the available borehole data at this structure, the bedrock surface varies between elevations 148 and 149, 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/or a perceived risk of damaging the piles by overdriving is apparent, consideration should be given to supplementing the field testing to prove the actual mobilized resistance. If lower mobilized pile

resistances are proven, options based on the most economical approaches may be considered (e.g., changes to the driving method and equipment, or addition of more piles).

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

The following general pile installation recommendations should be considered:

- The steel H piles should be installed and monitored in accordance with OPSS 903 requirements. The piles should be reinforced with Type I shoe flanges as shown in OPSD 3000.100, or approved alternatives.
- Survey of all the pile head elevations should be completed at the end of driving and just prior to forming the pile cap. Re-tapping of the piles will be necessary where uplift exceeding 5 mm is noted, or as directed by engineer.
- While unlikely to occur at this site, considering the general geologic conditions in the region, indications of natural gas venting, water, and fines washout should be monitored during driving. Provision to mitigate such occurrences (by heavy mud, grouting of the cavities, etc.) should be in place. It is recommended that the pile splicing be completed by butt-welding (OPSD 3000.150, Section A-A) to minimize the pathways for upward flow of artesian water along the piles to the surface.
- Consideration should be given to potential driving difficulties due to the presence of dense to very dense lower granular soils and potential presence of cobbles and boulders above the bedrock.
- Vibrations generated by piling should be monitored. It is not expected that the vibrations during piling will have a significant impact on the stability of temporary slopes. Nonetheless, if the vibration intensities at the toe and top of the slopes exceed 10 mm/s, appropriate mitigation measures (slope flattening or vibration dampening by dumping sand around the piles) should be considered.
- Noise monitoring should be carried out during pile driving at the site.

5.5.2 ULS and SLS Resistance to Lateral Loads

The ULS and SLS geotechnical resistances to lateral loads should be determined on the basis of field load tests. Both the ULS and SLS lateral load resistances are strongly dependent on the soil properties, structural configuration of the pile and pile foundation, load configuration and on the acceptable 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, a conventional SLS resistance of 60 to 70 kN along the strong axis of the HP310×110 can be considered. This conventional SLS resistance represents the lateral shear force applied on a free-head pile that causes a lateral deflection of 10 mm measured at the ground surface.

The ULS lateral resistance is defined as the lateral force applied to the pile shaft causing unstabilised pile displacements due to soil failure or pile structural failure. In the absence of field tests, the ULS lateral resistance may be assumed as 185 kN, and 85 kN along the strong axis and weak axis, respectively.

The above SLS and ULS estimate was conducted using the “p-y” model (L-Pile-5 model Ensoft 2010). The pile model was assumed embedded within firm silty clay below elevation 175. The “p-y” curves were generated using the Reese “Stiff-Clay without free water” and Matlock “Soft Clay” models in conjunction with the soil parameters defined in Tables 5-4 and 5-5.

Table 5-4: Soil Properties for Pile Interaction Assessment

Soils Around the Piles	Elevation Range	Design Bulk Unit Weight, kN/m ³	Undrained Shear Strength (S_u), kPa	ϵ_{50}
Silty Clay Crust	Above 177	22	75	0.005
Transition Clay	177 to 175	21	75 to 50	0.007
Upper Silty Clay - 1	175 to 165	20	50 to 45	0.010
Upper Silty Clay – 2	165 to 161	20	45	0.010
Lower Clayey Silt - 1	161 to 160	20	45 to 65	0.007
Lower Clayey Silt - 2	Below 160	22	65	0.005

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

Table 5-5: Fill Properties for Pile Interaction Assessment

Material	Soil Model in L-Pile	Design Bulk Unit Weight, kN/m ³	ϕ°	n_h , MPa/m
RSS Fill (LWF)	Sand (Reese)	22	35	5
RSS Fill (Granular*)	Sand (Reese)	21	35	10
RGM Granular (**)	Sand (Reese)	21	30	2

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

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

LWF: Lightweight Fill

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

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

It should be noted that during driving, significant soil disturbance and damage occur around the pile shaft forming sizeable gaps between the pile and the surrounding soils. These gaps cause significant reduction of the actual SLS and ULS resistances. Where the design relies on the lateral resistance provided by the soils, “repairs” to the disturbed soils must be undertaken (typically, the voids are grouted using non-shrink fills).

The abutment piles embedded within concrete filled CSP and compacted reinforced RSS fill will develop larger resistances to lateral loads.

Significant lateral loads in excess of the values previously cited should be resisted fully or partially by the use of battered piles. In this regard, batter piles are considered to be more effective in resisting horizontal loads, as a part of lateral load is converted into axial load and consequently the induced bending moments are less. For ease of constructability and to provide hammer energy sufficient for pile driving, batters are usually limited to no steeper than 1H:5V.

Horizontal Subgrade Reaction Method:

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 , is based on the following equations:

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

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

Where:

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

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

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, as indicated in Table 5-6. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed here.

The pile spacing in the direction of loading under the piers is 2.2 m and accordingly, the group reduction effect in this case is considered to be negligible. Under the abutments the two lines of piles are staggered and the group reduction effect is considered negligible.

Table 5-6: Lateral Resistance Reduction Factor for Pile Groups

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

d = pile diameter

Source: NAVFAC DM-7.2 (ref. R-13)

Alternative Nonlinear ‘p-y’ Curve Method:

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

The general procedure for computing p-y curves is summarized in the Canadian Foundation Engineering Manual (ref. R-8). A detailed description for the generation of the p-y curves can be found in the Technical Manual for the commercial software LPile Plus by Ensoft Inc (ref. R-15). For a given foundation configuration, pile size, and soil stratification, the soil properties required for the generation of the p-y curves are provided in Tables 5-4 and 5-5. “Stiff clay” p-y curves as given in the LPile manual should be developed appropriate for either static or cyclic loading conditions in absence of free water. For p-y curves below the water table, submerged unit weights in the soil mass shall be used.

The obtained p-y curves may require to be scaled by a factor (“modifier”) to account for batter and for group effects. The modifier factor applies to the “p” values.

In the case of batter of 1H:5V (abutments), the p-y curve modifier will be $B_m = 0.75$ and 1.25 for the batter in the direction of the lateral load, and opposite direction of the lateral load, respectively.

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

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

Where:

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

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

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

The closest spacing between the piles is 1550 mm at the piers and 1500 mm at the abutments. Accordingly, some reduction factors would apply only for the tailing piles.

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

5.5.3 Soil Pile Interaction Assessment

Downdrag Loads – Negative Skin Friction (NSF):

Potential for downdrag loads on piles was considered in conjunction with the anticipated ground movements (rebound and settlements) that are assumed to occur during and following excavation of the overburden of up to 10 m to accommodate the future depressed highways, followed by partial replacement of fills to construct the tunnel abutments.

Soil stress-deformation analyses described later in Section 5.6.3 were conducted using the SIGMA/W software. The estimated ground vertical movement (settlement/heave) in the vicinity of the pile shaft at representative stages: after RSS completion, after completion of the top backfill against the tunnel diaphragm (End of Construction – EC), and in long-term (LT) are presented in Figure F-11. The analysis indicates the following:

- Ground settlements are expected to occur along the pile shaft during construction of the RSS wall, tunnel and completion of the associated backfill and continue for long-term; and
- Ground rebound is expected to occur along Highway 401 and the south abutment after substantial completion of the ground surface loading.

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

- Maximum transient downdrag of 1,050 kN plus structural dead load only (pile cap and tunnel roof) occurring during completion of the backfilling against the tunnel diaphragm if the piles are installed before the completion of the RSS. If the abutment piles are driven after substantial

completion of the RSS abutment, the maximum transient downdrag load was estimated to reduce to 550 kN.

- Residual (long-term) downdrag of 250 kN plus total design dead loads (structural and topsoil/landscape materials over tunnel roof) after the completion of construction.

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

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

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

No downdrag is anticipated at the pier piles.

Shaft Bending due to Lateral Soil Displacement

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

- The ground lateral movement (Figure G-1 in Appendix G) 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 above soil deformation field was imposed as “loads” along the pile shaft. The calculation was conducted using the “p-y” model (L-Pile-5 model Ensoft 2010). The “p-y” curves were generated using the Reese method described in the Technical manual for L-PILE, using the soil parameters indicated in Tables 5-4 and 5-5.
- The pile head was assumed to be a free head.
- The pile was modelled with a 500 mm diameter collar section (CSP pipe filled with concrete around the pile shaft) within the RSS wall and RGM. Below the RSS wall, the pile section was HP section. 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 maximum unfactored bending moment in the shaft was 90 kN-m. The variations of the shear force with depth provide indications of the lateral forces transferred by the pile shaft to the surrounding soil within the RSS. The calculated maximum pile deflection at the underside of the pile cap is 17 mm.

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

The maximum computed moment in the pile under assumed pile head load equal to the conventional SLS resistance was 60 to 70 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 150 to 160 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. The structural designer should review the assumptions and analysis approach and satisfy themselves with these findings.

5.5.4 Pile Cap/Abutment Stem Anchoring

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

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

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

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

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

Where:

σ_v	vertical stress at the point of calculation including the effects of the dead loads and applicable live loads
$\Delta \sigma_H$	supplemental horizontal pressures from external lateral forces (if present, such as shear force at the bottom of footings resting on top of reinforced zone)
K_a	active earth pressure coefficient

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

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

Regular Backfill:	21 kN/m ³
LWF:	12 kN/m ³
EPS:	0.5 kN/m ³

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

Based on the above, and in conjunction with the proposed abutment configuration, the following unfactored lateral earth pressure loads were estimated (Table 5-8):

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

Abutment	Earth Pressure, kN/m		
	ELL	EDS	EB
North	5	15	5
South	5	15	10

Legend:

ELL (kN/m)	Earth pressure from live loads (assumed 9 kPa within landscape areas)
EDS (kN/m)	Earth pressure from Dead Surcharge load above the pile cap
EB (kN/m)	Earth pressure due to backfill behind the pile cap

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

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

5.6 RSS False Abutment Walls

The conceptual configurations developed for Tunnel T-5 that meet the geotechnical requirements are shown in Figure 5-2. The north and south abutments generally comprise RSS walls founded on a RGM foundation, approved clay backfill, EPS and LWF.

Along the north abutment at the, 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 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 and backfill material assumed in the geotechnical analyses are summarized in Tables 5-9 and 5-10.

Table 5-9: Assumed Proprietary Product Properties

Material		Unit Weight, kN/m ³	Limit Equilibrium Analyses		Stress Deformation Analyses	
			Friction Angle, °	Apparent Cohesion, kPa	Modulus of Elasticity, E, MPa	Poisson's ratio, μ
RSS	Regular GF	21	35	50	40	0.35
	LWF	12				
RGM		21	35	40	60	0.35
EPS		0.5	0	10	10	0.2

Table 5-10: Assumed Backfill Material Properties

Backfill Material	Unit Weight, kN/m ³	Undrained Shear Strength, kPa	Drained Angle of Internal Friction, °	Modulus of Elasticity, E, MPa	Poisson's ratio, μ
Compacted Clay Fill	21	50	30	20	0.35

5.6.1 Global Stability

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

Figures E-1 to E-6 illustrate the stability models for the south and north abutments. The global stability analyses have been carried out for short-term during construction (undrained soil properties), end of construction (undrained soil properties) and long-term (drained soil properties with stabilized water levels) loading conditions. The short-term analysis completed for temporary conditions using undrained soil properties represents the condition in which the pavement structure over the Highway 401 subgrade tunnel is not present or is removed (e.g., to simulate future pavement repairs). The drained analyses

assumed that all the components of the structure are present. The presence of the piles was not considered in the stability models (somewhat conservative approach). Surcharge of 9 kPa for short-term and long-term model was applied at the top of ground surface, while tension crack was assumed for short-term only.

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

As mentioned before, the abutment configurations were determined in consideration of the global stability and geotechnical bearing of the false abutments using the applicable soil characteristics and the design strength profiles. The calculated factors of safety (FS) for circular slip surfaces are in excess of 1.3 and 1.5 for short-term and long-term loading conditions, respectively as shown in Figures E-1 to E-9 and summarized in Table 5-11.

Table 5-11: Summary of Abutment Slope Stability Analyses

Abutment	Loading Conditions			Figure
	Short-term during Construction ⁽¹⁾	End of Construction ⁽²⁾	Long-term ⁽³⁾	
North Wall	1.43 (1.32) ⁽⁴⁾	1.48 (1.36)	1.55 (1.48)	E-1 to E-3
South Wall – East and Central Segments	1.38 (1.27) ⁽⁴⁾	1.47 (1.35)	1.55 (1.47)	E-4 to E-6
South Wall – West Segment (weak clay)	1.41 (1.29) ⁽⁴⁾	1.46 (1.34)	1.62 (1.51)	E-7 to E-9

(*) Values outside parentheses refer to circular failure surfaces and the values in parentheses refer to non-circular failure surface

(1) Short-term (temporary) undrained response without pavement box over EBR5/WBR5 subgrade

(2) Undrained response with pavement box over EBR5/WBR5 subgrade

(3) Drained response with all design components present

(4) RSS toe berm and granular base over EBR5 and WBR5 must be placed before any backfill is placed above the deck seat level.

As shown on Figures E-7 to E-9, extension of EPS outside the RSS footprint and into the backfill was required to achieve the minimum required FS as per the design criteria for the west segment of the south abutment.

5.6.2 Stress Deformation Analyses

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

The configuration of the calculation model is presented in Figure F-1. The calculation model typically assumed the following loading steps:

- a) Definition of the initial (in-situ) stress condition for level ground assuming an average bulk unit weight of 21 kN/m^3 and an at-rest earth pressure coefficient K_0 of 0.75 for the soil deposit⁹;
- b) Bulk excavation to the subgrade level under the highway pavement;
- c) Construction of the RSS structure and the associated backfill;
- d) Completion of the remaining fill above the RSS structure;
- e) Completion of the pavement structure for Highway 401; and
- f) Dissipation of excess pore pressure.

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

The SDA were carried for drained (effective stress) soil behaviour using a fully coupled stress-pore pressure analysis (coupled stress-deformation and seepage dissipation equations). Hydraulic conductivity properties described in Table 5-3 were assigned to the different soil layers. The phreatic surface was assumed to correspond to the initial groundwater level at elevation 179.0 and then follow the excavation and subgrade surfaces.

Modified Cam-Clay constitutive models were considered for the unweathered firm to soft clayey silt below the transition zone, and the elastic-plastic Mohr-Coulomb model for the remaining soil layers (i.e., crust, transition, and backfill). The drained Modified Cam-Clay model required as input the critical state friction angle, pre-consolidation pressure, initial void ratio, primary compression and unloading compression indices. The latter was selected as the rebound compression index given in Table 4-2. The drained elastic-plastic Mohr-Coulomb model required as input the peak friction angle, the drained initial Young's modulus, and a Poisson's ratio.

The construction stages represented by excavation, completion of the RSS and completion of the entire abutment followed by the placement of the pavement box were assumed to occur rapidly (1 day/stage). Accordingly, there is not sufficient time to allow the dissipation of any tangible proportion of the excess pore water pressures generated by the soil unloading/reloading of the listed construction stages. Hence, the state of stress and deformations at the end of each of the first "4 days" largely correspond to undrained conditions. After the completion of the entire construction, the model is allowed to dissipate the excess pore-pressures over a period of time until a steady-state pore pressure condition is achieved.

The SIGMA model was developed for the north abutment where the height of the retained soils measured from the top of finished grade to the bottom of the RSS is the highest (11.8 m) and will provide the upper limits for the deformation estimates.

Figures F-1 and F-2 show the calculated cumulative settlement/heave for the end of construction (4 days) undrained conditions and the long-term drained loading conditions (14,000 days), respectively. Figure F-3 illustrates the stabilized pore water pressure contours at the end of dissipation (long-term) period.

⁹ Based on published information and experience in Windsor area with DMT

5.6.3 Serviceability Limit States (SLS) Assessment

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

The ground movements generated by the construction loads are anticipated to stabilize within approximately 10 to 15 years following completion of construction.

Due to the relatively smooth changes in the geometry of the tunnel, the above settlement changes along Highway 401 are anticipated to be gradual in longitudinal profile.

Table 5-12: Summary of Calculated Cumulative Deformations

Parameter	End of RSS Construction	End of Construction (Undrained)	Long-term (Drained)	Remarks
Settlements on Top of Ground at Distances (m) from the Edge of Deck of (†)				Figures F-4 & F-5
0 m†	N/A	60 mm (*)	40 mm	
5 m	N/A	70 mm (*)	45 mm	
10 m	N/A	75 mm (*)	55 mm	
20 m	40 mm (*)	50 mm (*)	35 mm	
Settlement at the top of RSS facing (mm)	60 mm (*)	110 mm (*)	90 mm	Figure F-6
Lateral displacement at the base of RSS facing	< 10 mm	< 25 mm	< 10 mm	Figure F-7
Rotation of the RSS facing	0.001	< 0.002	0.002	
Maximum Heave (rebound) at Highway 401	30 mm	25 mm	85 mm	Figure F-8

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

(†) Distances measured perpendicular to the tunnel abutment.

(*) Settlements compensated (corrected) during construction.

Note: The abutment design and soil properties assumed represent the north abutment configuration.

All the calculated ground movement and deformations calculated presented in this report are estimates based on soil deformation and compressibility properties interpreted from laboratory tests and empirical correlations. In this regard, the reported values are approximate and should be considered only as an approximate/preliminary 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.

Figures F-9 and F-10 show soil settlement and lateral soil displacements along the pile line determined from SDA, which were used in pile calculations in Section 5.5.

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

5.6.4 RSS Wall External Stability

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

Bearing Capacity: The following net bearing capacity values (q_u) were determined for the native subgrade soils at the two abutments for short-term (undrained) loading conditions (Table 5-13). Short-term (undrained) bearing capacity is based on an average cohesion of 55 kPa within the assumed zone of influence of the RSS wall foundation for the base-line. For the western segment of the south abutment cohesion for the underlying clay was assumed equal to 45 kPa. The long-term (drained) bearing capacity is based on a friction angle of 30°.

Table 5-13: Subgrade Ultimate Bearing Capacity

Abutment	Assumed Lowest Subgrade Elevation ⁽¹⁾	Loading Condition	q_u , kPa ⁽²⁾
South Wall – West Segment	171.5	Short-term	240
		Long-term	790
South Wall – East & Central Segment	172.5	Short-term	290
		Long-term	800
North Wall	172.5	Short-term	290
		Long-term	930

(1) Below RGM foundation

(2) All reported values are based on 1.5 m thick RGM

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

$$H_n = A'c' + V \tan \delta > 1.5 H_f$$

Where:

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

c' = cohesion/adhesion at sliding interface

δ = friction angle at sliding interface

V = vertical force (kN)

H_f = design horizontal load (kN)

Allowance for buoyancy should be made, where applicable. The following soil properties (Table 5-14) at the interfaces between the RSS, RGM and silty clay subgrade can be used in the design:

Table 5-14: Soil properties for use in ULS at Sliding

Interface	Undrained (Short-Term)		Drained (Long-Term)	
	δ , degrees	c, kPa	δ' , degrees	c', kPa
RSS to RGM	30	0	30	0
RGM to Silty Clay at South-West Segment	0	45		
RGM to Silty Clay at North & South-East Segments	0	55		

Based on geotechnical analyses discussed in Sections 5.6.1 to 5.6.4, tentative abutment configurations and dimensions were determined (Table 5-15). As noted previously in Section 5.6.1, the abutment configurations and dimensions indicated in these analyses are the minimum required and are to be finalized by proprietary suppliers. The final design of the abutments is to be developed in consultation with the suppliers of the proprietary components. The proposed abutment configurations are shown in Figure 5-2.

Table 5-15: Tentative Abutment Dimensions

Abutment	Station	Assumed Total Height ¹ , m	RSS Structure (Width \times Height ²), m	RGM (Min Thickness \times Width ³), m	LWF required within ⁴	EPS Volume, m ³ /m
South	14+510W to	10.2	5.5 \times 4.8	1.5 \times 7.0	Pile cap straps and 3.3 m within RSS	58.5
	14+540W to	9.6	5.5 \times 4.7	1.5 \times 7.0	Pile cap straps and 0.5 m within RSS	19.0
North	14+510W to	11.5	7.5 \times 6.9	1.5 \times 9.0	Pile cap straps and 4.0 m within RSS	34.5
	14+540W to	10.9	7.0 \times 6.5	1.5 \times 8.5	Pile cap straps and 3.0 m within RSS	35.5
	14+600W to	10.1	6.5 \times 5.7	1.5 \times 8.0	Pile cap straps and 1.0 m within RSS	8.5

- (1) Maximum height, measured from top of finished grade at tunnel edge to the base of the RSS structure
- (2) Measured between underside of the stem (pile cap) and bottom of RSS
- (3) Width measured at the base of RGM
- (4) In general, the use of RGM, LWF and EPS is required to meet the design compliance for undrained short-term condition.
- (5) 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.

5.6.5 RGM Foundation – Loads & Design

RGM foundation comprising Granular B Type II was considered under the RSS false abutment walls to improve the load distribution to the bearing soils and satisfy the ULS bearing capacity requirements for undrained conditions at the north and south abutments. A simplified analytical approach was used considering that the RGM foundation distributes the vertical pressures at the base of the RSS walls to the subgrade below the RGM within a 45 degree angle. The following loads (Table 5-16) were estimated to act on top of the RGM on the basis of conventional calculation of the bearing pressures under gravity retaining walls.

Table 5-16: Estimated load on 1.5 m thick RGM at the Underside of RSS

Abutment Location	Average Unfactored Bearing Pressure, kPa
South Wall – West Segment	115
South Wall – East & Central Segments	150
North Wall	170

Based on the above loads on RGM, an estimated factored horizontal tensile load of 40 to 70 kN per meter of RGM was estimated across the entire RGM height of 1.5 m at the north, south/east and south/west abutments, respectively. For cost estimates, this tensile load can be accommodated by 3 layers of UX1100HS, or equivalent.

5.7 Wingwalls

The wingwalls constitute of tapered RSS walls parallel to the tunnel centre line and outside the tunnel footprint, and return walls perpendicular to the abutment.

Global stability analyses have been carried out on RSS tapered walls (highest section) and return wall at four corners of the tunnel. The calculated factors of safety against global instability are in excess of 1.3 and 1.5 for short-term and long-term conditions, respectively.. A summary of calculated factors of safety is listed in Table 5-17. Figures E-10 to E-21 in Appendix E illustrate the stability models for the wing walls.

Table 5-17: Calculated Factors of Safety for Wing Wall Global Instability

Wing Wall Location	Maximum Wall Height, m	RSS Width, m	Minimum Calculated Factor of Safety		
			Undrained		Drained
			Short-Term	End-of-Construction	Long-Term
North West Tapered Wall	6.9	7.5	1.42 (1.27) ¹	1.47 (1.32)	1.61 (1.53)
North East Tapered Wall	5.7	6.5	1.52 (1.35)	1.68 (1.48)	1.69 (1.57)
South West Tapered Wall	5.3	5.5	1.34 (1.25) ¹	1.46 (1.37)	1.68 (1.57)
South East Tapered Wall	3.5	5.0	1.45 (1.28)	1.61 (1.41)	1.56 (1.42)
South Return Wall	4.9	5.0	1.74 (1.66)	1.94 (1.83)	2.04 (1.86)
North Return Wall	4.4	4.5	1.69 (1.63)	1.83 (1.76)	2.17 (2.06)

(*) Values outside the parentheses refer to circular failure surface and the values in the parentheses refer to non-circular failure surfaces.

(1) RSS toe berm and granular base over WBR5/EBR5 must be placed before any backfill is placed above the deck seat level.

As show in Figures E-14 and E-15, 50 m² of EPS has been utilized as back fill in the south west tapered wall to meet the global stability and bearing capacity requirements at this corner of tunnel, where weak clays have been encountered during investigations.

Similar to the abutment RSS walls, the RSS tapered and return walls have been checked for external stability. In case of tapered walls, bearing capacity checks have been carried out for two configurations, namely long and short panels. The long panel refers to the 5 m long RSS wall immediately outside the tunnel footprint, while the short panel (assumed 1.5 m lower) represent the remainder of tapered wall before it connects to the highway returning walls. Table 5-18 shows the recommended RGM and LWF requirements based on the external stability of the RSS walls.

The wingwall configurations and dimensions indicated in these analyses and shown in Figure 5-3 show the minimum width required and are to be finalized by proprietary suppliers. The design of the abutments is to be developed in consultation with the proprietary component suppliers.

Table 5-18: Tentative Wing Wall Dimensions

Wing Wall Location		Maximum Wall Height, m	RGM Thickness, m	RSS Width, m	LWF required within RSS
North West Tapered Wall	Long	6.9	1.5	7.5	Yes
	Short	5.4	1.0	5.0	No
North East Tapered Wall	Long	5.7	1.5	6.5	No
	Short	4.2	1.0	4.5	No
South West Tapered Wall	Long	5.3	1.5	5.5	Yes
	Short	3.8	1.0	4.5	No
South East Tapered Wall	Long	3.5	-	5.0	No
	Short	2.0	-	3.5	No
South Return Wall		4.9	-(*)	5.0	Yes
North Return Wall		4.4	-(*)	4.5	Yes

(*) Minimum 300 mm granular mat required to be placed above approved subgrade prior to RSS construction.

5.8 Backfilling

Behind the concrete abutment and wing walls, non-frost susceptible free draining granular fill should be placed in accordance with the CHBDC (ref. R-9).

The granular backfill should be compacted in maximum 200 mm thick loose lifts in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to ensure positive drainage of the backfill. Other aspects of the abutment backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150.

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.

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

Table 5-19: Soil Parameters for Earth Pressure Calculations

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

Note: Values are given for level backfill and ground surface behind the wall compacted to > 95% Standard Proctor maximum dry density. The coefficients of lateral earth pressure should be adjusted if there is sloping ground at the back of the wall.

Group I Soils: Coarse grained soils (e.g. Granular A and B Type 2)

Group II Soils: Finer grained than Group I noncohesive soils (e.g. Granular B Type1, pit run, etc)

Group III Soils: Finer grained soils (e.g. approved site generated silty clay)

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

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: ϕ = Friction angle of backfill material,

β = Slope of the backfill surface.

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 CHBDC.

For retained backfill that is placed and compacted in layers, the lateral force caused by compaction shall 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.

5.9 Permanent Subdrainage System

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

Use of free draining granular soils within the reinforced soil mass within the RSS structures and the RGM will ensure that these structures will act as “natural” drains conveying the seepage resulting 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 RSS and RGM materials, a filter layer may be required at the interface between the native soils and imported granular materials. Given the grain size uniformity, the LWF should be wrapped in filter fabric to prevent the migration of the fines from adjacent fills and soils.

A simplified steady-state model (Appendix H) was used to estimate seepage rates associated with the long-term drawdown of the groundwater along a typical cross-section of Tunnel T-5. SEEP/W 2007 software was used for this analysis. Groundwater recharge from infiltrations from ground surface sources was also considered. The rate of recharge was estimated on the basis of saturated hydraulic conductivity of the soils in conjunction with the assumption that no mounding of the long-term groundwater should occur. The initial groundwater table was assumed at elevations 179. A ground surface infiltration of 5×10^{-4} m/day (20% infiltration coefficient \times 918.3 mm/yr) was accommodated by trial-and-error approach to ensure a sustained groundwater level consistent with the hydraulic conductivity of the subsurface soils.

Based on the above, the flow rate from groundwater seepage across the entire tunnel cross section was estimated to be 2.4 litre/day per meter length of the tunnel. This is an approximate estimate and the actual quantities could differ significantly from this magnitude. The above flow 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 General Construction Requirements

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

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

- All excavation works should be carried out in accordance with the guidelines outlined in Occupational Health and Safety Act (OHSA) and OPSS 902. The native undisturbed soils may be classified as Type 3 soils. The excavations below the original ground levels may intersect water bearing backfill within trenches of active and/or abandoned utilities. In these cases, Type 4 soil conditions may occur and should be addressed accordingly.
- The silty clay soils at the project site are highly susceptible to rapid deterioration when exposed to elements, weathering and/ or subjected to direct construction traffic.
- Temporary slopes, permanent slopes, and subgrade areas must be appropriately protected at all times against surface erosion due to runoff, desiccation, freeze-thaw effects, etc.
- To protect the integrity of subgrade for foundations and pavements, the final excavation lift above the design elevation should not be less than 500 mm and should be carried out only when the Contractor is ready to prepare and cover the subgrade with the materials specified in the design same day the final excavation is exposed and approved. No construction traffic should be permitted over subgrade without approved protective covers.
- The final excavation lift above the design subgrade should be carried out using buckets equipped with smooth lips. Once exposed, the subgrade must be immediately inspected. Upon approval, the subgrade should be immediately protected; depending on the type of construction, geofabrics, granular mats, a skim coat (minimum 75 mm thick) of lean concrete protection (mud mat), etc. should be used.
- As indicated earlier, pore pressures, heave/settlement behaviour and presence of gassy soils below the excavation should be monitored diligently during excavation. If the presence of gassy soils is evidenced (for example, dissolved gas bubbles coming out of solution and softening of the excavation face), the excavation should be carried out in small (say 1 m) depth increments and sufficient time to dissipate the pore pressures should be allowed at each excavation stage.
- Regular monitoring and inspections of the condition of the temporary slopes for signs of instability, deterioration, sloughing, etc should be carried out by qualified personnel. Appropriate mitigation measures should be implemented.

- Excavations in this area should be limited in size in the area and appropriate monitoring of the residence should take place. Monitoring should consist of a precondition survey along with regular surveying conducted of the nearby utilities, residences, etc.

6.2 Construction Dewatering

The design of the dewatering system should comply with the OPSS 517 and 518 provisions.

Due to the relatively low permeability of the silty clay deposit, minor groundwater seepage is anticipated, 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 excavations slopes. In adverse conditions, the runoff and seepage from perched groundwater and sand/silt pockets can be significant and accompanied by piping and wash-outs of the fines causing sloughing of the slopes.

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

Effective drainage is an important aspect in the life expectancy and performance of any abutment wall, wing wall, or pavement structure associated with the tunnel. Permanent sub-drainage should be installed behind abutment and wing walls. Free draining granular material (Granular B Type 1 or approved equivalent) should be installed immediately adjacent to walls to prevent water pressures acting on the walls and to permit downward flow of surface water down into the wall sub-drains. The subdrains should be surrounded by approved granular material and discharged via gravity flow to the storm drain or road ditch system along Highway 401.

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

6.3 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 monitoring of the response of the excavations (ground movement) and backfilling during construction. Detailed plans and procedures should be submitted to HMQ for approval at least 3 month prior to commencement of the monitoring of the works.

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

6.4 Corrosion Potential

Analytical testing was carried out on samples of the silt and clay stratum obtained in Boreholes BH T5-1 (Sample 25), BH T5-2 (Sample 24) and BH T5-3 (Sample 11). Table 6-2 summarizes the results of various analyses carried out on the soil samples to assess the potential for corrosion on concrete.

Table 6-1: Results of Analytical Testing on Soils

Location of Soil Samples	Elevation of Soil Sample	pH	Redox Potential, mV	Resistivity, ohm.cm	Sulphide, mg/kg	Sulphate, mg/kg
Borehole BH T5-1 (Sample 25)	150.8	7.83	246	1900	<0.2	582
Borehole BH T5-2 (Sample 24)	153.9	7.89	260	1860	<0.2	387
Borehole BH T5-3 (Sample 11)	170.8	8.01	234	1830	<0.2	520

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 soil would be considered to have a potential for corrosion to buried metallic elements AWWA (ref. R-2).

The above results and recommendations should be reviewed by a corrosion specialist.

6.5 Construction Quality Control

To ensure that construction is carried out in a manner consistent with the intent of the recommendations set forth in this report, a program of geotechnical inspection and testing should be developed and implemented throughout the construction phase. In addition, related laboratory testing should be carried out in conjunction with the fieldwork to monitor compliance with the various materials and project specifications.

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

7 Limitations of Report

The work performed in this report was carried out in accordance with the Standard Terms and Conditions made part of our contract. The conclusions and recommendations presented herein are based solely upon the scope of services and time and budgetary limitations described in our contract.

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

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

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

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

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

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

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

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

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

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

8 Closure

The design for Tunnel T-5 was developed by Dr. Siavash Farhangi, P.Eng. 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. (Technical Director) who also provided the senior review of the report. Mr. Matt Oldewening, P.Eng. managed the geotechnical investigation and Mr. Brian Lapos, P.Eng. is the project manager.

Mr. Zuhtu Ozden, P.Eng. and Dr. Andrew Smith of Coffey Geotechnics provided peer review. The cooperation received from Ms. Biljana Rajlic, P.Eng. and Mr. Philip Murray, P.Eng. of Hatch Mott McDonald and Mr. Daniel Muñoz, P.Eng. of PIC during the design study is gratefully acknowledged.

Yours truly,

**AMEC Environment & Infrastructure,
a division of AMEC Americas Limited**



Siavash Farhangi, Ph.D., P.Eng.
Senior Geotechnical Engineer



Dan Dimitriu, Ph.D., P.Eng,
Associate Geotechnical Engineer



Narendra S. Verma, Ph.D., P.Eng, F.ASCE, D.GE.
Principal Geotechnical Engineer
(Designated MTO RAQS Contact)

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. 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-8. Canadian Geotechnical Society, 2006, Canadian Foundation Engineering Manual (CFEM), 4th Edition.
- R-9. Canadian Standard Association, 2006, Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-06 S6.1.06.
- R-10. Canadian Standard Association, 2009, Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete CAN/CSA-A23.
- R-11. 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-12. 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-13. Department of the Navy, 1986, Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2, Naval Facilities Engineering Command.
- R-14. 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-15. Ensoft Inc., 2004. LPILE Technical Manual.

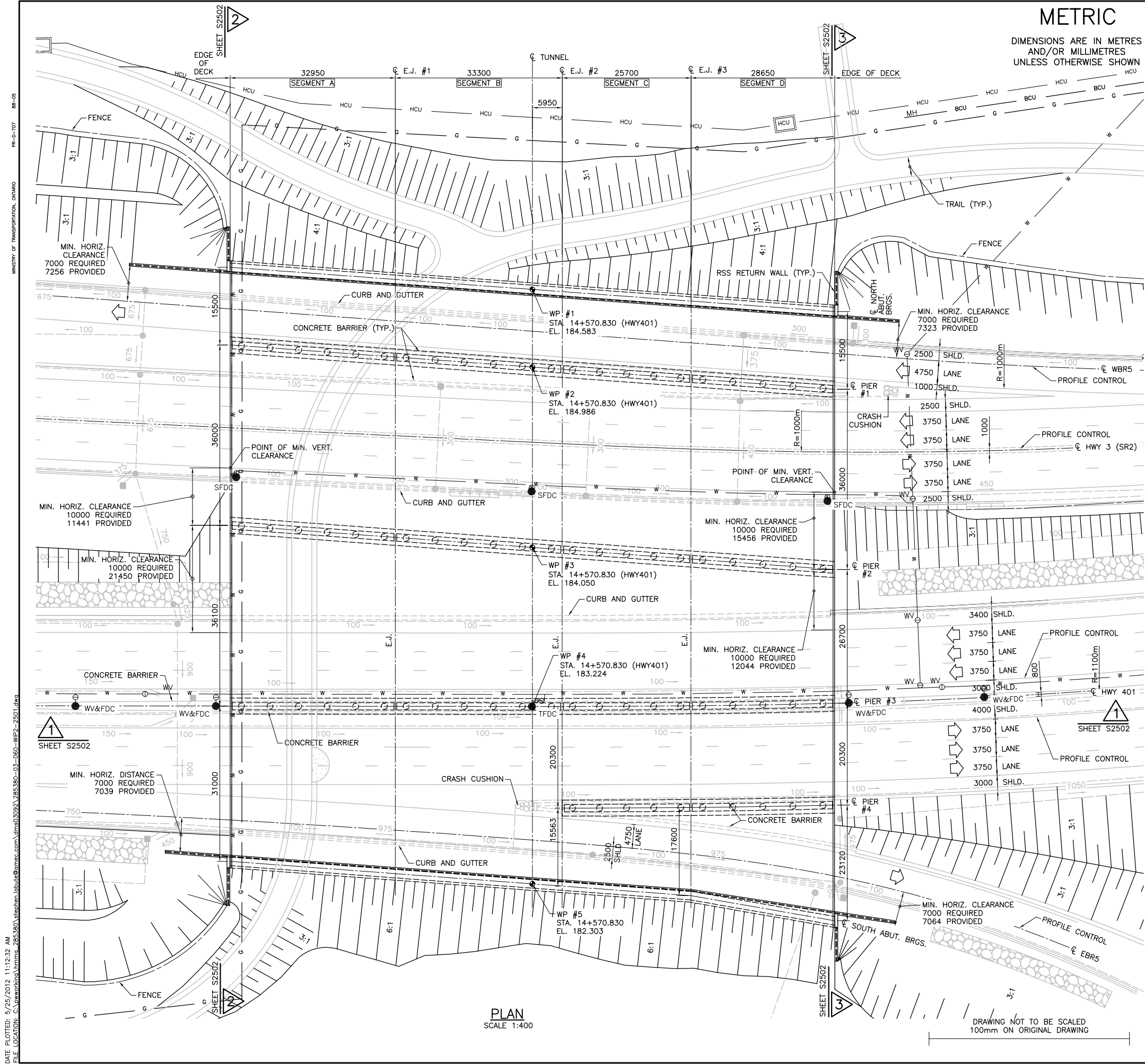
- R-16. Golder Associates Ltd., 2007, Preliminary foundation investigation and design report, Detroit River International Crossing Bridge Approach Corridor, Geocres No. 40J6-18, October.
- R-17. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Geocres No. 40J6-27, June.
- R-18. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Subsurface Conditions – Baseline Report, Geocres No. 40J6-28, June.
- R-19. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Subsurface Conditions – Interpretation Report, Geocres No. 40J6-28, Revision December.
- R-20. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 1 – Soil Chemistry Data, Geocres No. 40J6-27, February.
- R-21. 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-22. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 3 – Supplementary Cone Penetration Testing, Geocres No. 40J6-27, February.
- R-23. Golder Associates Ltd., 2009, Windsor-Essex Parkway, Geotechnical Data Report, Addendum No. 4 – Supplementary Geotechnical Investigation, March.
- R-24. 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-25. Grozic, J.L., Robertson, P.K., and Morgenstern, N.R., 1999, The behaviour of loose gassy sand, Canadian Geotechnical Journal, 36, 482-492.
- R-26. Hudec, P.P., 1998, Geology and Geotechnical Properties of Glacial Soils in Windsor.
- R-27. 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-28. 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-29. Kenney, T.C., 1959, Discussion of Geotechnical Properties of Glacial Lake Clays, by T.H. Wu, Journal of the Soil Mechanics and Foundations Division, A SCE, Vol. 85, No. SM 3, PP. 67 – 79.
- R-30. 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-31. 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-32. 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-33. 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-34. Leroueil, S., Magnan, J-P., and Tavenas, F., 1990, Embankments on Soft clays, Ellis Horwood.
- R-35. 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-36. 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-37. Lunne, T., Robertson, P.K., and Powel, J., 1997, Cone Penetration Testing in Geotechnical Practice.
- R-38. Ministry of Transportation Ontario, 1990, Pavement Design and Rehabilitation Manual, SDO-90-01.
- R-39. National Highway Institute, Federal Highway Administration, November 2009, Design of Mechanically Stabilized Earth Walls and Reinforced Walls and Reinforced Soil Slopes – Volume I, FHWA-NHI-10-024.
- R-40. 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-41. Sobkowicz, J.C. and Morgenstern, N.R., 1984, The undrained equilibrium behaviour of gassy sediments, Canadian Geotechnical Journal, Vol. 21, pp. 439-448.
- R-42. Terzaghi, K., Peck, R.B., and Mesri, G., 1990, Soil Mechanics in Engineering Practice, John Wiley and Sons, NY.
- R-43. Windsor-Essex Mobility Group, 2010, Design Submission, Section 5.1.3 – Geotechnical Design.
- R-44. 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-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

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



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

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

NEW CONSTRUCTION
HWY 401
OAKWOOD TUNNEL T-5
GENERAL ARRANGEMENT

SHEET
S2501
Phase 2
IFC

NOTES:
1. FOR GENERAL NOTES SEE
SHEET S2503.

PROFILE OF HWY3 (SR2)
N.T.S.

PROFILE OF WBR 5
N.T.S.

PROFILE OF HWY 401
N.T.S.

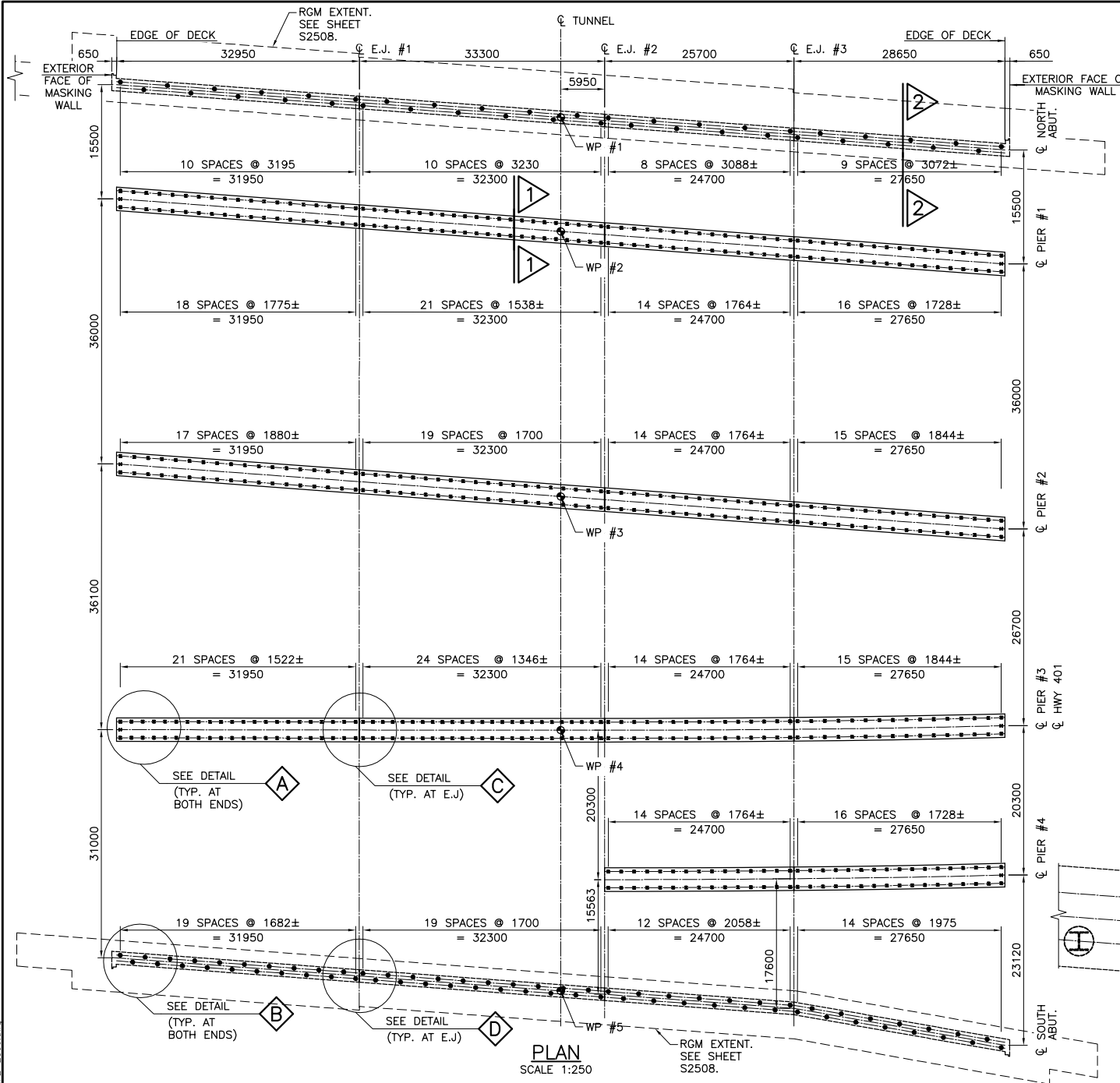
PROFILE OF EBR 5
N.T.S.

NOT FOR
CONSTRUCTION

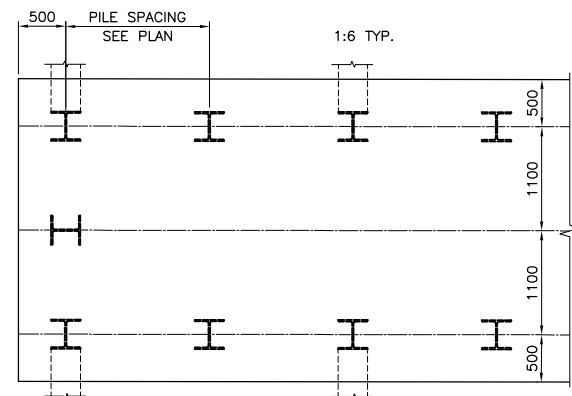
REVISIONS	
DATE	REV. BY
16-MAY-12	0 MY
ISSUED FOR CONSTRUCTION	

DESIGN	YEL	CHK	BR	CODE	CAN/CSA	S6-06	LOAD	CL-625-ONT
DRAWN	DM	CHK	MAS	SITE	6-705	DATE	18-JUL-11	
285380-03-060-WIP2-2501								

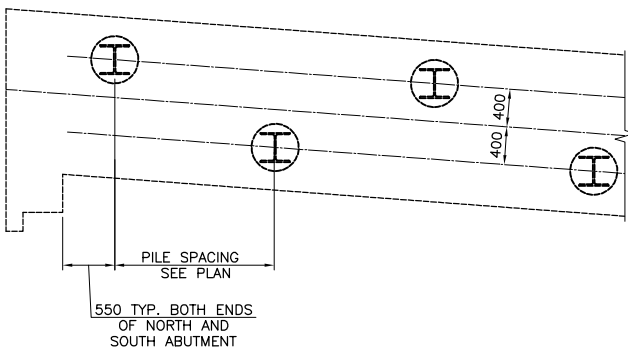
DATE PLOTTED: 5/25/2012 11:13:27 AM
FILE LOCATION: C:\working\mimg\285380\stephen.libault\ames.com\dms03092\285380-03-061-WP2-2506.dwg
MINISTRY OF TRANSPORTATION, ONTARIO
PR-D-707
BB-05



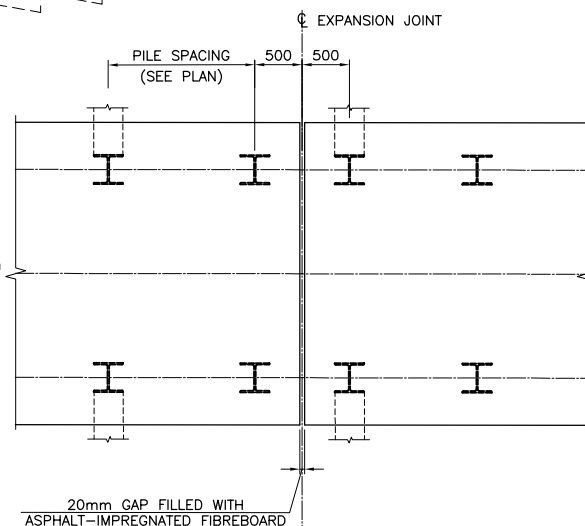
PLAN
SCALE 1:250



SCALE 1:40

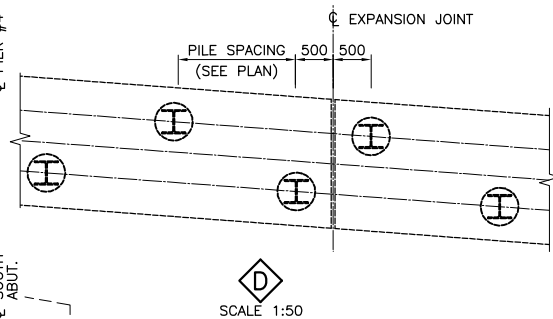


SCALE 1:40



SCALE 1:40

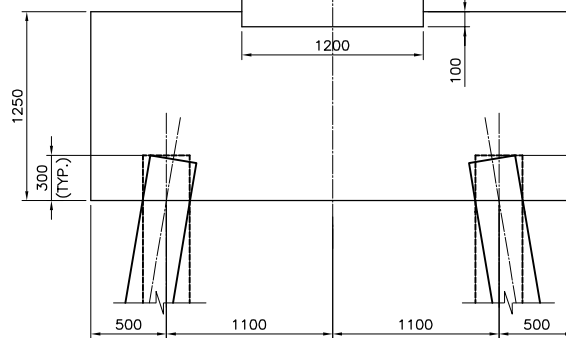
TABLE 2506-2 PILE DATA			
LOCATION	No. REQUIRED	ESTIMATED LENGTH (m)	BATTER
N. ABUTMENT	41	33.4	VERTICAL
PIER #1	74	25.7	VERTICAL
	74	26.1	1:6
PIER #2	70	24.8	VERTICAL
	70	25.1	1:6
PIER #3	80	23.6	VERTICAL
	78	23.9	1:6
PIER #4	33	23.8	VERTICAL
	32	24.1	1:6
S. ABUTMENT	68	30.3	VERTICAL



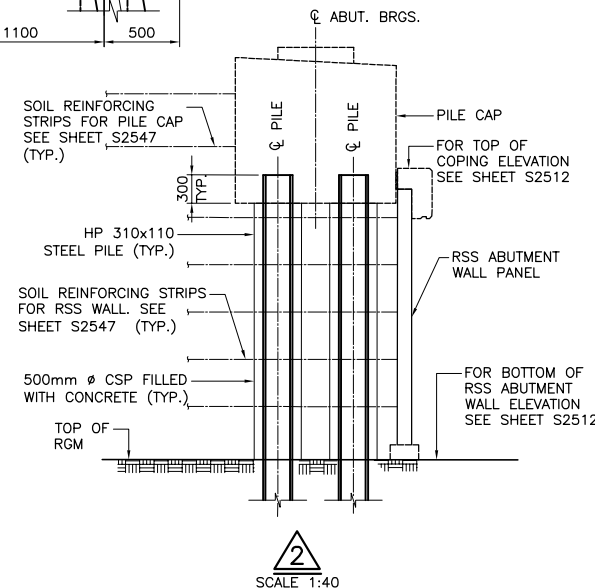
SCALE 1:50

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



SCALE 1:25



SCALE 1:40

LAYOUT NOTES:

- LAYOUT ϕ TUNNEL AS PER WP'S.
- LAYOUT THE LIMITS OF TUNNEL BY OFFSETTING ϕ TUNNEL EAST AND WEST BY HALF THE TOTAL LENGTH OF TUNNEL.
- LAYOUT ϕ E.J.'S BY OFFSETTING THE LIMITS OF TUNNEL BY THE SEGMENT LENGTHS SHOWN ON PLAN.
- LAYOUT ϕ PIER #3 ALONG ϕ HWY 401.
- LAYOUT ϕ PIER #4 BY TRANSLATING ϕ HWY 401 SOUTH BY A CONSTANT DISTANCE AS SHOWN ON PLAN.
- LAYOUT ϕ 'S OF NORTH ABUTMENT, PIER #1 AND PIER #2 AS TANGENT LINES WITH END DISTANCES TO ϕ HWY401 AT TUNNEL LIMITS AS SHOWN ON PLAN.
- LAYOUT ϕ SOUTH ABUTMENT AS TWO TANGENT LINES JOINED AT ϕ E.J. #3, WITH END DISTANCES TO ϕ PIER #3 AND ϕ PIER #4 AS SHOWN ON PLAN.

TABLE 2506-1 WORKING POINT DATA		
LOCATION	NORTHING	EASTING
WP #1	4 680 003.303	331 982.712
WP #2	4 679 998.793	331 967.882
WP #3	4 679 988.319	331 933.440
WP #4	4 679 979.080	331 903.062
WP #5	4 679 968.775	331 869.177

APPLICABLE STANDARD DRAWINGS:

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

NOT FOR
CONSTRUCTION

Parkway
Infrastructure
Engineers



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

NEW CONSTRUCTION
HWY 401
OAKWOOD TUNNEL T-5
FOUNDATION LAYOUT



SHEET
S2506

Phase 2
IFC

NOTES:

- FOR GENERAL NOTES SEE SHEET S2503.
- 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 ULTIMATE GEOTECHNICAL RESISTANCE, AND PREVENT DAMAGE 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 GAS 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. CSP PIPES AND PROVIDE BLOCKING OF PILES IN CSP PIPES.
- CONSTRUCT RETAINED SOIL SYSTEM WALLS AND BACKFILL TO UNDERSIDE OF ABUTMENTS. BACKFILL AS PER DWG. S2508.
- FILL 500mm DIA. CSP PIPES WITH CONCRETE.
- CONSTRUCT ABUTMENTS TO UNDERSIDE OF BEARING PEDESTALS.
- CONTRACTOR TO PROVIDE SUITABLE STABILITY DURING CONSTRUCTION.
- SEE ABUTMENT LAYOUT 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 LAYOUT FOR CONTINUATION.

REVISIONS		DATE	REV.	BY	DESCRIPTION
16-MAY-12	0	MY	ISSUED FOR CONSTRUCTION		
DESIGN	YEL	CHK	BR	CODE CAN/CSA S6-06	LOAD CL-625-ONT
DRAWN	DM	CHK	MAS	SITE 6-705	DATE 18-JUL-11

DOC: 285380-03-061-WP2-2506

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



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

NEW CONSTRUCTION
HWY 401
OAKWOOD TUNNEL T-5
BOREHOLE LOCATIONS & SOIL STRATA

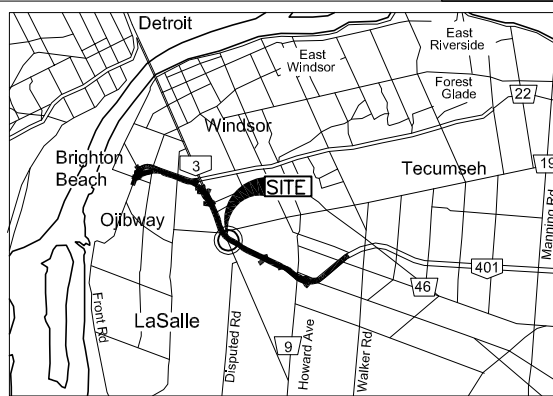


SHEET

G2504

Phase 2

IFC



KEY PLAN

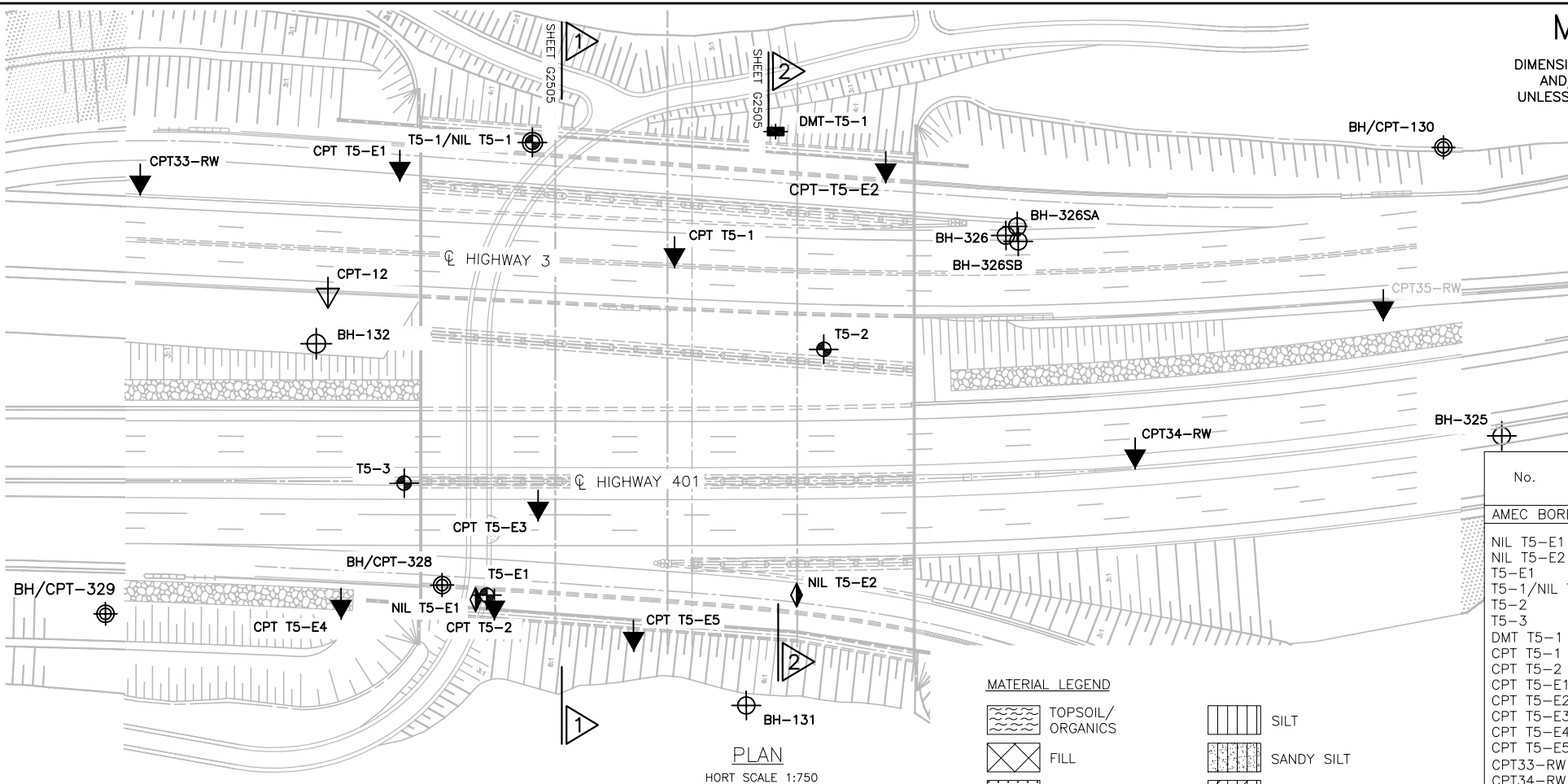
SCALE
1 0 2 4Km

LEGEND

- BOREHOLE CURRENT INVESTIGATION
- BOREHOLE AND NILCON VANE CURRENT INVESTIGATION
- NILCON VANE CURRENT INVESTIGATION
- CPT - CURRENT INVESTIGATION
- DMT - CURRENT INVESTIGATION
- BOREHOLE PREVIOUS INVESTIGATION
- PREVIOUS INVESTIGATIONS
- CPT - PREVIOUS INVESTIGATION
- BOREHOLE, CPT AND NILCON VANE SPT N-VALUE
- BLOWS/0.3M UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
- SEAL
- STANDPIPE
- 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

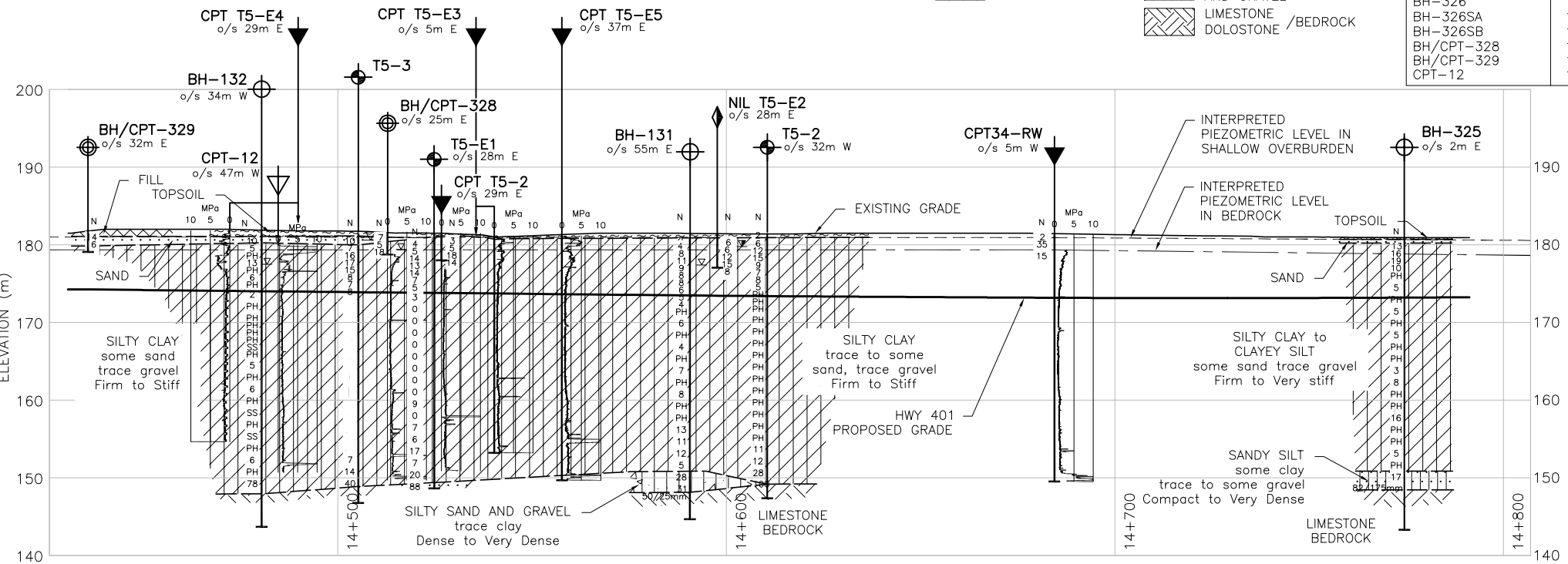
- THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
- THE INTERPRETED STRATIGRAPHY REPRESENTS SIMPLIFIED SUBSURFACE CONDITIONS. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN DEFINED AT BOREHOLE LOCATIONS ONLY. CONDITIONS BETWEEN BOREHOLE LOCATIONS COULD DIFFER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



MATERIAL LEGEND

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

No.	ELEVATION	CO-ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC BOREHOLES			
NIL T5-E1	181.3	4680015.5	331861.8
NIL T5-E2	181.1	4679940.9	331885.7
T5-E1	181.1	4680013.2	331863.7
T5-1/NIL T5-1	181.3	4680034.6	331972.6
T5-2	181.3	4679952.3	331945.1
T5-3	181.5	4680040.4	331884.0
DMT T5-1	181.1	4679978.7	331992.4
CPT T5-1	181.2	4679994.0	331958.7
CPT T5-2	180.9	4680010.9	331863.0
CPT T5-E1	181.4	4680064.3	331958.7
CPT T5-E2	181.0	4679950.8	331992.9
CPT T5-E3	181.2	4680008.0	331889.1
CPT T5-E4	181.9	4680046.8	331852.3
CPT T5-E5	181.2	4679976.4	331865.6
CPT33-RW	181.4	4680121.7	331932.6
CPT34-RW	181.3	4679865.7	331958.2
PREVIOUS BOREHOLES			
BH/CPT-130	180.8	4679821.8	332036.1
BH-131	180.8	4679944.8	331856.4
BH-132	181.5	4680070.8	331937.0
BH-325	180.8	4679787.7	331972.9
BH-326	181.8	4679917.6	331984.5
BH-326SA	181.8	4679915.6	331987.4
BH-326SB	181.8	4679914.3	331984.0
BH/CPT-328	181.6	4680024.3	331862.9
BH/CPT-329	182.0	4680100.8	331832.3
CPT-12	181.6	4680072.0	331924.0



PROFILE ALONG CL OF HWY 401

HORT SCALE 1:750
VERT SCALE 1:375

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS	27-MAR-12				SF	ISSUED FOR CONSTRUCTION
	DATE	REV.	BY	DESCRIPTION		
DESIGN	SF	CHK	NSV	CODE CAN/CSA S6-06	LOAD	CL 625-ON
DRAWN	MM	CHK	DD	SITE	6-705	DATE 18-JUL-11

DATE PLOTTED: 5/25/2012 11:06:49 AM
FILE LOCATION: C:\working\hmm\285380\401-WP2-2505.dwg
MINISTRY OF TRANSPORTATION, ONTARIO
PR-D-707
BB-05

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

Parkway Infrastructure Engineers

amc
Hatch Mott MacDonald

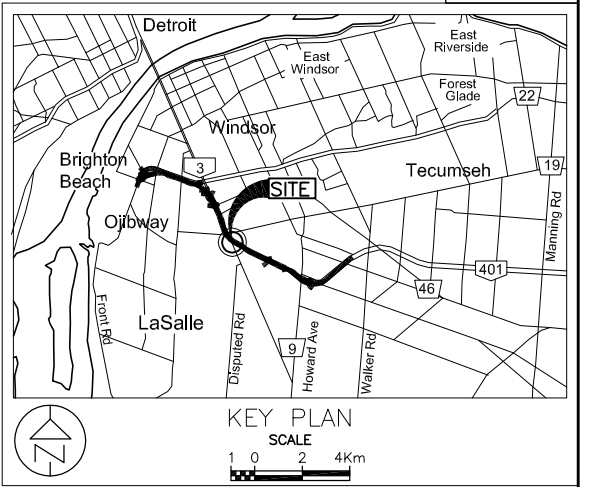
Windsor-Essex
Parkway Project
RFP No. 09-54-1007

NEW CONSTRUCTION

HWY 401
OAKWOOD TUNNEL T-5
SOIL STRATIGRAPHY

SHEET
G2505

Phase 2
IFC



LEGEND

BOREHOLE
CURRENT INVESTIGATION

BOREHOLE AND NILCON VANE
CURRENT INVESTIGATION

NILCON VANE
CURRENT INVESTIGATION

CPT - CURRENT INVESTIGATION

DMT - CURRENT INVESTIGATION

BOREHOLE
PREVIOUS INVESTIGATION

CPT -PREVIOUS INVESTIGATION

BOREHOLE, CPT AND NILCON VANE
SPT N-VALUE

BLOWS/0.3M UNLESS
OTHERWISE STATED
(STD. PEN. TEST, 475 J/BLOW)

SEAL

STANDPIPE

DRY BOREHOLE DRY DURING DRILLING

WATER LEVEL DURING DRILLING

WATER LEVEL (SHALLOW PIEZO)

WATER LEVEL (DEEP PIEZO)

MHSg - MAGNETIC
HEAVE/SETTLEMENT
GAUGE

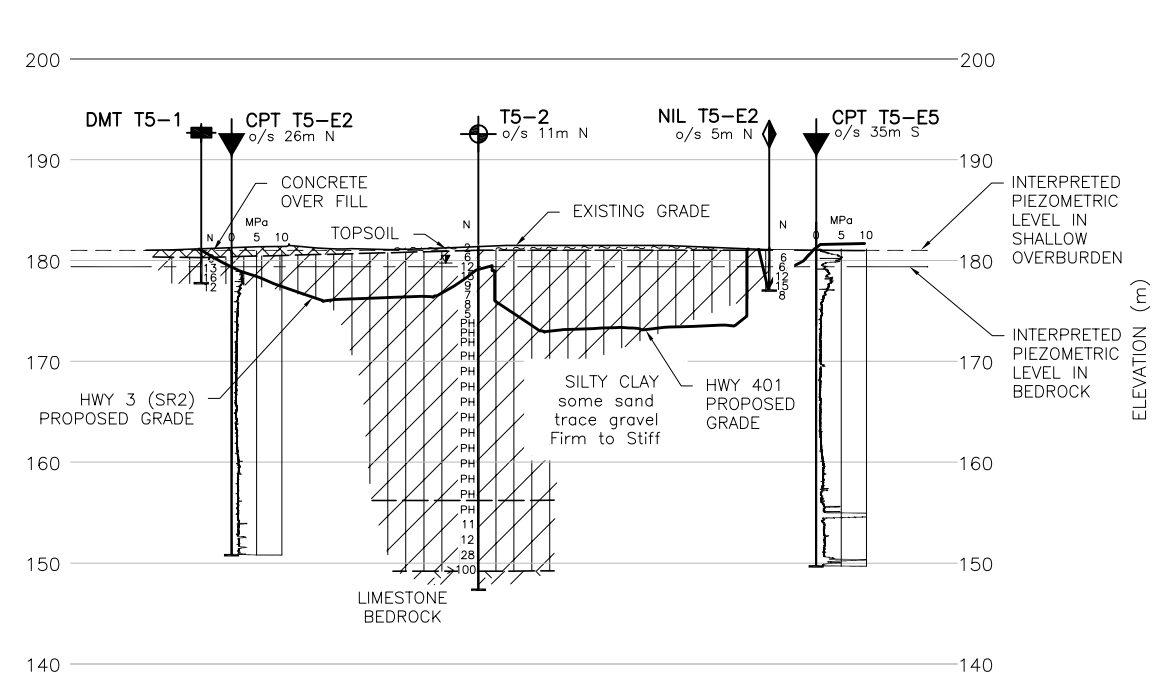
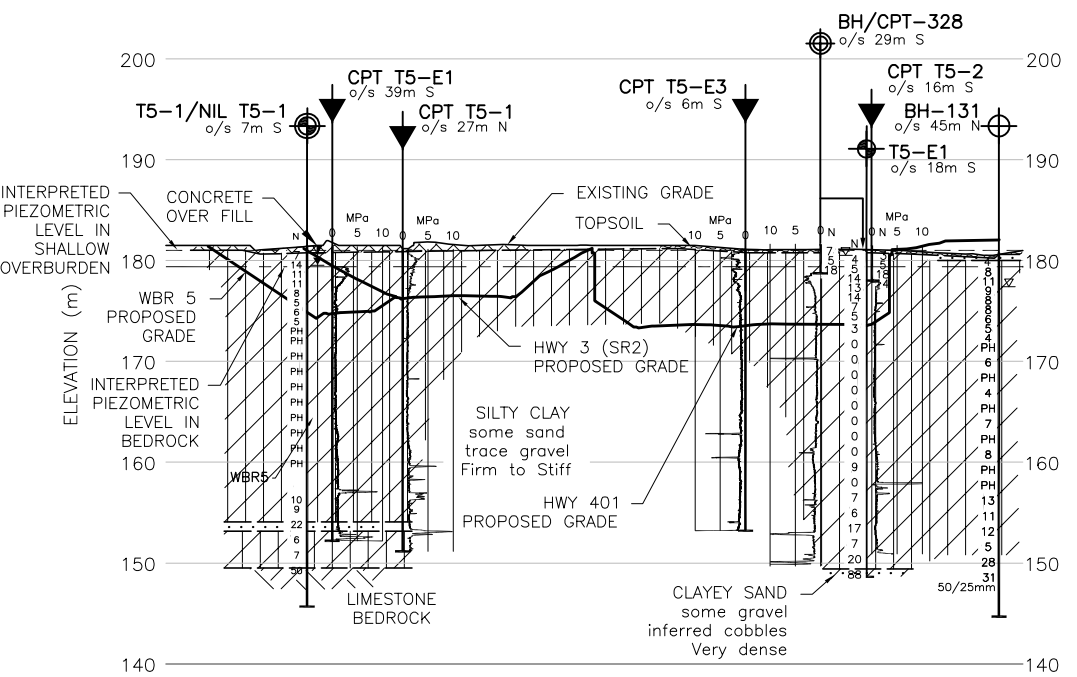
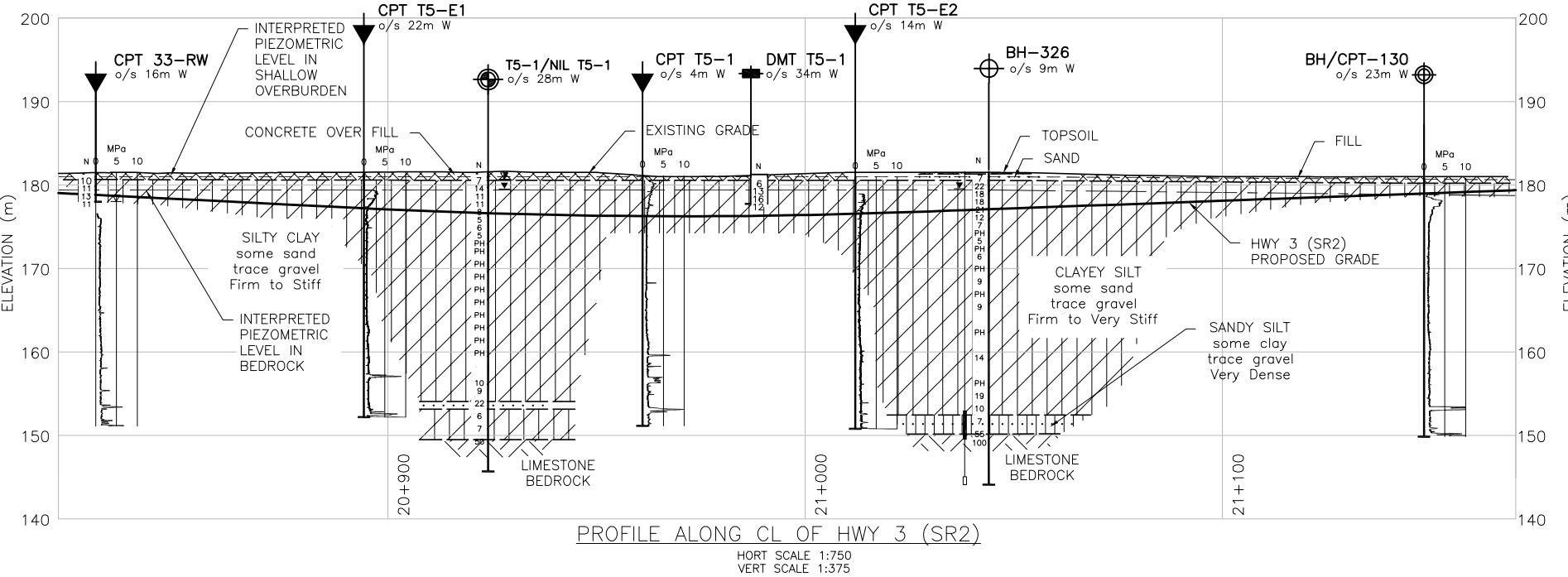
CPT, qc

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.

REVISIONS		DATE	REV.	BY	DESCRIPTION
27-MAR-12		0	SF	ISSUED FOR CONSTRUCTION	
DESIGN	SF	CHK	NSV	CODE CAN/CSA S6-06	LOAD CL-625-ON
DRAWN	MM	CHK	DD	SITE 6-705	DATE 18-JUL-11

DOC: 285380-04-091-WP2-2505



No.	ELEVATION	CO-ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
MEC BOREHOLES			
L T5-E1	181.3	4680015.5	331861.8
L T5-E2	181.1	4679940.9	331885.7
-E1	181.1	4680013.2	331863.7
-1/NIL T5-1	181.3	4680034.6	331972.6
-2	181.3	4679952.3	331945.1
-3	181.5	4680040.4	331884.0
MT T5-1	181.1	4679978.7	331992.4
PT T5-1	181.2	4679994.0	331958.7
PT T5-2	180.9	4680010.9	331863.0
PT T5-E1	181.4	4680064.3	331958.7
PT T5-E2	181.0	4679950.8	331992.9
PT T5-E3	181.2	4680008.0	331889.1
PT T5-E4	181.9	4680046.8	331852.3
PT T5-E5	181.2	4679976.4	331865.6
PT33-RW	181.4	4680121.7	331932.6
PT34-RW	181.3	4679865.7	331958.2



No.	ELEVATION	CO—ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
PREVIOUS BOREHOLES			
BH/CPT—130	180.8	4679821.8	332036.1
BH—131	180.8	4679944.8	331856.4
BH—132	181.5	4680070.8	331937.0
BH—325	180.8	4679787.7	331972.9
BH—326	181.8	4679917.6	331984.5
BH—326SA	181.8	4679915.6	331987.4
BH—326SB	181.8	4679914.3	331984.0
BH/CPT—328	181.6	4680024.3	331862.9
BH/CPT—329	182.0	4680100.8	331832.3
CPT—12	181.6	4680072.0	331924.0

LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

TOPSOIL/
ORGANICS

FILL

SAND

SILTY CLAY

SILTY SAND

SILT

SANDY SILT

CLAYEY SILT

SAND AND GRAVEL

SILTY SAND
AND GRAVEL

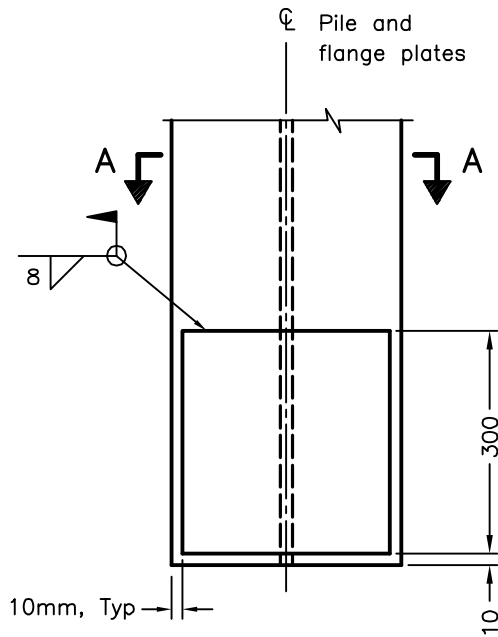
LIMESTONE
DOLOSTONE /BEDROCK

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

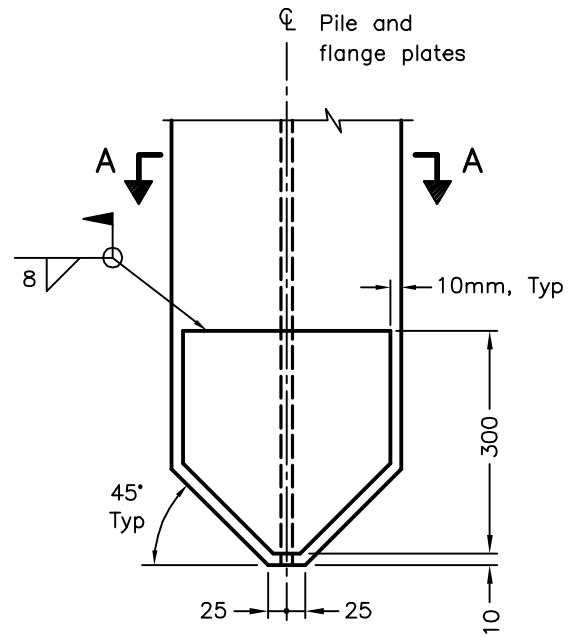
OPSDs

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

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

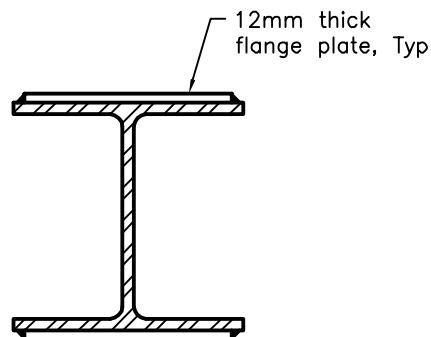


TYPE I



TYPE II

ELEVATION



PILE DRIVING SHOE
SECTION A-A

NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

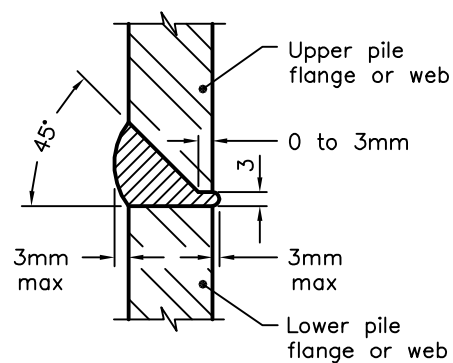
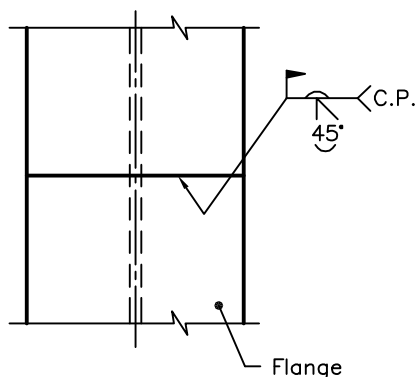
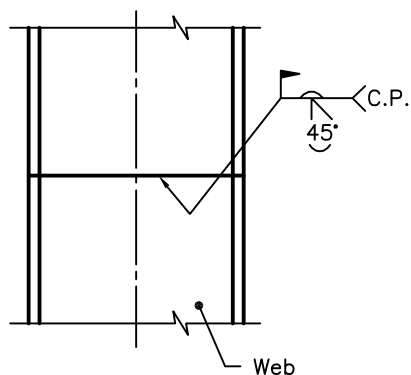
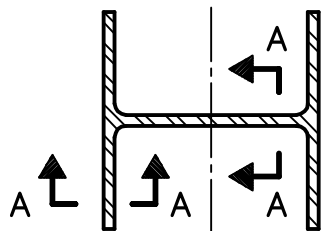
Rev 2

FOUNDATION
PILES

STEEL H-PILE DRIVING SHOE

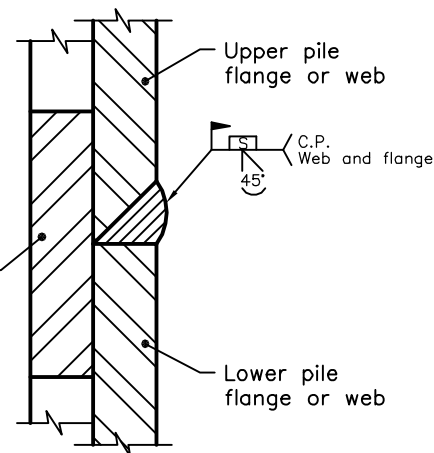
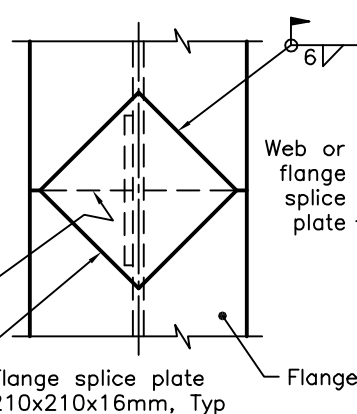
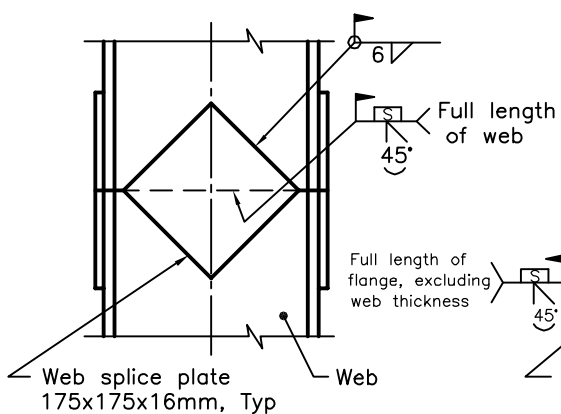
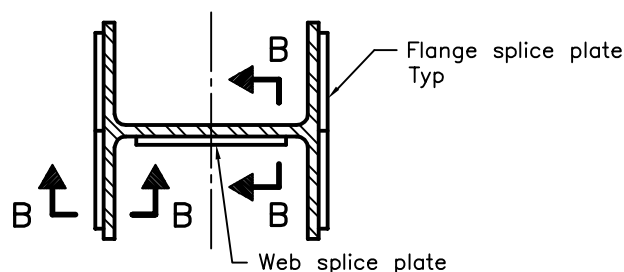
OPSD 3000.100





BUTT WELD

SECTION A-A



BUTT WELD WITH SPLICE PLATES

SECTION B-B

NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

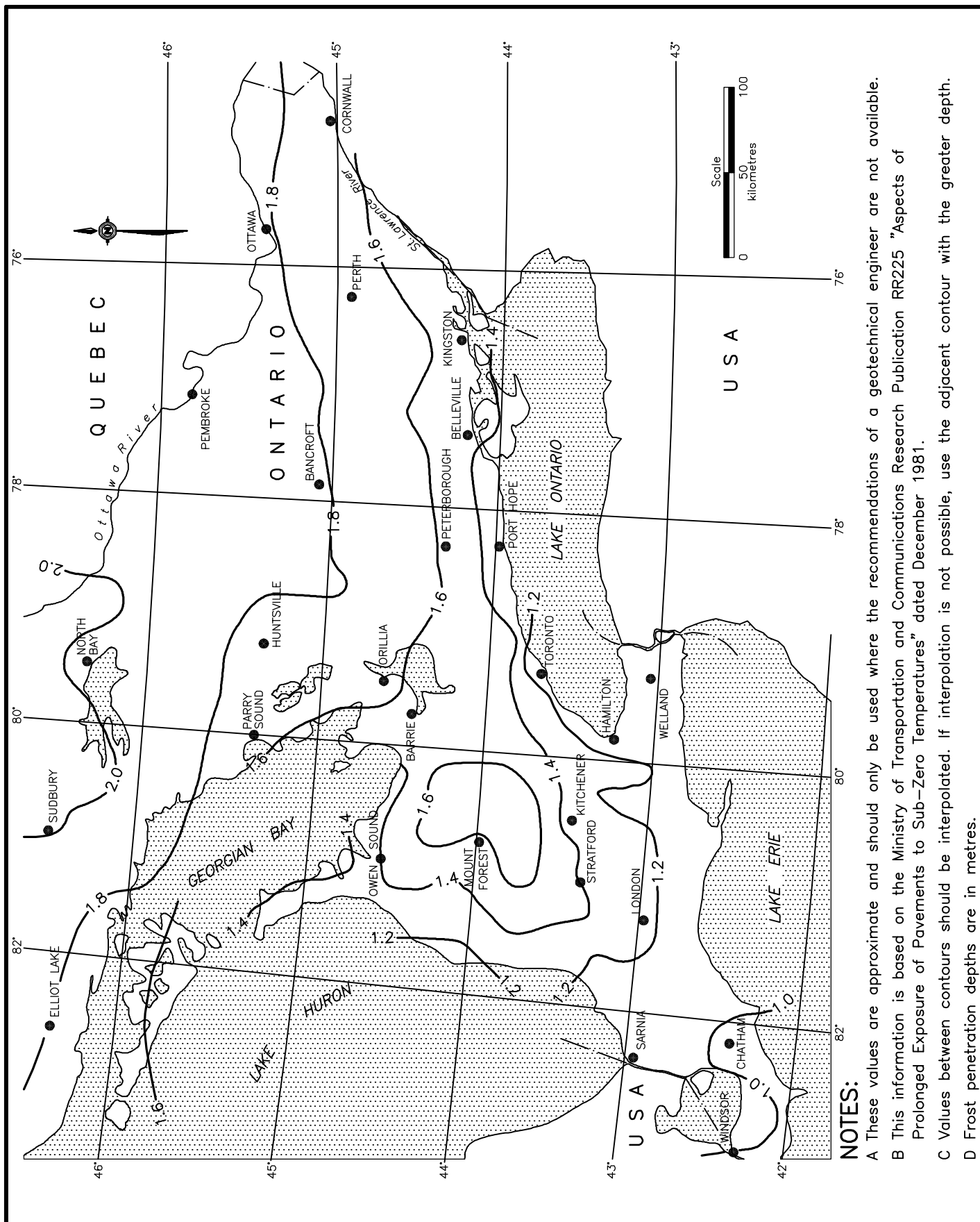
Rev

1

**FOUNDATION
PILES
STEEL H-PILE SPLICE**

OPSD 3000.150





NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

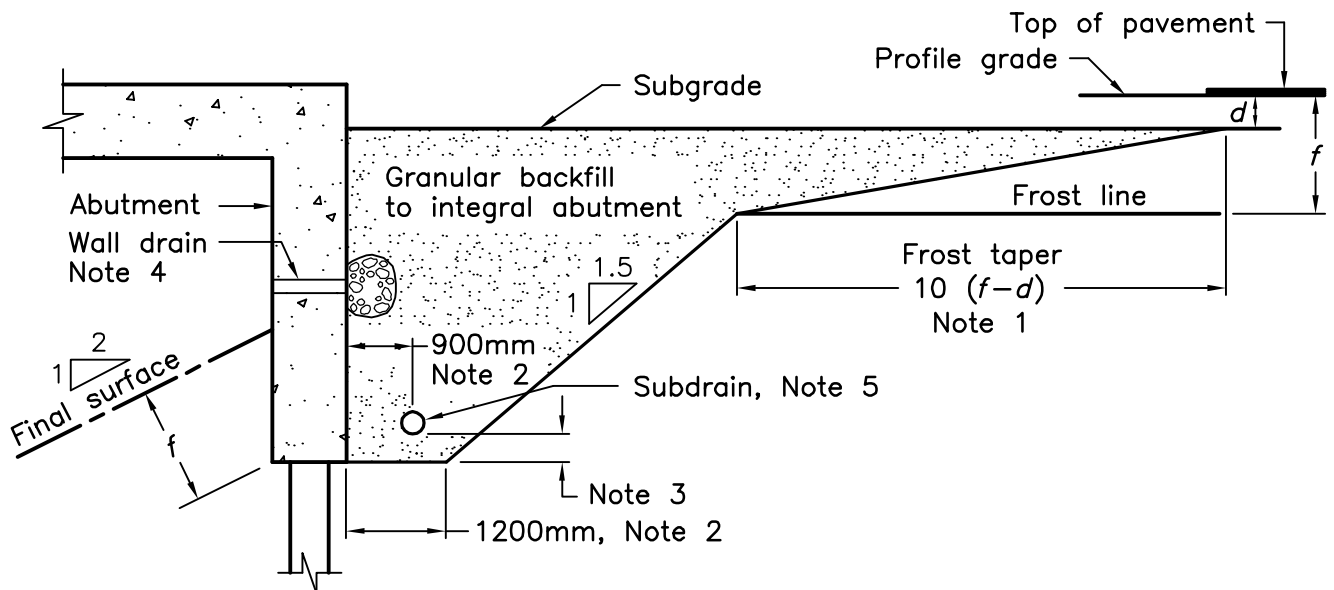
Nov 2010

Rev 1

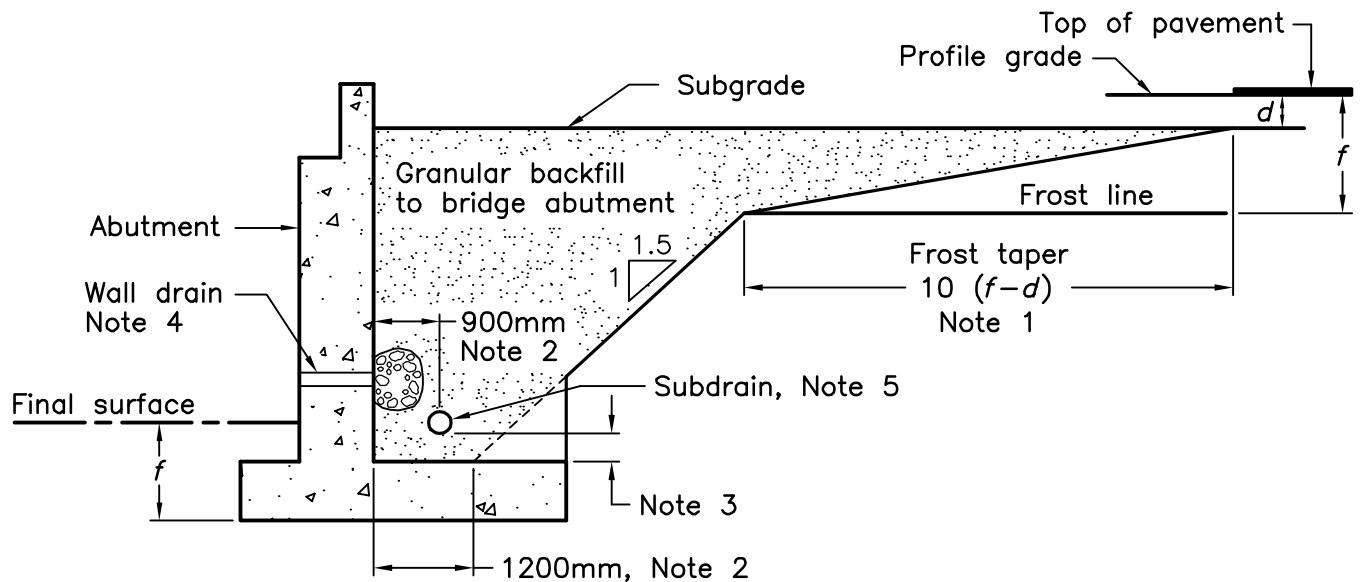
**FOUNDATION
FROST PENETRATION DEPTHS
FOR SOUTHERN ONTARIO**



OPSD 3090.101



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

Rev 1



WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD 3101.150

Figures

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

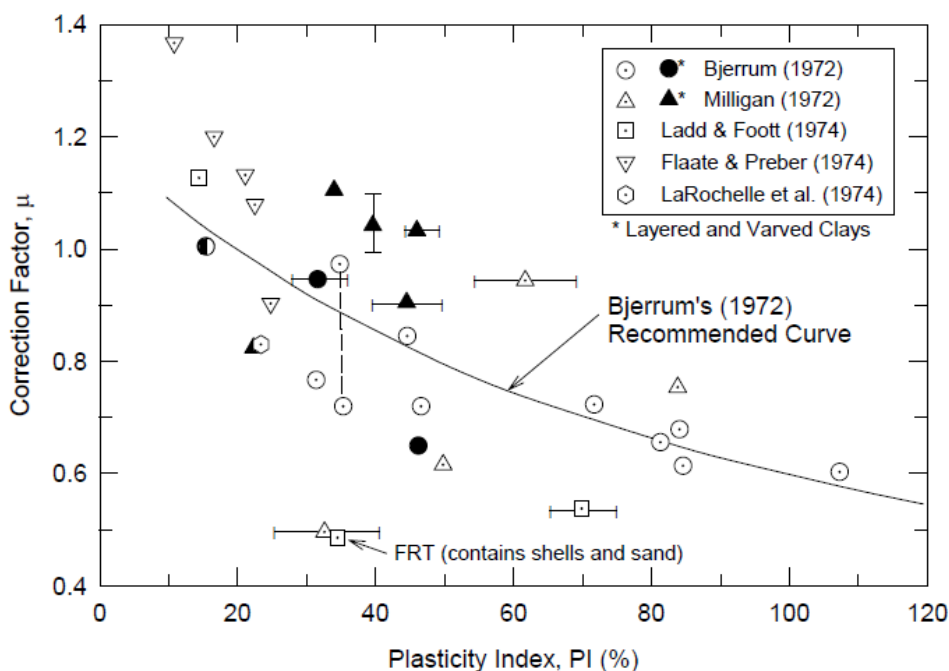
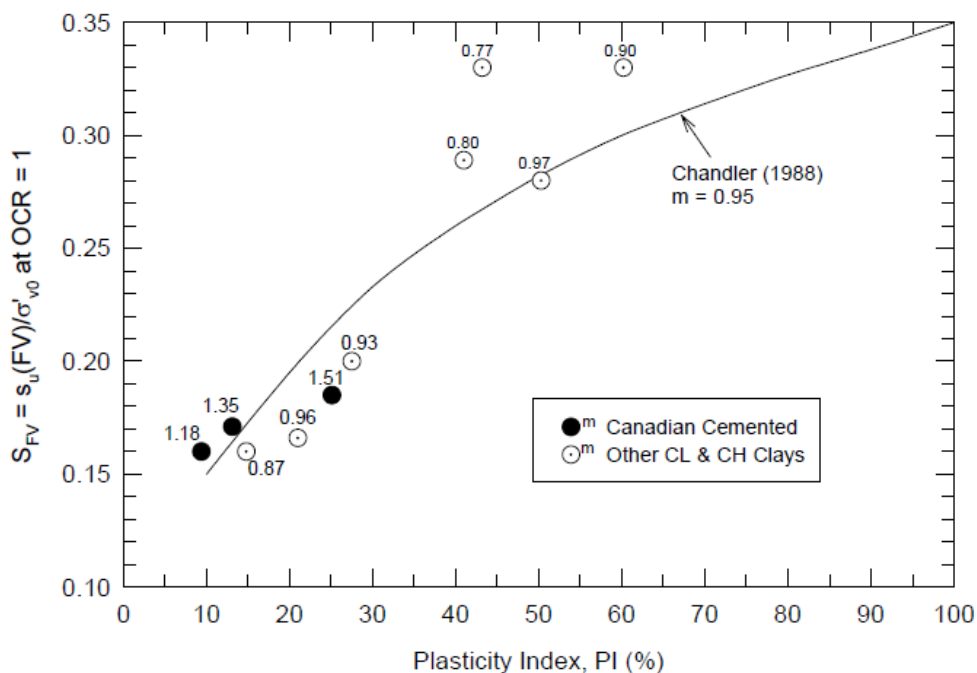
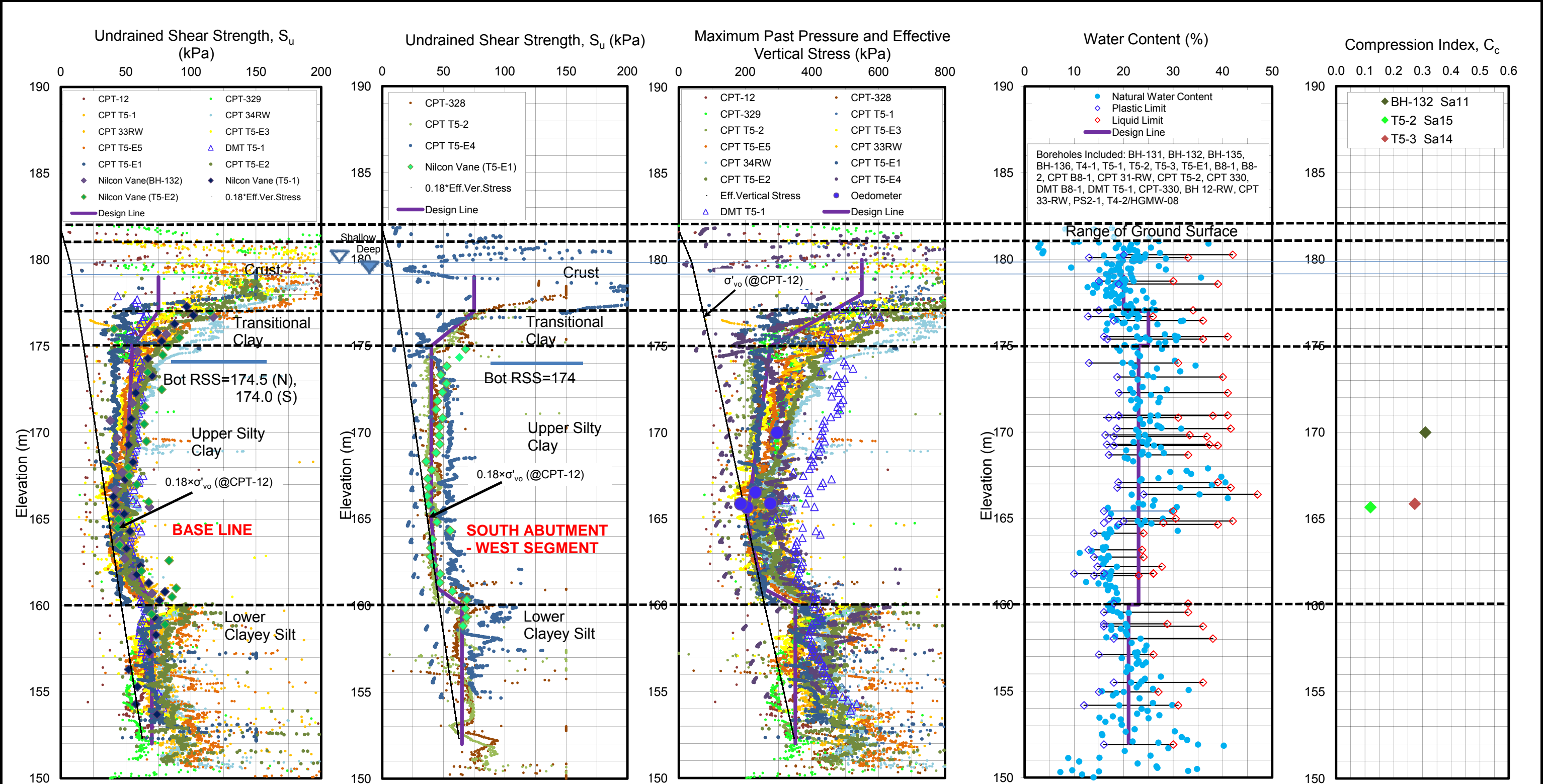


Figure 3-2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays
(Ladd & DeGroot, 2004)





Notes:

1. Shear strength profiles were estimated from CPT data using the equation $S_u = (q_t - \sigma_{VO}) / N_{KT}$. The cone factor N_{KT} was estimated by comparing the CPT profiles with a nearby Nilcon Vane profile.
2. Maximum past pressure profiles estimated using SHANSEP method. $OCR = [(S_u / \sigma'_{VO}) / S]^{1/m}$
3. Soil properties based on AMEC 2011 and Golder Investigations (ref. R-17 to R-23) results.


 Earth & Environmental	PROJECT: WINDSOR ESSEX PARKWAY				
	TITLE: SOIL PROPERTIES PROFILES TUNNEL T-5				
	DATE: Sep 2011	JOB NO.: SW8801.1002	CAD FILE:	FIGURE NO.: 3-3	REV.
CLIENT :					

Figure 4-1: Compressibility Parameters at WEP

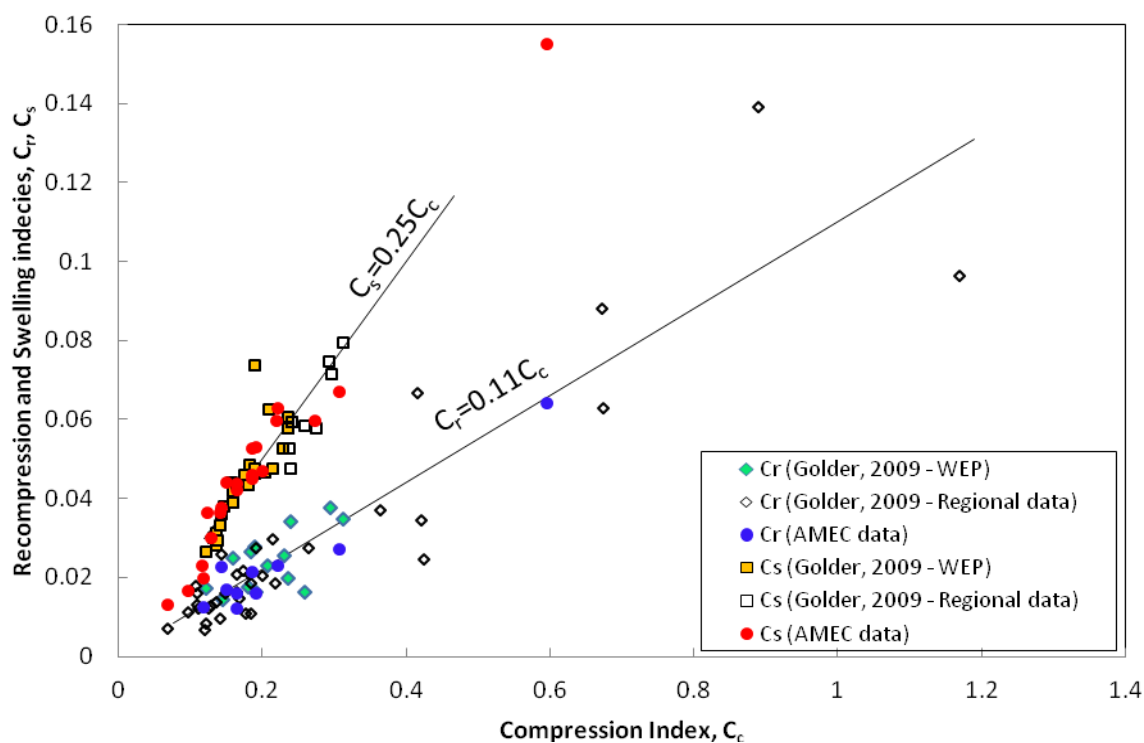
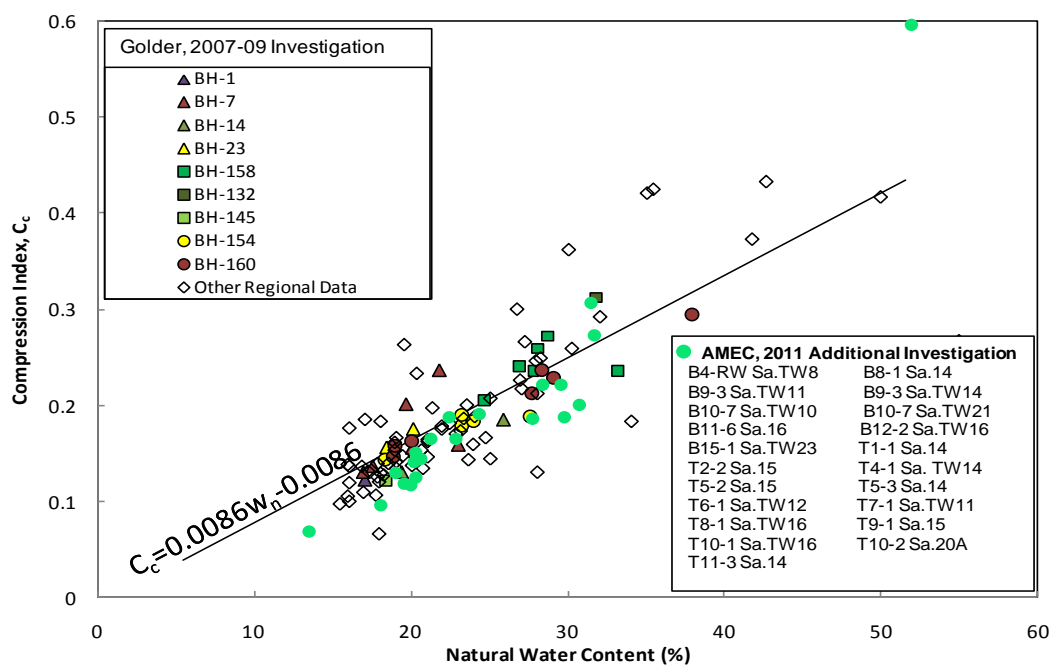


Figure 4-2: C_α versus C_c Relationship at WEP

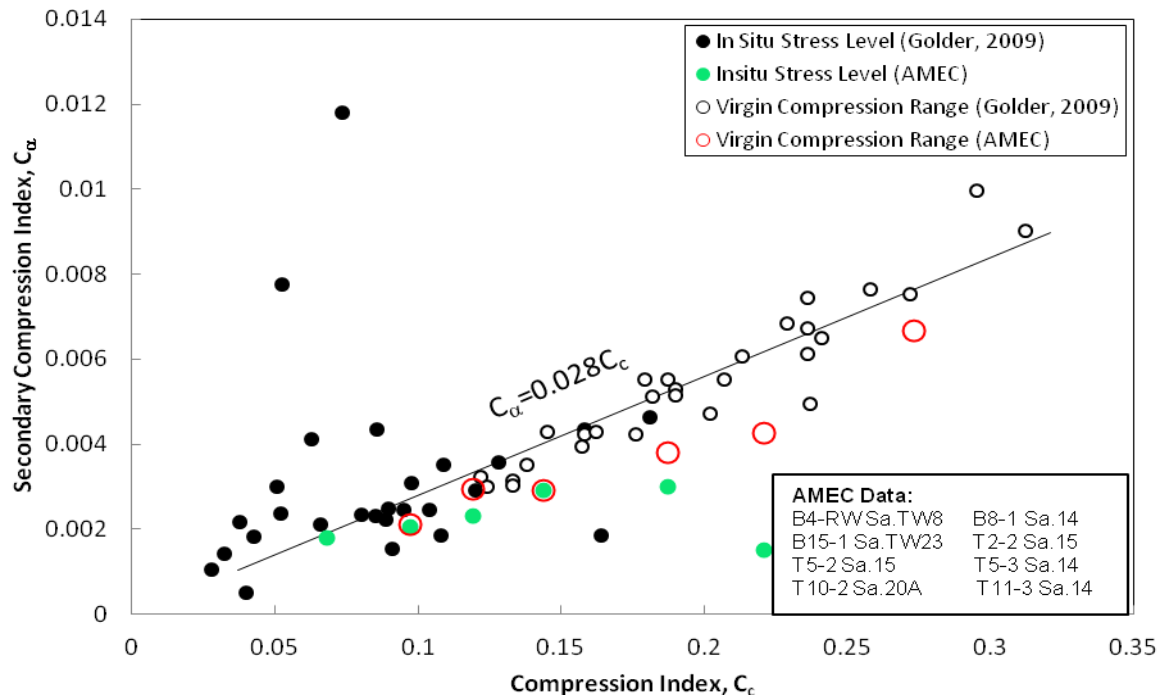


Figure 4-3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP

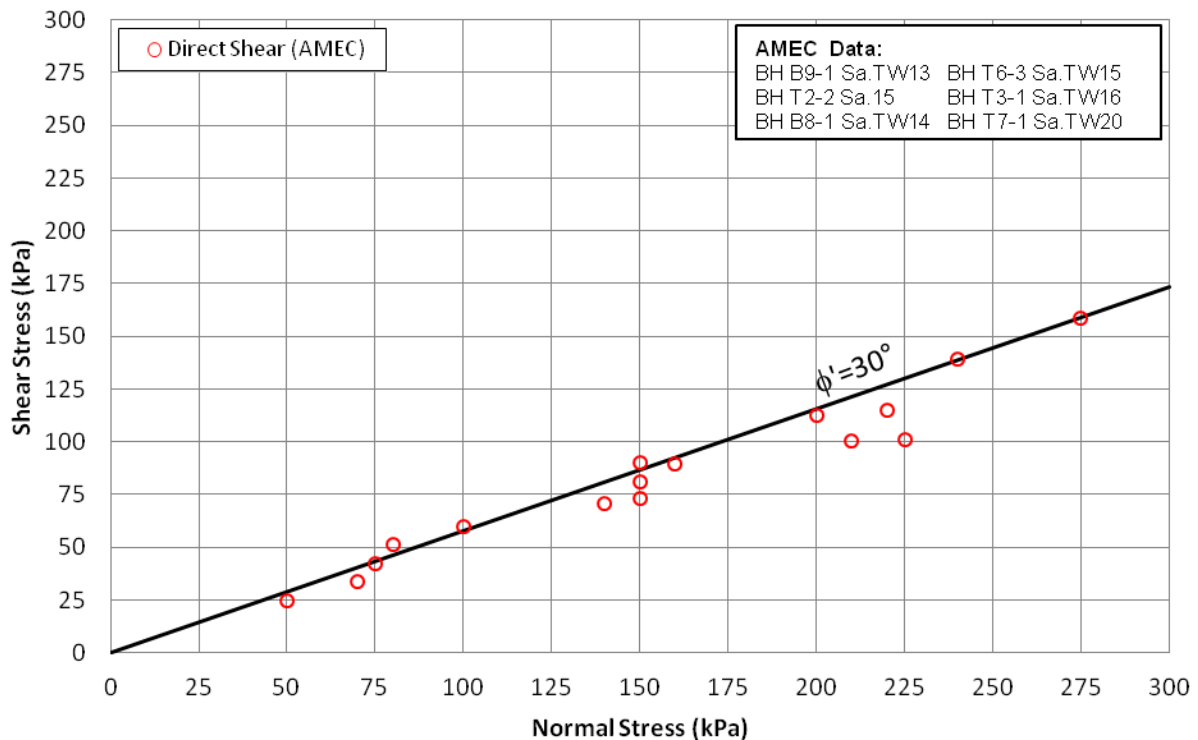
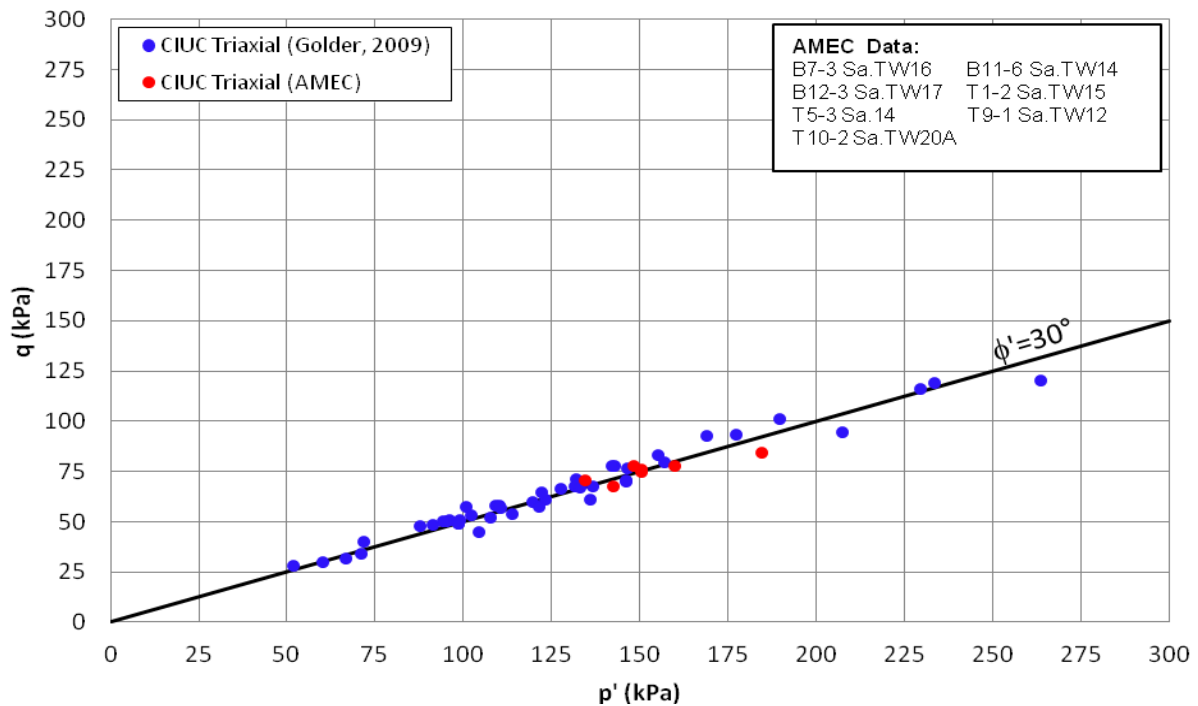


Figure 4-4: Relationship between $\sin \phi'$ and Plasticity Index for Normally Consolidated Soils
(Kenney, 1959)

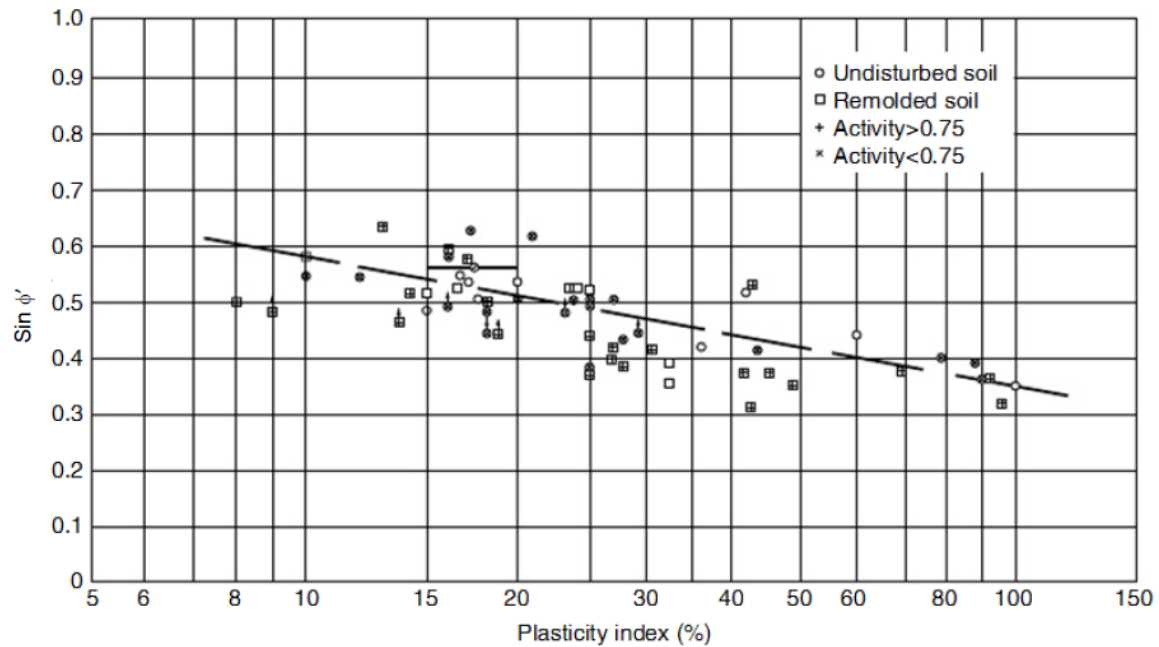


Figure 4-5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests

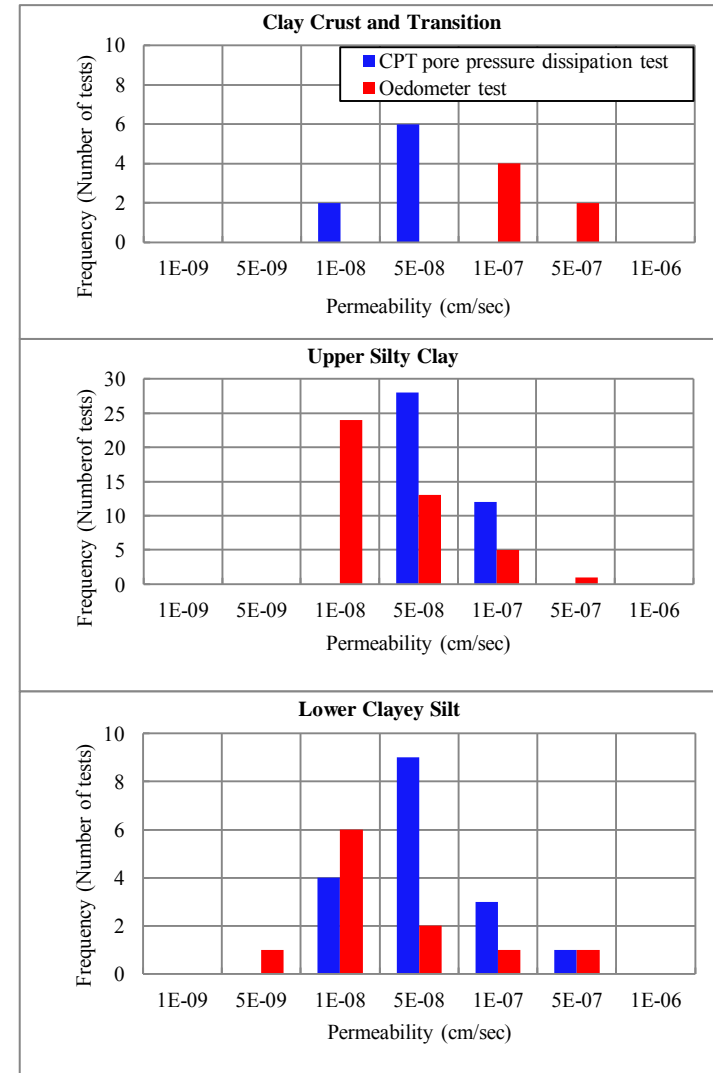
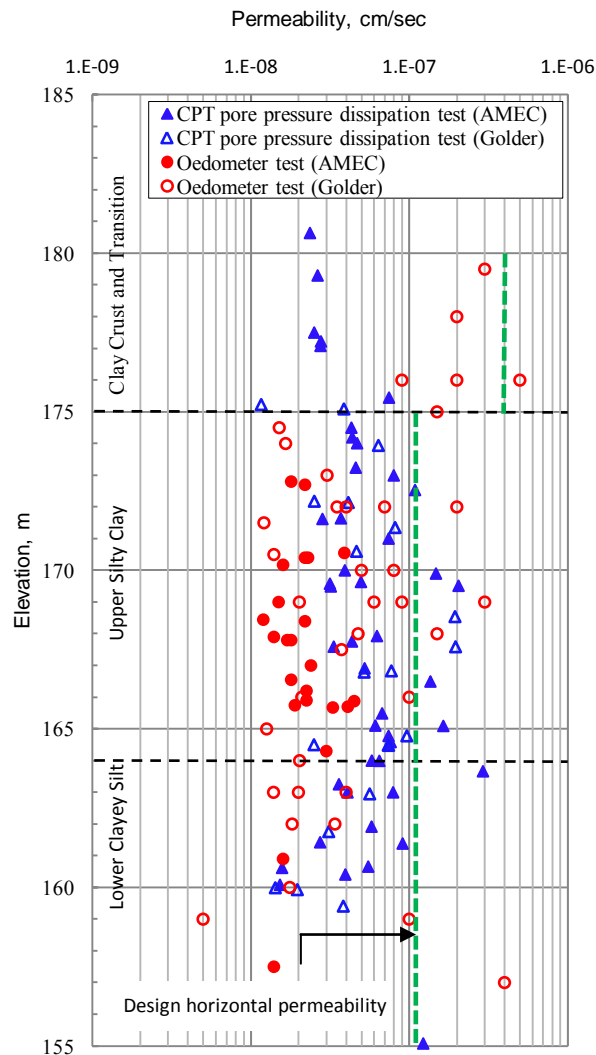
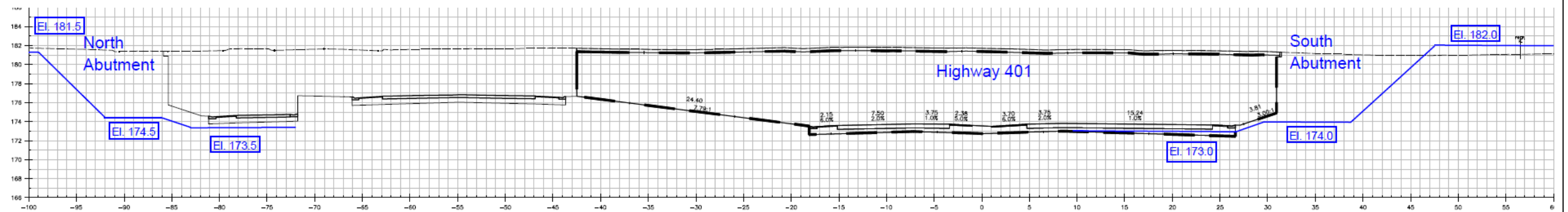
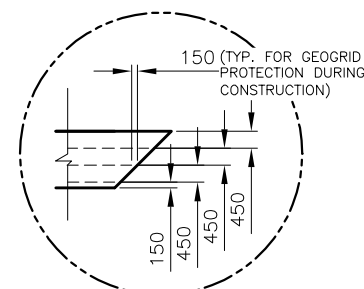
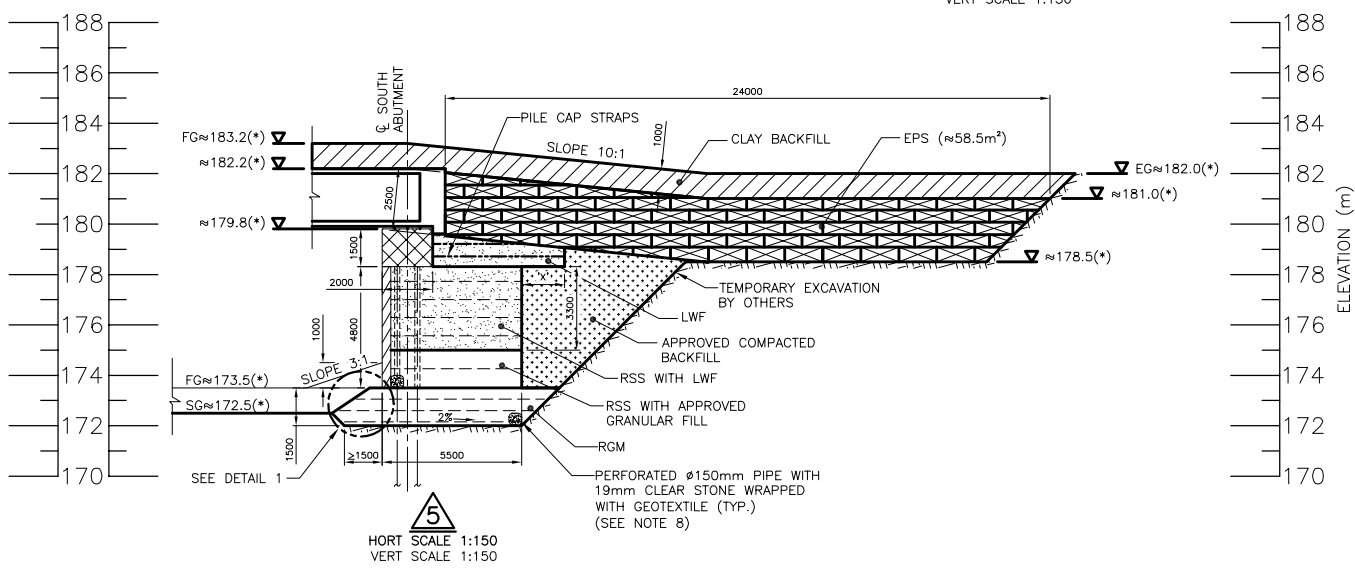
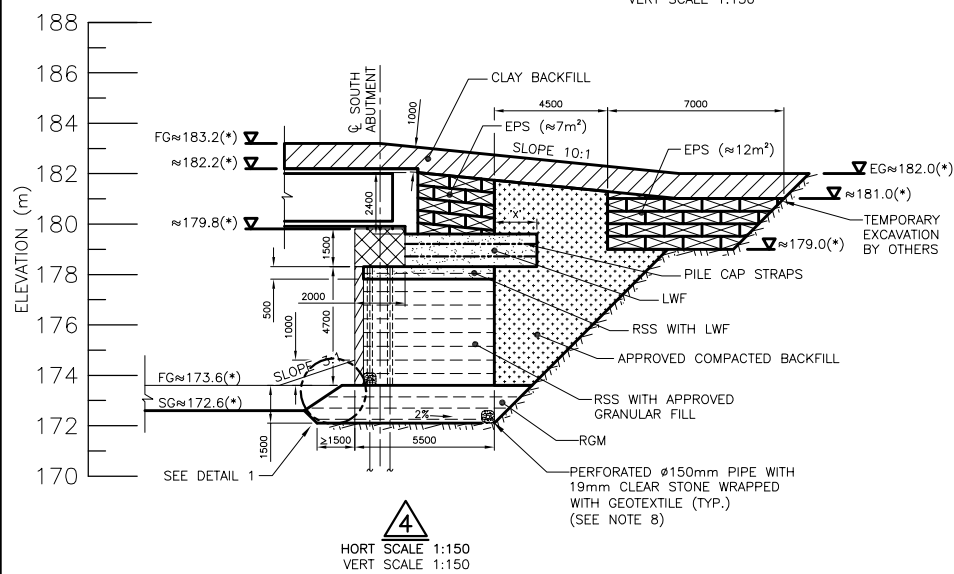
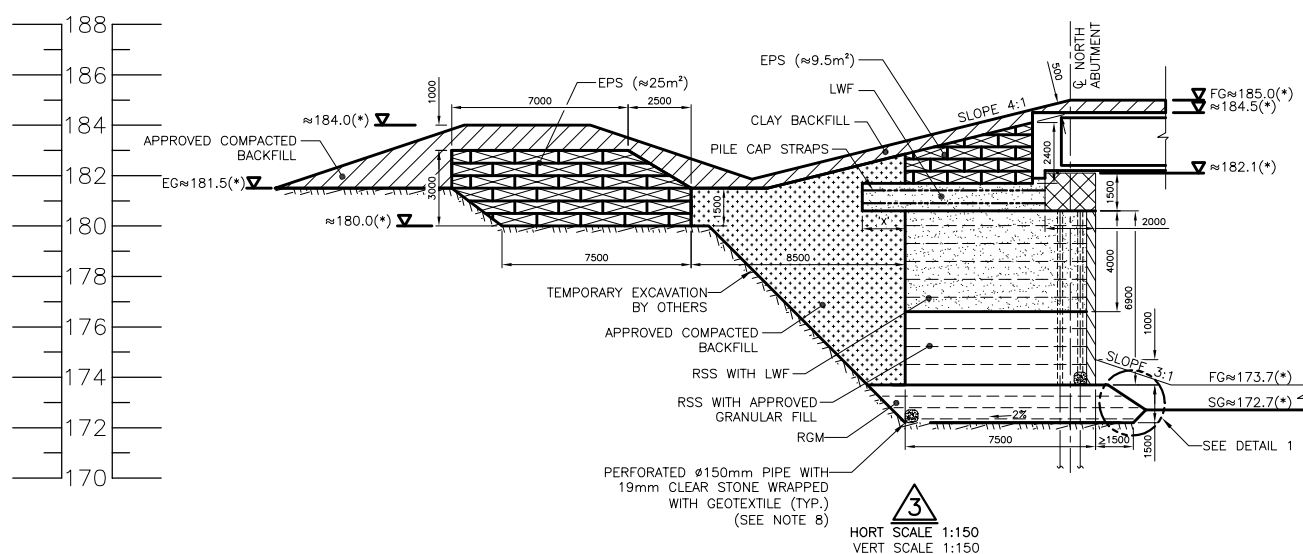
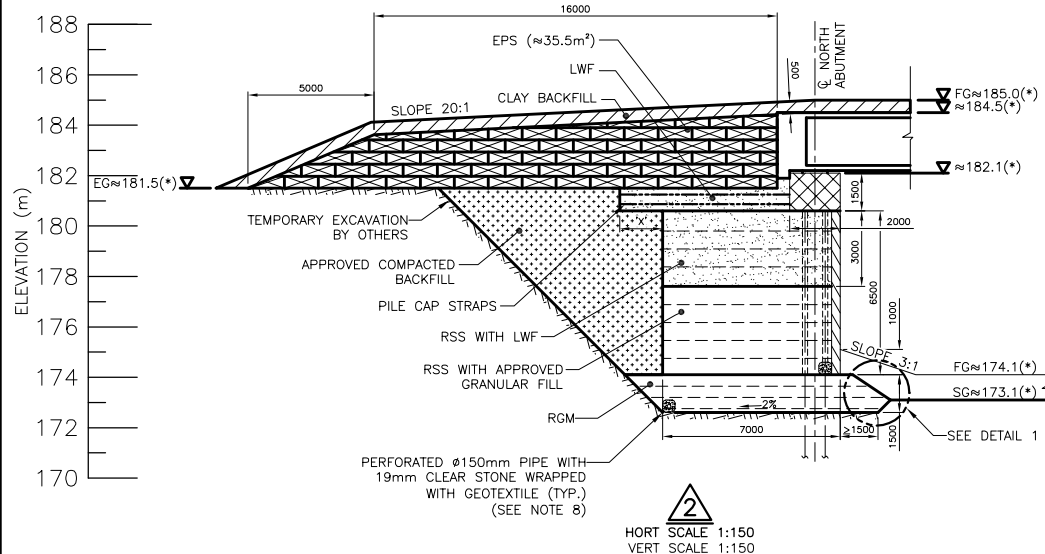
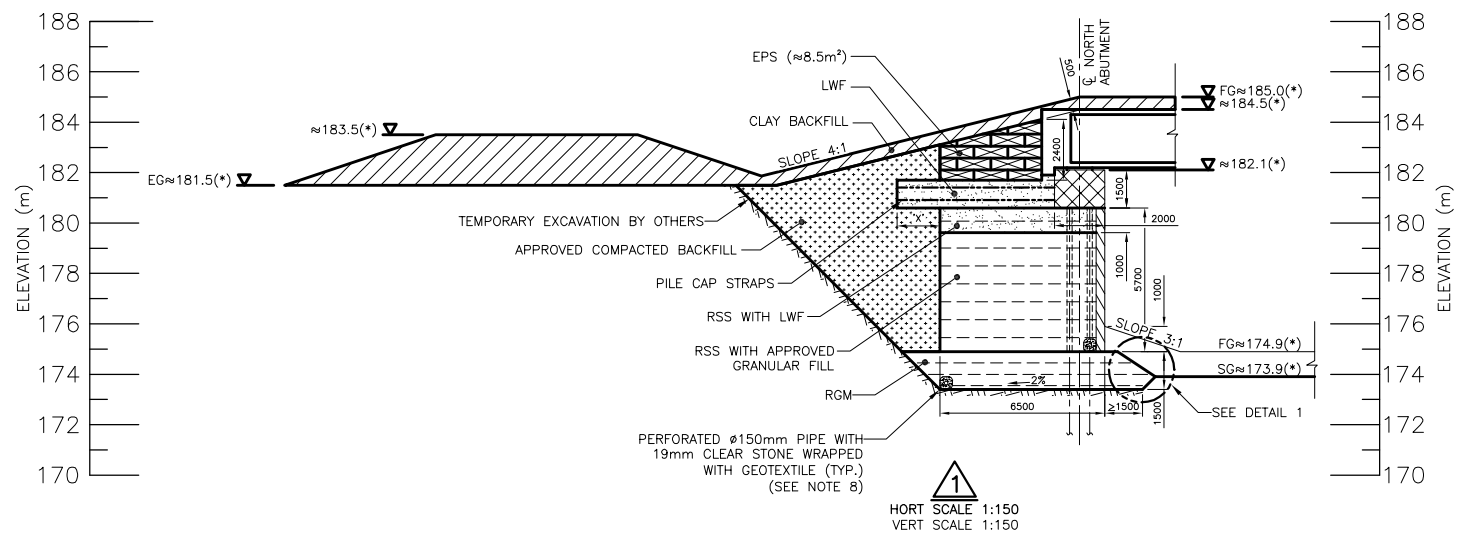
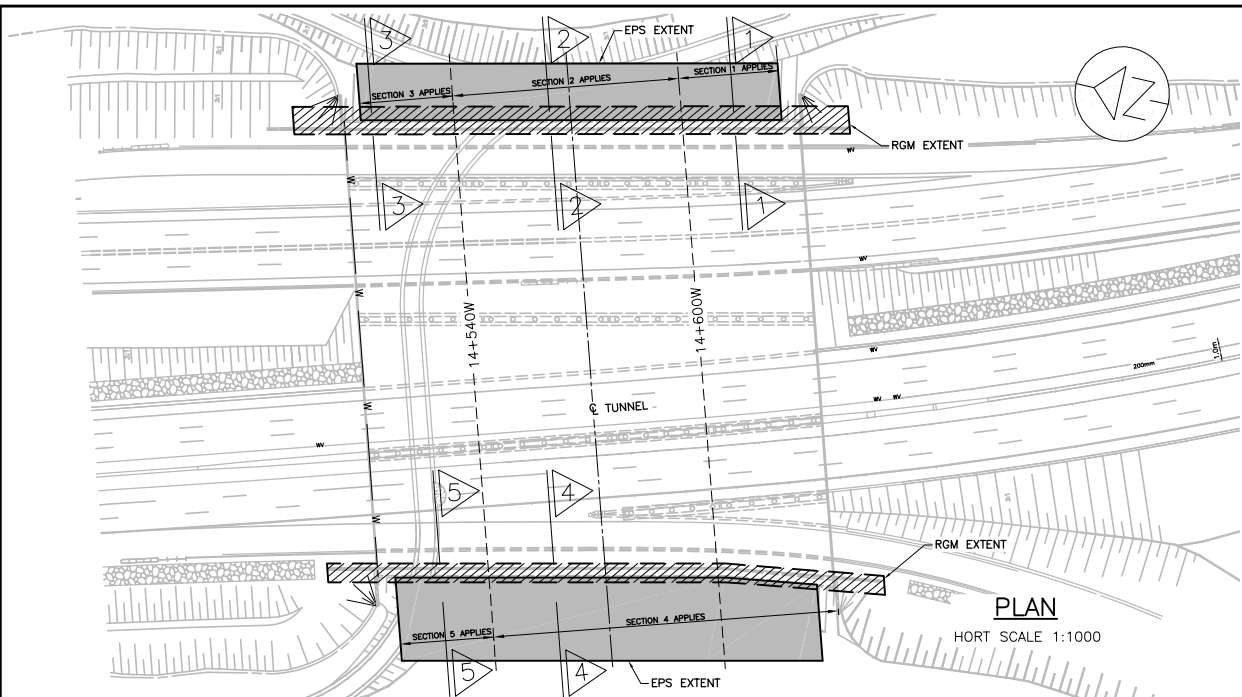


Figure 5-1: Tunnel T-5 – Typical Excavation Profile



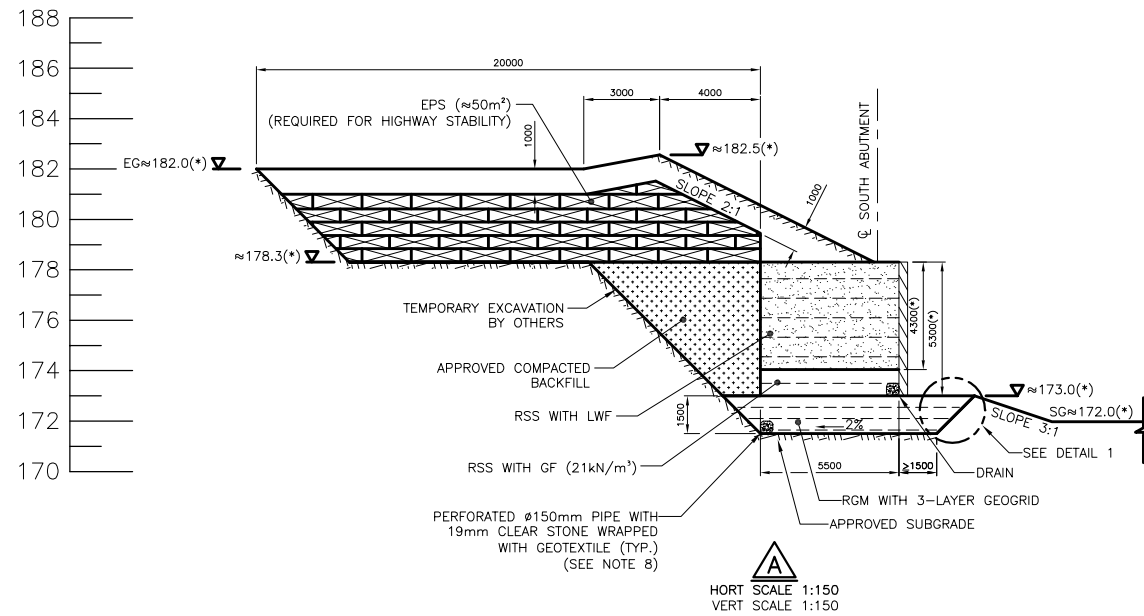
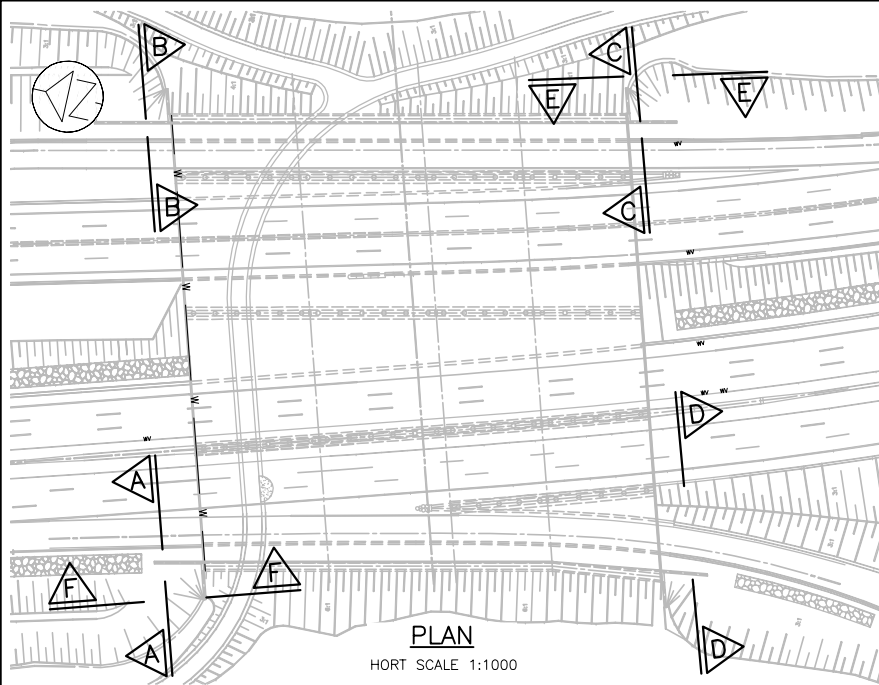


- NOTES:**
- THIS DRAWING IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
 - THIS DRAWING ILLUSTRATES THE GENERAL ARRANGEMENTS AT SELECTED REPRESENTATIVE LOCATIONS OF THE ABUTMENT WALLS OF TUNNEL T-5 BASED ON GEOTECHNICAL DESIGN ANALYSES.
 - THE ILLUSTRATED RSS WALL WIDTH REPRESENTS THE MINIMUM WIDTH BASED ON GEOTECHNICAL REQUIREMENTS. THE DESIGN OF THE RSS WALL IS TO BE DEVELOPED BY OTHERS.
 - TUNNEL ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGNS WERE OBTAINED FROM STRUCTURAL DRAWINGS AVAILABLE IN MARCH 2012. ABUTMENT ELEVATIONS VARY ALONG THE TUNNEL.
 - CLAY SUBGRADE IS SUSCEPTIBLE TO DISTURBANCE AND LOSS OF STRENGTH DUE TO WATER INFLOW/PONDING, CONSTRUCTION TRAFFIC AND THE LIKE. SUITABLE EXCAVATION METHODS, DEWATERING AND SUBGRADE PROTECTION MUST BE EXERCISED.
 - CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS. EXCAVATED CLAY SURFACES ARE SUSCEPTIBLE TO DETERIORATION AND EXPERIENCE DEFORMATIONS AND INSTABILITY; THEY ARE TO BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED AND TREATED AS REQUIRED.
 - RSS TOE BERM AND GRANULAR BASE OVER EBR5 AND WBR5 SUBGRADE MUST BE PLACED BEFORE ANY BACKFILL IS PLACED ABOVE THE DECK SEAT LEVEL.
 - SEE ACCOMPANYING DRAWINGS FOR APPLICABLE CONSTRUCTION NOTES.

LEGEND:

RSS - REINFORCED SOIL STRUCTURE
GF - GRANULAR FILL
RGM - REINFORCED GRANULAR MAT (LONG-TERM ALLOWABLE LOAD CAPACITY OF GEOTEXTILE SHALL BE MINIMUM 24kN/m)
EPS - EXPANDED POLYSTYRENE
LWF - LIGHT WEIGHT FILL (ULTRALIGHT WATER-COOLED IRON FURNACE SLAG)
"X" - LENGTH OF PILE CAP STRAPS TO BE DETERMINED BY SUPPLIER
(*) - VARIES

UNFACTORED HORIZONTAL LOADS ON PILE CAP:
EARTH PRESSURE LOADS:
- DEAD LOADS (NORTH/SOUTH) = 20/25kN/m
- LIVE LOADS = 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 WING WALLS OF TUNNEL T-5 BASED ON GEOTECHNICAL DESIGN ANALYSES.
 - THE ILLUSTRATED RSS WALL WIDTH REPRESENTS THE MINIMUM WIDTH BASED ON GEOTECHNICAL REQUIREMENTS. THE DESIGN OF THE RSS WALL IS TO BE DEVELOPED BY OTHERS.
 - TUNNEL ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGNS WERE OBTAINED FROM STRUCTURAL DRAWINGS AVAILABLE IN MARCH 2012. ABUTMENT ELEVATIONS VARY ALONG THE TUNNEL.
 - CLAY SUBGRADE IS SUSCEPTIBLE TO DISTURBANCE AND LOSS OF STRENGTH DUE TO WATER INFLOW/PONDING, CONSTRUCTION TRAFFIC AND THE LIKE. SUITABLE EXCAVATION METHODS, DEWATERING AND SUBGRADE PROTECTION MUST BE EXERCISED.
 - CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS. EXCAVATED CLAY SURFACES ARE SUSCEPTIBLE TO DETERIORATION AND EXPERIENCE DEFORMATIONS AND INSTABILITY; THEY ARE TO BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED AND TREATED AS REQUIRED.
 - RSS TOE BERM AND GRANULAR BASE OVER EBR5 AND WBR5 SUBGRADE MUST BE PLACED BEFORE ANY BACKFILL IS PLACED ABOVE THE DECK SEAT LEVEL.
 - SEE ACCOMPANYING DRAWINGS FOR APPLICABLE CONSTRUCTION NOTES.
 - SEE TABLE BELOW FOR RSS AND RGM DIMENSIONS FOR SHORT TAPERED WALLS.

WING WALL LOCATION	MAX. WALL HEIGHT (m)	RGM THICKNESS (m)	RSS WIDTH (m)	LWF REQUIRED WITHIN RSS
NORTH WEST TAPERED WALL LONG (B)	6.9	1.5	7.5	YES
NORTH WEST TAPERED WALL SHORT	5.4	1.0	5.0	NO
NORTH EAST TAPERED WALL LONG (C)	5.7	1.5	6.5	NO
NORTH EAST TAPERED WALL SHORT	4.2	1.0	4.5	NO
SOUTH WEST TAPERED WALL LONG (A)	5.3	1.5	5.5	YES
SOUTH WEST TAPERED WALL SHORT	3.8	1.0	4.5	NO
SOUTH EAST TAPERED WALL LONG (D)	3.5	-	5.0	NO
SOUTH EAST TAPERED WALL SHORT	2.0	-	3.5	NO

LEGEND:

RSS - REINFORCED SOIL STRUCTURE

GF - GRANULAR FILL

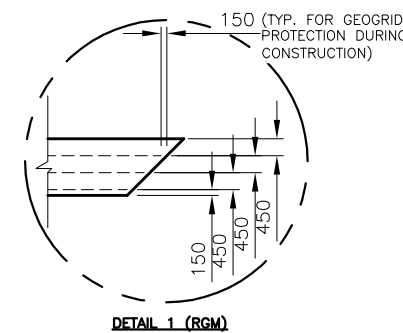
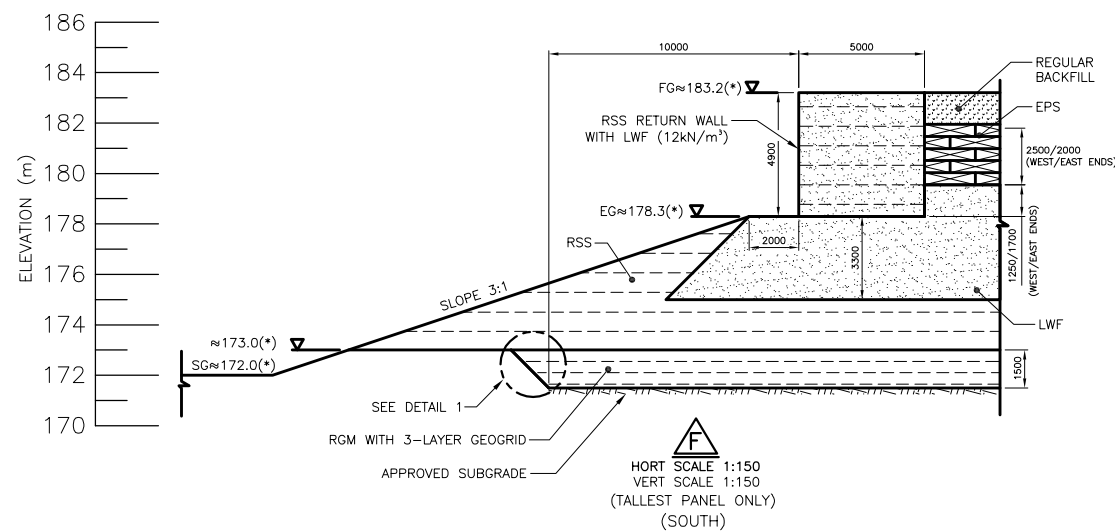
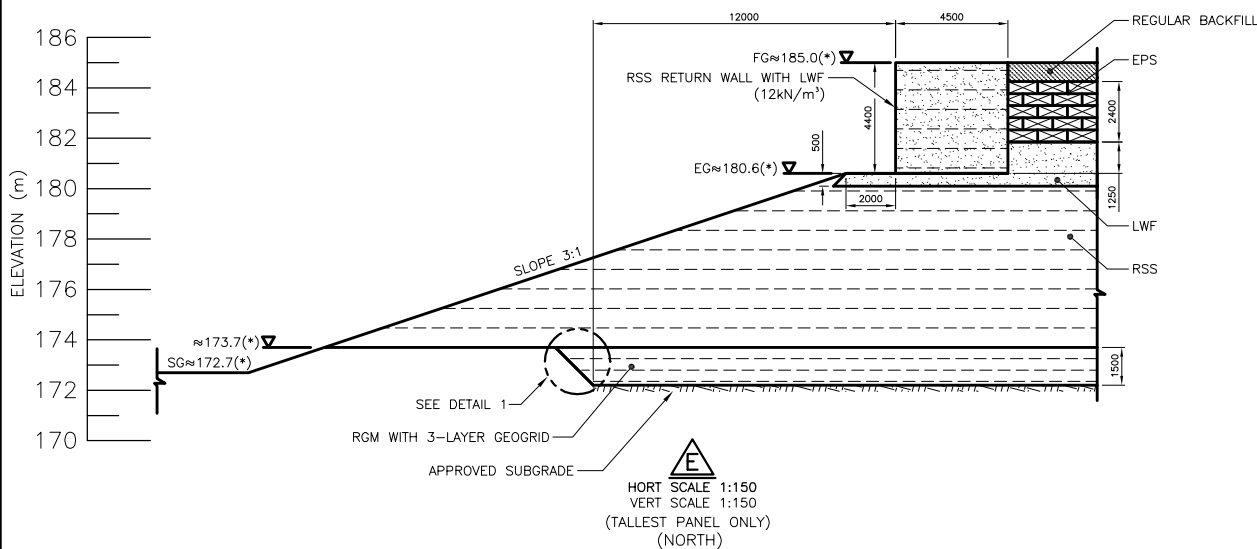
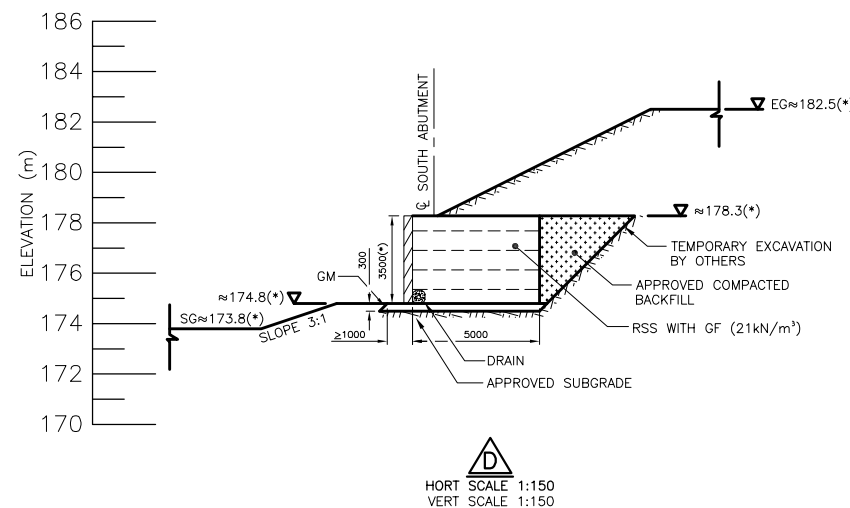
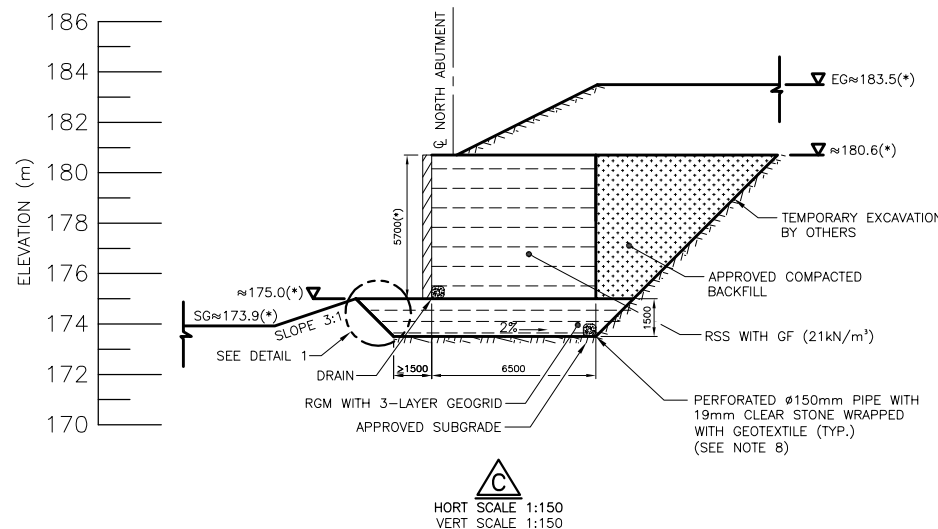
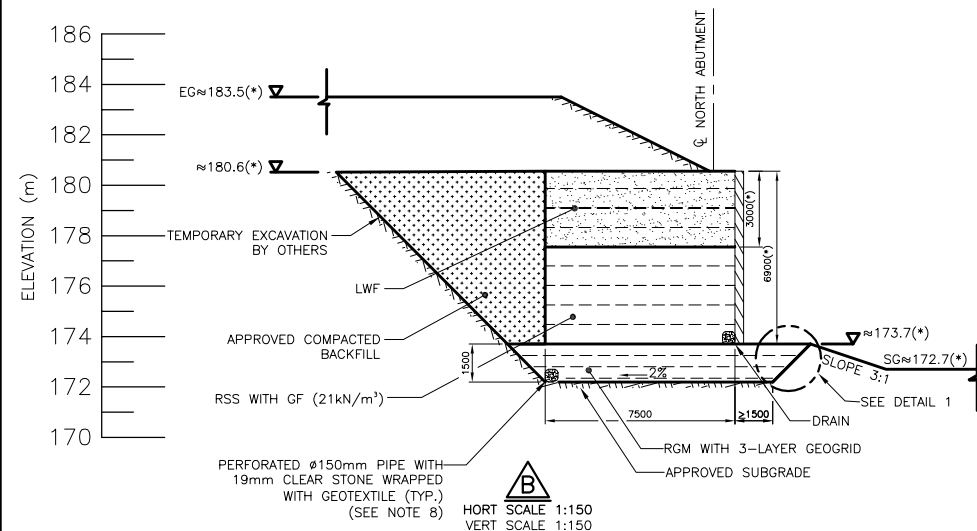
RGM - REINFORCED GRANULAR MAT (LONG-TERM ALLOWABLE LOAD CAPACITY OF GEOGRID SHALL BE MINIMUM 24kN/m)

GM - GRANULAR MAT

EPS - EXPANDED POLYSTYRENE

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

(*) - VARIES



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

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

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

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.

BEDROCK DESCRIPTION

STRENGTH CLASSIFICATION

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

JOINT SPACING CLASSIFICATION

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

ROCK QUALITY CLASSIFICATION

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

Reference: Deere et al, 1967

WEATHERING CLASSIFICATION

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

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

TERMINOLOGY

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

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

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

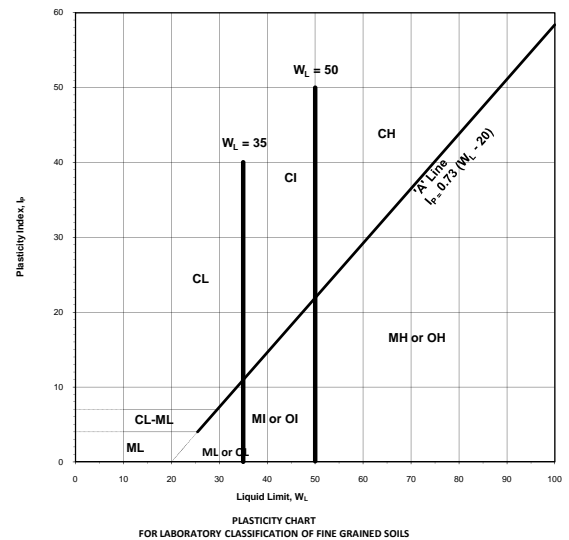
MTC SOIL CLASSIFICATION
Based on MTC Soil Classification Manual



MAJOR DIVISION					GROUP SYMBOL	TYPICAL DESCRIPTION	INFORMATION REQUIRED FOR DESCRIBING SOILS	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 PARTICULAR SIZE	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GIVE TYPE, NAME, IF NECESSARY, INDICATE APPROX % OF SAND & GRAVEL ; MAX SIZE; ANGULARITY, SURFACE CONDITION, & HARDNESS OF THE COARSE GRAINS, LOCAL OR GEOLOGICAL NAME & OTHER PERTINENT DESCRIPTIVE INFORMATION, & SYMBOL IN PARENTHESIS.	$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN 1 AND 3									
			PREDOMINANTLY ONE SIZE OF A RANGE OF SIZES WITH STONE INTERMEDIATE SIZES MISSING	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES											
		GRAVEL WITH FINES (APPLICABLE AMOUNT OF FINES)	NON PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND- SILT MIXTURES											
			PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES											
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (LITTLE OR NO FINES)	WIDE RANGE IN GRAIN SIZE & SUBSTANTIAL AMOUNT OF ALL INTERMEDIATE PARTICLE SIZES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	FOR UNDISTURBED SOILS ADD INFORMATION ON STRATIFICATION, DEGREE OF COMPACTNESS, CEMENTATION, MOISTURE CONDITION & DRAINAGE CHARACTERISTICS		NOT MEETING ALL GRADATION REQUIREMENTS FOR GW								
			PREDOMINANTLY ONE SIZE OR A RANGE OF SIZES WITH SOME INTERMEDIATE SIZE MISSING	SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES											
		SANDS WITH FINES (APPLICABLE AMOUNT OF FINES)	NON PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES											
			PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES											
	FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	IDENTIFICATION PROCEDURE ON FRACTION SMALLER THAN 425µm					USE GRAIN SIZE CURVE IN IDENTIFYING THE FACTORS AS GIVEN UNDER FIELD IDENTIFICATION	DETERMINE PERCENTAGE OF GRAVEL & SAND FROM GRAIN SIZE CURVE, DEPENDING ON PERCENTAGE OF FINES (FRACTION SMALLER THAN 75 µm) COARSE GRAINED SOILS ARE CLASSIFIED AS FOLLOWS: LESS THAN 5% GW, GP, SW, SP MORE THAN 12% GM, GC, SM, SC 5% TO 12% BORDER LINE CASES REQUIRE USE OF DUAL SYMBOL.	$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN 1 AND 3	NOT MEETING ALL GRADATION FOR SW	ATTERBERG LIMITS BELOW A- LINE OR I_p LESS THAN 4 ABOVE A-LINE WITH I_p BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS					
		LIQUID LIMIT LESS THAN 35	DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)											
NONE			QUICK	NONE	ML	INORGANIC SILTS & SANDY SILTS OR SLIGHTLY PLASTICITY, ROCK FLOUR										
MEDIUM TO HIGH			NONE TO VERY SLOW	MEDIUM	CL	SILTY CLAYS (INORGANIC), GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS										
SLIGHT TO MEDIUM			SLOW	SLIGHT	OL	ORGANIC SILT OF LOW PLASTICITY, ORGANIC SANDY SILTS										
LIQUID LIMIT BETWEEN 35 AND 50		NONE TO SLIGHT	SLOW TO QUICK	SLIGHT	MI	INORGANIC COMPRESSIBLE FINE SANDY SILT WITH CLAY OF MEDIUM PLASTICITY, CLAYEY SILTS										
		HIGH	NONE	MEDIUM TO HIGH	CI	SILTY CLAYS (INORGANIC) OF MEDIUM PLASTICITY										
		SLIGHT TO MEDIUM	VERY SLOW	SLIGHT	OI	ORGANIC SILTY CLAYS OF MEDIUM PLASTICITY										
		SLIGHT TO MEDIUM	SLOW TO NONE	MEDIUM	MH	INORGANIC SILTS, HIGHLY COMPRESSIBLE MICACEOUS OR DIATOMEACACOUS FINE SANDY SILTS, ELASTIC SILTS										
LIQUID LIMIT GREATER THAN 50		HIGH TO VERY HIGH	NONE	HIGH	CH	CLAYS (INORGANIC) OF HIGH PLASTICITY, FAT CLAYS										
		MEDIUM TO HIGH	NONE TO VERY SLOW	SLIGHT TO MEDIUM	OH	ORGANIC CLAYS OF HIGH PLASTICITY										
												FOR UNDISTURBED SOILS AND INFORMATION ON STRUCTURE, STRATIFICATION, CONSISTANCY IN UNDISTURBED AND REMOLDED STATES, MOISTURE & DRAINAGE CONDITION.				
HIGH ORGANIC SOILS				READILY IDENTIFIED BY COLOUR, ODOUR, SPONGY FEEL & FREQUENTLY BY FIBROUS TEXTURE	Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS										

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

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

**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 T5-1

1 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4680034.7, E331972.6 ORIGINATED BY TA
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 30 Apr 11 - 2 May 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE								
						● POCKET PEN.	× LAB VANE	WATER CONTENT (%)									
						20	40	60	80	100	10	20	30	GR SA SI CL			
181.3	Pavement Surface																
0.0	275mm																
181.0	CONCRETE																
0.3	FILL																
180.5	crushed limestone sand and gravel																
180.4	Grey																
0.9	FINE SAND		1AB	SS	7												
	Poorly graded																
	Brown																
	SILTY CLAY																
	Trace sand, trace gravel		2	SS	14												
	Firm to stiff																
	Brown																
	Grey																
	-Trace pink clay nodules		3	SS	11												
	-Varved to approx 4.5m		4	SS	11												
			5	SS	8												
			6	SS	5												
			7	SS	6												
	-Some zones of sandy clay below approx. 6m		8	SS	5												
			9	TW	PH												
				VT													
			10	TW	PH												
			11	TW	PH												
				VT													
			12	TW	PH												
168.2	SILTY CLAY																
13.1	Some pink and black clay nodules																
	Trace embedded sand and gravel																
	Grey		13	TW	PH												
	-Increased sand content and fine gravel																
				VT													

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 26/03/12

METRIC

[illegible]

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 26/03/12

METRIC

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

METRIC

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 26/03/12

RECORD OF BOREHOLE No T5-2

3 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679949, E331946.8 ORIGINATED BY SD
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 14 Apr 11 - 17 Apr 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE	20						40	60	80
29.9	SANDY SILT Compact to very dense Grey (continued)						151													
			26	SS	28		150													
149.0	-Shattered Limestone and shale fragments		27	SS	>100		149										-core barrel blocked at 32.00mm RQD = 56% TCR = 100% SCR = 65%			
32.3	LIMESTONE. Thinly laminated, moderately porous; Medium to fine-grained ; medium to semi-hard Light grey to brown		28	NQ			148										RQD = 14% TCR = 78% SCR = 37%			
			29	NQ																
147.2	-Numerous fractures																			
34.1	END OF BOREHOLE						147													
	Water level measured in Piezometer VWP #P11 at elevation 179.9m on May 16, 2011						146													
	Water level measured in Piezometer VWP #P11 at elevation 179.6m on June 25, 2011						145													
	Water level measured in Piezometer VWP #P11 at elevation 179.5m on July 10, 2011						144													
	Water level measured in Piezometer VWP #P11 at elevation 179.5m on July 23, 2011						143													
	Water level measured in Piezometer VWP #P21 at elevation 179.6m on May 16, 2011						142													
	Water level measured in Piezometer VWP #P21 at elevation 179.3m on June 25, 2011						141													
	Water level measured in Piezometer VWP #P21 at elevation 179.2m on July 10, 2011						140													
	Water level measured in Piezometer VWP #P21 at elevation 179.1m on July 23, 2011						139													
							138													
							137													

-core barrel blocked at 32.00mm
 RQD = 56%
 TCR = 100%
 SCR = 65%

 RQD = 14%
 TCR = 78%
 SCR = 37%

RECORD OF BOREHOLE No T5-3

1 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4680036.9, E331884.3 ORIGINATED BY SD
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 18 Apr 11 - 19 Apr 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
181.6	Ground Surface							20 40 60 80 100						
0.0	300mm Black organic							○ UNCONFINED + FIELD VANE						
181.3	TOPSOIL							● POCKET PEN. × LAB VANE						
0.3	SAND Some silt Compact Brown Wet		1	SS	10		181	20 40 60 80 100						
180.1	SILTY CLAY Some sand, trace gravel Stiff to very stiff Grey		2	SS	7		180							
1.5			3	SS	16		179							
	Trace pink nodules		4	SS	17		178							
			5	SS	15		177							
	Variable laminated and unstructured		6	SS	8		176							
176.0	Dense sand nodules		7	SS	7		175							
5.6	SILTY CLAY Medium Plasticity Some pink clay nodules Grey		8	SS	8		174							
174.6	CLAYEY SILT Some sand, trace gravel Grey		9	TW	PH		173							
7.0			10	TW	PH		172							
			VT				171							
			11	TW	PH		170							
			VT				169							
			12	TW	PH		168							
			VT				167							
168.2	SILTY CLAY Pink and black clay nodules Grey		13	TW	PH		166							
13.4			VT				165							

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 26/03/12

METRIC

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




ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 26/03/12

RECORD OF BOREHOLE No T5-E1

1 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4680013.2, N331863.7 ORIGINATED BY TP
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 16 May 11 - 17 May 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED ● POCKET PEN.	+ FIELD VANE × LAB VANE									
181.1 0.0	Ground Surface TOPSOIL Silty Clay Black organic																	
180.2 0.9	SILTY CLAY to CLAYEY SILT Firm to stiff Mottled brown and grey		1AB	SS	4													
			2	SS	5													
			3	SS	14													
178.2 2.9	SILTY CLAY Some sand, trace gravel Firm to stiff Grey		4	SS	13													
			5	SS	14													
			6	SS	7													
	-Pink clay nodules		VT															
			7	SS	5													
			8	SS	3													
			VT															
		9	SS	0														
		10	SS	0														
		VT																
		11	SS	0														
168.0 13.1	SILTY CLAY Medium Plasticity Soft Grey		12	SS	0													
	-Some fine sand nodules		VT															

Continued Next Page

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 26/03/12

METRIC

Continued Next Page

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

METRIC



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

RECORD OF BOREHOLE No CPT T5-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679993.7, E331958.7 ORIGINATED BY KH
DIST HWY WEP BOREHOLE TYPE Track Mounted Drill - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 28 Sep 11 - 28 Sep 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE								
181.2 0.0	Mottled Brown CLAYEY SILT		A	SA			181									
180.6 0.6	Topsoil with sand and oxidation, trace organics Moist		B	SA			180									
180.0 1.2	Grey SILTY CLAY Oxidation and gravel Moist						179									
178.8 2.4	Grey SILTY CLAY		C	SA			178									
178.2 3.0	Trace gravel and oxidation Moist						177									
END OF BOREHOLE							176									
							175									
							174									
							173									
							172									
							171									
							170									
							169									
							168									
							167									

RECORD OF BOREHOLE No CPT T5-2

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4680010.9, E331863 ORIGINATED BY TA
DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 28 Apr 11 - 28 Apr 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED	+	FIELD VANE						
								● POCKET PEN.	×	LAB VANE						
180.9	Ground Surface															
0.0	TOPSOIL															
180.6	300mm black organic clay															
0.3	CLAYEY SILT															
	to															
	SILTY CLAY															
	Some gravel		1	SS	3											
	Mottled brown and grey															
179.6																
1.4	SILTY CLAY															
	Firm to very stiff															
	Mottled brown and grey to brown		2	SS	5											
			3	SS	18											
178.1																
2.8	SILTY CLAY															
	Stiff															
	Grey		4	SS	14											
177.4																
3.5	END OF SAMPLED BOREHOLE (Continued with CPT to refusal)															
	Borehole dry on completion															
							177									
							176									
							175									
							174									
							173									
							172									
							171									
							170									
							169									
							168									
							167									
							166									

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

RECORD OF BOREHOLE No DMT T5-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4679978.7, E331992.4 ORIGINATED BY LEC
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 29 Apr 11 - 29 Apr 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE	WATER CONTENT (%)					
181.1	Pavement Surface						20	40	60	80	100	10	20	30			
0.0	250mm																
180.8	CONCRETE																
0.3	FILL																
180.2	Crushed limestone sand and gravel																
0.9	Grey																
	SILTY CLAY		1AB	SS	6												
	Trace rootlets and oxidized, sand-lined fissures																
	Some sand, trace gravel		2	SS	13												
	Firm to very stiff																
	Mottled brown and greenish grey		3	SS	16												
	-Sand seams																
	Grey																
	-Pink clay nodules and sand inclusions		4	SS	12												
177.6	END OF SAMPLED BOREHOLE (Continued with DMT)																
3.5	Borehole dry on completion																
							177										
							176										
							175										
							174										
							173										
							172										
							171										
							170										
							169										
							168										
							167										

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

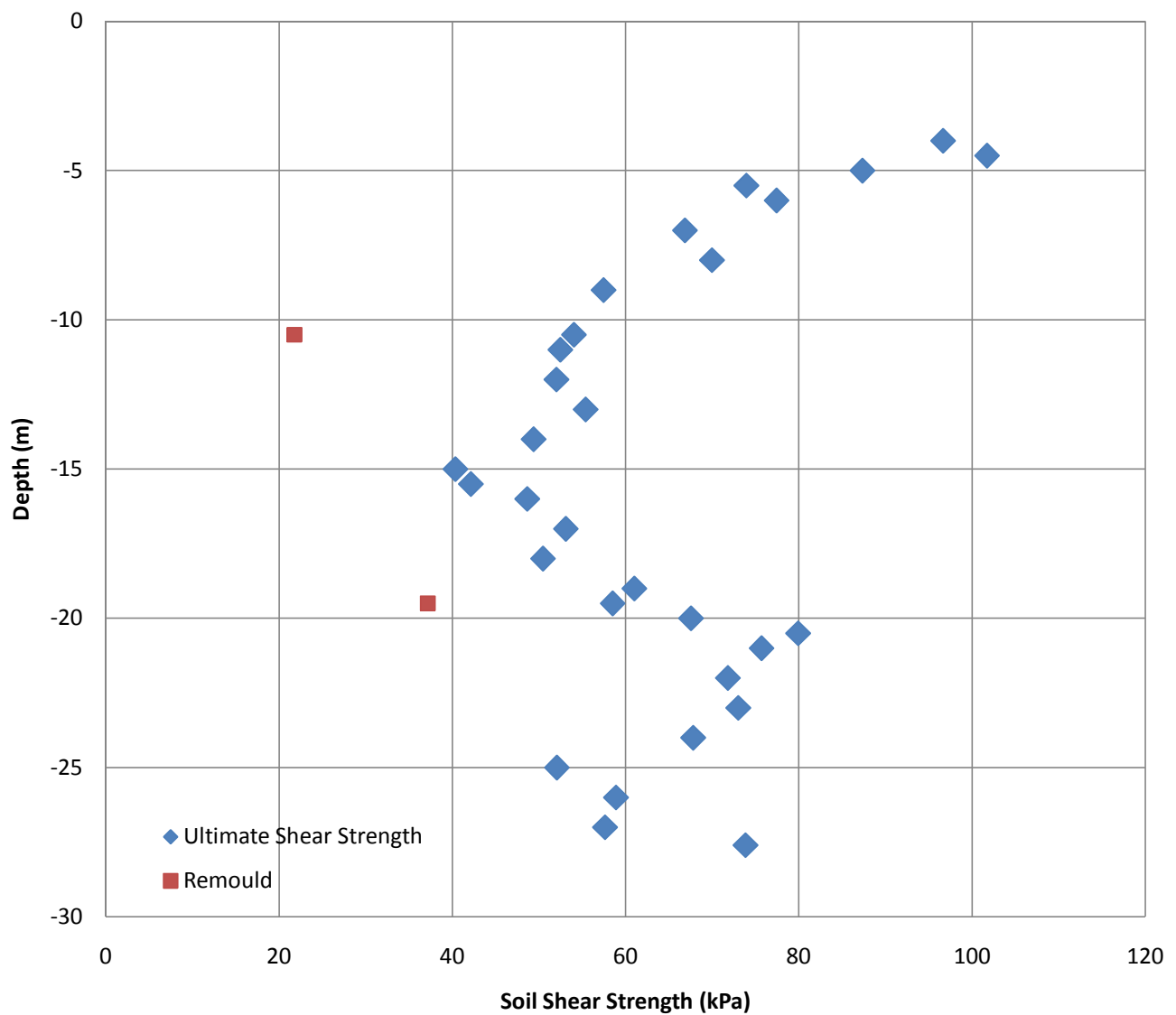
RECORD OF NILCON VANE TEST T5-1

Project : Windsor-Essex Parkway
Location: 4680038N; 331971E

Test Date: 5/3/2011
Predrill Depth : 4 m

Sheet 1 of 1
Datum Geodetic

NIL T5-1



Engineer: SO

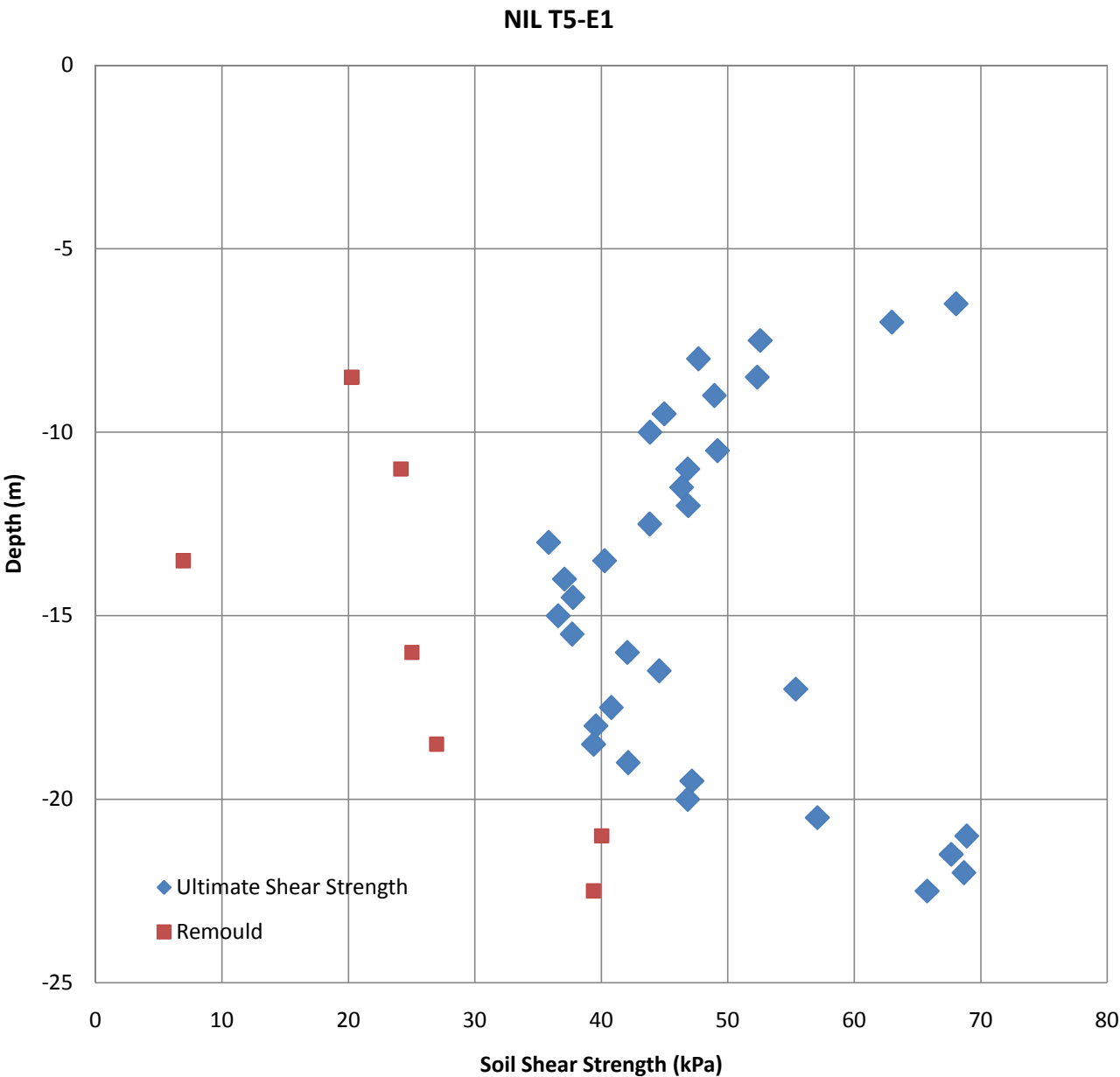
Checked: DD

RECORD OF NILCON VANE TEST T5-E1

Project : Windsor-Essex Parkway
Location: 4680016N; 331862E

Test Date: 5/25/2011
Predrill Depth : 6.5 m

Sheet 1 of 1
Datum Geodetic



Engineer:

Checked: DD

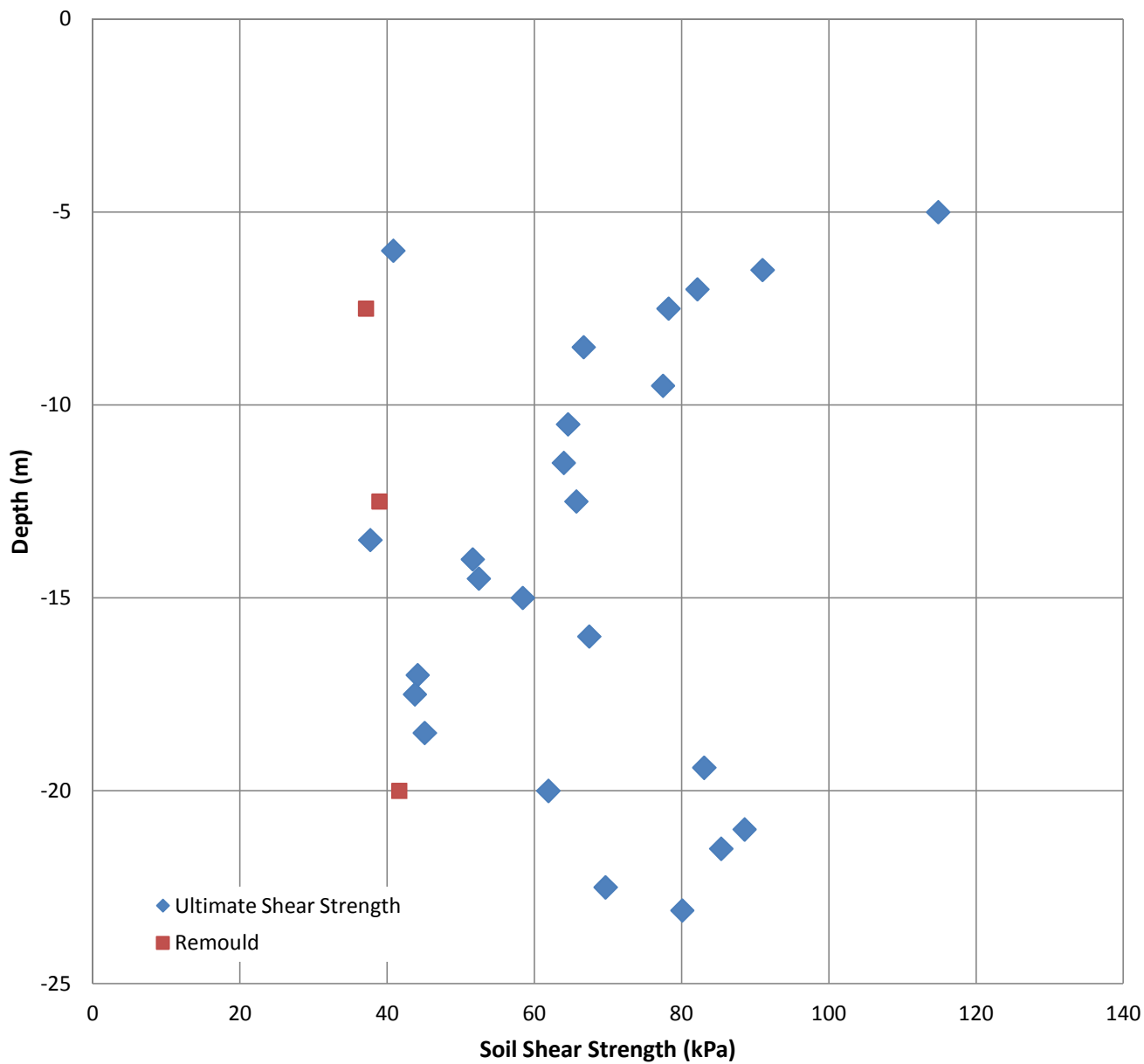
RECORD OF NILCON VANE TEST T5-E2

Project : Windsor-Essex Parkway
 Location: 4679941N; 331886E

Test Date: 9/8/2011
 Predrill Depth : 5 m

Sheet 1 of 1
 Datum Geodetic

NIL T5-E2



Engineer:

Checked: DD

RECORD OF CONE PENETRATION TEST CPT T5-1

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 4/28/2011 - 4/28/2011

SHEET 1 OF 2

LOCATION 4679994N; 331959 E

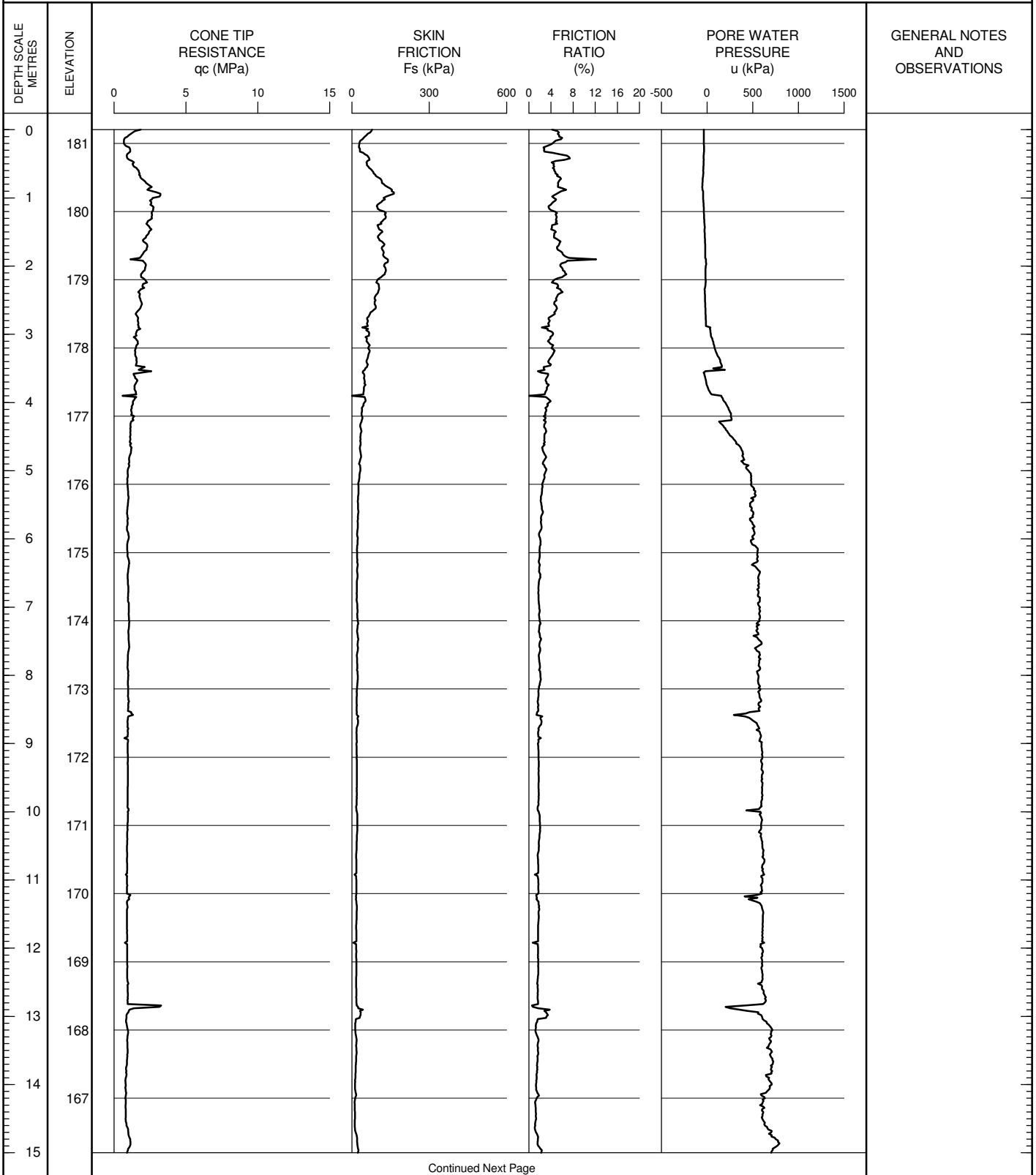
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2

PREDRILL DEPTH: 0

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: NR

RECORD OF CONE PENETRATION TEST CPT T5-1

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 4/28/2011 - 4/28/2011

SHEET 2 OF 2

LOCATION 4679994N; 331959 E

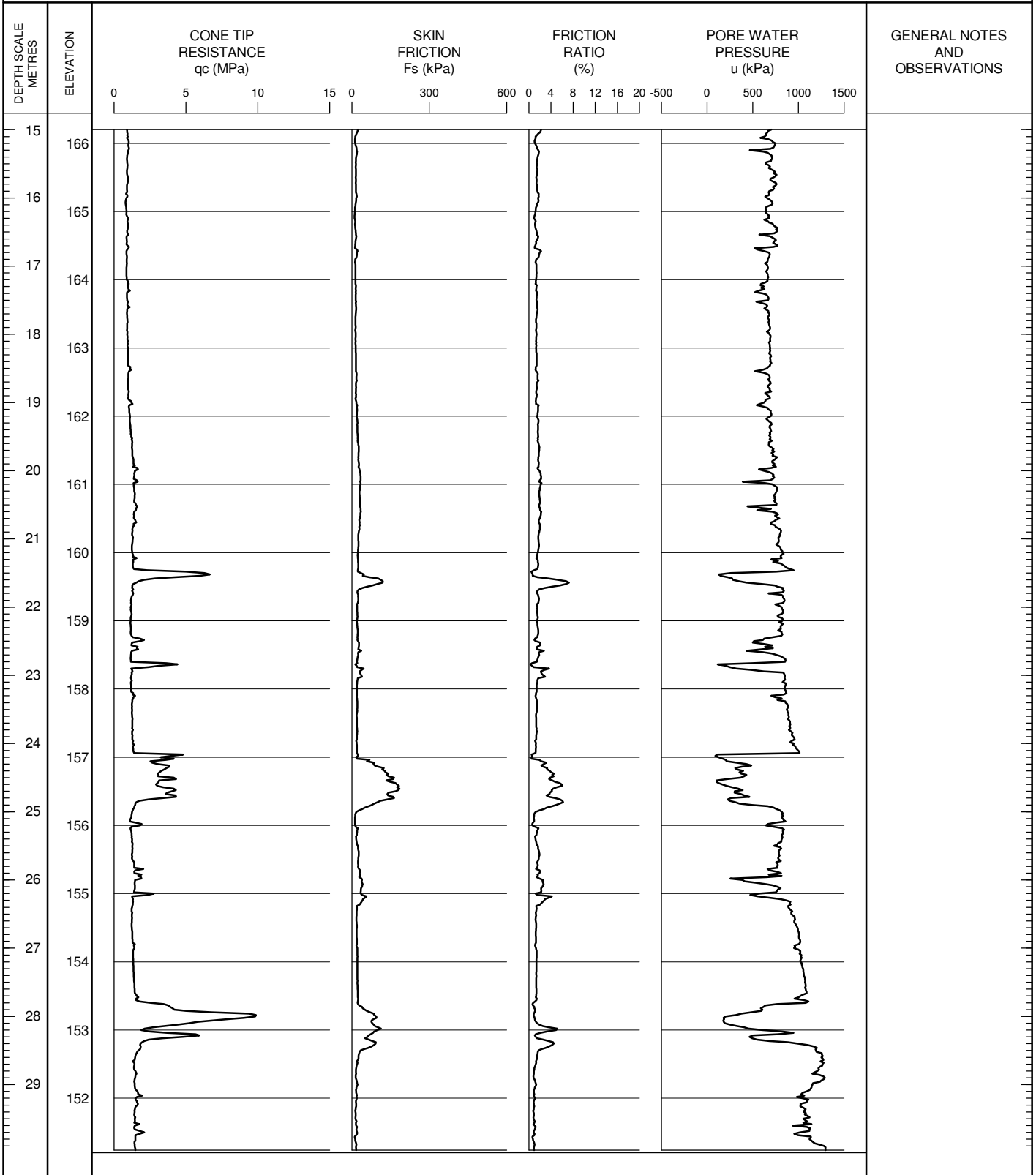
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2

PREDRILL DEPTH: 0

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



WEP CPT LOG CPT T5.GPJ ONTARIO.MOT.GDT 19/08/11

OPERATOR: TA

CHECKED: NR

RECORD OF CONE PENETRATION TEST CPT T5-2

METRIC

PROJECT Windsor-Essex Parkway

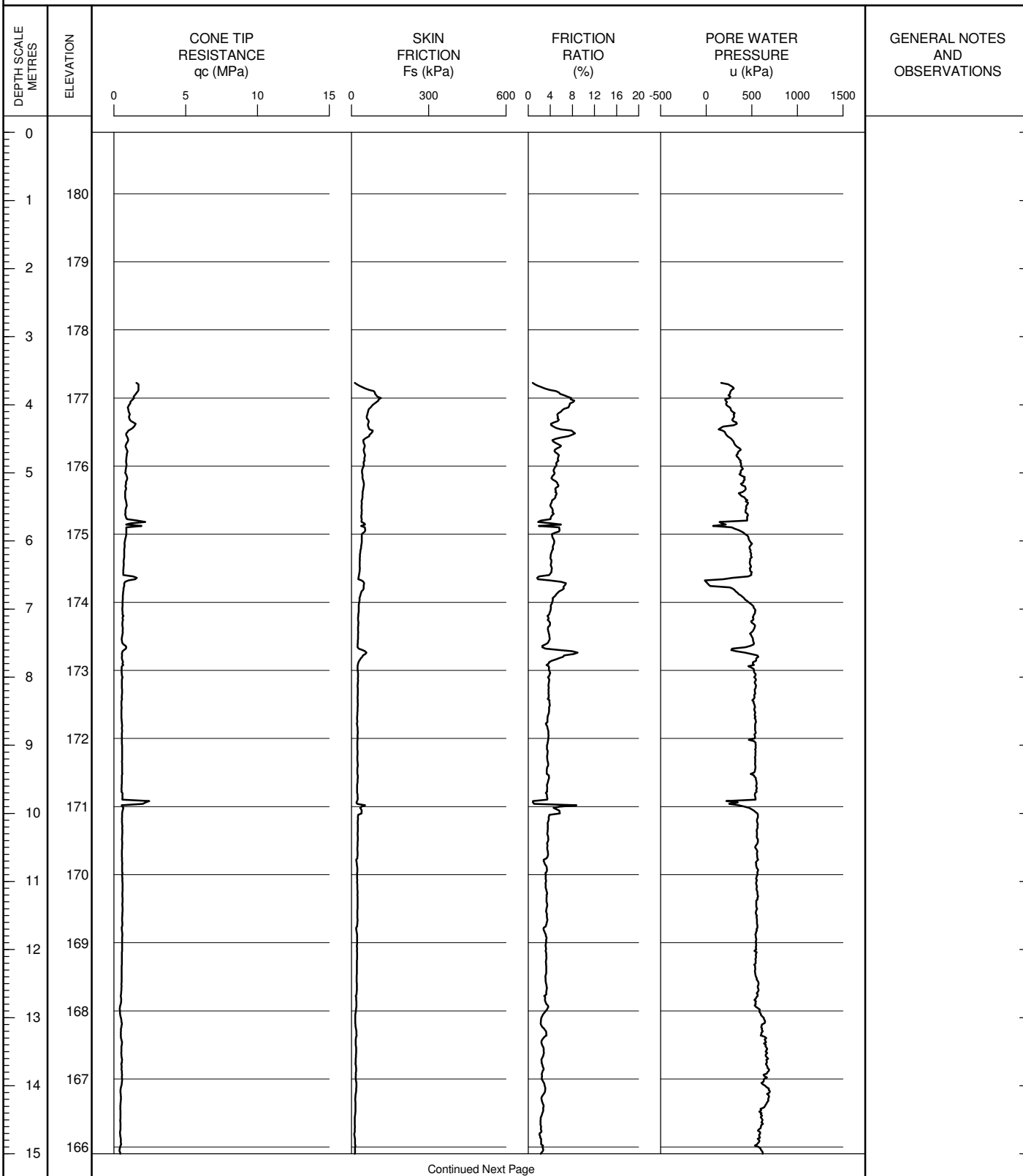
TEST DATE 4/28/2011 - 4/28/2011

SHEET 1 OF 3

LOCATION 4680011N; 331863 E

DATUM Geodetic

GROUND SURFACE ELEVATION: 180.9 PREDRILL DEPTH: 3.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: NR

RECORD OF CONE PENETRATION TEST CPT T5-2

METRIC

PROJECT Windsor-Essex Parkway

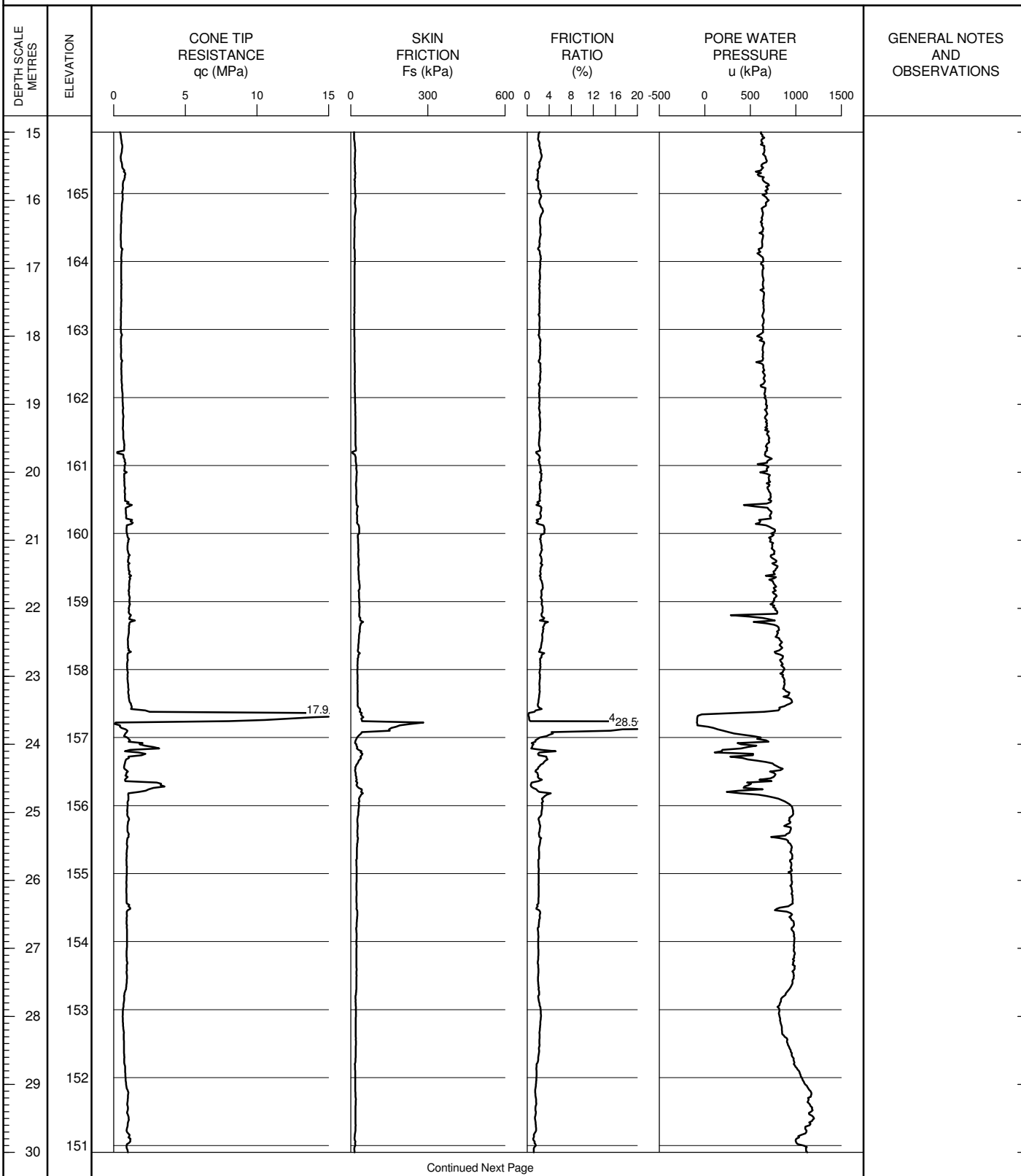
TEST DATE 4/28/2011 - 4/28/2011

SHEET 2 OF 3

LOCATION 4680011N; 331863 E

DATUM Geodetic

GROUND SURFACE ELEVATION: 180.9 PREDRILL DEPTH: 3.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEP CPT LOG CPT T5.GPJ ONTARIO.MOT.GDT 19/08/11

OPERATOR: TA

CHECKED: NR

RECORD OF CONE PENETRATION TEST CPT T5-2

METRIC

PROJECT Windsor-Essex Parkway

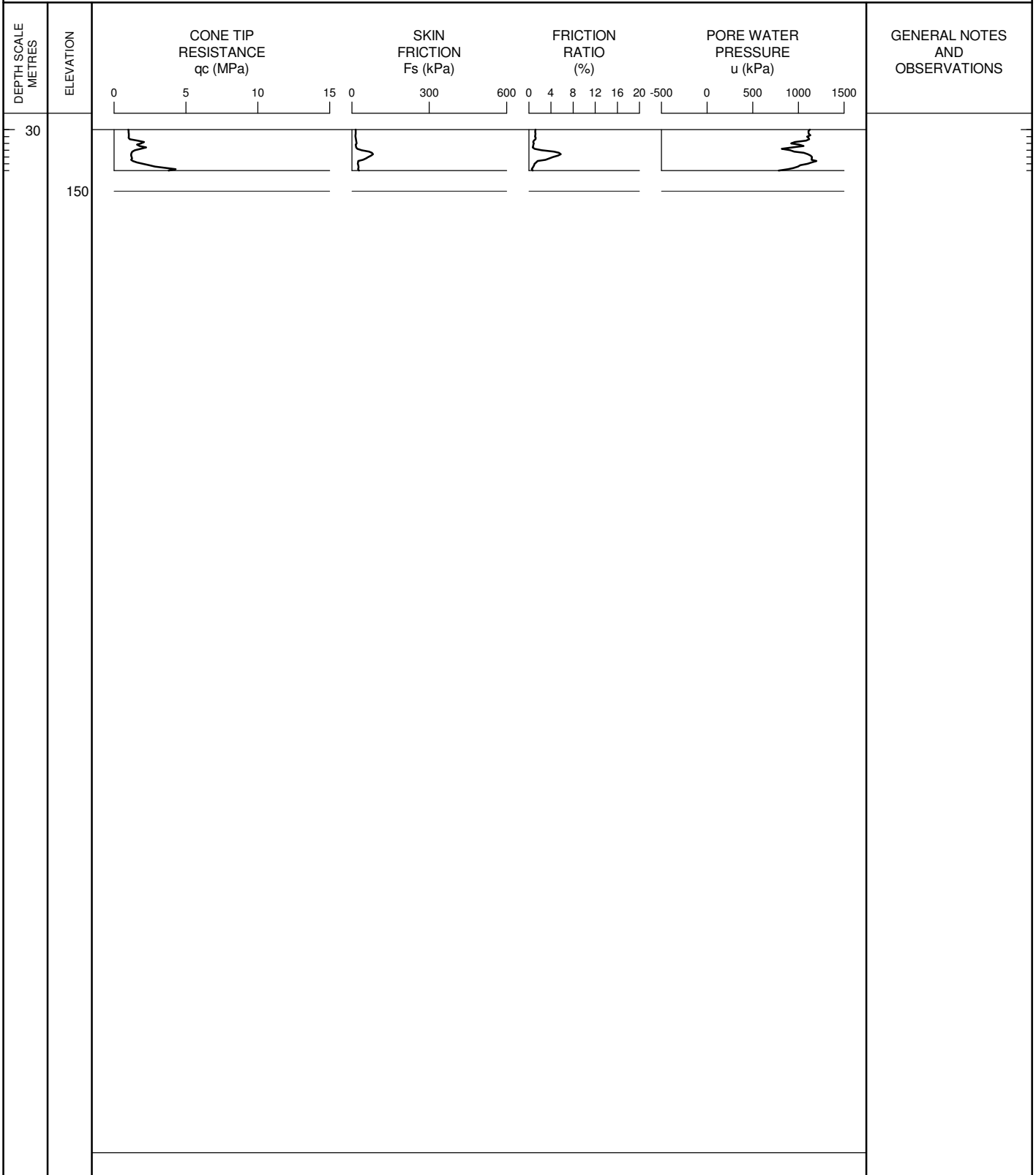
TEST DATE 4/28/2011 - 4/28/2011

SHEET 3 OF 3

LOCATION 4680011N; 331863 E

DATUM Geodetic

GROUND SURFACE ELEVATION: 180.9 PREDRILL DEPTH: 3.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: NR

RECORD OF CONE PENETRATION TEST CPT T5-E1

METRIC

PROJECT Windsor-Essex Parkway

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

SHEET 1 OF 2

LOCATION N4680064.3; E331958.7

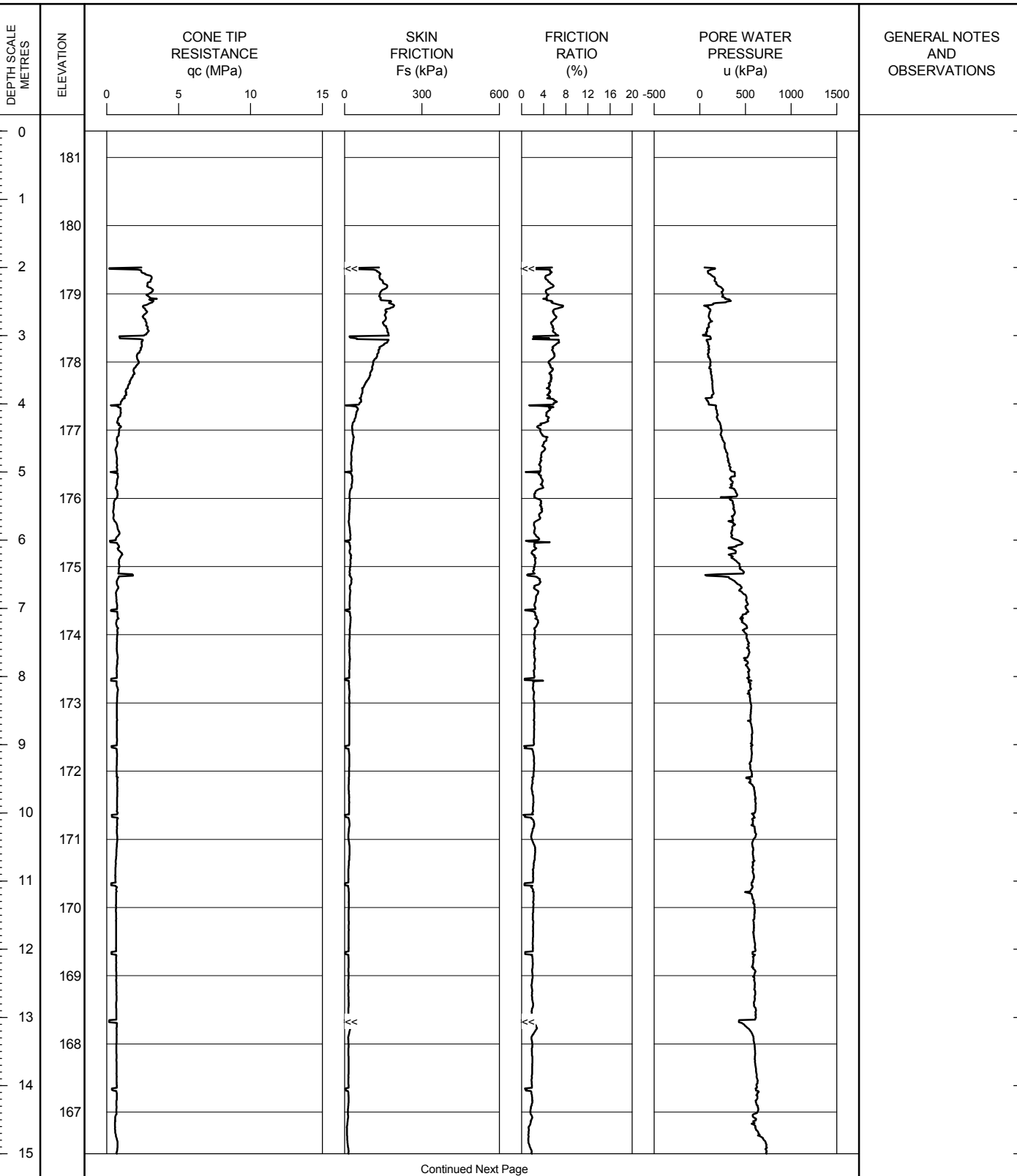
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.4

PREDRILL DEPTH: 1.98

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E1

METRIC

PROJECT Windsor-Essex Parkway

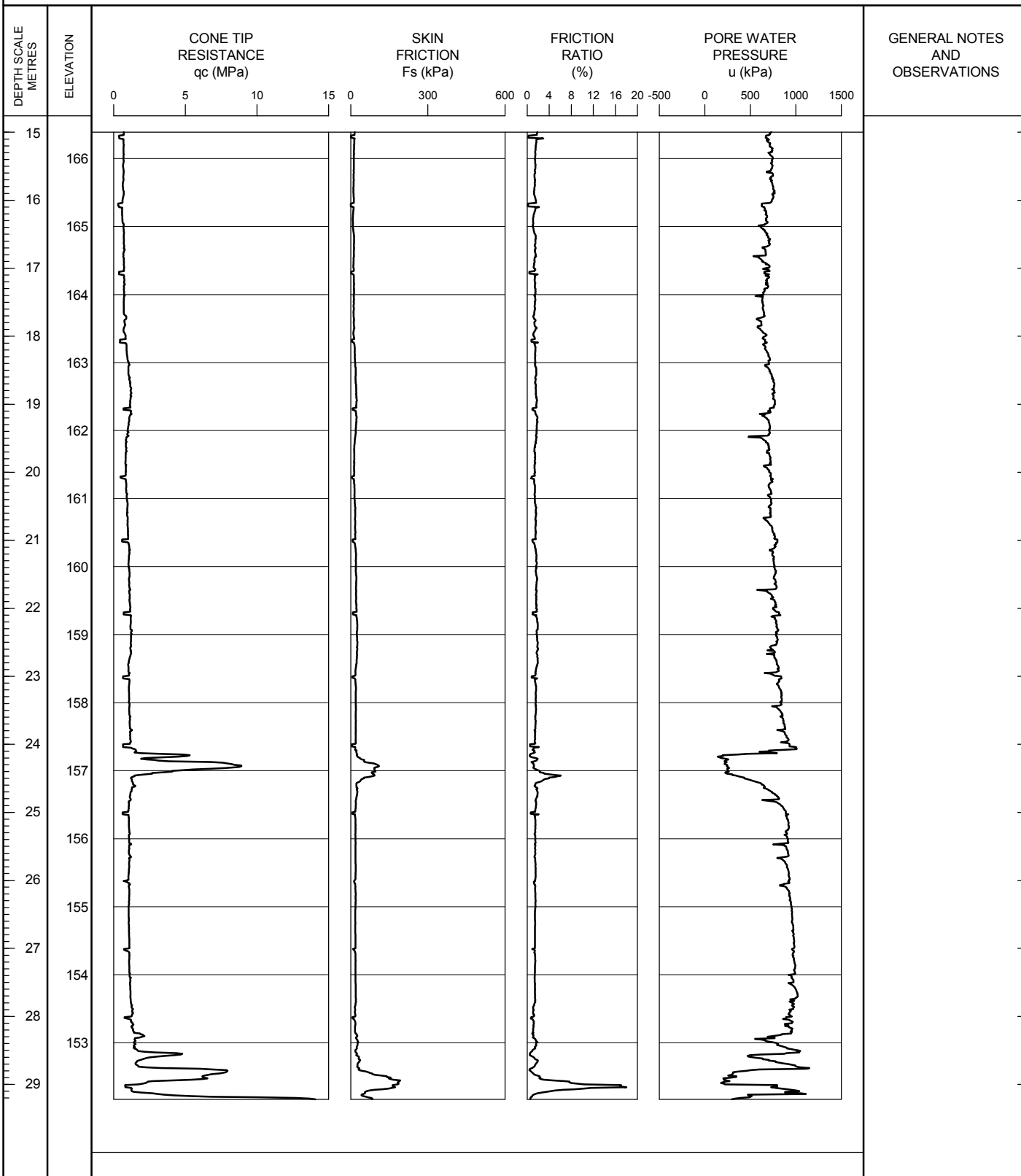
TEST DATE 9/8/2011 - 9/8/2011

SHEET 2 OF 2

LOCATION N4680064.3; E331958.7

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.4 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEF CPT LOG CPT T5-E1.GPJ ONTARIO MOT.GDT 21/12/11

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E2

METRIC

PROJECT Windsor-Essex Parkway

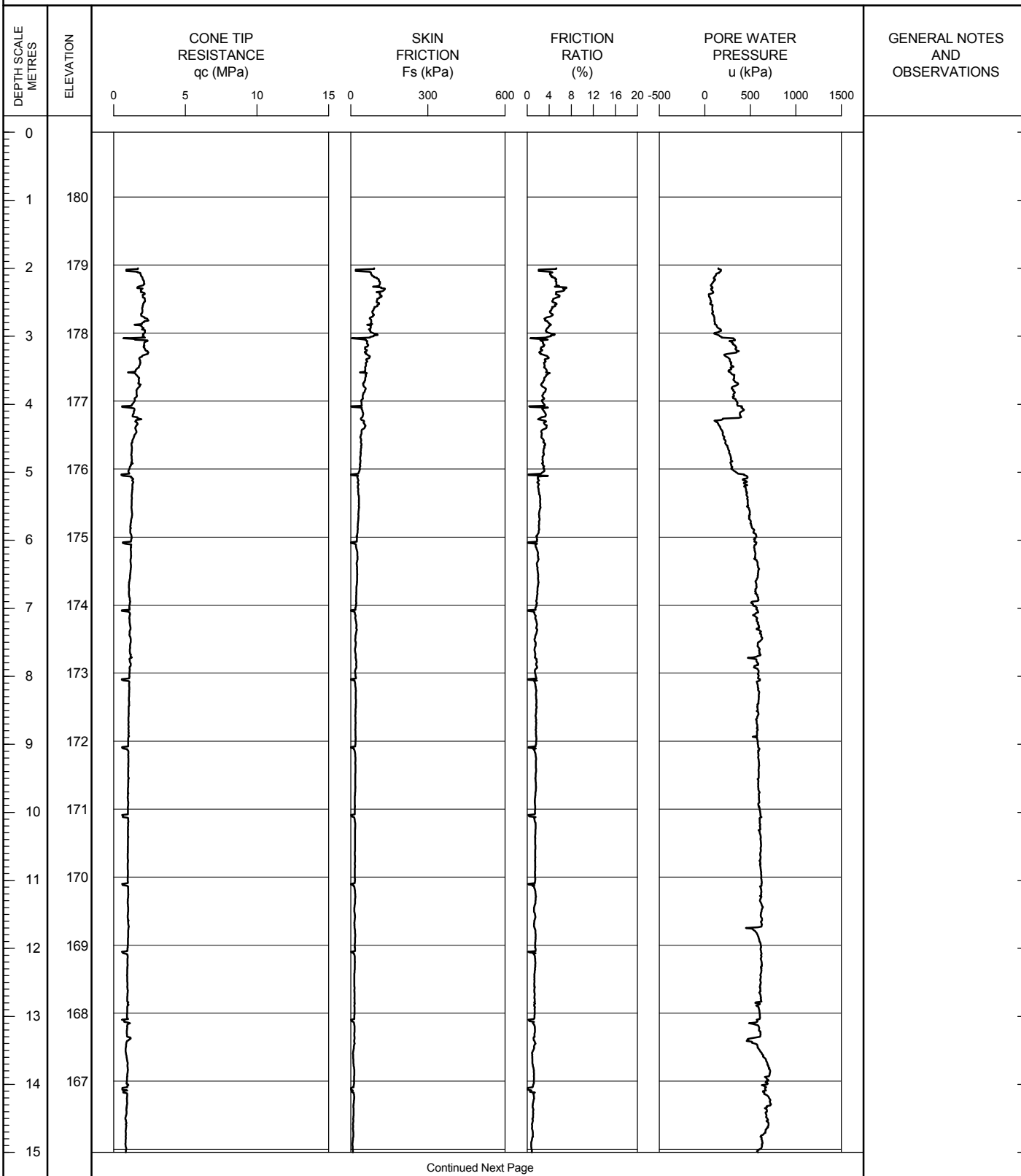
TEST DATE 9/7/2011 - 9/7/2011

SHEET 1 OF 3

LOCATION N4679950.8; E331992.9

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.0 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEF CPT LOG CPT T5-E2.GPJ ONTARIO MOT.GDT 21/12/11

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E2

METRIC

PROJECT Windsor-Essex Parkway

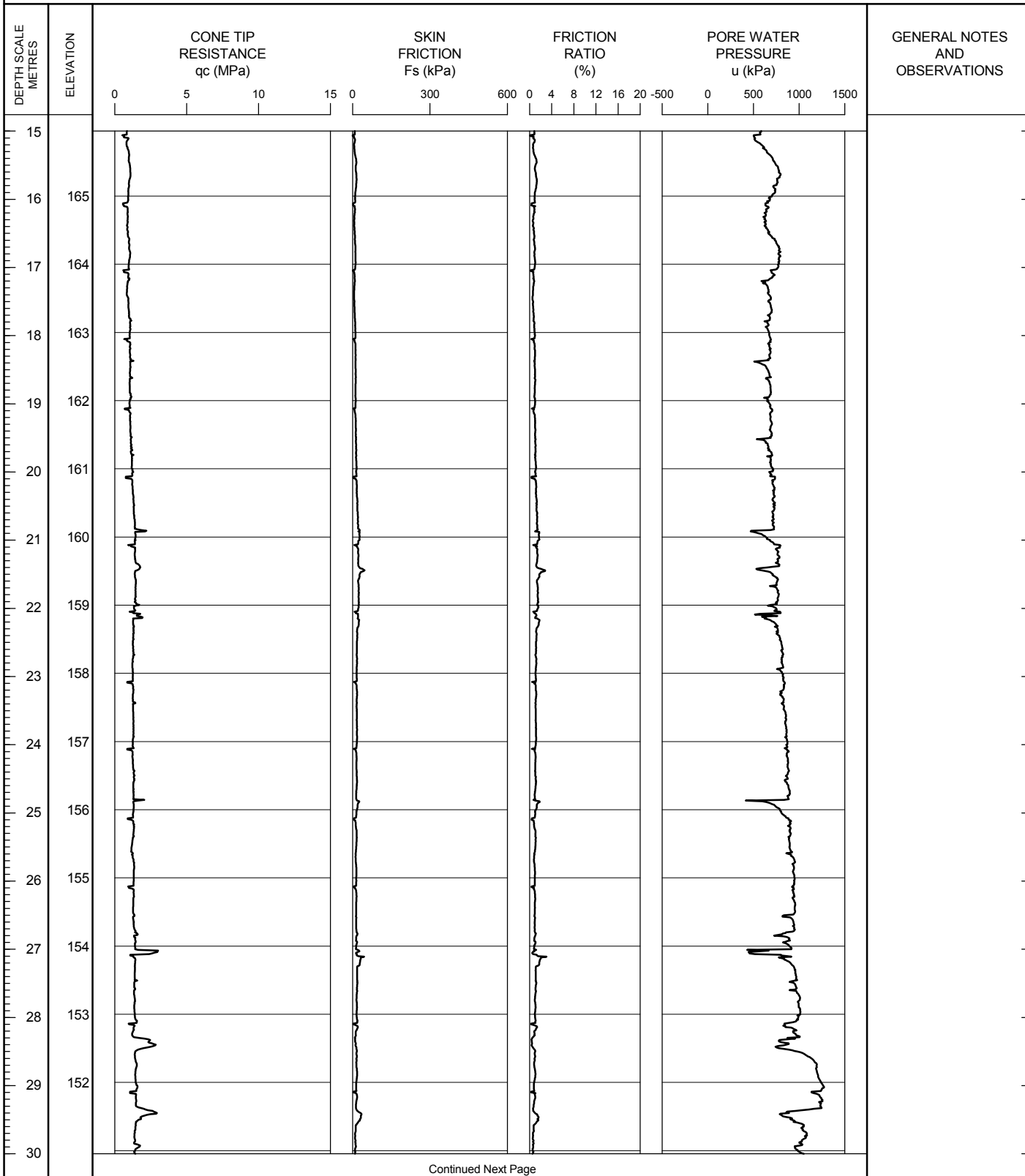
TEST DATE 9/7/2011 - 9/7/2011

SHEET 2 OF 3

LOCATION N4679950.8; E331992.9

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.0 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEP CPT LOG CPT T5-E2.GPJ ONTARIO MOT.GDT 21/12/11

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E2

METRIC

PROJECT Windsor-Essex Parkway





TEST DATE 9/7/2011 - 9/7/2011

SHEET 3 OF 3

LOCATION N4679950.8; E331992.9

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.0 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0

DEPTH SCALE METRES	ELEVATION	CONE TIP RESISTANCE qc (MPa)	SKIN FRICTION Fs (kPa)	FRICTION RATIO (%)	PORE WATER PRESSURE u (kPa)	GENERAL NOTES AND OBSERVATIONS
		0 5 10 15	0 300 600	0 4 8 12 16 20 -500	0 500 1000 1500	
30						

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E3

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 9/1/2011 - 9/1/2011

SHEET 1 OF 2

LOCATION N4680008.0; E331889.1

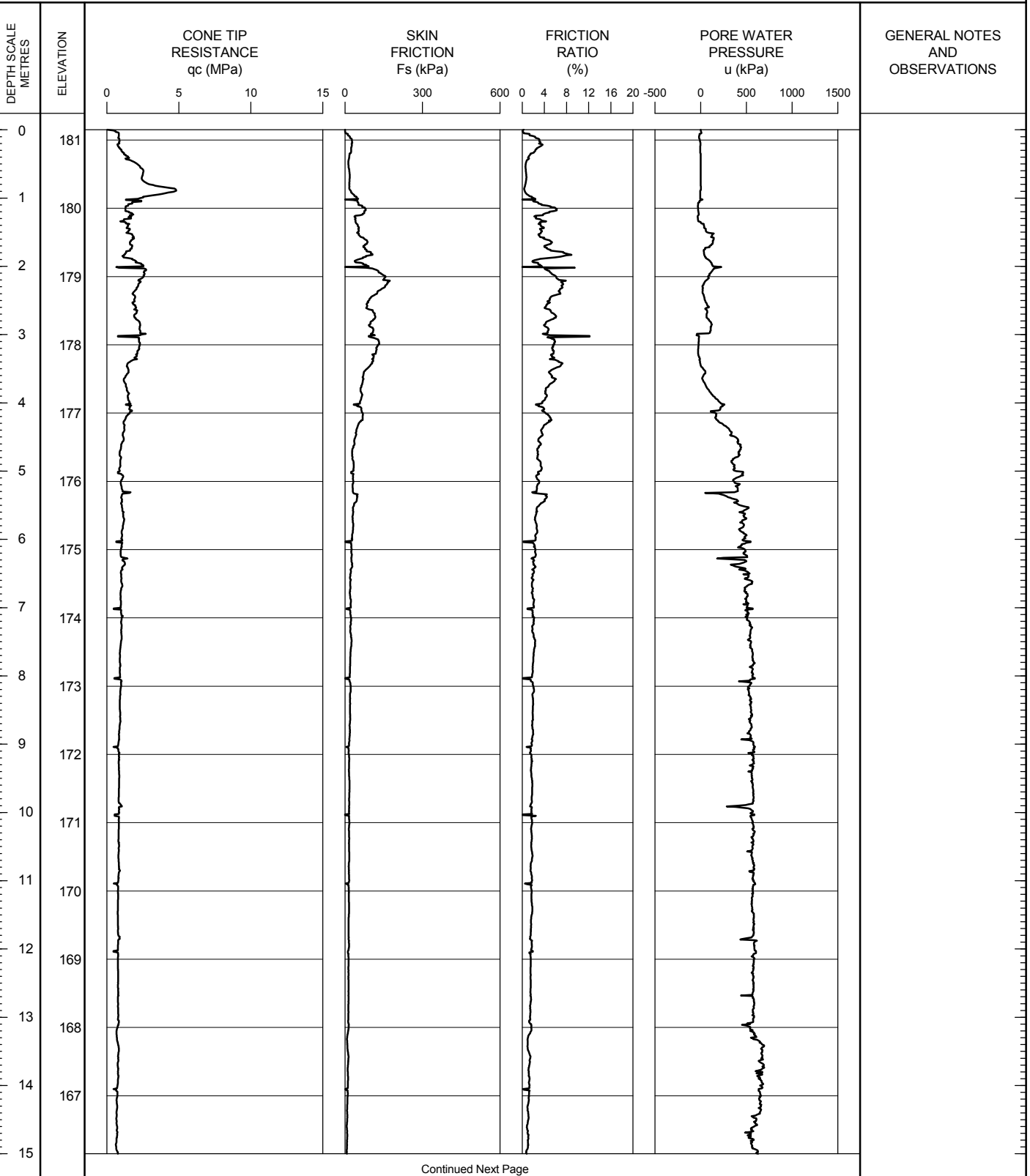
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2

PREDRILL DEPTH: 0

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E3

METRIC

PROJECT Windsor-Essex Parkway

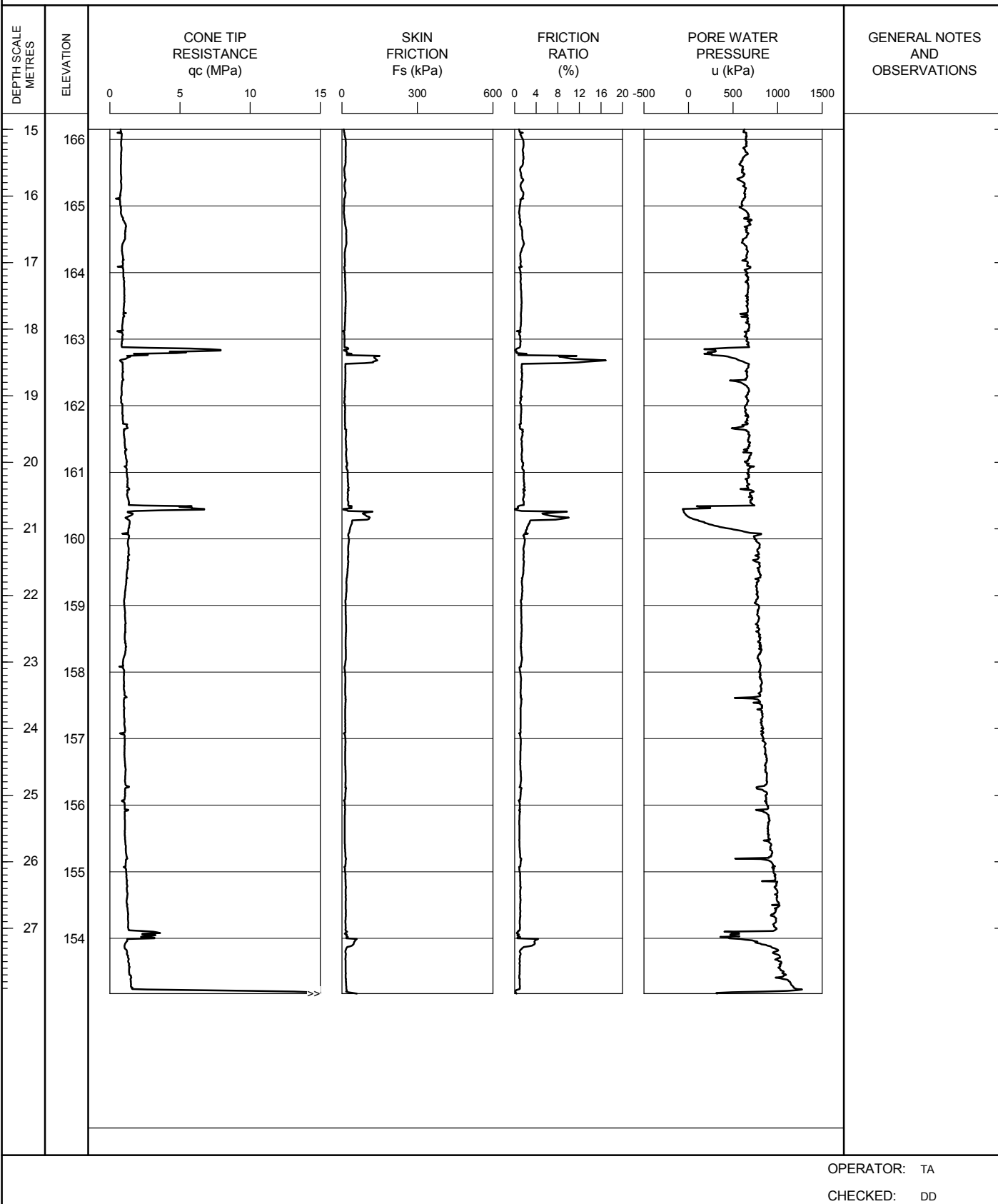
TEST DATE 9/1/2011 - 9/1/2011

SHEET 2 OF 2

LOCATION N4680008.0; E331889.1

DATUM Geodetic

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



RECORD OF CONE PENETRATION TEST CPT T5-E4

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 9/6/2011 - 9/6/2011

SHEET 1 OF 2

LOCATION N4680046.8; E331852.3

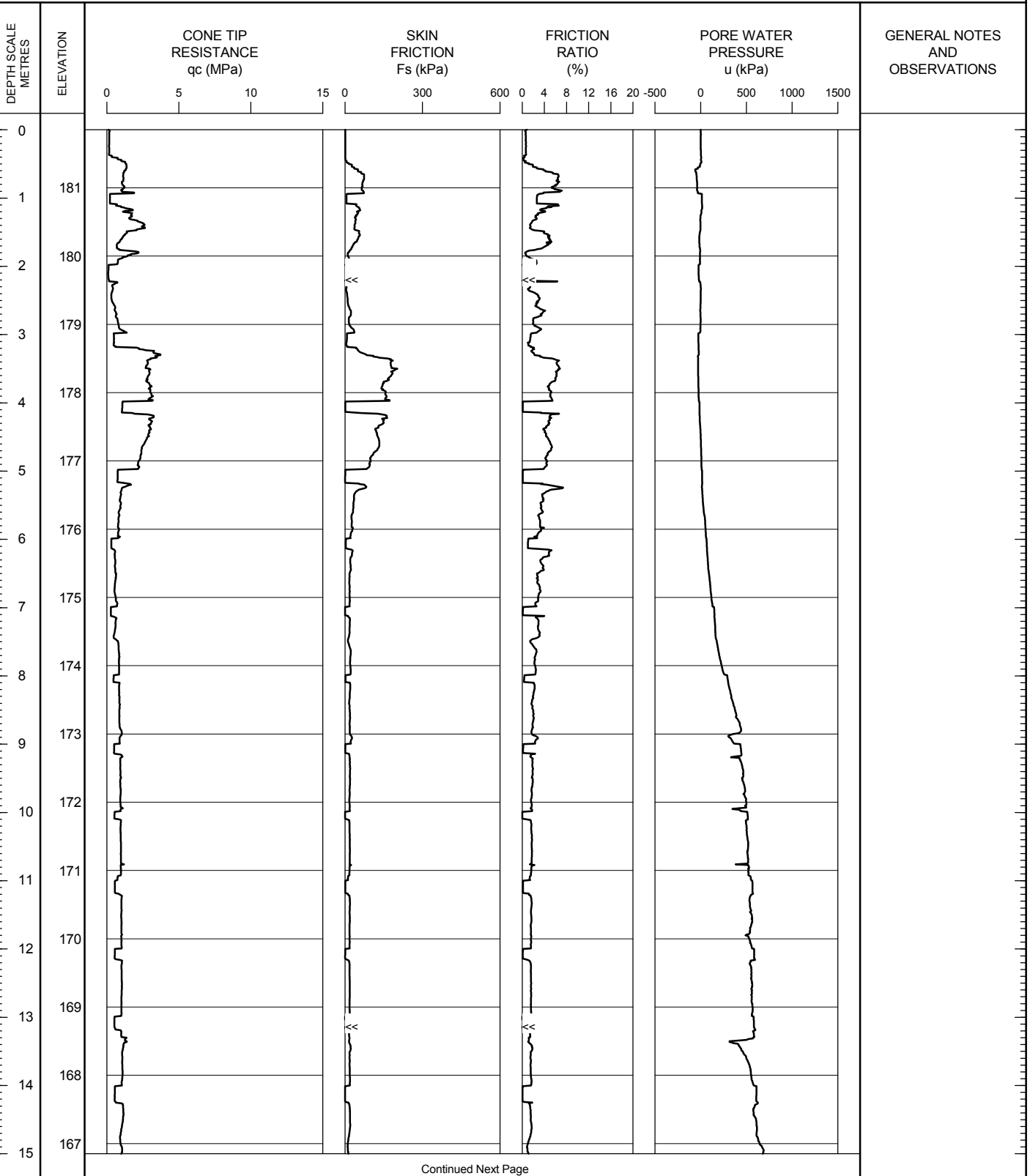
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.9

PREDRILL DEPTH: 0

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E4

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 9/6/2011 - 9/6/2011

SHEET 2 OF 2

LOCATION N4680046.8; E331852.3

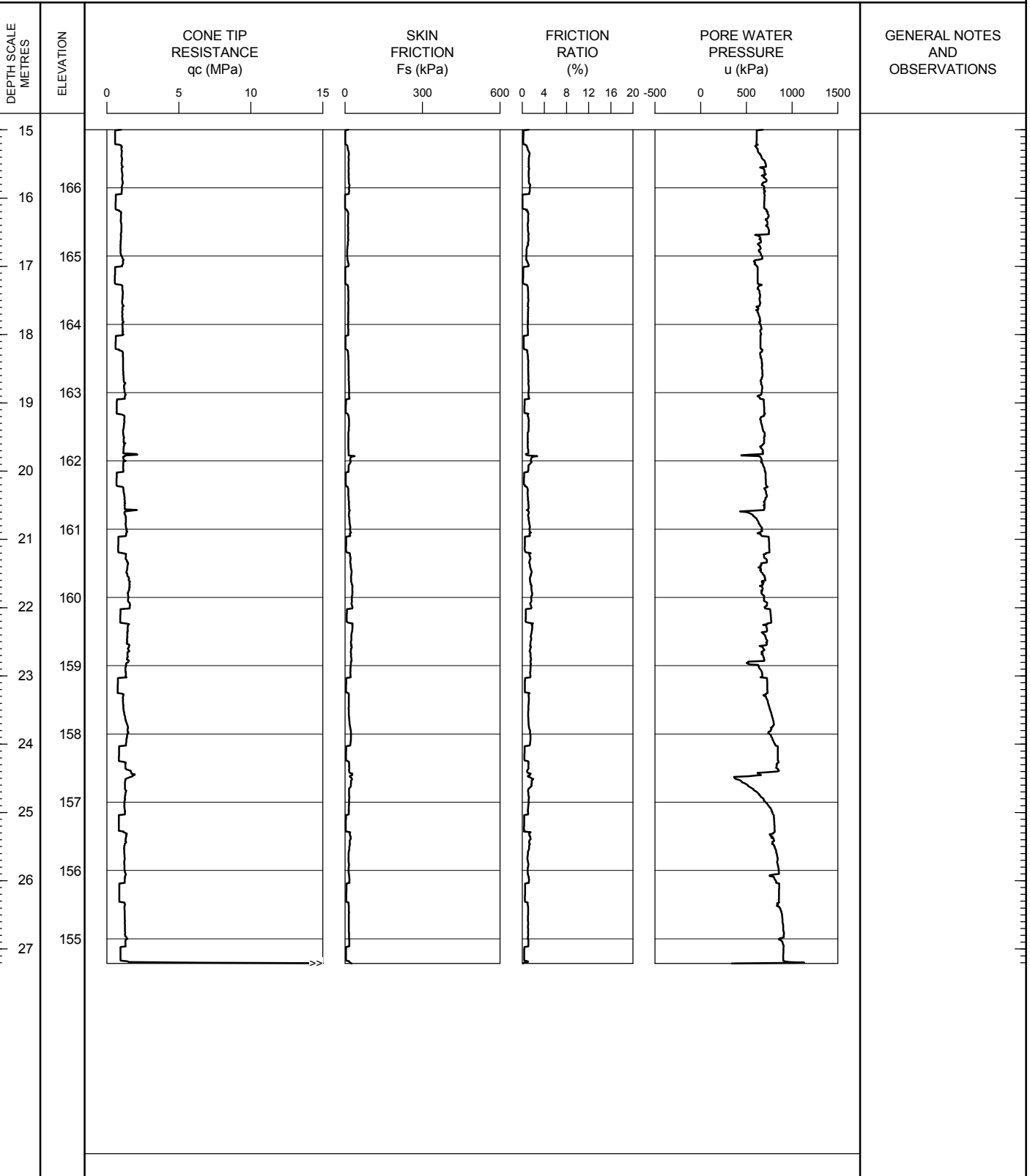
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.9

PREDRILL DEPTH: 0

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



WEP CPT LOG CPT T5.GPJ ONTARIO MOT.GDT 21/12/11

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E5

METRIC

PROJECT Windsor-Essex Parkway

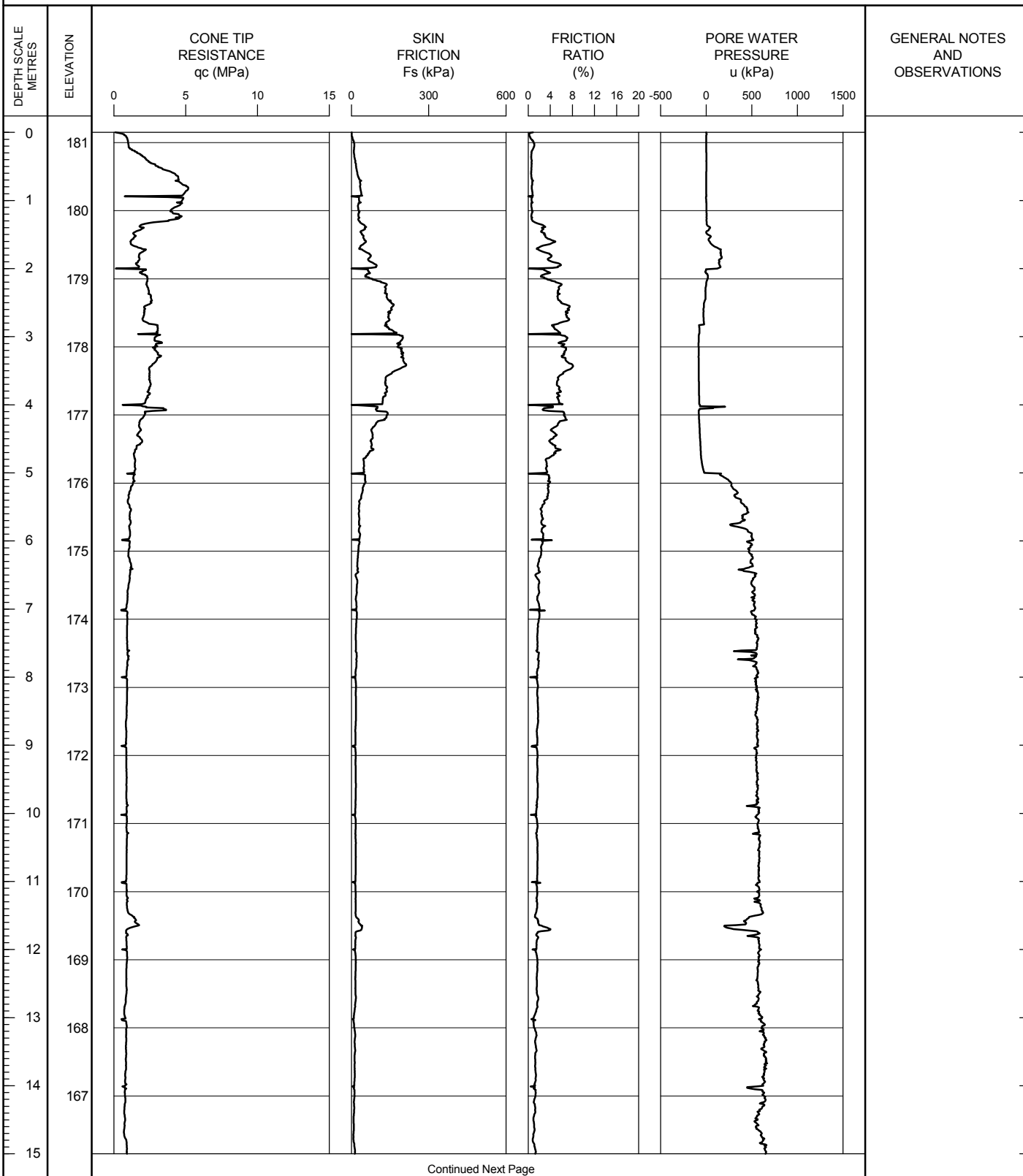
TEST DATE 9/1/2011 - 9/1/2011

SHEET 1 OF 3

LOCATION N4679976.4; E331865.6

DATUM Geodetic

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



WEF CPT LOG CPT T5.GPJ ONTARIO.MOT.GDT 21/12/11

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E5

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 9/1/2011 - 9/1/2011

SHEET 2 OF 3

LOCATION N4679976.4; E331865.6

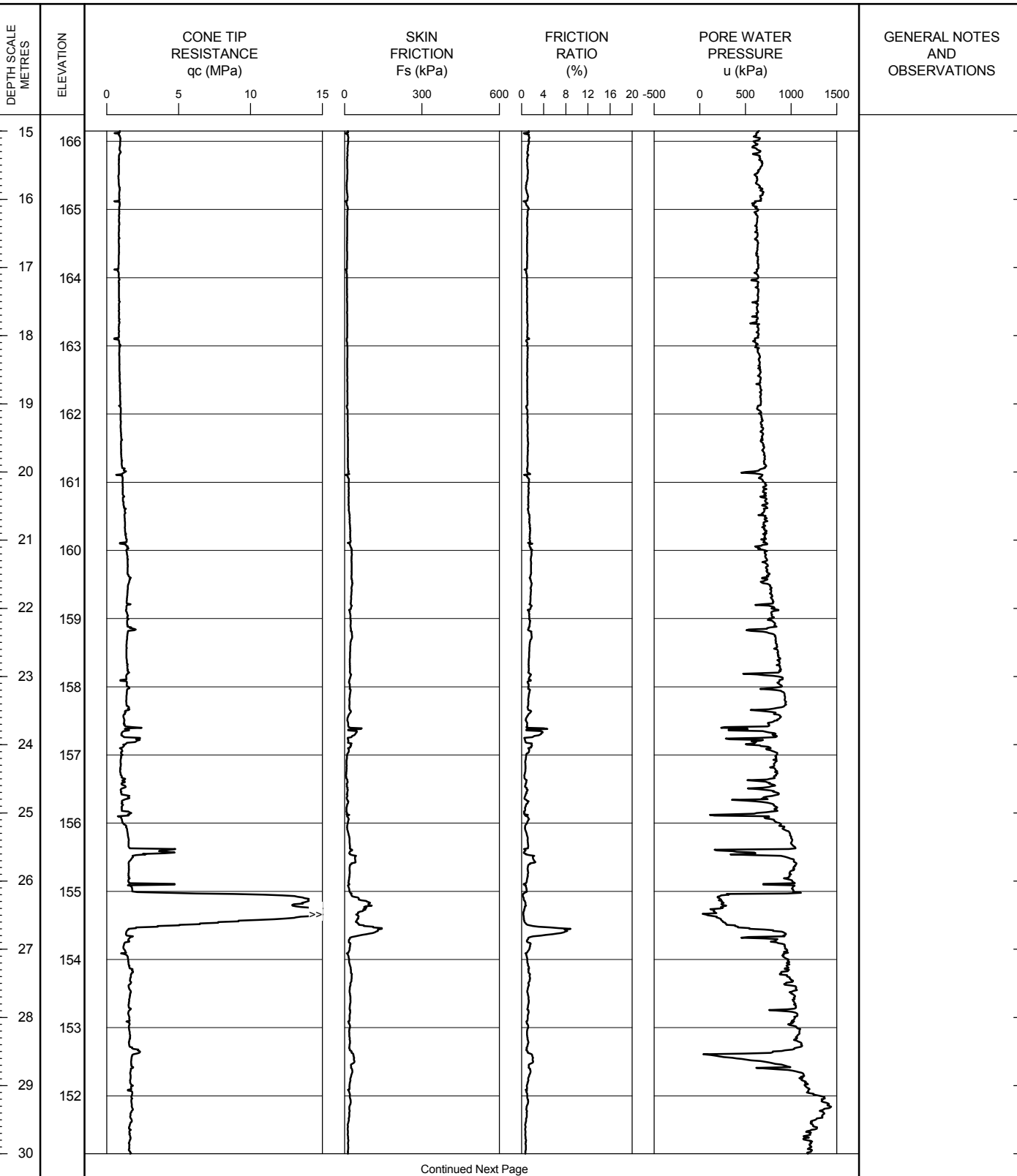
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.2

PREDRILL DEPTH: 0

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



WEP CPT LOG CPT T5.GPJ ONTARIO.MOT.GDT 21/12/11

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT T5-E5

METRIC

PROJECT Windsor-Essex Parkway

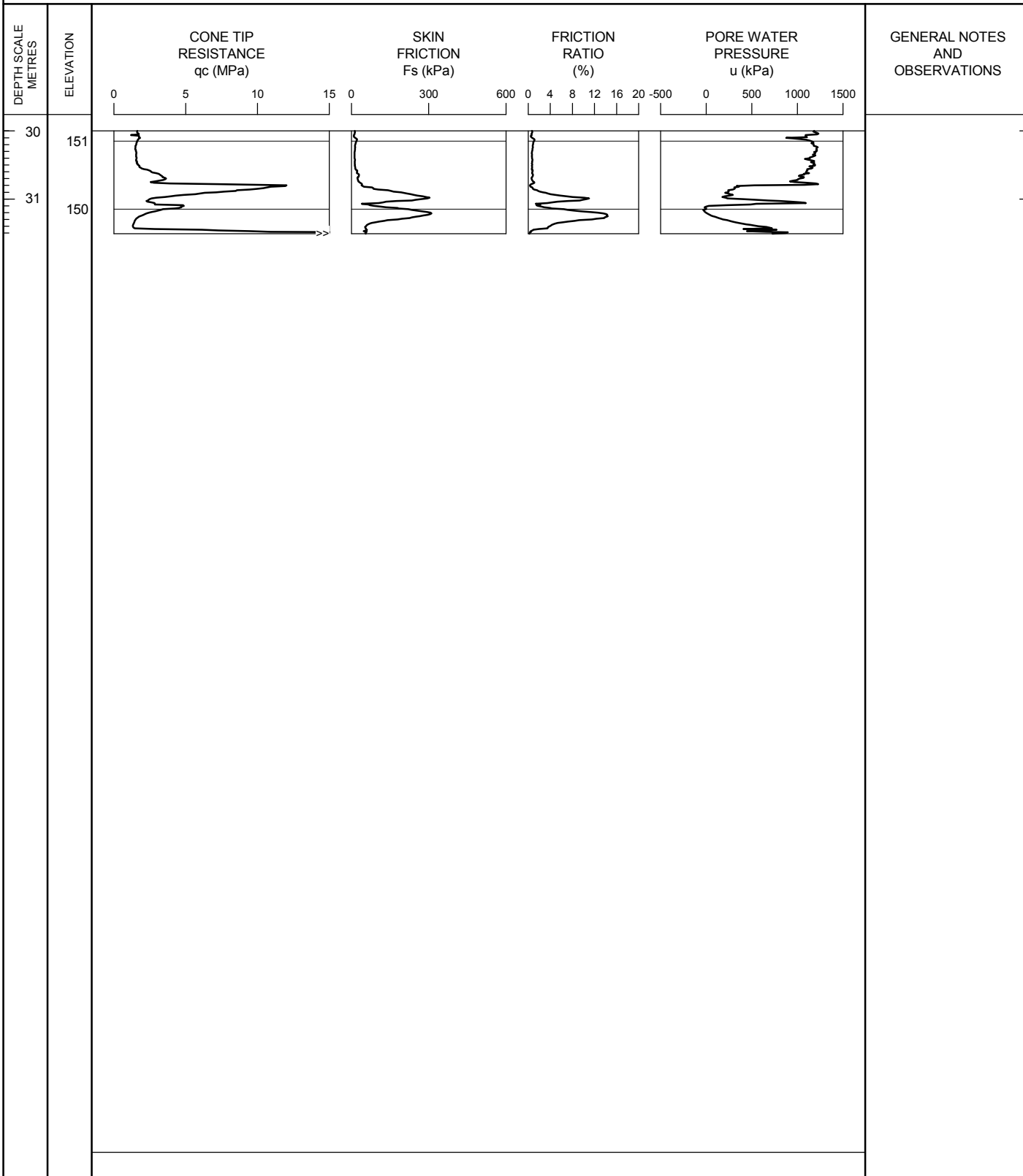
TEST DATE 9/1/2011 - 9/1/2011

SHEET 3 OF 3

LOCATION N4679976.4; E331865.6

DATUM Geodetic

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



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 33-RW

METRIC

PROJECT Windsor-Essex Parkway

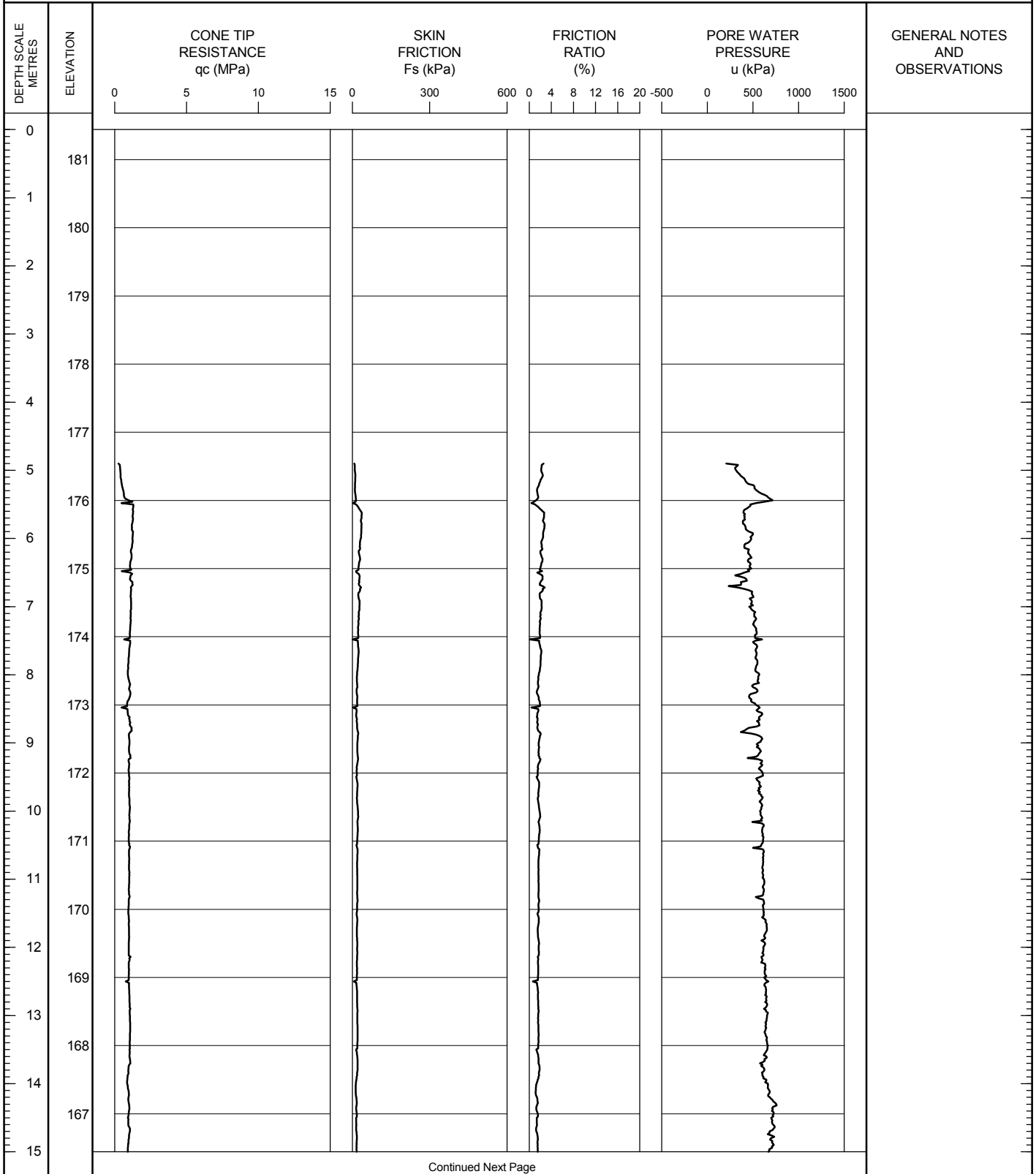
TEST DATE 4/29/2011 - 4/30/2011

SHEET 1 OF 3

LOCATION N4680121.7; E331932.6

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.4 PREDRILL DEPTH: 3.5 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: NR

RECORD OF CONE PENETRATION TEST CPT 33-RW

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 4/29/2011 - 4/30/2011

SHEET 2 OF 3

LOCATION N4680121.7; E331932.6

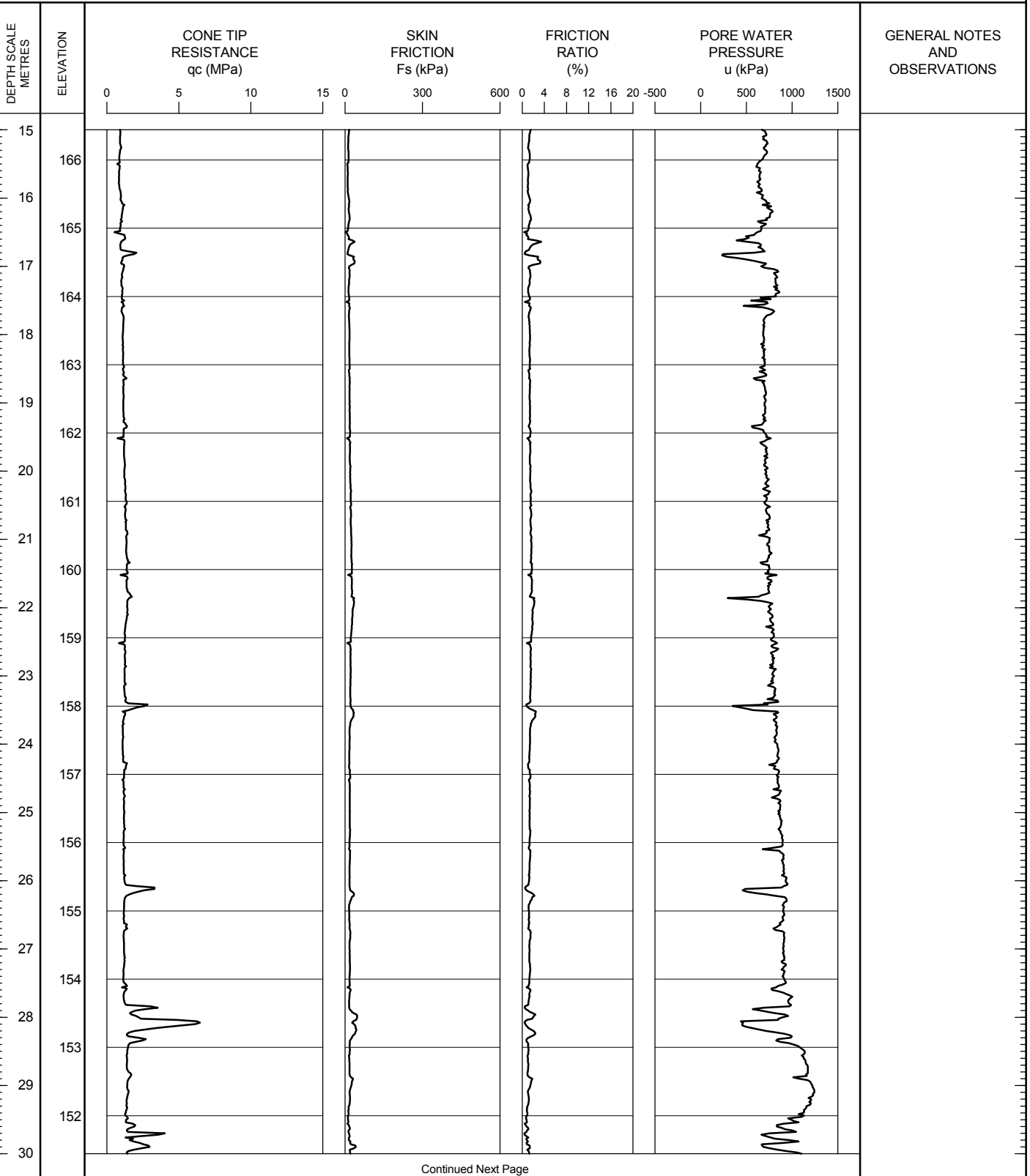
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.4

PREDRILL DEPTH: 3.5

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: NR

METRIC

SHEET 3 OF 3

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.4 PREDRILL DEPTH: 3.5 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0

RECORD OF CONE PENETRATION TEST CPT 34-RW

METRIC

PROJECT Windsor-Essex Parkway

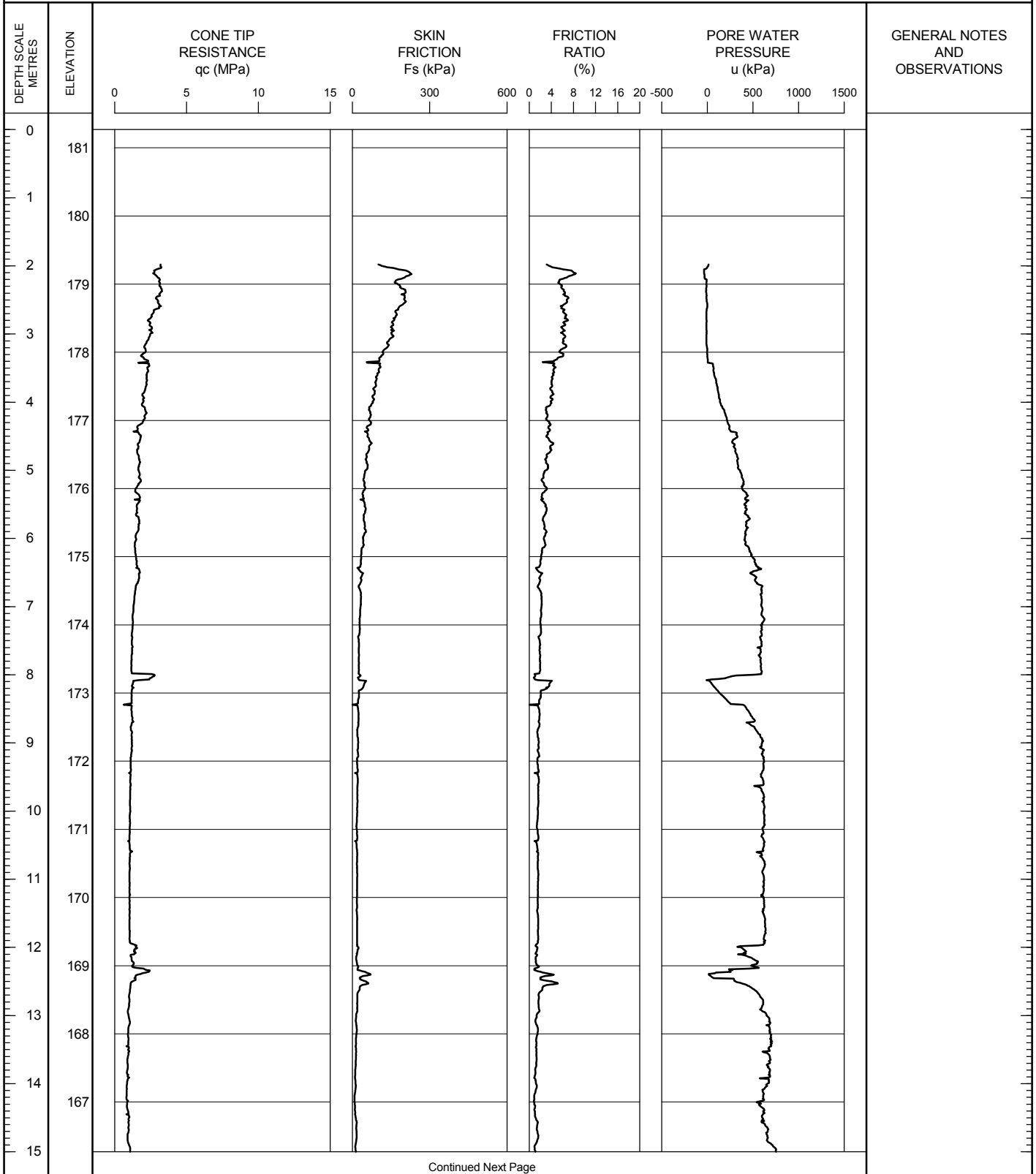
TEST DATE 8/23/2011 - 8/23/2011

SHEET 1 OF 3

LOCATION N4679865.7; E331958.2

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.3 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 34-RW

METRIC

PROJECT Windsor-Essex Parkway

TEST DATE 8/23/2011 - 8/23/2011

SHEET 2 OF 3

LOCATION N4679865.7; E331958.2

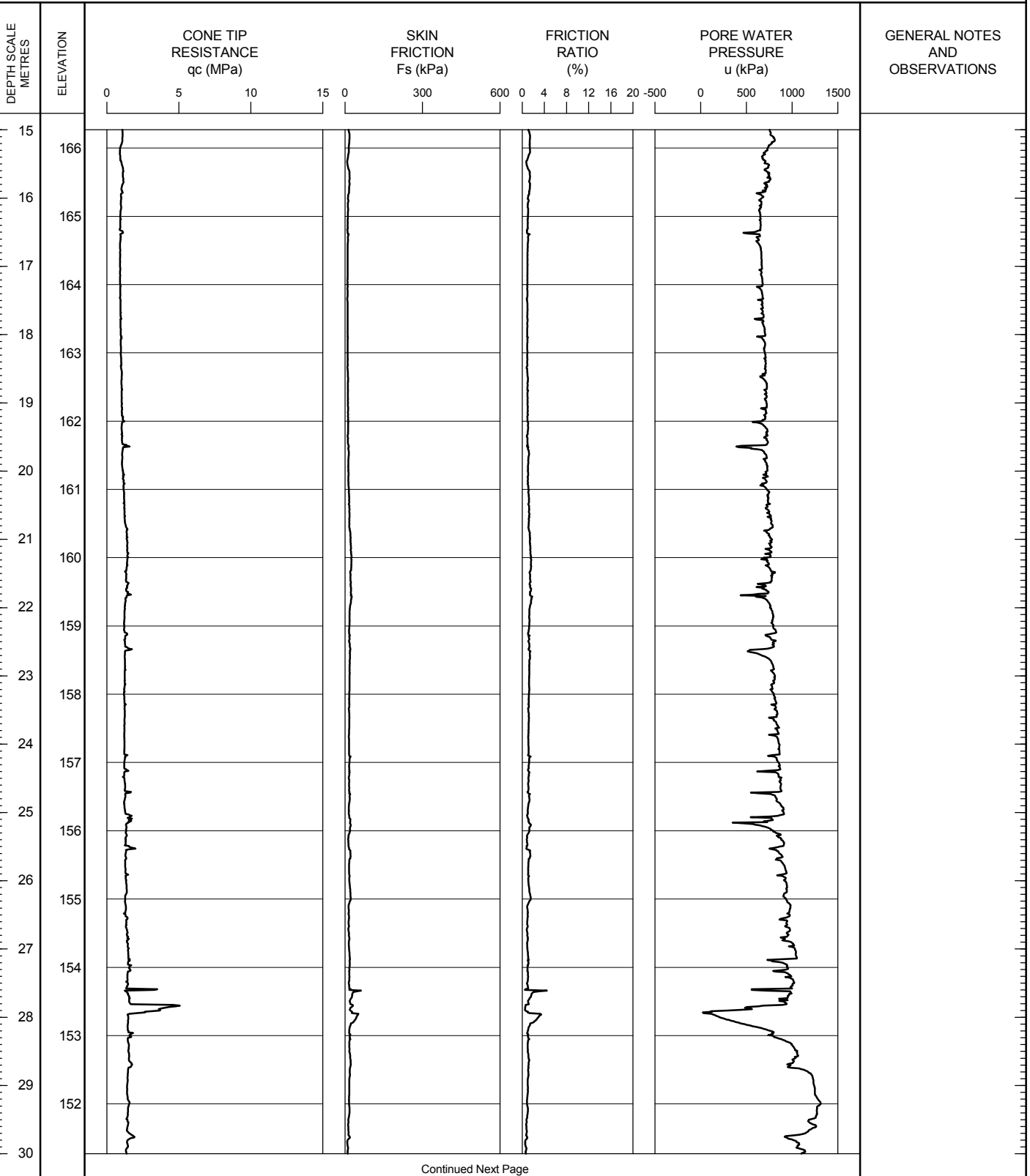
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.3

PREDRILL DEPTH: 1.98

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 34-RW

METRIC

PROJECT Windsor-Essex Parkway

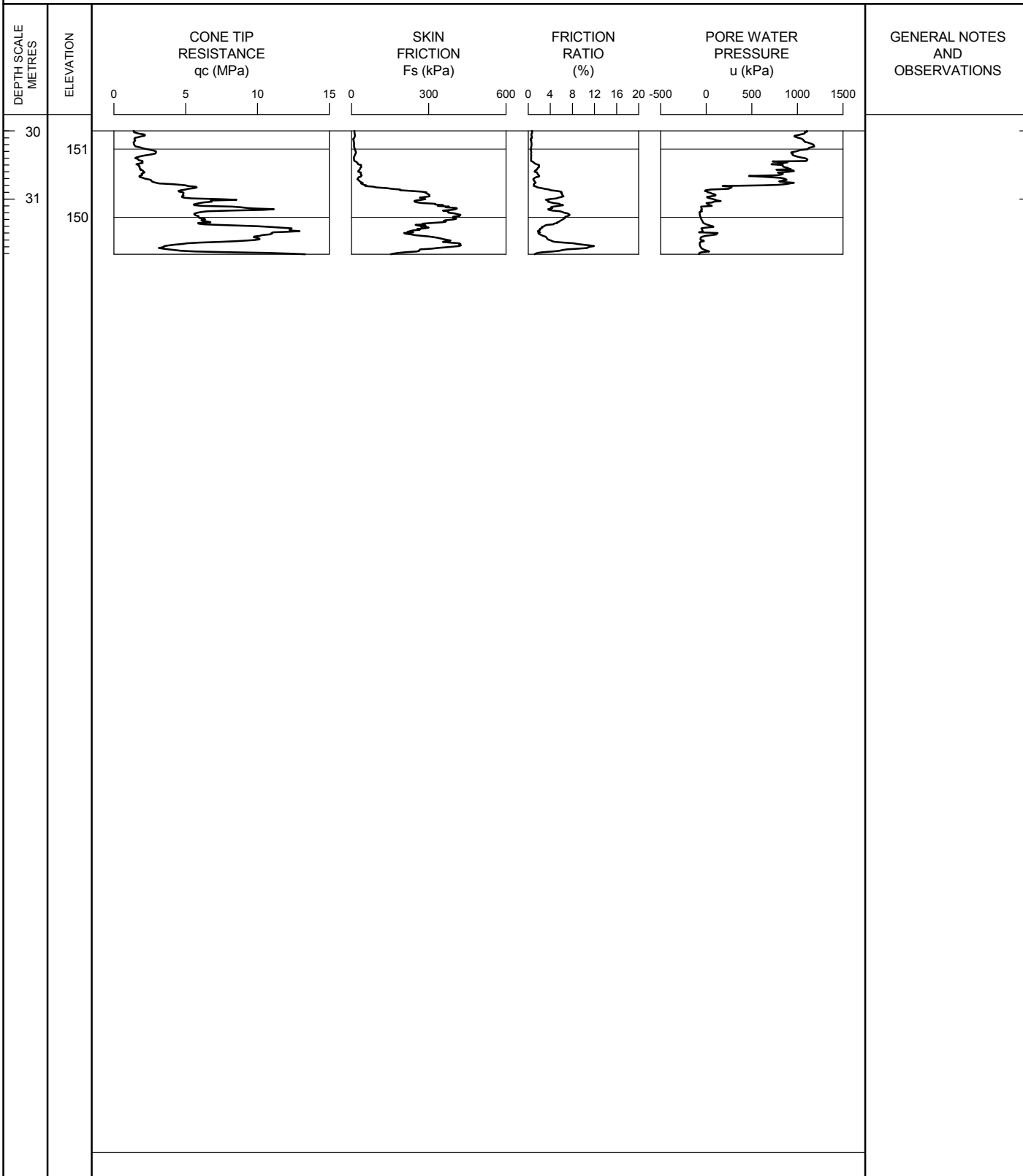
TEST DATE 8/23/2011 - 8/23/2011

SHEET 3 OF 3

LOCATION N4679865.7; E331958.2

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.3 PREDRILL DEPTH: 1.98 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



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

OPERATOR: TA

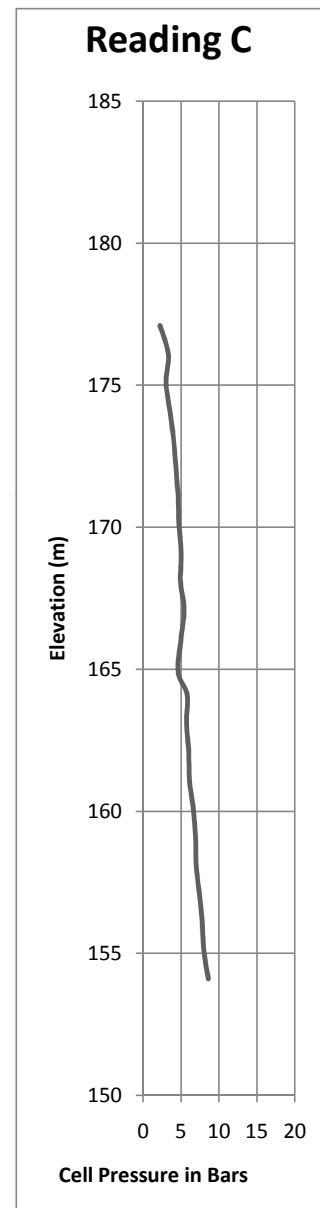
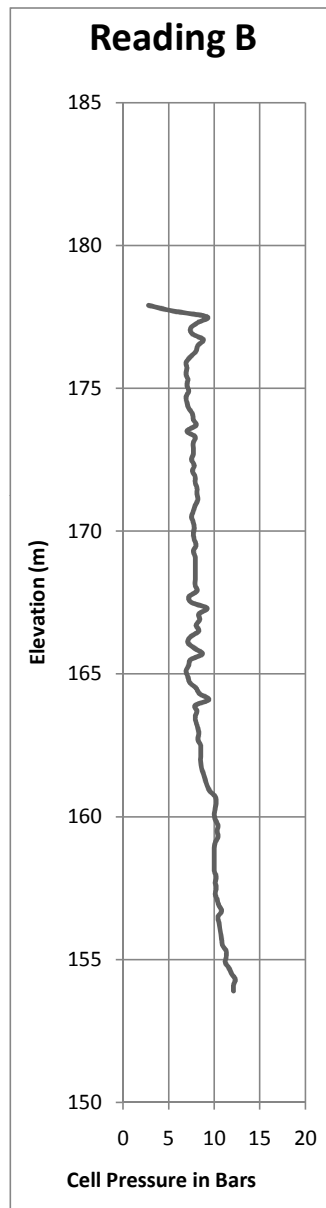
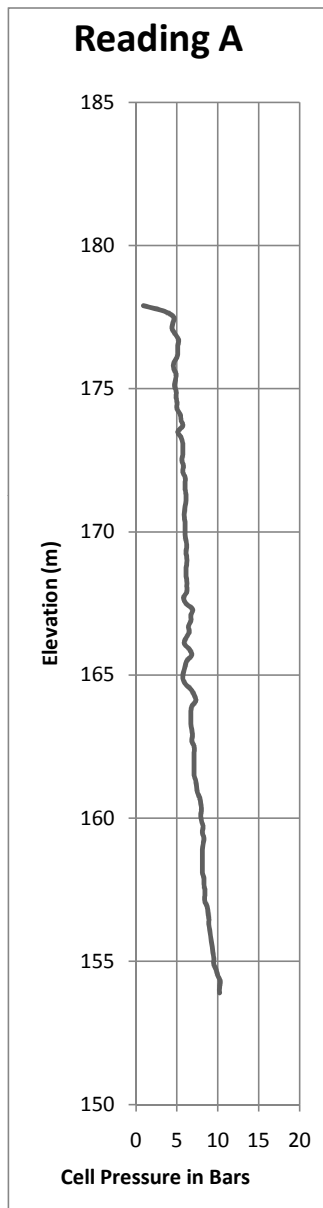
CHECKED: DD

RECORD OF DILATOMETER TEST DMT T5-1

Project : Windsor-Essex Parkway
Location: 4679979N; 331992E
Ground Surface Elevation : 181.1

Test Date: 4/29/2011
Predrill Depth : 3 m
Delta A: 0.19 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.35 Bar



Operator: LC

Checked: DD

Appendix B: Borehole and CPT Logs from Previous Investigations

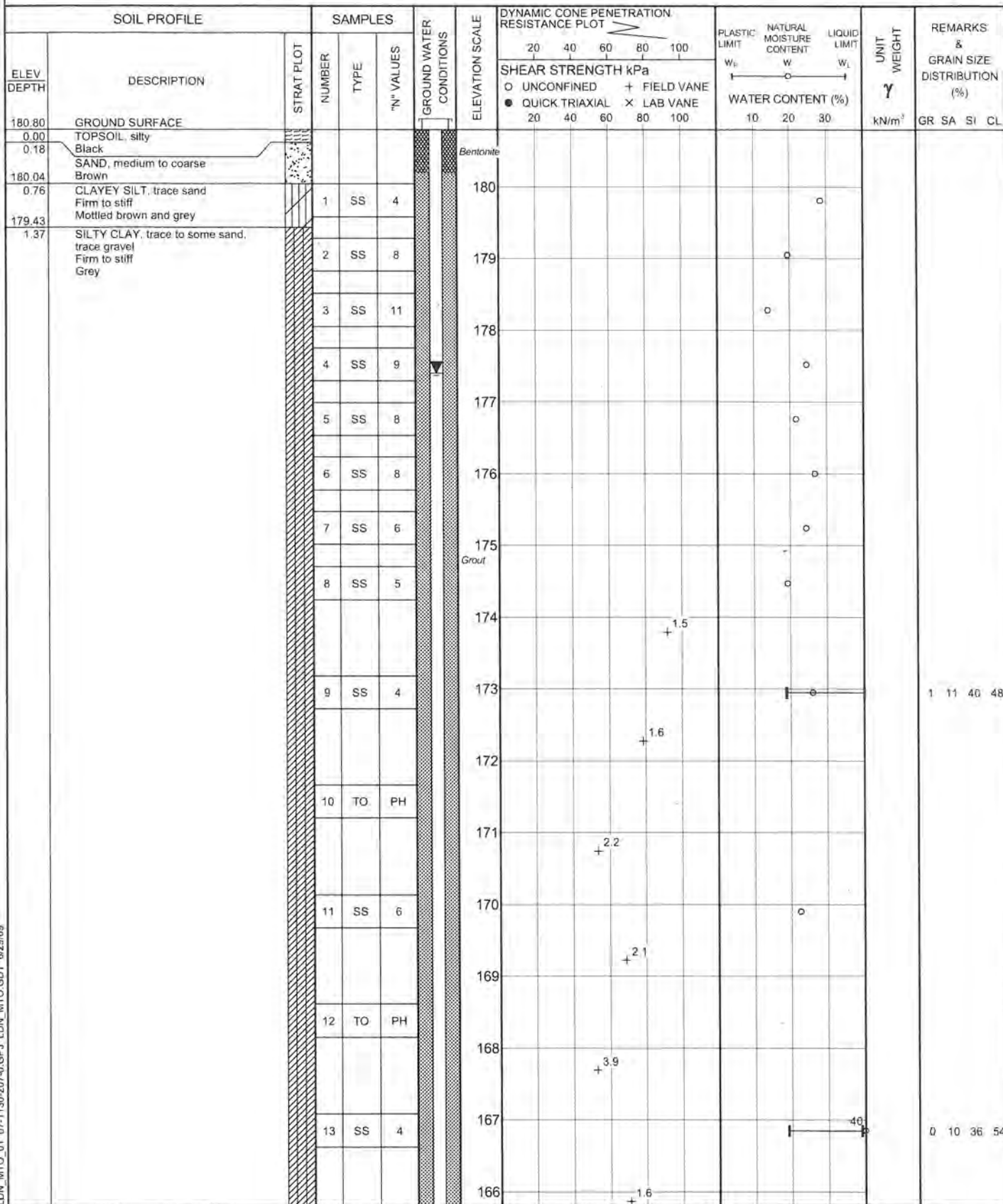
Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

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

PROJECT 07-1130-207-0 **RECORD OF BOREHOLE No CPT-130** 1 OF 1 **METRIC**
W.P. _____ LOCATION N 4679821.8 :E 332036.1 ORIGINATED BY CC
DIST WEST HWY 401/3 BOREHOLE TYPE POWER AUGER, SOLID STEM COMPILED BY SJL
DATUM GEODETIC DATE September 4, 2008 CHECKED BY SJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
180.82	GROUND SURFACE							20 40 60 80 100						
0.00	FILL, crushed sand and gravel, trace silt Compact Brown		1	SS	16			○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
180.29								20 40 60 80 100						
0.61	FILL, sand with slag Compact Black		2	SS	14		180							
	CLAYEY SILT, trace sand, trace gravel Stiff		3	SS	11									
178.99	Mottled brown and grey						179							
1.83	END OF BOREHOLE													
	Borehole dry during drilling on September 4, 2008.													

PROJECT 07-1130-207-0 RECORD OF BOREHOLE No 131 1 OF 4 METRIC
W.P. LOCATION N 4679944 8 E 331856.4 ORIGINATED BY SM
DIST WEST HWY 401/3 BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC COMPILED BY BRS
DATUM GEODETIC DATE August 8, 2008 - August 13, 2008 CHECKED BY SJB



Continued Next Page

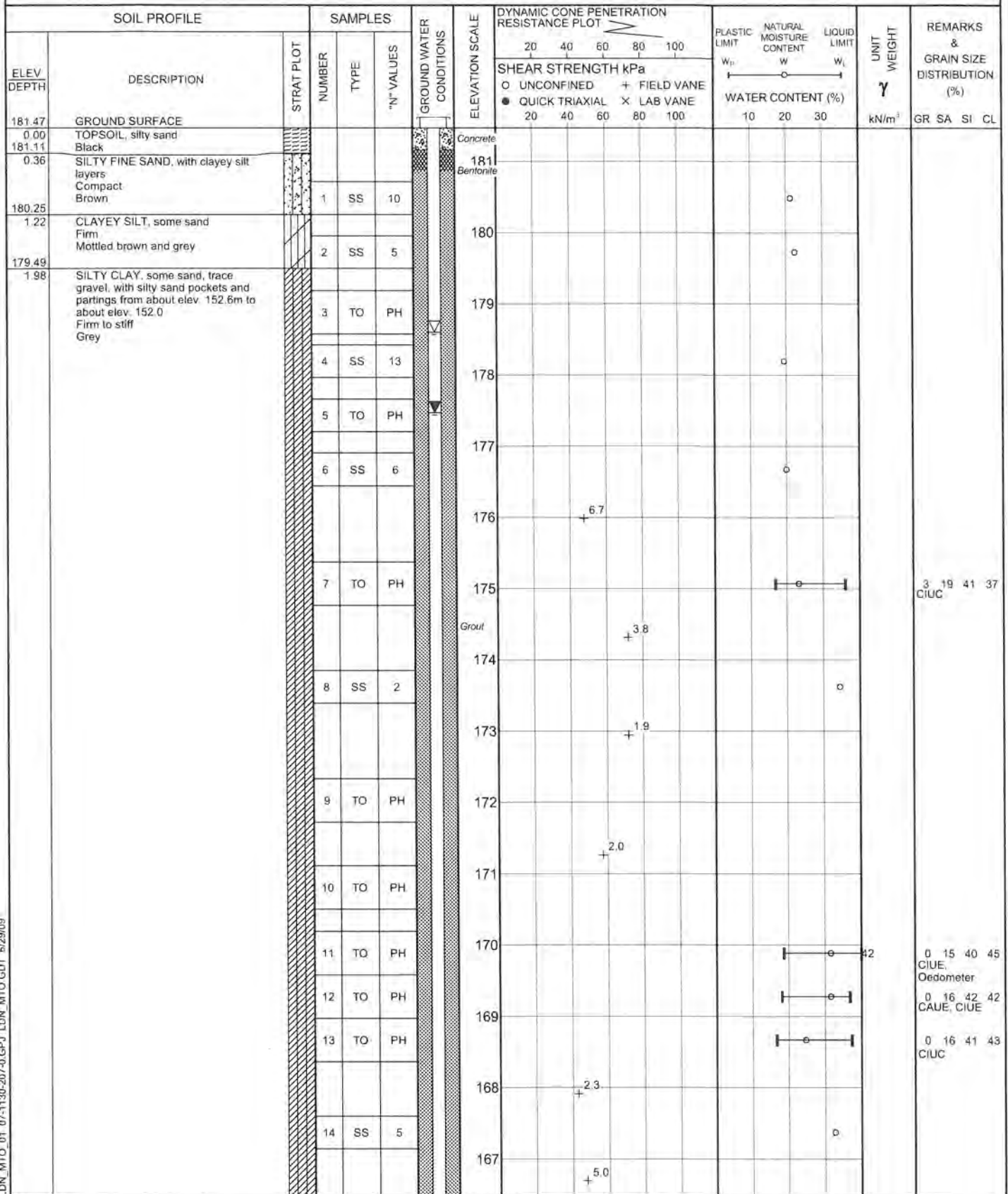
+ 3 × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>07-1130-207-0</u>		RECORD OF BOREHOLE No 131		3 OF 4	METRIC
W.P. _____		LOCATION <u>N 4679944.8 ; E 331856.4</u>		ORIGINATED BY <u>SM</u>	
DIST <u>WEST</u> HWY <u>401/3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>BRS</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 8, 2008 - August 13, 2008</u>		CHECKED BY <u>SJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						× LAB VANE		
29.95	SILTY SAND AND GRAVEL, trace clay Dense to very dense Grey																	
			24	SS	28													
			25	SS	31													
148.10	LIMESTONE, fresh, medium strong, thinly laminated to bedded, very fine to medium grained, faintly porous Brown to light grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		26	SS														
32.70			27	NQ RC	50/ 25mm													
			28	NQ RC														
			29	NQ RC														
144.68	END OF BOREHOLE																	
36.12	Borehole dry during drilling between August 8 and 12, 2008. Water level measured in deep piezometer at elev. 177.91m on September 22, 2008. Water level measured in deep piezometer at elev. 177.54m on November 11, 2008. Water level measured in deep piezometer at elev. 177.40m on January 28, 2009.																	

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/30/09

PROJECT 07-1130-207-0 RECORD OF BOREHOLE No 132 1 OF 4 METRIC
W.P. LOCATION N 4680070.8 :E 331910.3 ORIGINATED BY CC
DIST WEST HWY 401/3 BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC COMPILED BY BRS
DATUM GEODETIC DATE July 28, 2008 - July 29, 2008 CHECKED BY SJB

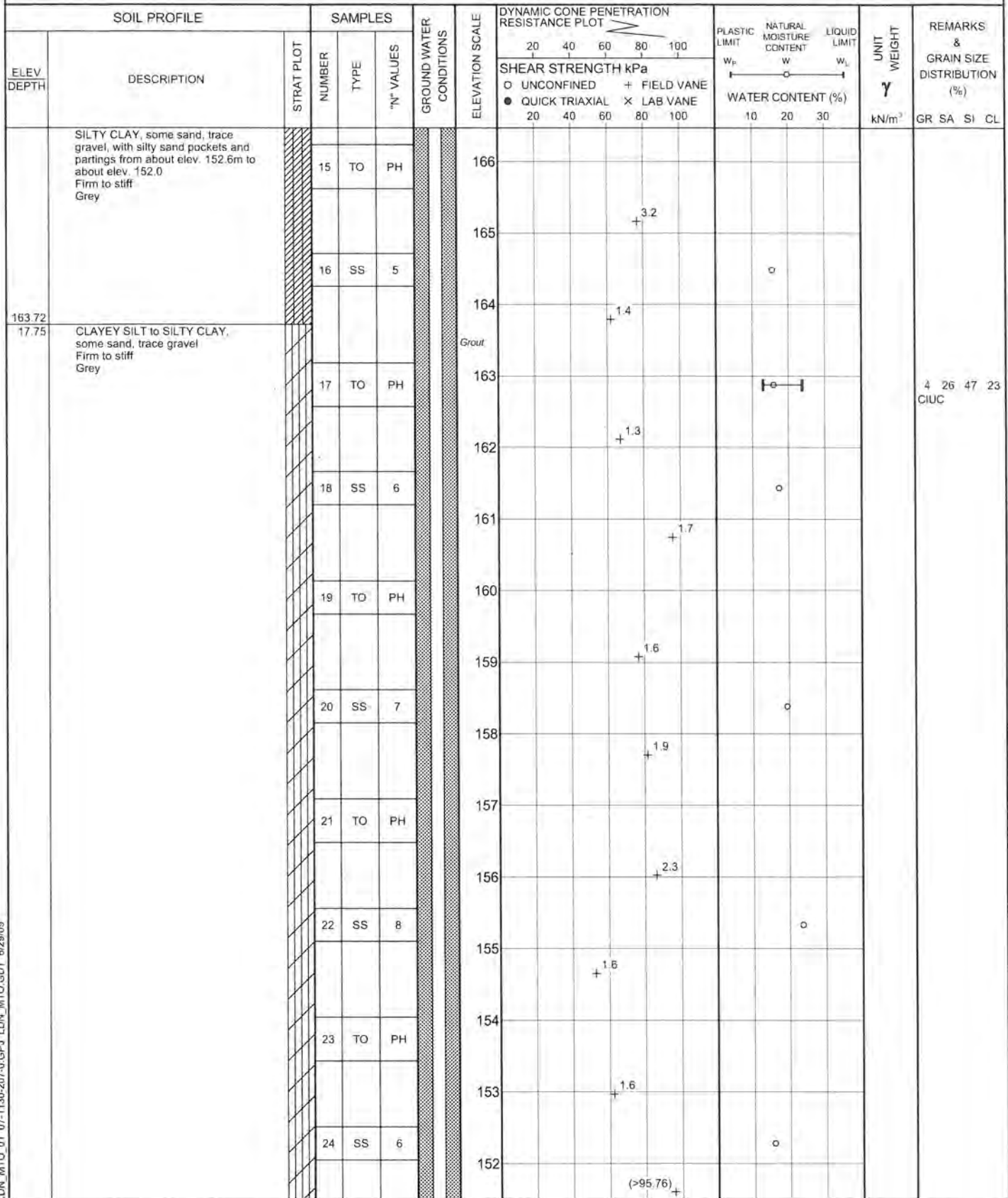


LDN MTO.01 07-1130-207-0.GPJ LDN MTO.GDT 5/29/09

Continued Next Page

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

PROJECT <u>07-1130-207-0</u>		RECORD OF BOREHOLE No 132		2 OF 4	METRIC
W.P. _____		LOCATION <u>N 4680070.8 : E 331910.3</u>		ORIGINATED BY <u>CC</u>	
DIST <u>WEST</u> HWY <u>401/3</u>		BOREHOLE TYPE <u>POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC</u>		COMPILED BY <u>BRS</u>	
DATUM <u>GEODETIC</u>		DATE <u>July 28, 2008 - July 29, 2008</u>		CHECKED BY <u>SJS</u>	



LDN MTO_01 07-1130-207-0.GPJ LDN MTO.GDT 8/29/09

Continued Next Page

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

RECORD OF BOREHOLE No 132

3 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4680070.8, E 331910.3

ORIGINATED BY CC

DIST WEST HWY 401/3

BOREHOLE TYPE

POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY BRS

DATUM GEODETIC

DATE

July 28, 2008 - July 29, 2008

CHECKED BY *SJB*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
	CLAYEY SILT to SILTY CLAY, some sand, trace gravel Firm to stiff Grey		25	TO	PH		151								
149.46															
32.01	SILTY SAND, trace gravel Very dense Grey		26	SS	78		149								(47)
148.04															
33.43	LIMESTONE, fresh, medium strong, weakly laminated to laminated, very fine to fine grained, faintly porous to porous Brown to grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		27	NQ RC			148								
			28	NQ RC			147								
			29	NQ RC			146								
143.67							145								
37.80	END OF BOREHOLE Water level in borehole at about elev. 178.6m during drilling on July 28, 2008. Water level measured in deep piezometer at elev. 177.97m 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. 177.48m on January 28, 2009.						144								

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 132

SHEET 4 OF 4

LOCATION: N 4680070.8 :E 331910.3

DRILLING DATE: July 28, 2008 - July 29, 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 % 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										NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
				DEPTH (m)					RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec	DIPWELL CORE AXIS	TYPE AND SURFACE DESCRIPTION	DIAMETRAL POINT LOAD INDEX (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
									TOTAL CORE %	SOLID CORE %																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
									0 10 20 30 40 50	0 10 20 30 40 50			0 10 20 30 40 50	0 10 20 30 40 50																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
		ROCK SURFACE		148.04																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	

DEPTH SCALE

1 : 75



LOGGED: SG

CHECKED: SJB

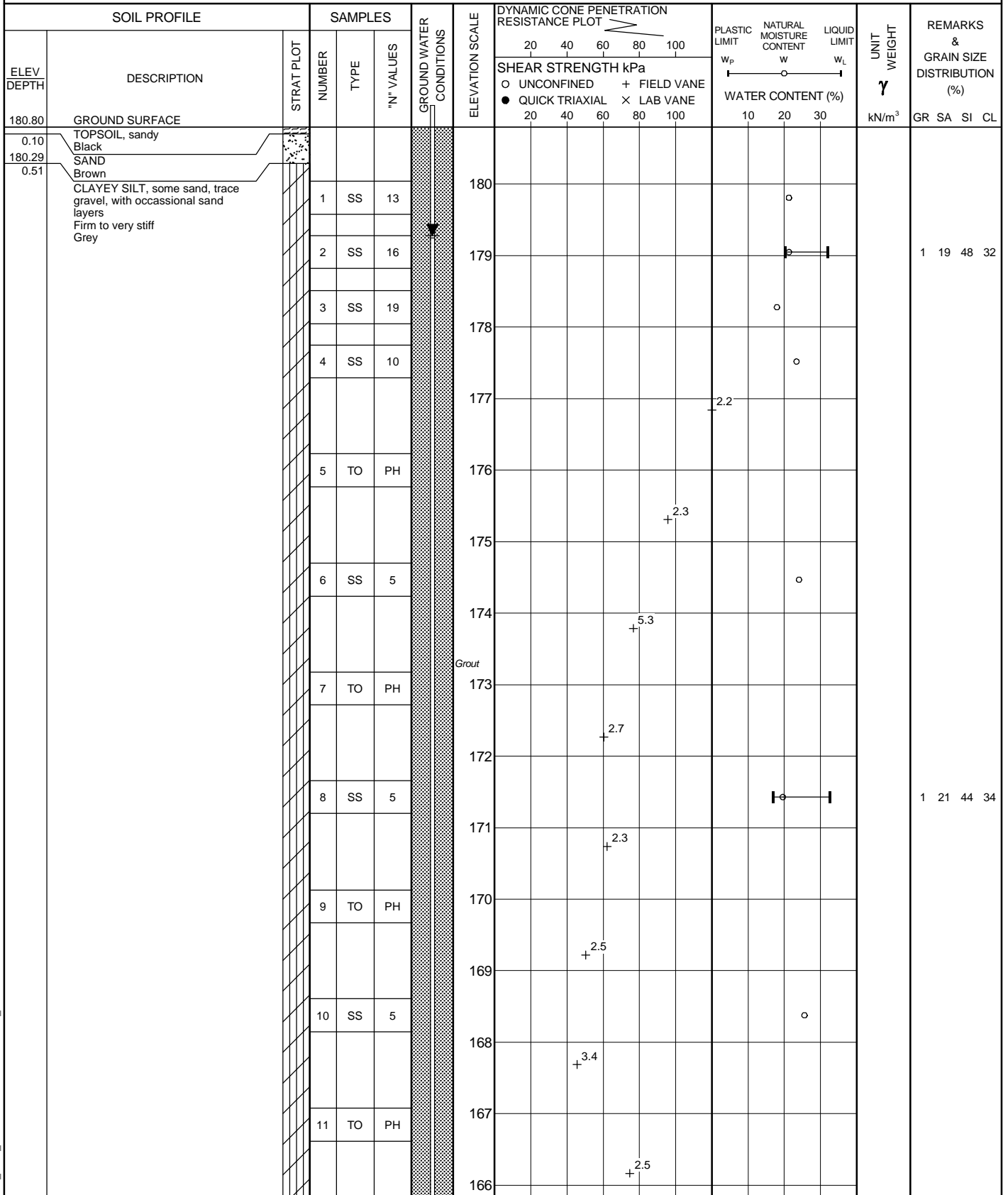
PROJECT <u>07-1130-207-0</u>		RECORD OF BOREHOLE No 132A		1 OF 1	METRIC
W.P. _____		LOCATION <u>N 4680070.8 ; E 331910.3</u>		ORIGINATED BY <u>CC</u>	
DIST <u>WEST</u> HWY <u>401/3</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM</u>		COMPILED BY <u>BRS</u>	
DATUM <u>GEODETIC</u>		DATE <u>July 29, 2008</u>		CHECKED BY <u>SJS</u>	

[illegible]

DN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

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

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No 325		1 OF 4		METRIC	
W.P. _____		LOCATION <u>N 4679787.7 ; E 331972.9</u>		ORIGINATED BY <u>SM</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 16, 2009 - December 17, 2009</u>		CHECKED BY _____			

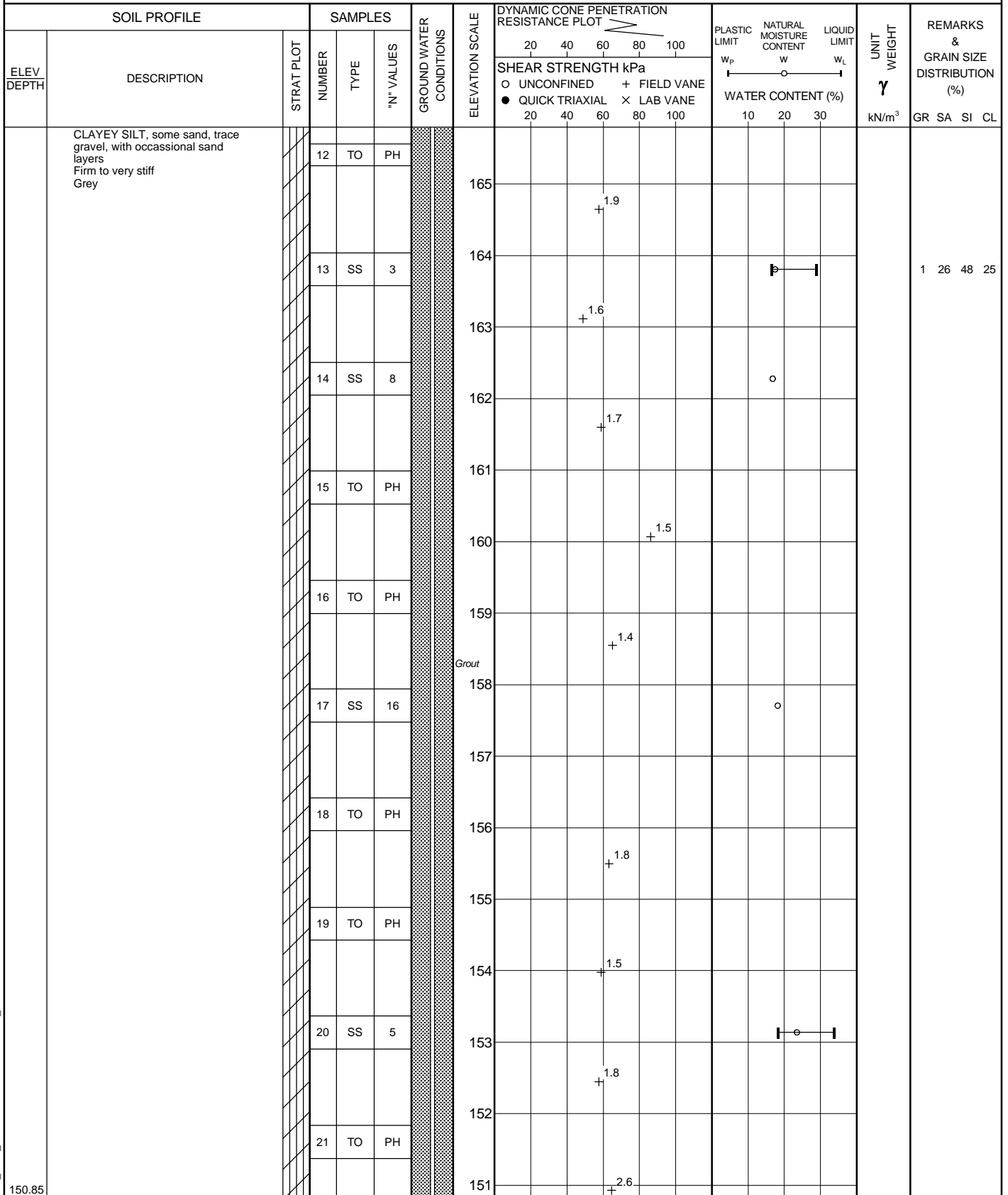


LDN_MTO_06 09-1132-0080.GPJ LDN_MTO.GDT 11/03/10

Continued Next Page

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

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No 325		2 OF 4	METRIC
W.P. _____		LOCATION <u>N 4679787.7 ; E 331972.9</u>		ORIGINATED BY <u>SM</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 16, 2009 - December 17, 2009</u>		CHECKED BY _____	



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

Continued Next Page

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

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No 325		3 OF 4		METRIC	
W.P. _____		LOCATION <u>N 4679787.7 ; E 331972.9</u>		ORIGINATED BY <u>SM</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 16, 2009 - December 17, 2009</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
29.95	SANDY SILT, some clay, trace to some gravel Compact to very dense Grey		22	SS	17												10 40 38 12
148.48	LIMESTONE, fresh, medium strong, weakly laminated to laminated, very fine to fine grained, faintly porous Light grey to brown (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		23	SS	82/175mm												
32.32			24	NQ RC	-		88	78	78								
			25	NQ RC	-		97	95	94								
			26	NQ RC	-		100	98	86								
			27	NQ RC	-		100	95	88								
143.31	END OF BOREHOLE Borehole dry during drilling between December 14 and 17, 2009. Water level measured at elev. 179.35 on February 24, 2010. Water level measured at elev. 179.28 on January 6, 2010.																
37.49																	

PROJECT: 09-1132-0080

RECORD OF DRILLHOLE: 325

SHEET 4 OF 4

LOCATION: N 4679787.7 ;E 331972.9

DRILLING DATE: December 16, 2009 - December 17, 2009

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (m/min)	COLOUR FLUSH	ELEVATION															NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
				DEPTH					RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
				(m)					TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION		10 ⁶	10 ⁴	10 ²	2	4	6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
									80 60 40 20	80 60 40 20	80 60 40 20	5 10 15 20	0 30 60 90	10 ⁶	10 ⁴	10 ²	2	4	6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				

DEPTH SCALE

1 : 75



LOGGED: SG

CHECKED:

LDN_ROCK_03 09-1132-0080-ROCK.GPJ GLDR LDN.GDT 11/03/10 DATA INPUT: LMK

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No 326		1 OF 4		METRIC	
W.P. _____		LOCATION <u>N 4679917.6 ; E 331984.5</u>		ORIGINATED BY <u>DB</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>November 25, 2009 - November 30, 2009</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT <div style="display: flex; justify-content: space-around; font-size: small;"> 20 40 60 80 100 </div>	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
181.78	GROUND SURFACE												
0.00	TOPSOIL, silty Black						Filter sand						
181.35													
0.43	SAND, fine to medium, some silt Loose Brown		1	SS	7								0 86 11 3
180.56													
1.22	SILTY CLAY, some sand, trace gravel Very stiff Brown		2	SS	22		Bentonite				42		
179.65													
2.13	CLAYEY SILT, some sand, trace gravel Firm to very stiff Grey		3	SS	18								
			4	SS	18								1 17 42 40
			5	SS	21								
			6	SS	12								
			7	SS	7								4 20 42 34
			8	TO	PH								
			9	SS	5		Grout	1.6					
			10	TO	PH			2.2					
			11	SS	6			1.4					
			12	TO	PH			1.9					
			13	SS	9			1.3					1 21 41 37
			14	TO	PH			1.6					

Continued Next Page

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

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No 326		2 OF 4	METRIC
W.P. _____		LOCATION <u>N 4679917.6 ;E 331984.5</u>		ORIGINATED BY <u>DB</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>November 25, 2009 - November 30, 2009</u>		CHECKED BY _____	

SOIL PROFILE					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT
					<div><div>20406080100</div><div>SHEAR STRENGTH kPa</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div><div>20406080100</div></div> <div><div>PLASTIC LIMITNATURAL MOISTURE CONTENTLIQUID LIMIT</div><div>W_PW W_L</div><div>WATER CONTENT (%)</div><div>102030</div></div> <div><div>UNIT WEIGHT</div><div>γ</div><div>kN/m³</div></div> <div><div>REMARKS & GRAIN SIZE DISTRIBUTION (%)</div><div>GR SA SI CL</div></div>
	CLAYEY SILT, some sand, trace gravel Firm to very stiff Grey				
			15SS9		166+1.5
					165○
					164+1.3
			16TO PH		163
					162
					161+1.3
					160○
			17SS14		159
					158+1.1
					157(>143.6)
			18TO PH		156
					155
			19SS19		
153.96 27.82	SILTY CLAY, some sand, trace gravel Stiff Grey				154○
			20SS10		153
152.44 29.34					152

Continued Next Page

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

PROJECT 09-1132-0080		RECORD OF BOREHOLE No 326		3 OF 4	METRIC
W.P. _____		LOCATION N 4679917.6 ; E 331984.5		ORIGINATED BY DB	
DIST WEST HWY 401 / 3		BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY LMK/DMB	
DATUM GEODETIC		DATE November 25, 2009 - November 30, 2009		CHECKED BY _____	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE							
	CLAYEY SILT, some sand, trace clay Very stiff Grey		21	SS	7													
150.16																		
31.62	SANDY SILT, some clay, trace gravel Very dense Grey		22	SS	55													
149.16																		
32.62	LIMESTONE, fresh, medium strong, weakly laminated, very fine to fine grained, faintly porous Light grey to brown (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		23	SS	100													
			24	NQ RC	-													
			25	NQ RC	-													
			26	NQ RC	-													
			27	NQ RC	-													
			28	NQ RC	-													
144.10	END OF BOREHOLE																	
37.68	Borehole dry during drilling between November 25 and 30, 2009. Water level measured at elev. 179.55 on February 24, 2010. Water level measured at elev. 179.52 on January 6, 2010.																	

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DATUM: GEODETIC



**Golder
Associates**






CHECKED:

_LDN_ROCK_03 09-1132-0080-ROCK.GPJ GLDR_LDN.GDT 12/03/10 DATA INPUT: LMK

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No CPT-328		1 OF 1	METRIC
W.P. _____		LOCATION <u>N 4680024.3 ; E 331862.9</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>December 18, 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 γ kN/m³	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 x LAB VANE					w _p w w _L							
181.64	GROUND SURFACE							20	40	60	80	100								
0.00	FILL, clayey silt, some sand, trace gravel, trace topsoil, trace rootlets Brown						▽													
181.03								181												
0.61	FILL, silty sand, some clay, trace topsoil Brown		1	SS	7															
0.76								180												
180.43	TOPSOIL, sandy Firm Black																			
1.21	CLAYEY SILT, some sand, trace gravel, with occasional sand pockets and silt partings Firm to very stiff Brown		2	SS	5															
178.74			3	SS	18		179													
2.90	END OF BOREHOLE																			
	Groundwater encountered at about elev. 179.4m during drilling on December 18, 2009.																			

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No CPT-329		1 OF 1		METRIC	
W.P. _____		LOCATION <u>N 4680100.8 ; E 331832.3</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>December 18, 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					w _p	w	w _L						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE													
181.98	GROUND SURFACE							20	40	60	80	100									
0.00	FILL, clayey silt, some sand, trace gravel, trace topsoil, trace rootlets Stiff Brown						181														
181.07																					
0.99	FILL, silty sand, some topsoil Compact Dark brown		1	SS	14																
180.61							180														
1.37	TOPSOIL, sandy Stiff Black		2	SS	6																
179.85																					
2.13	SILTY FINE SAND Loose Brown																				
179.08	CLAYEY SILT, some sand, trace gravel, with occasional silt partings Stiff to very stiff Grey		3	SS	15																
2.90	END OF BOREHOLE Borehole dry during drilling on December 18, 2009.																				

NILCON FIELD VANE SHEAR TEST RESULTS**Windsor-Essex Parkway**

Depth (m)	Elevation (m)	Undrained Shear Strength (kPa)			Sensitivity
		Natural	Post-Peak	Remoulded	
12.7	168.1	49	26	21	2.4
13.7	167.1	38	15	11	3.3
14.7	166.1	42	34	23	1.8
15.7	165.1	55	40	30	1.8
16.7	164.1	40	28	21	1.9
17.7	163.1	76	59	55	1.4
18.7	162.1	74	60	66	1.1

Field Vane Location 132 (Borehole BH-132)

4.8	176.7	117	79	60	1.9
5.8	175.7	81	51	49	1.7
6.8	174.7	59	15	13	4.4
7.8	173.7	62	42	26	2.4
8.8	172.7	57	25	19	3.0
9.8	171.7	55	25	25	2.2
10.8	170.7	38	28	17	2.2
11.8	169.7	43	25	21	2.1
12.8	168.7	42	26	23	1.8
13.8	167.7	47	38	25	1.9
14.8	166.7	42	19	15	2.8
15.8	165.7	68	25	21	3.3
16.8	164.7	45	26	26	1.7
17.8	163.7	45	38	45	1.0
18.8	162.7	49	32		
19.8	161.7	55	38	43	1.3
20.8	160.7	70	34	62	1.1

Field Vane Location 135 (Borehole BH-135)

4.7	177.3	125	91	59	2.1
5.7	176.3	68	47	28	2.4
6.7	175.3	68	45	30	2.3
7.7	174.3	59	34	25	2.4
8.7	173.3	55	30	15	3.6
9.7	172.3	55	36	21	2.6
10.7	171.3	38	21	19	2.0
11.7	170.3	47	30	32	1.5
12.7	169.3	59	36	30	1.9
13.7	168.3	42	21	25	1.7
14.7	167.3	45	36	8	6.0
15.7	166.3	36	9	15	2.4
16.7	165.3	55	19	15	3.6
17.7	164.3	40	8	19	2.1
18.7	163.3	30	19	19	1.6
19.7	162.3	36	25	30	1.2

Field Vane Location 145 (Borehole BH-145)

4.6	177.7	146	104	100	1.5
5.6	176.7	130	104	91	1.4
6.6	175.7	108	76	79	1.4

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-12

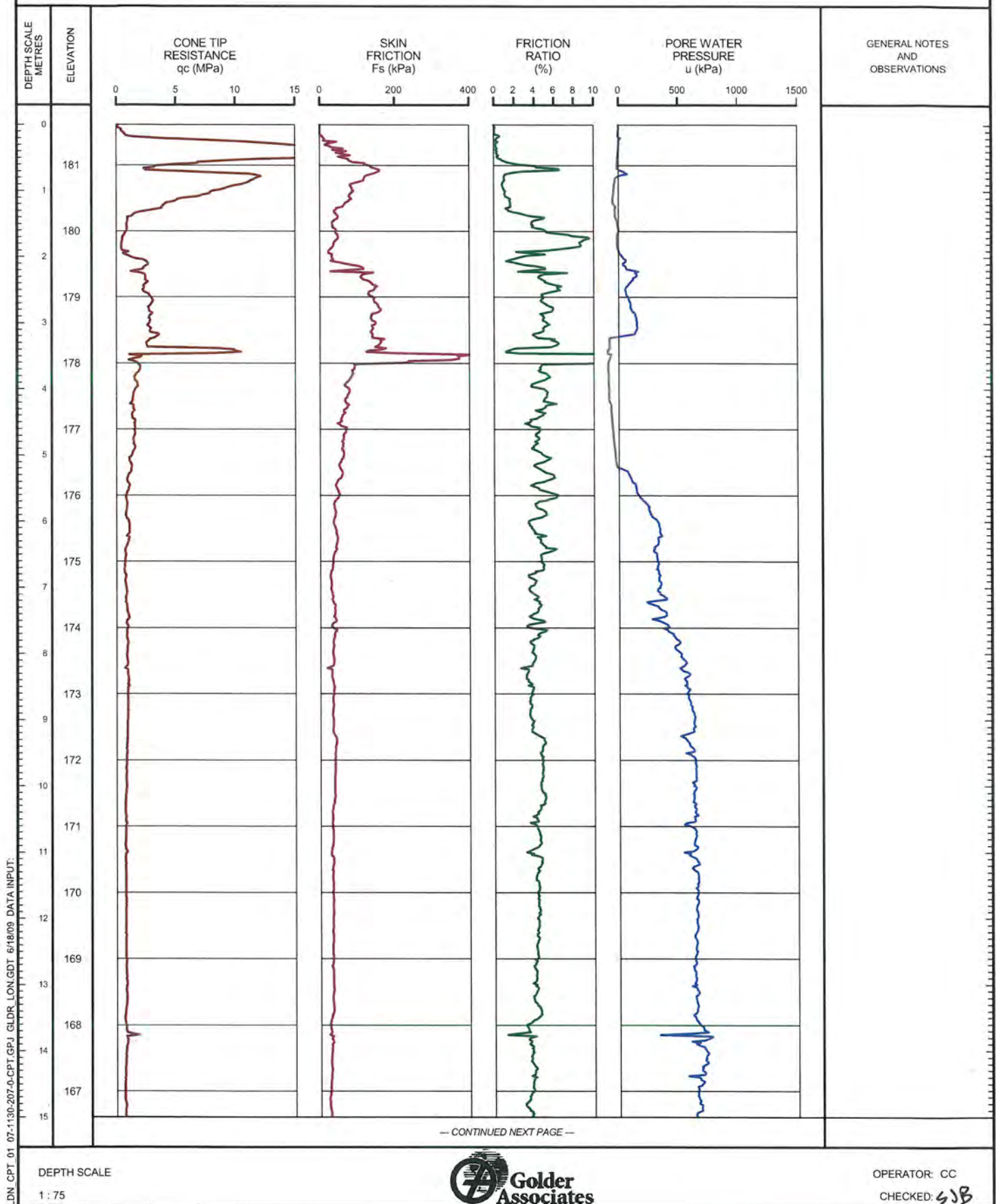
SHEET 1 OF 2

LOCATION: N 4680072.0 ; E 331924.0

TEST DATE: November 10, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



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

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-12

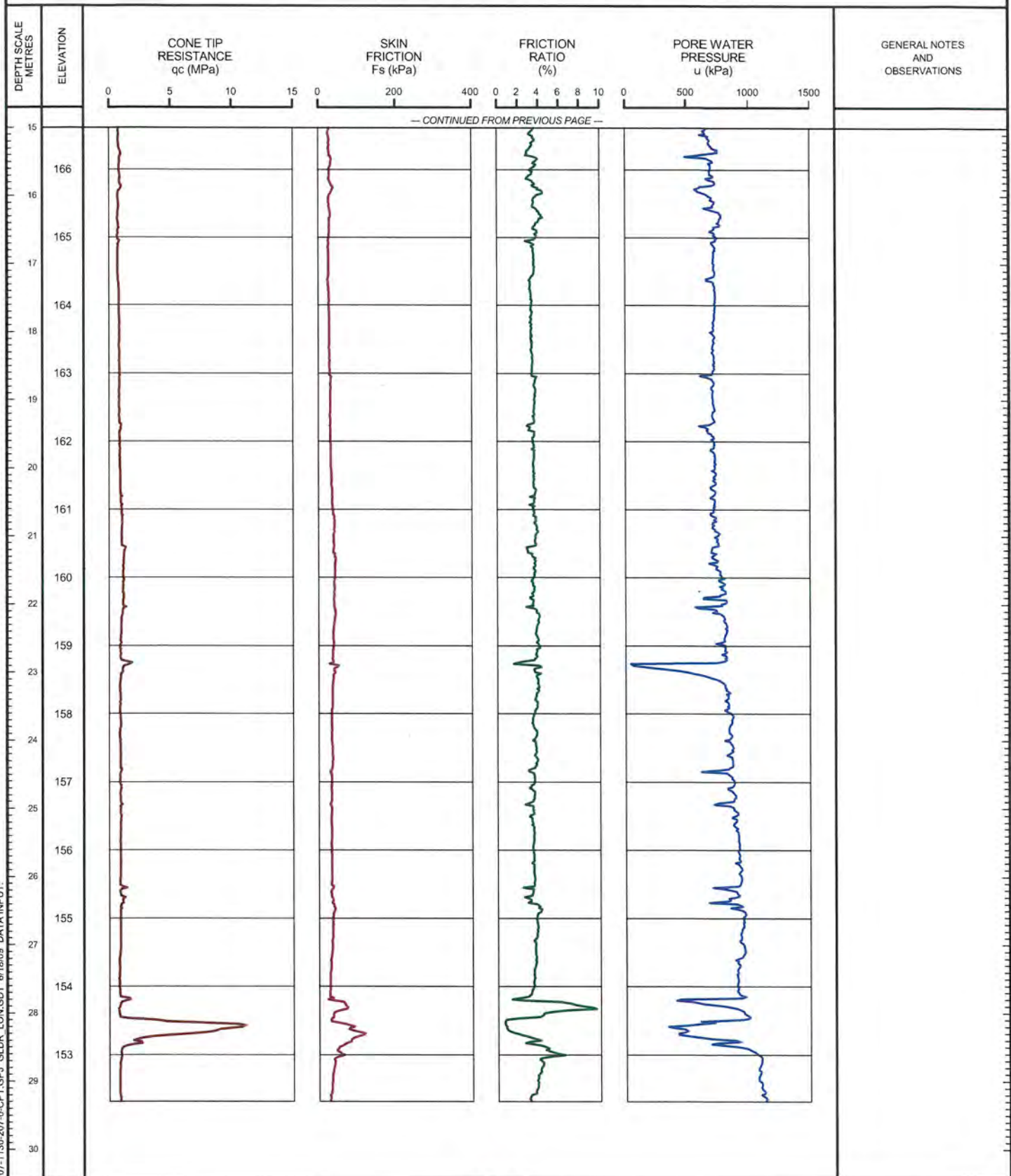
SHEET 2 OF 2

LOCATION: N 4680072.0 ;E 331924.0

TEST DATE: November 10, 2006

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



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

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: *Sub*

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-130

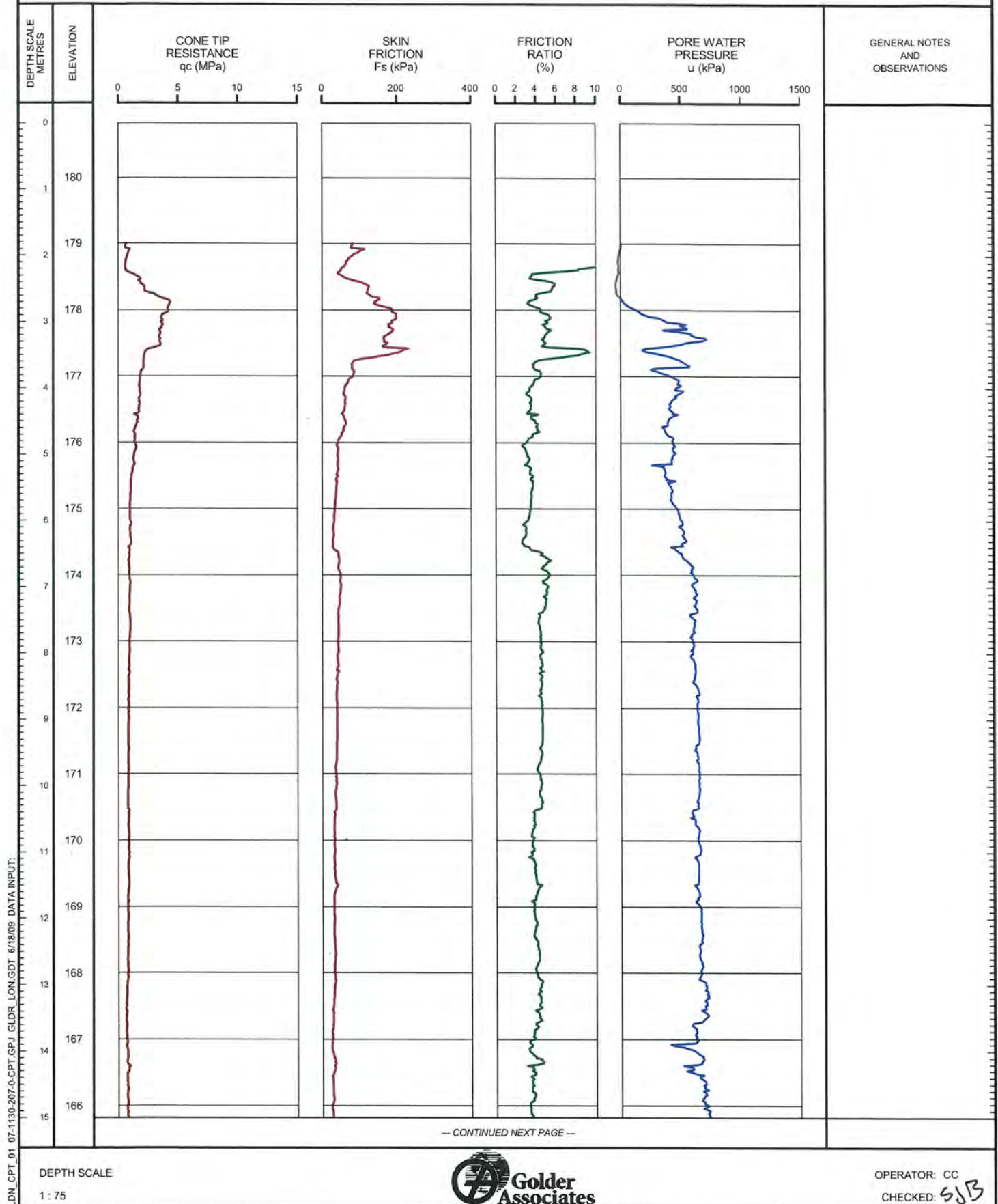
SHEET 1 OF 3

LOCATION: N 4679821.8 :E 332036.1

TEST DATE: September 4, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 1.83m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-130

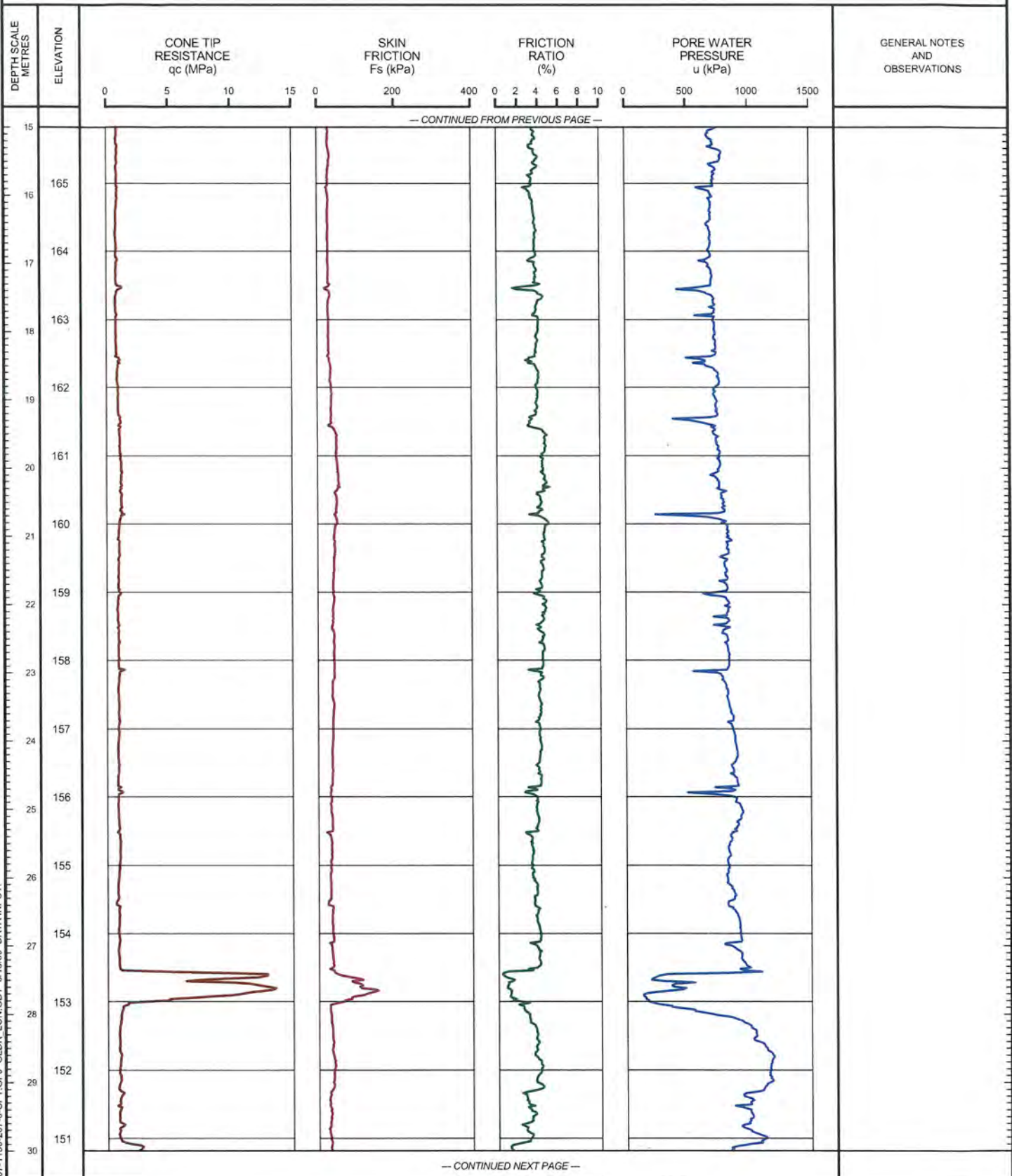
SHEET 2 OF 3

LOCATION: N 4679821.8 ; E 332036.1

TEST DATE: September 4, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 1.83m 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: SJB

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-130

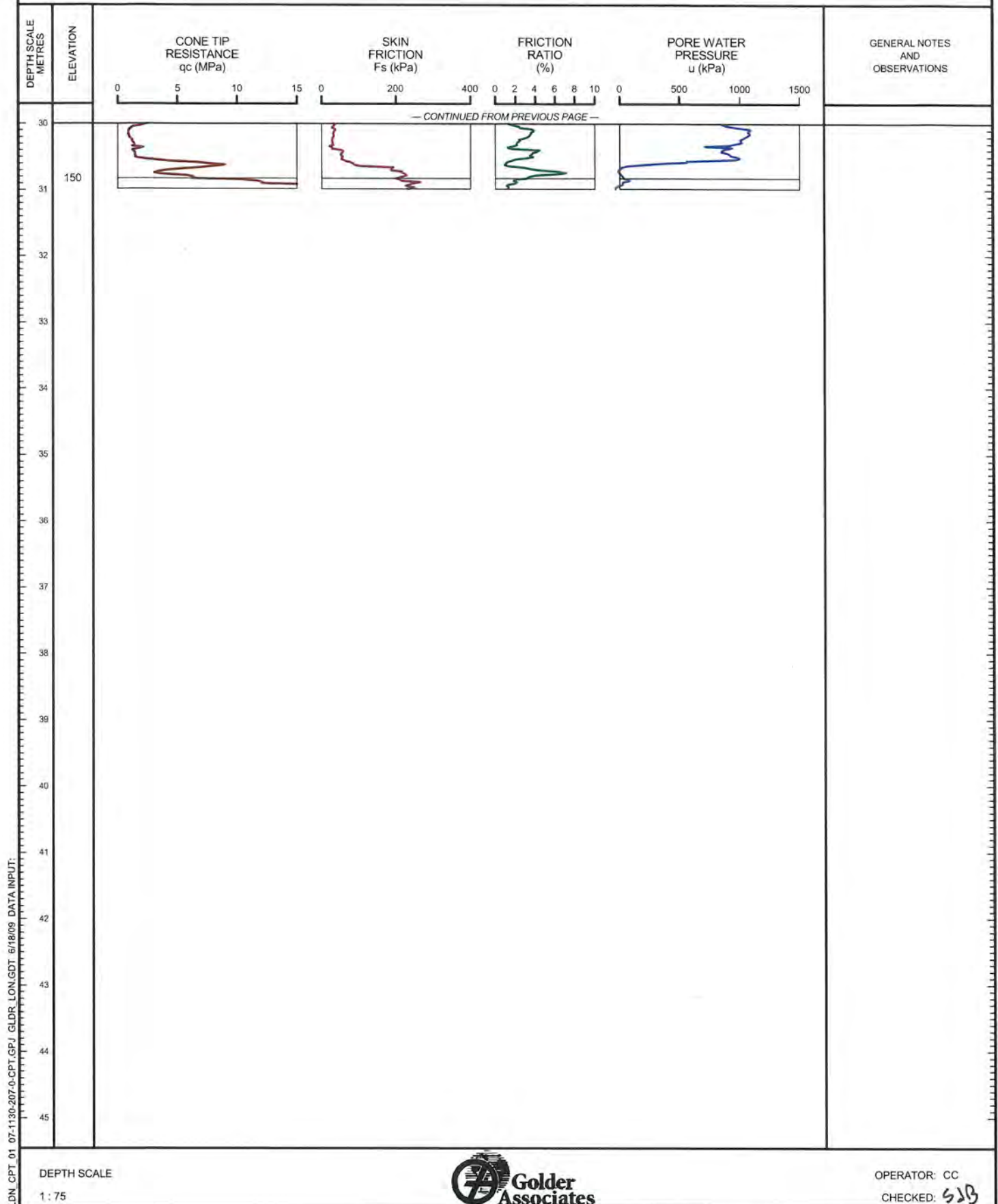
SHEET 3 OF 3

LOCATION: N 4679821.8 :E 332036.1

TEST DATE: September 4, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 1.83m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-328

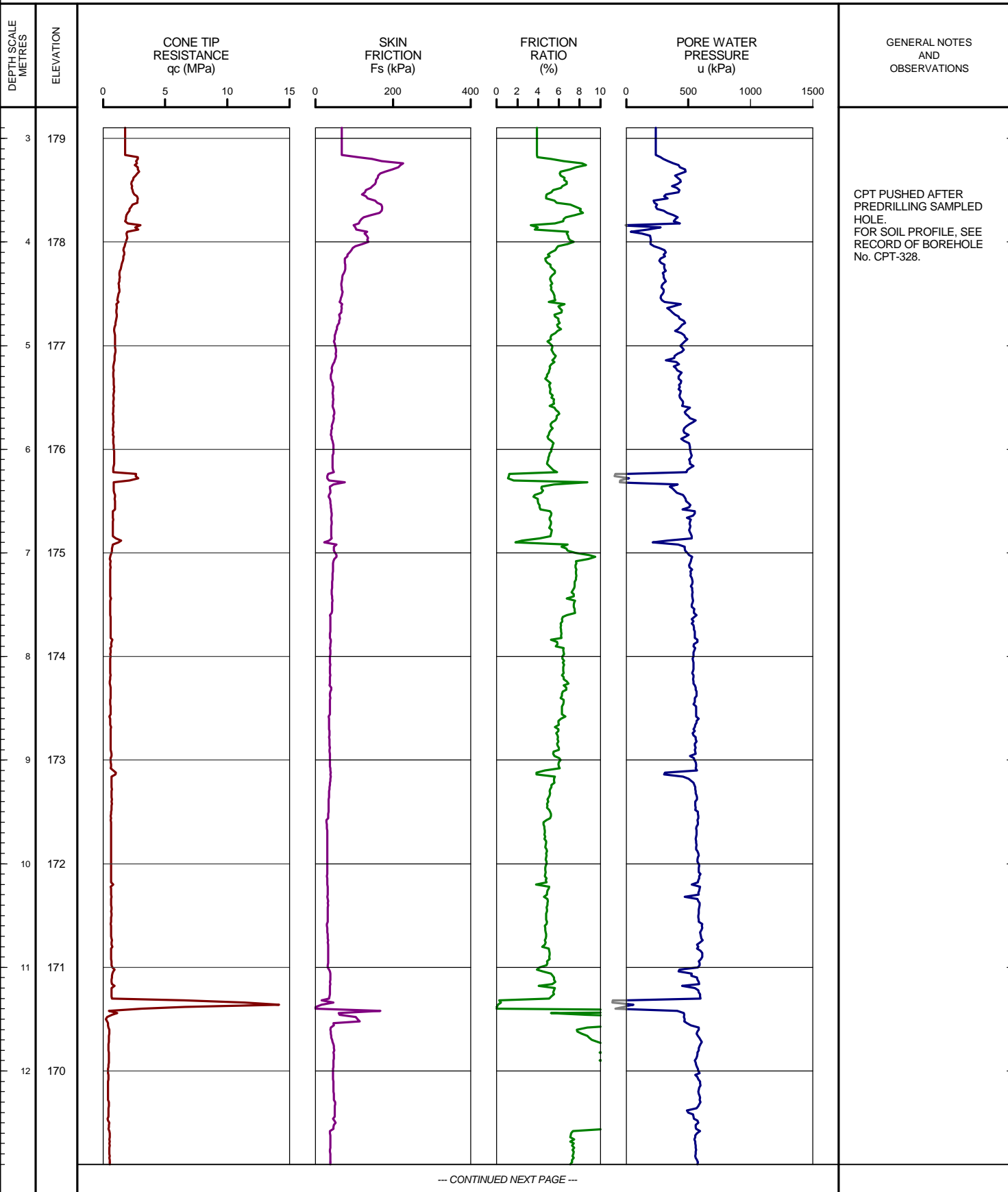
SHEET 1 OF 3

LOCATION: N 4680024.3 ;E 331862.9

TEST DATE: January 26, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.64m PREDRILL DEPTH: 2.90m 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:

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-328

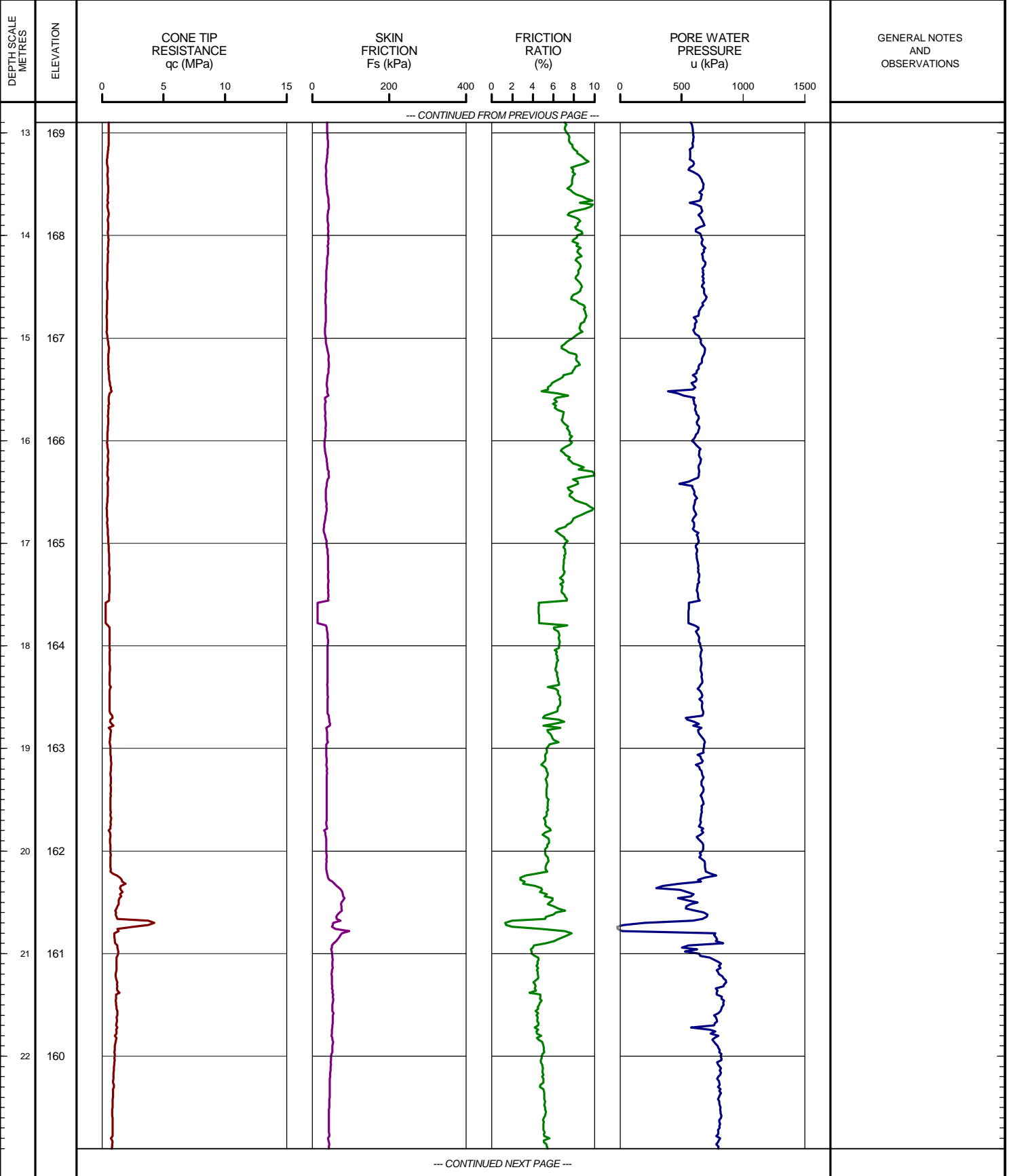
SHEET 2 OF 3

LOCATION: N 4680024.3 ;E 331862.9

TEST DATE: January 26, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.64m PREDRILL DEPTH: 2.90m 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:

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-328

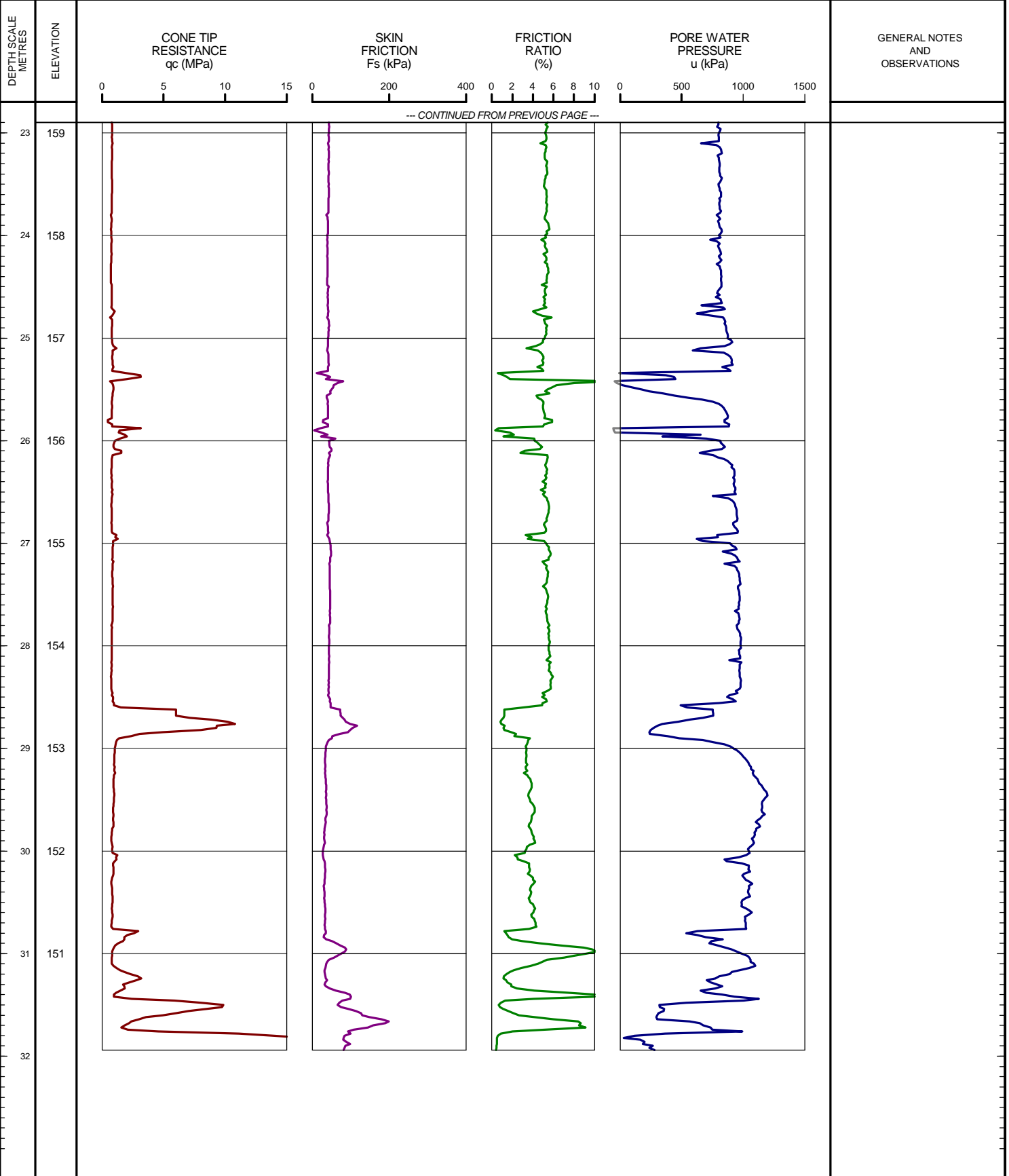
SHEET 3 OF 3

LOCATION: N 4680024.3 ;E 331862.9

TEST DATE: January 26, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.64m PREDRILL DEPTH: 2.90m 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-329

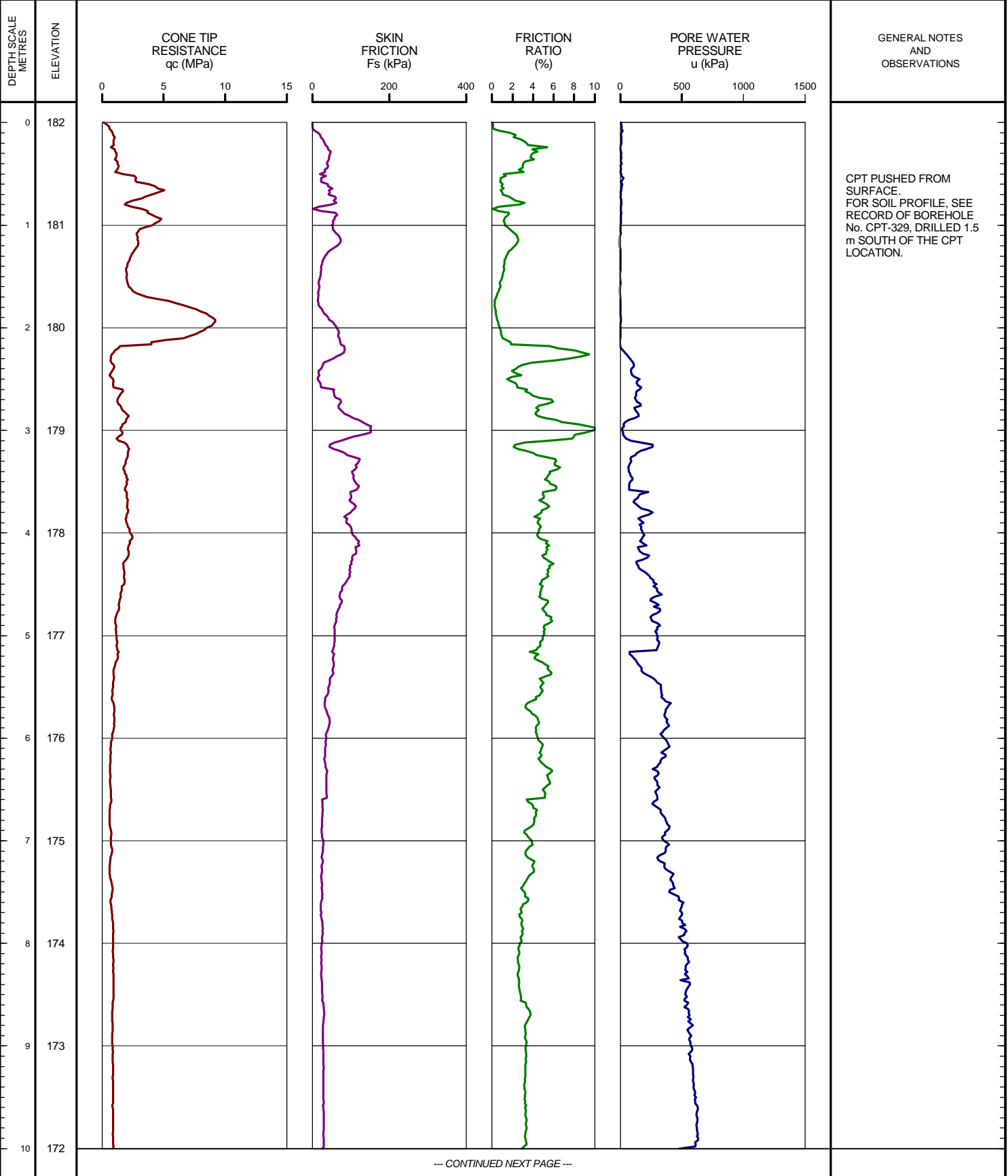
SHEET 1 OF 4

LOCATION: N 4680100.8 ;E 331832.3

TEST DATE: December 21, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.98m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



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-329

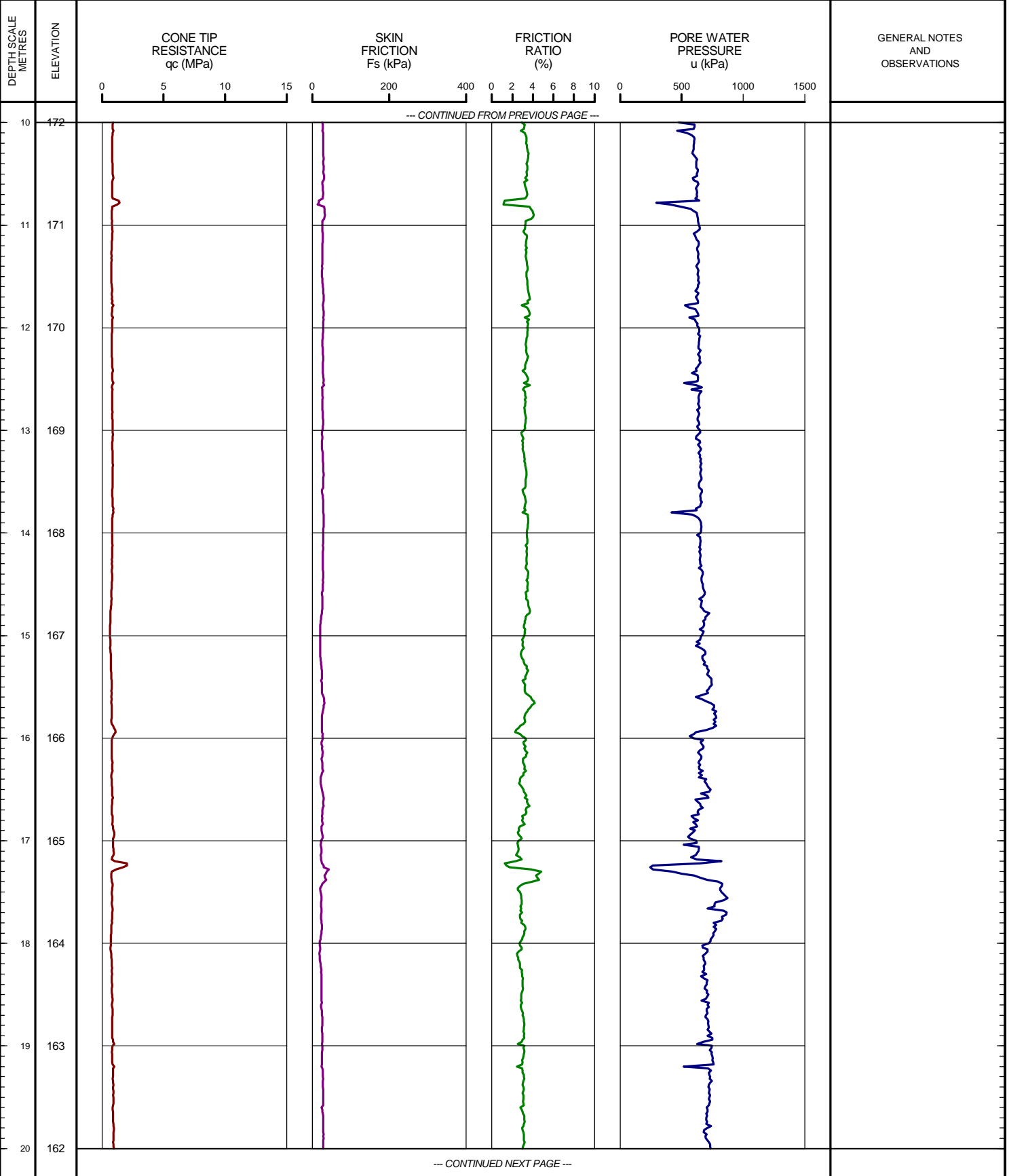
SHEET 2 OF 4

LOCATION: N 4680100.8 ;E 331832.3

TEST DATE: December 21, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.98m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LON_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-329

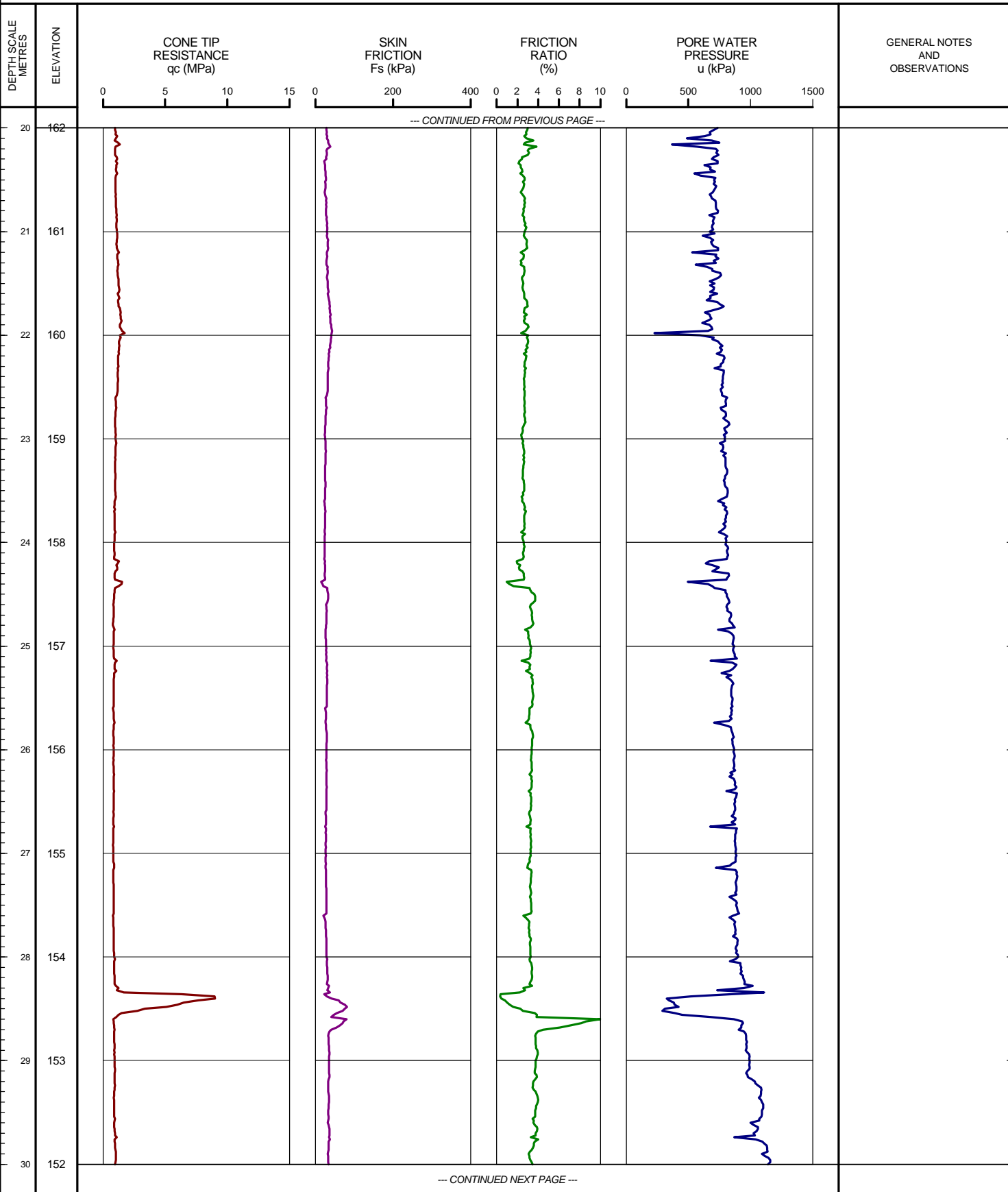
SHEET 3 OF 4

LOCATION: N 4680100.8 ;E 331832.3

TEST DATE: December 21, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.98m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



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-329

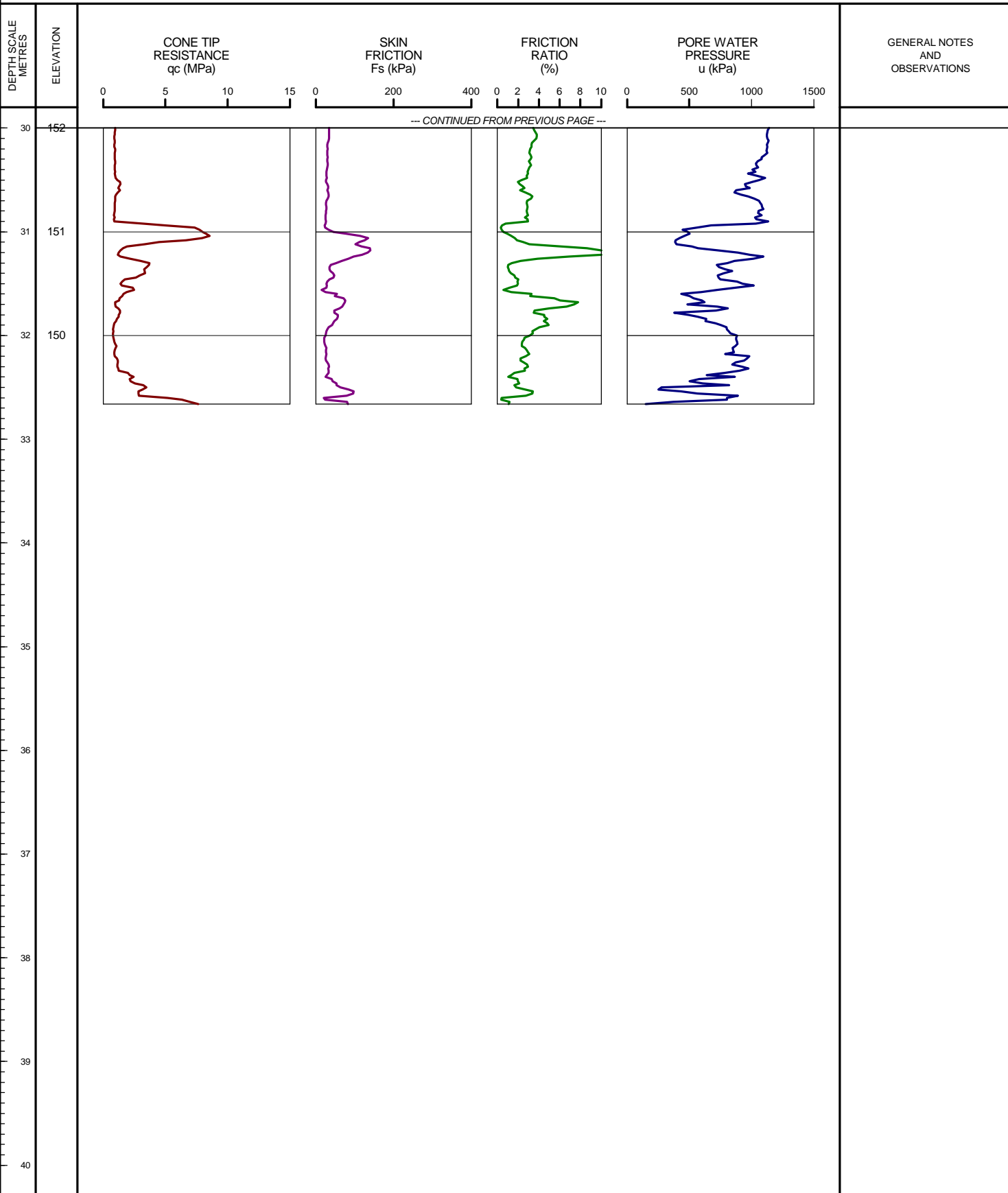
SHEET 4 OF 4

LOCATION: N 4680100.8 ;E 331832.3

TEST DATE: December 21, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.98m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LON_CPT_01 09-1132-0080-CPT.GPJ GLDR_LON.GDT 02/23/10 DATA INPUT:

DEPTH SCALE

1 : 50



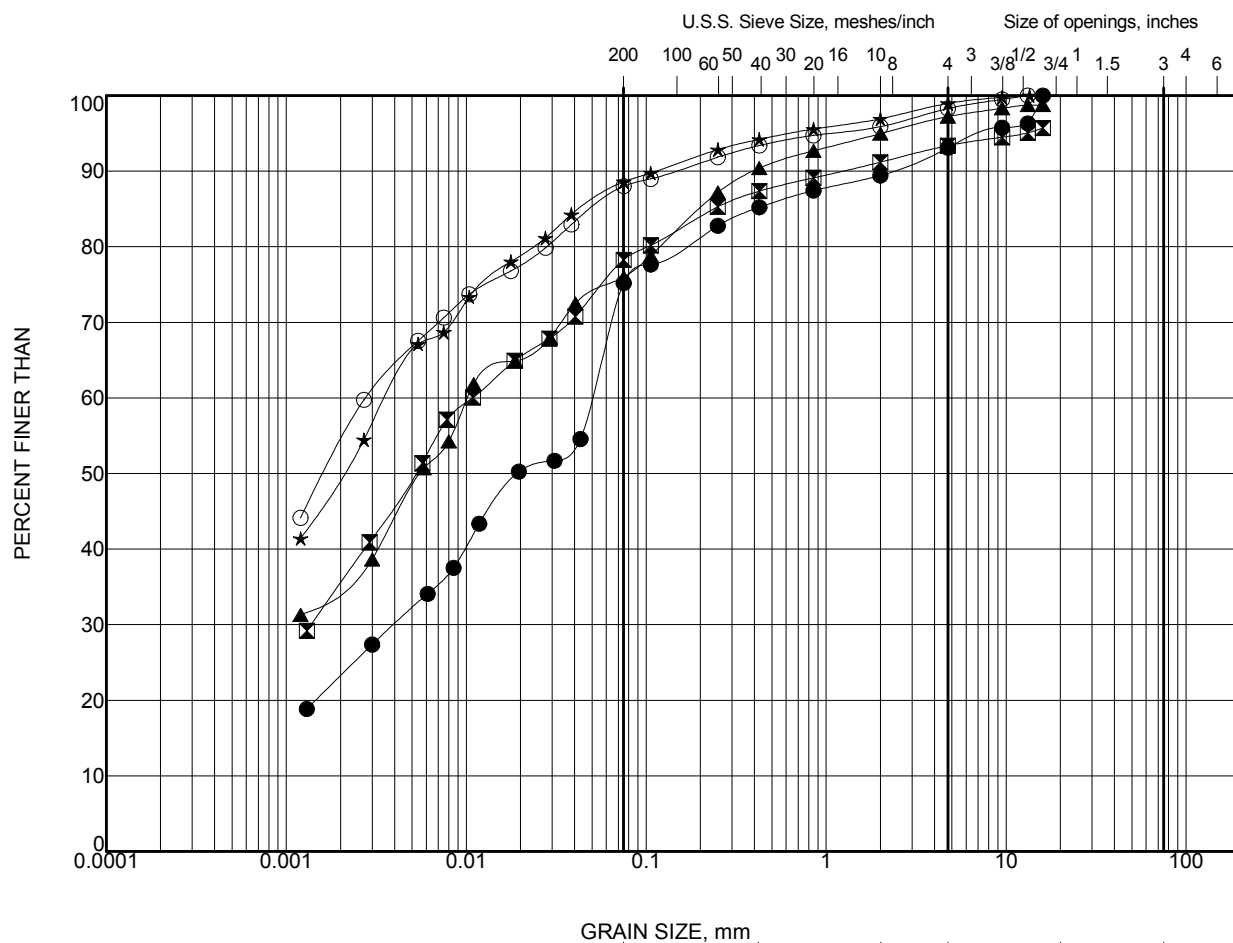
OPERATOR: TA

CHECKED:

Appendix C: Geotechnical Laboratory Test Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

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

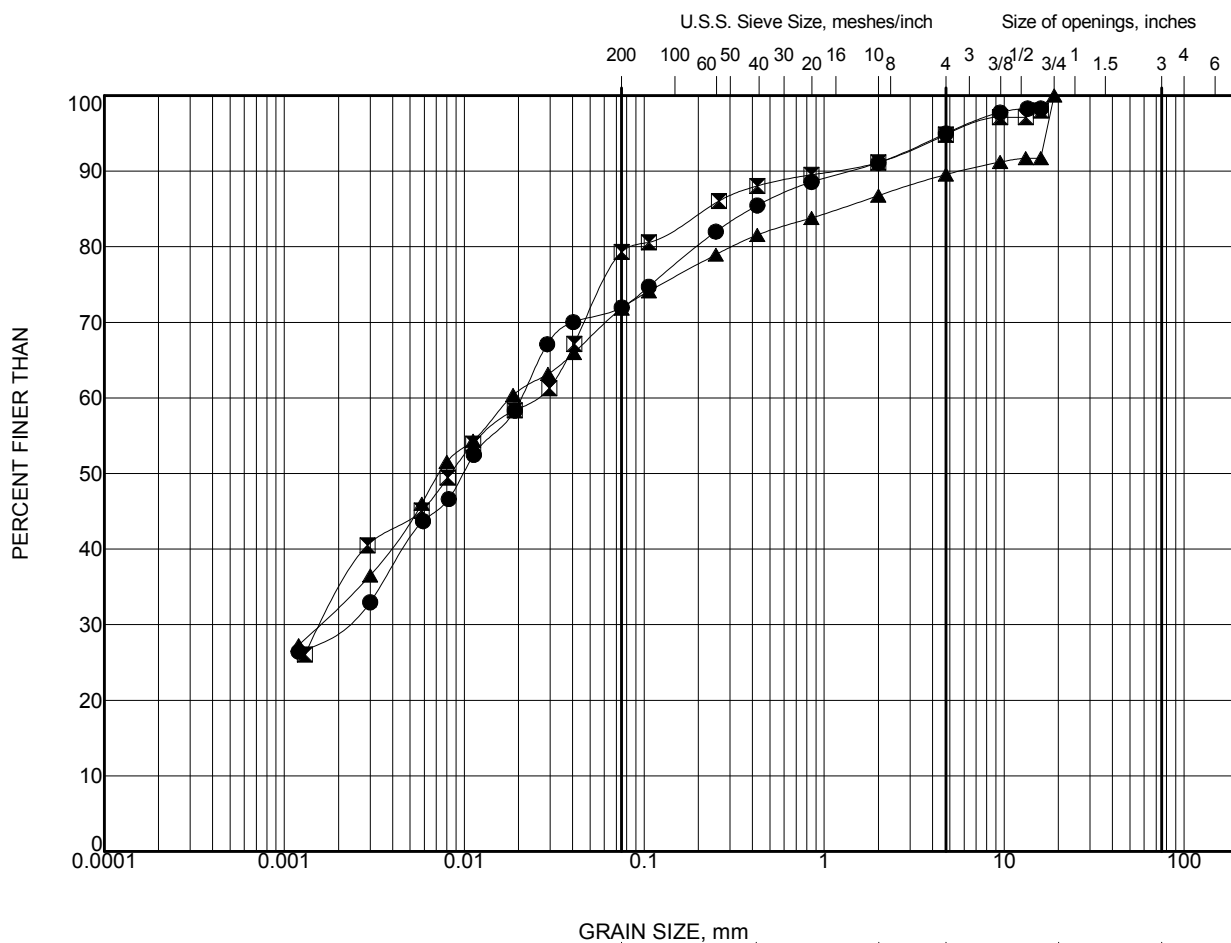


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T5-2	16	16.8
■	T5-2	20	22.9
▲	T5-3	9	7.6
★	T5-3	13	13.7
○	T5-3	16	16.8




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

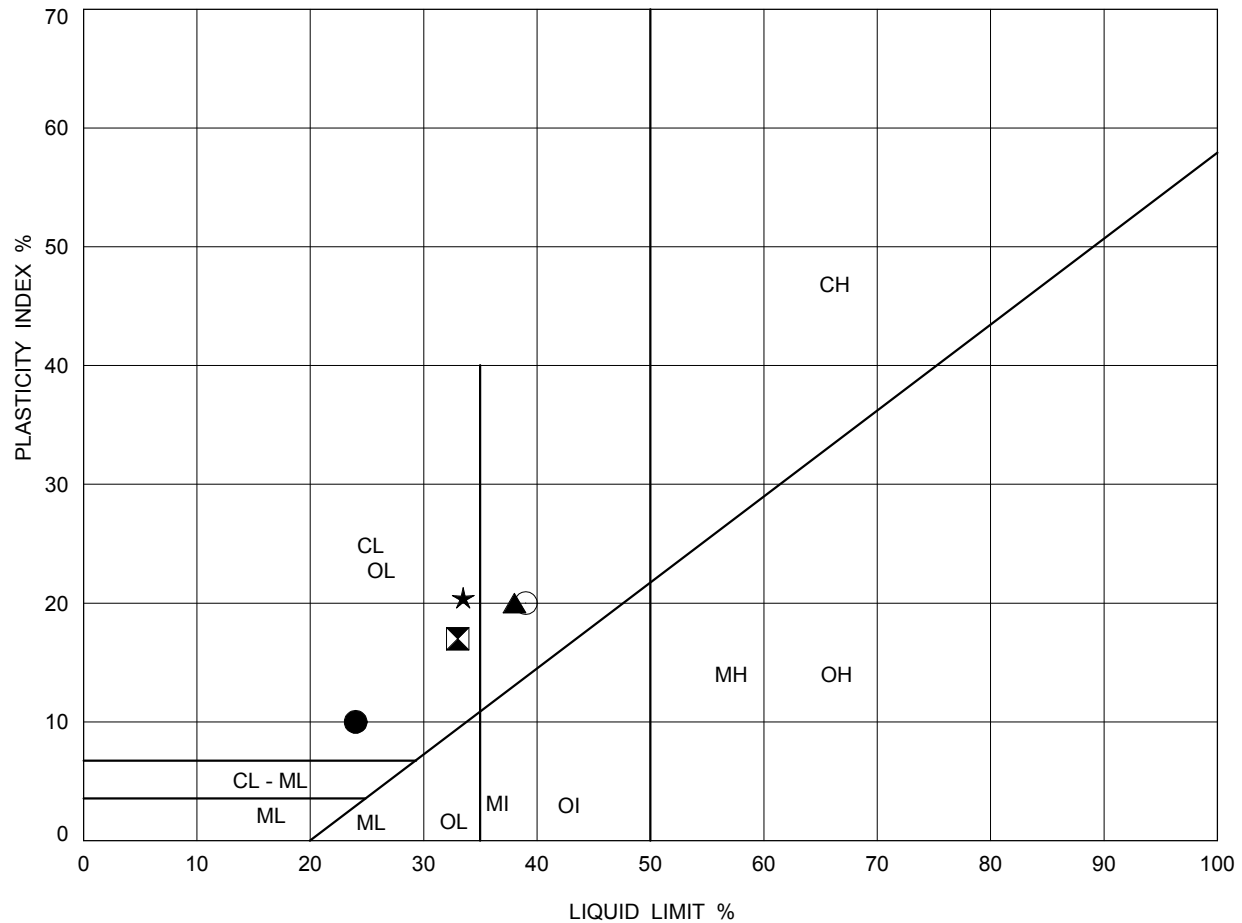


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	T5-3	20	19.8
■	T5-3	23	22.9
▲	T5-3	26	27.4

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






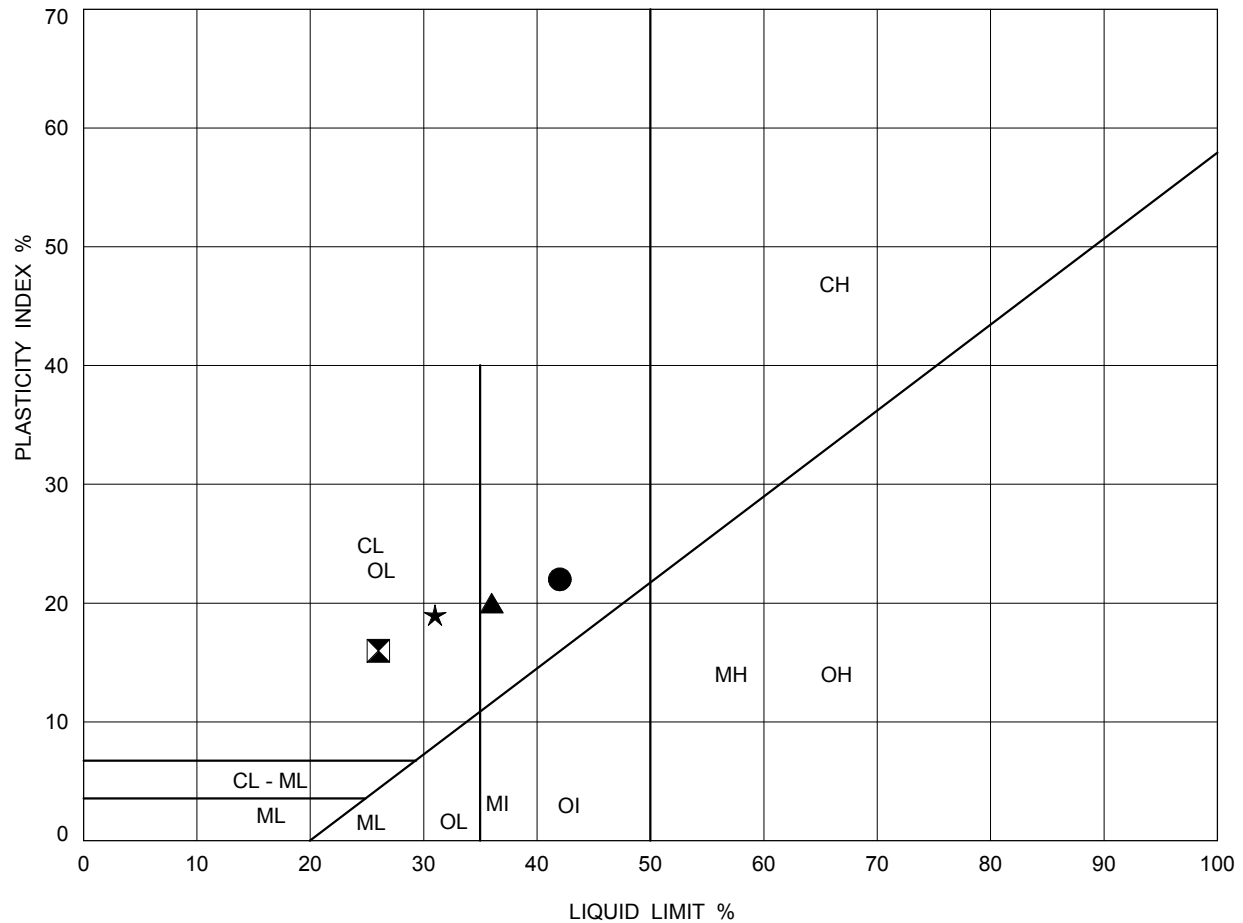
SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND:



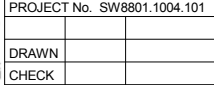
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T5-2	16	16.8	24	14	10
⊠	T5-2	18	19.8	33	16	17
▲	T5-2	20	22.9	38	18	20
★	T5-3	2	1.5	33	13	20
○	T5-3	4	3	39	19	20

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



LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	T5-3	16	16.8	42	20	22
⊠	T5-3	20	19.8	26	10	16
▲	T5-3	23	22.9	36	16	20
★	T5-3	26	27.4	31	12	19

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

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP** Job No.: **SW8801.1004.101**
 Client: **Hatch Mott MacDonald Limited**
 Date: **17-May-11** Sample ID: **T5-2_Sa15** Depth(m): **15.25 to 16.00**

Test Data

Ring # :	B	Ring Height (in) =	0.755	Wt of dry filter paper (g)	0.70
Wet soil + Ring Wt (g)			205.07	Wt of ring (g)	76.53
Wet soil + Wet Paper + Ring (g)			201.82	Wet Paper (g)	1.91
Dry Soil + Dry Paper + Ring (g)			184.75	Ring Dia (in)	2.498
Initial moisture Content (%)			19.55	Final moisture Content (%)	14.75
Area of Ring (in ²)			4.90	Initial Volume (in ³)	3.7002
Initial Bulk Density (kg/m ³)			2120	Initial Dry Density (kg/m ³)	1773
Specific Gravity of Soil			2.72	Equiv. Thick. of solids (mm)	12.502
Final Bulk Density (kg/m ³)			2230	Final Dry Density (kg/m ³)	1865
Initial gauge reading for Load 1			0.2562	Gauge reading for last Loading	0.1902
Initial Voids Ratio			0.534	Final Void Ratio	0.400
Initial Degree of Saturation (%)			100	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	5.0	7.5	11.5	17.0	25.0	38.0	55.0
Load (tsf)	0.052	0.078	0.120	0.177	0.260	0.395	0.572
Gauge Reading (in)	0.2547	0.2531	0.2505	0.2475	0.2446	0.24158	0.2378
(H-Hs) mm	6.636	6.597	6.530	6.454	6.380	6.304	6.208
Voids ratio	0.531	0.528	0.522	0.516	0.510	0.504	0.497
t ₉₀ (min)		20.25	12.60	12.25	11.22	7.29	9.00
C _v (m ² /day)		0.006	0.009	0.009	0.010	0.015	0.012
k' (MPa)		1.231	1.139	1.383	2.059	3.200	3.330
M _v (mm ² / N)		0.8123	0.8778	0.7231	0.4857	0.3125	0.3003

Trial #	8	9	10	11	12	13	14
Load (kPa)	85	130.0	190.0	130.0	85.0	55.0	38.0
Load (tsf)	0.884	1.352	1.976	1.352	0.884	0.572	0.395
Gauge Reading (in)	0.233	0.2269	0.2205	0.2208	0.2211	0.2219	0.2229
(H-Hs) mm	6.086	5.931	5.769	5.776	5.783	5.804	5.829
Voids ratio	0.487	0.474	0.461	0.462	0.463	0.464	0.466
t ₉₀ (min)	4.20	3.80	5.29				
C _v (m ² /day)	0.025	0.028	0.019				
k' (MPa)	4.604	5.399	6.835				
M _v (mm ² / N)	0.2172	0.1852	0.1463				

Trial #	15	16	17	18	19	20	21
Load (kPa)	25.0	17.0	11.5	17.0	25.0	38.0	55.0
Load (tsf)	0.26	0.177	0.120	0.177	0.260	0.395	0.572
Gauge Reading (in)	0.22382	0.2251	0.2260	0.2258	0.2252	0.2244	0.2235
(H-Hs) mm	5.853	5.884	5.909	5.902	5.887	5.867	5.844
Voids ratio	0.468	0.471	0.473	0.472	0.471	0.469	0.467
t ₉₀ (min)							
C _v (m ² /day)							
k' (MPa)							
M _v (mm ² / N)							

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: WEP **Job No.:** SW8801.1004.101
Client: Hatch Mott MacDonald Limited
Date: 17-May-11 **Sample ID:** T5-2_Sa15 **Depth(m):** 15.25 to 16.00

Trial #	22	23	24	25	26	27	28
Load (kPa)	85	130.0	190.0	275.0	430.0	650.0	975.0
Load (tsf)	0.884	1.352	1.976	2.860	4.472	6.760	10.140
Gauge Reading (in)	0.2222	0.2208	0.2183	0.2117	0.2023	0.1916	0.1813
(H-Hs) mm	5.811	5.776	5.712	5.543	5.305	5.034	4.773
Voids ratio	0.465	0.462	0.457	0.443	0.424	0.403	0.382
t90 (min)				3.80	4.20	4.00	2.89
Cv (m ² /day)				0.026	0.023	0.024	0.032
k' (MPa)				9.166	11.715	14.482	21.784
Mv (mm ² / N)				0.1091	0.0854	0.0691	0.0459

Trial #	29	30	31	32	33	34	35
Load (kPa)	1450	725.0	360.0	180.0	90.0	45.0	22.5
Load (tsf)	15.08	7.540	3.744	1.872	0.936	0.468	0.234
Gauge Reading (in)	0.1715	0.1727	0.1737	0.1763	0.1791	0.1825	0.1865
(H-Hs) mm	4.524	4.553	4.580	4.644	4.717	4.802	4.905
Voids ratio	0.362	0.364	0.366	0.371	0.377	0.384	0.392
t90 (min)	3.61						
Cv (m ² /day)	0.025						
k' (MPa)	32.964						
Mv (mm ² / N)	0.0303						

Trial #	36						
Load (kPa)	11.5						
Load (tsf)	0.1196						
Gauge Reading (in)	0.1902						
(H-Hs) mm	4.997						
Voids ratio	0.400						
t90 (min)							
Cv (m ² /day)							
k' (MPa)							
Mv (mm ² / N)							

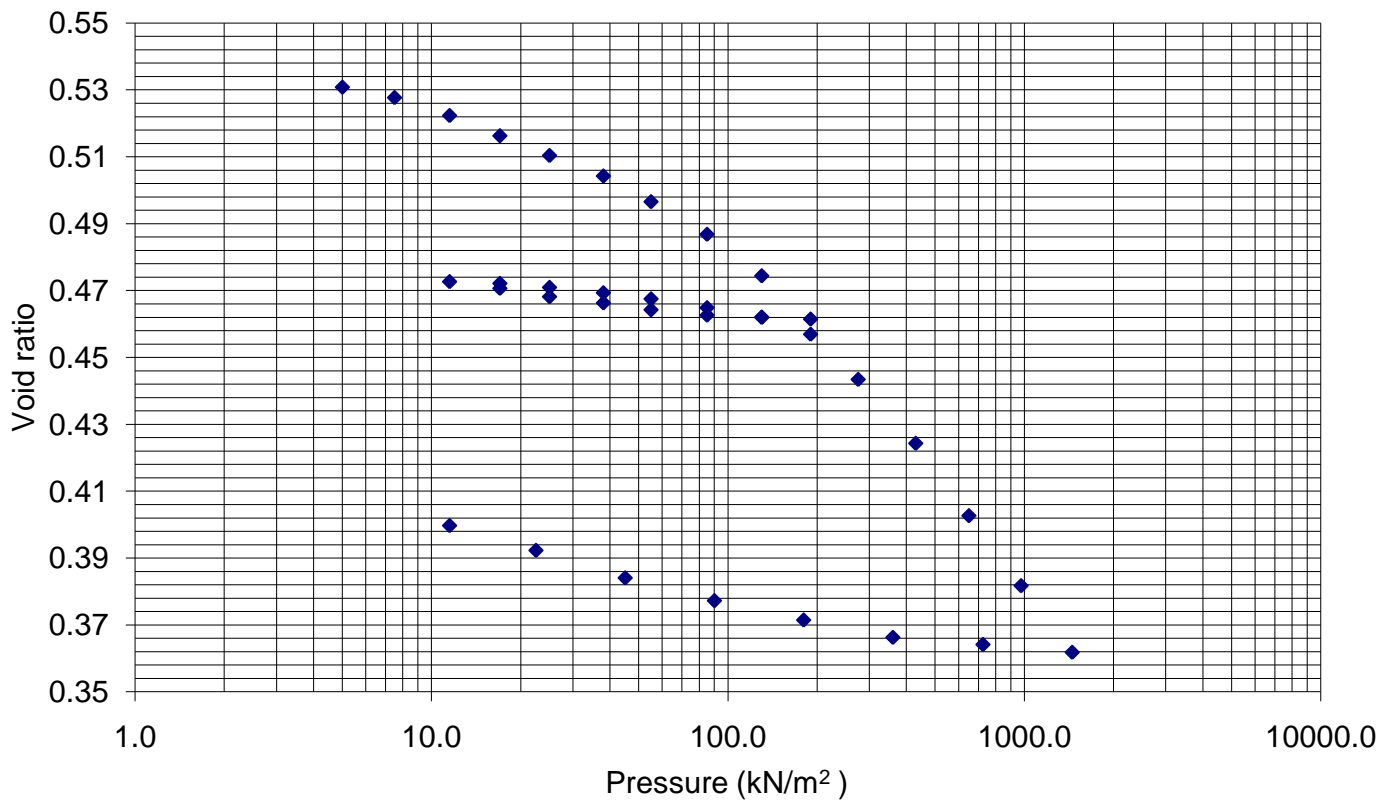
ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP**
 Client: **Hatch Mott MacDonald Limited**
 Date: **17-May-11**

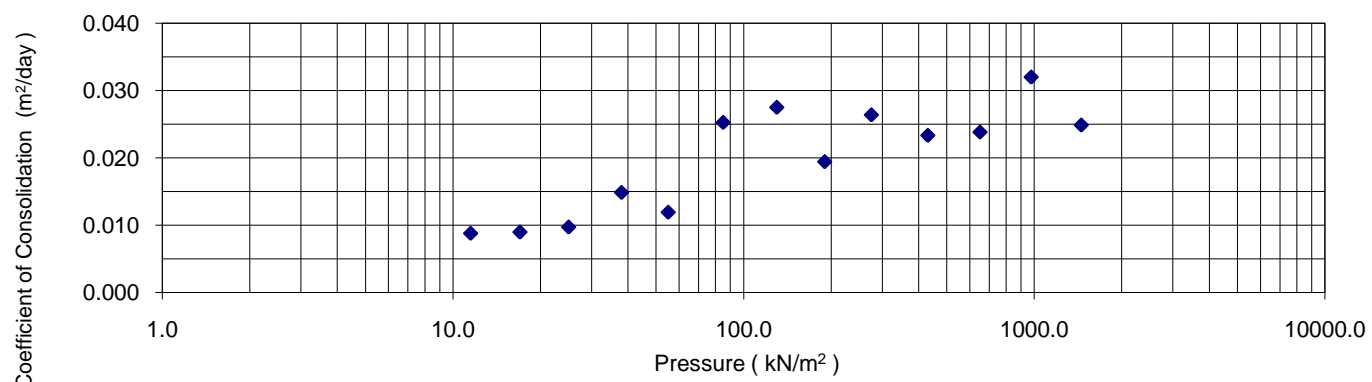
Job No.: **SW8801.1004.101**
 Sample ID: **T5-2_Sa15**
 Depth(m): **15.25 to 16.00**

σ'_v versus e and c_v

Void Ratio Vs Pressure



Coefficient of Consolidation Vs Pressure



ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project:

WEP

Client:

Hatch Mott MacDonald Limited

Date:

17-May-11

Sample ID: T5-2_Sa15

Job No.:

SW8801.1004.101

Depth(m):

15.25 to 16.00

Strain Energy Data

Presssure (kN/m ²)	c _v (m ² /day)	Void ratio
5.0		0.531
7.5		0.528
11.5	0.009	0.522
17.0	0.009	0.516
25.0	0.010	0.510
38.0	0.015	0.504
55.0	0.012	0.497
85.0	0.025	0.487
130.0	0.028	0.474
190.0	0.019	0.461
130.0		0.462
85.0		0.463
55.0		0.464
38.0		0.466
25.0		0.468
17.0		0.471
11.5		0.473
17.0		0.472
25.0		0.471
38.0		0.469
55.0		0.467
85.0		0.465
130.0		0.462
190.0		0.457
275.0	0.026	0.443
430.0	0.023	0.424
650.0	0.024	0.403
975.0	0.032	0.382
1450.0	0.025	0.362
725.0		0.364
360.0		0.366
180.0		0.371
90.0		0.377
45.0		0.384
22.5		0.392
11.5		0.400

Presssure (kN/m ²)	Height mm	Total Work (kJ/m ³)
5.0	19.177	0.000
7.5	19.138	0.013
11.5	19.071	0.046
17.0	18.995	0.103
25.0	18.922	0.184
38.0	18.845	0.312
55.0	18.749	0.549
85.0	18.627	1.004
130.0	18.472	1.898
190.0	18.310	3.299
130.0	18.317	3.237
85.0	18.325	3.194
55.0	18.345	3.116
38.0	18.371	3.051
25.0	18.420	2.968
17.0	18.451	2.932
11.5	18.476	2.913
17.0	18.469	2.918
25.0	18.454	2.935
38.0	18.434	2.969
55.0	18.411	3.027
85.0	18.378	3.152
130.0	18.343	3.362
190.0	18.279	3.913
275.0	18.110	6.062
430.0	17.872	10.709
650.0	17.601	18.883
975.0	17.340	30.959
1450.0	17.091	48.366
725.0	17.120	46.507
360.0	17.147	45.662
180.0	17.211	44.642
90.0	17.284	44.074
45.0	17.369	43.742
22.5	17.472	43.542
11.5	17.564	43.452

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP**

Client: **Hatch Mott MacDonald Limited**

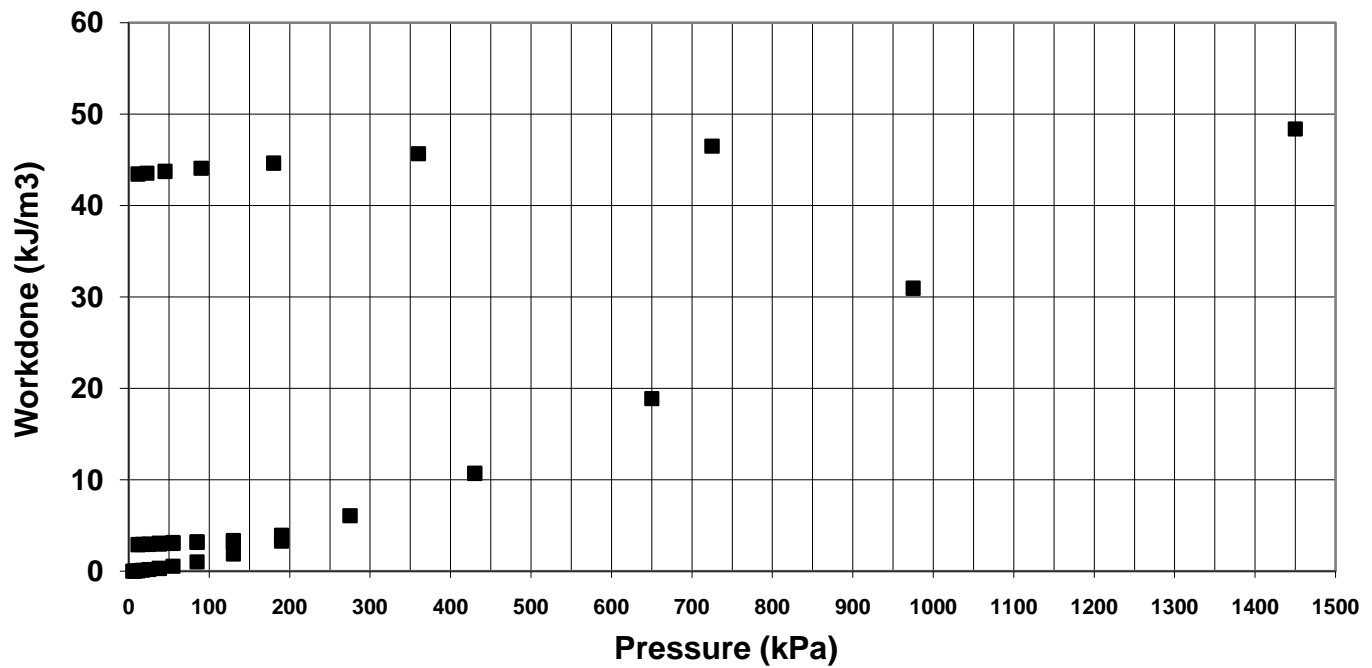
Date: **17-May-11**

Job No.: **SW8801.1004.101**

Sample ID: **T5-2_Sa15**

Depth(m): **15.25 to 16.00**

Strain Energy Method for Preconsolidation Pressure



ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP** Job No.: **SW8801.1004.101**
 Client: **Hatch Mott MacDonald Limited**
 Date: **17-May-11** Sample ID: **T5-3_Sa14** Depth(m): **15.25 to 16.00**

Test Data

Ring # :	A	Ring Height (in) =	0.750	Wt of dry filter paper (g)	0.69
Wet soil + Ring Wt (g)			191.50	Wt of ring (g)	76.58
Wet soil + Wet Paper + Ring (g)			187.60	Wet Paper (g)	1.81
Dry Soil + Dry Paper + Ring (g)			164.53	Ring Dia (in)	2.498
Initial moisture Content (%)			31.70	Final moisture Content (%)	25.15
Area of Ring (in ²)			4.90	Initial Volume (in ³)	3.6757
Initial Bulk Density (kg/m ³)			1908	Initial Dry Density (kg/m ³)	1449
Specific Gravity of Soil			2.74	Equiv. Thick. of solids (mm)	10.072
Final Bulk Density (kg/m ³)			2033	Final Dry Density (kg/m ³)	1544
Initial gauge reading for Load 1			0.2578	Gauge reading for last Loading	0.1768
Initial Voids Ratio			0.891	Final Void Ratio	0.687
Initial Degree of Saturation (%)			97	Final Degree of Saturation (%)	100

Trial #	1	2	3	4	5	6	7
Load (kPa)	5.0	7.5	11.5	17.0	25.0	38.0	55.0
Load (tsf)	0.052	0.078	0.120	0.177	0.260	0.395	0.572
Gauge Reading (in)	0.2575	0.2571	0.2558	0.2540	0.2519	0.2487	0.2452
(H-Hs) mm	8.970	8.960	8.927	8.882	8.828	8.747	8.657
Voids ratio	0.891	0.890	0.886	0.882	0.876	0.868	0.859
t ₉₀ (min)		1.00	2.25	6.25	2.25	1.82	1.96
C _v (m ² /day)		0.111	0.049	0.018	0.049	0.060	0.055
k' (MPa)		4.686	2.306	2.311	2.816	3.023	3.548
M _v (mm ² / N)		0.2134	0.4337	0.4327	0.3551	0.3308	0.2819

Trial #	8	9	10	11	12	13	14
Load (kPa)	85	130.0	190.0	130.0	85.0	55.0	38.0
Load (tsf)	0.884	1.352	1.976	1.352	0.884	0.572	0.395
Gauge Reading (in)	0.2396	0.2319	0.2227	0.2232	0.2244	0.2258	0.2277
(H-Hs) mm	8.516	8.319	8.086	8.099	8.129	8.165	8.213
Voids ratio	0.845	0.826	0.803	0.804	0.807	0.811	0.815
t ₉₀ (min)	1.96	2.56	3.61				
C _v (m ² /day)	0.054	0.041	0.028				
k' (MPa)	3.986	4.266	4.733				
M _v (mm ² / N)	0.2509	0.2344	0.2113				

Trial #	15	16	17	18	19	20	21
Load (kPa)	25.0	17.0	11.5	17.0	25.0	38.0	55.0
Load (tsf)	0.26	0.177	0.120	0.177	0.260	0.395	0.572
Gauge Reading (in)	0.22919	0.2312	0.2339	0.2336	0.2327	0.2313	0.2297
(H-Hs) mm	8.251	8.302	8.371	8.364	8.339	8.304	8.264
Voids ratio	0.819	0.824	0.831	0.830	0.828	0.824	0.820
t ₉₀ (min)							
C _v (m ² /day)							
k' (MPa)							
M _v (mm ² / N)							

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP** Job No.: **SW8801.1004.101**
 Client: **Hatch Mott MacDonald Limited**
 Date: **17-May-11** Sample ID: **T5-3_Sa14** Depth(m): **15.25 to 16.00**

Trial #	22	23	24	25	26	27	28
Load (kPa)	85	130.0	190.0	285.0	430.0	650.0	975.0
Load (tsf)	0.884	1.352	1.976	2.964	4.472	6.760	10.140
Gauge Reading (in)	0.22708	0.2239	0.2193	0.2087	0.1926	0.1715	0.1517
(H-Hs) mm	8.198	8.117	8.000	7.730	7.320	6.785	6.283
Voids ratio	0.814	0.806	0.794	0.768	0.727	0.674	0.624
t90 (min)				4.00	7.84	6.25	6.25
Cv (m ² /day)				0.025	0.012	0.014	0.013
k' (MPa)				6.359	6.297	7.150	10.905
Mv (mm ² / N)				0.1573	0.1588	0.1399	0.0917

Trial #	29	30	31	32	33	34	35
Load (kPa)	1450	725.0	360.0	180.0	90.0	45.0	22.5
Load (tsf)	15.08	7.540	3.744	1.872	0.936	0.468	0.234
Gauge Reading (in)	0.13375	0.1360	0.1391	0.1444	0.1516	0.1590	0.1668
(H-Hs) mm	5.827	5.884	5.963	6.097	6.280	6.468	6.665
Voids ratio	0.579	0.584	0.592	0.605	0.624	0.642	0.662
t90 (min)	5.06						
Cv (m ² /day)	0.016						
k' (MPa)	17.039						
Mv (mm ² / N)	0.0587						

Trial #	36						
Load (kPa)	11.5						
Load (tsf)	0.1196						
Gauge Reading (in)	0.1768						
(H-Hs) mm	6.919						
Voids ratio	0.687						
t90 (min)							
Cv (m ² /day)							
k' (MPa)							
Mv (mm ² / N)							

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP**

Job No.: **SW8801.1004.101**

Client: **Hatch Mott MacDonald Limited**

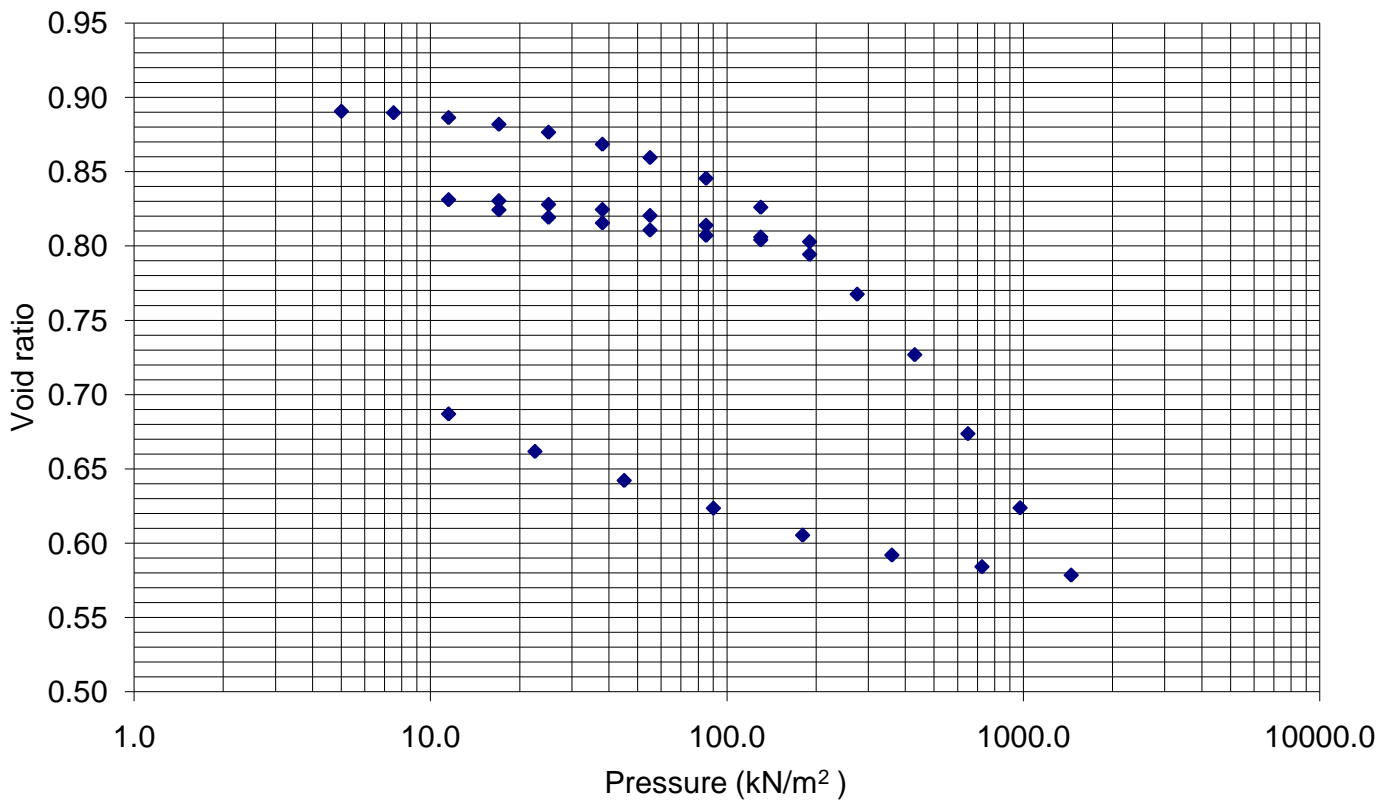
Date: **17-May-11**

Sample ID: **T5-3_Sa14**

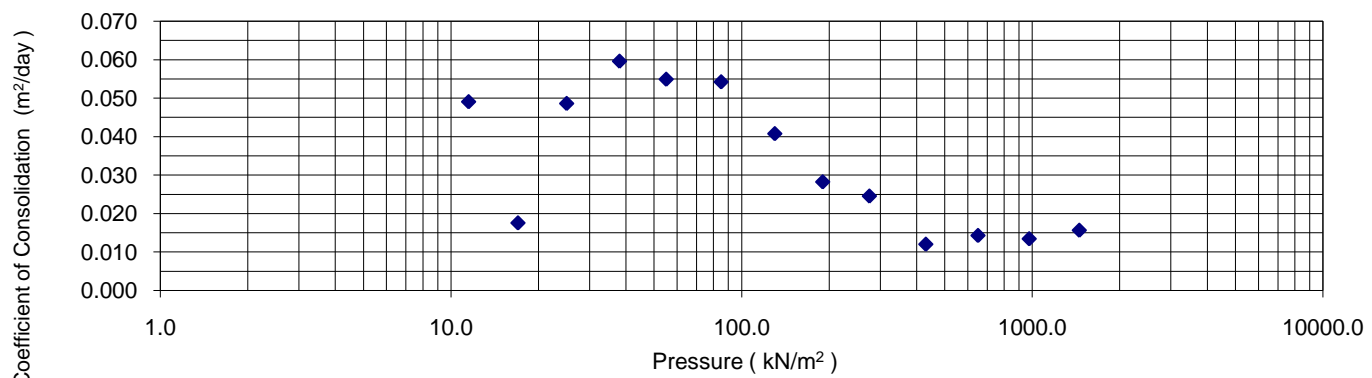
Depth(m): **15.25 to 16.00**

σ'_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: **17-May-11** Sample ID: **T5-3_Sa14** Depth(m): **15.25 to 16.00**

Strain Energy Data

Presssure (kN/m ²)	c _v (m ² /day)	Void ratio
5.0		0.891
7.5		0.890
11.5	0.049	0.886
17.0	0.018	0.882
25.0	0.049	0.876
38.0	0.060	0.868
55.0	0.055	0.859
85.0	0.054	0.845
130.0	0.041	0.826
190.0	0.028	0.803
130.0		0.804
85.0		0.807
55.0		0.811
38.0		0.815
25.0		0.819
17.0		0.824
11.5		0.831
17.0		0.830
25.0		0.828
38.0		0.824
55.0		0.820
85.0		0.814
130.0		0.806
190.0		0.794
275.0	0.025	0.768
430.0	0.012	0.727
650.0	0.014	0.674
975.0	0.013	0.624
1450.0	0.016	0.579
725.0		0.584
360.0		0.592
180.0		0.605
90.0		0.624
45.0		0.642
22.5		0.662
11.5		0.687

Presssure (kN/m ²)	Height mm	Total Work (kJ/m ³)
5.0	19.050	0.000
7.5	19.040	0.003
11.5	19.007	0.020
17.0	18.962	0.054
25.0	18.908	0.113
38.0	18.826	0.249
55.0	18.736	0.471
85.0	18.595	0.998
130.0	18.399	2.132
190.0	18.166	4.159
130.0	18.179	4.048
85.0	18.209	3.870
55.0	18.245	3.732
38.0	18.293	3.609
25.0	18.379	3.460
17.0	18.430	3.402
11.5	18.499	3.349
17.0	18.492	3.354
25.0	18.467	3.382
38.0	18.432	3.442
55.0	18.392	3.544
85.0	18.326	3.796
130.0	18.245	4.267
190.0	18.128	5.291
275.0	17.858	8.754
430.0	17.449	16.846
650.0	16.913	33.409
975.0	16.411	57.544
1450.0	15.955	91.230
725.0	16.012	87.352
360.0	16.091	84.684
180.0	16.226	82.421
90.0	16.408	80.899
45.0	16.596	80.126
22.5	16.793	79.726
11.5	17.047	79.469

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D 2435)

Project: **WEP**

Job No.: **SW8801.1004.101**

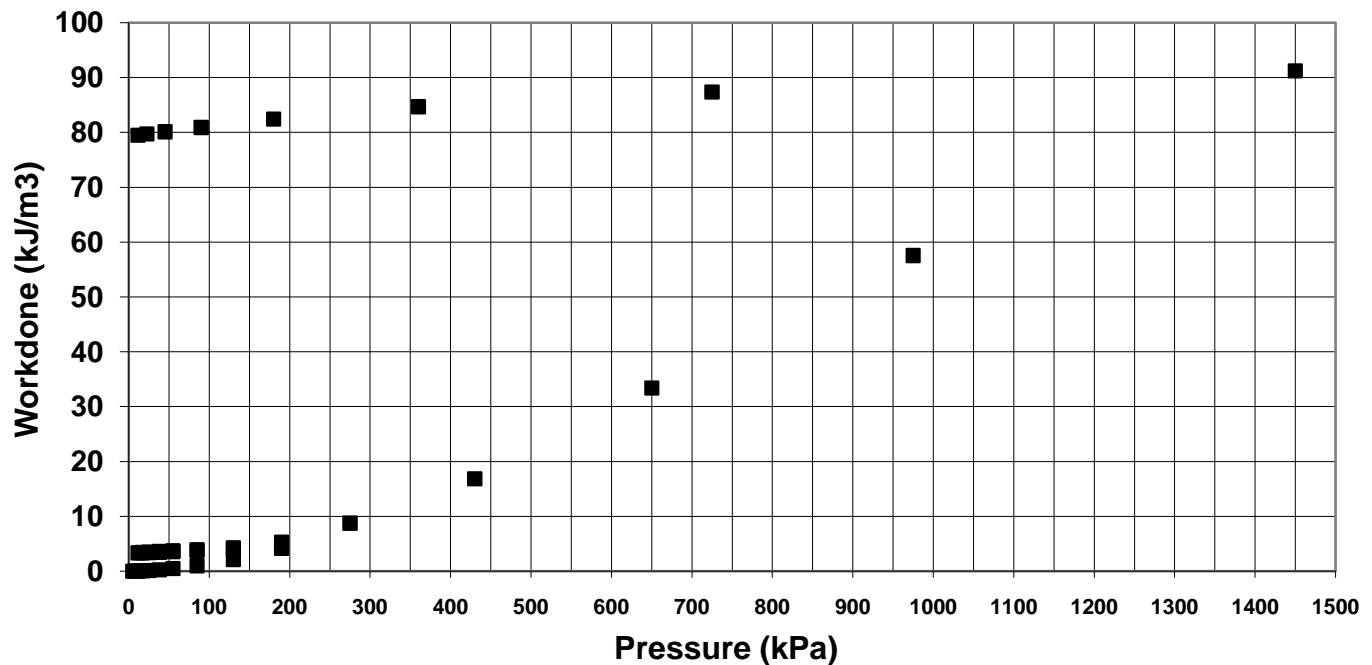
Client: **Hatch Mott MacDonald Limited**

Date: **17-May-11**

Sample ID: **T5-3_Sa14**

Depth(m): **15.25 to 16.00**

Strain Energy Method for Preconsolidation Pressure



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

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

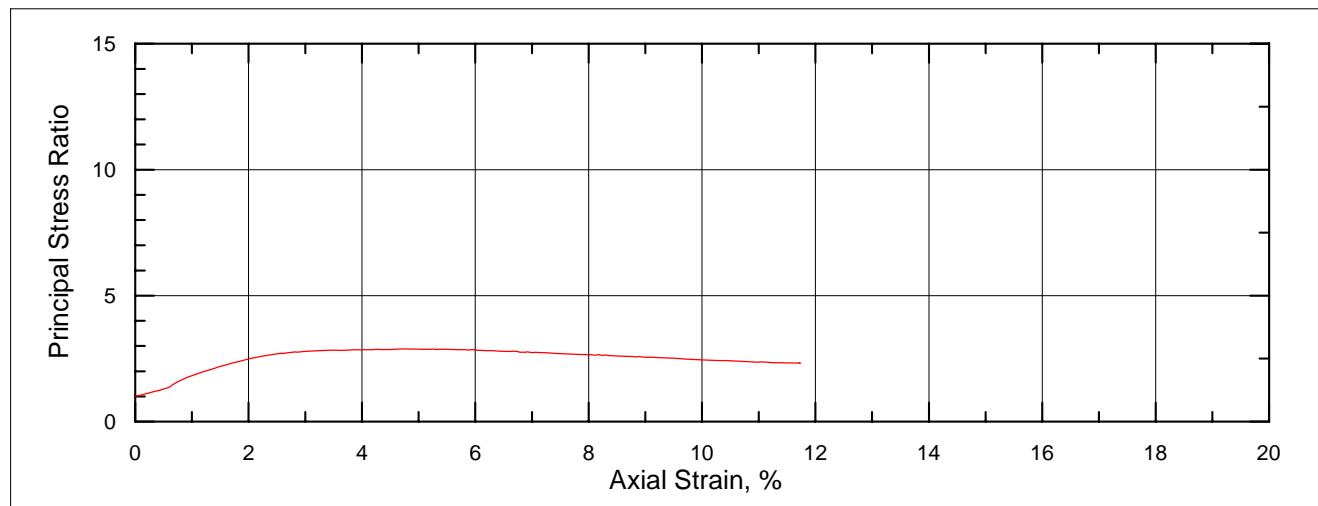
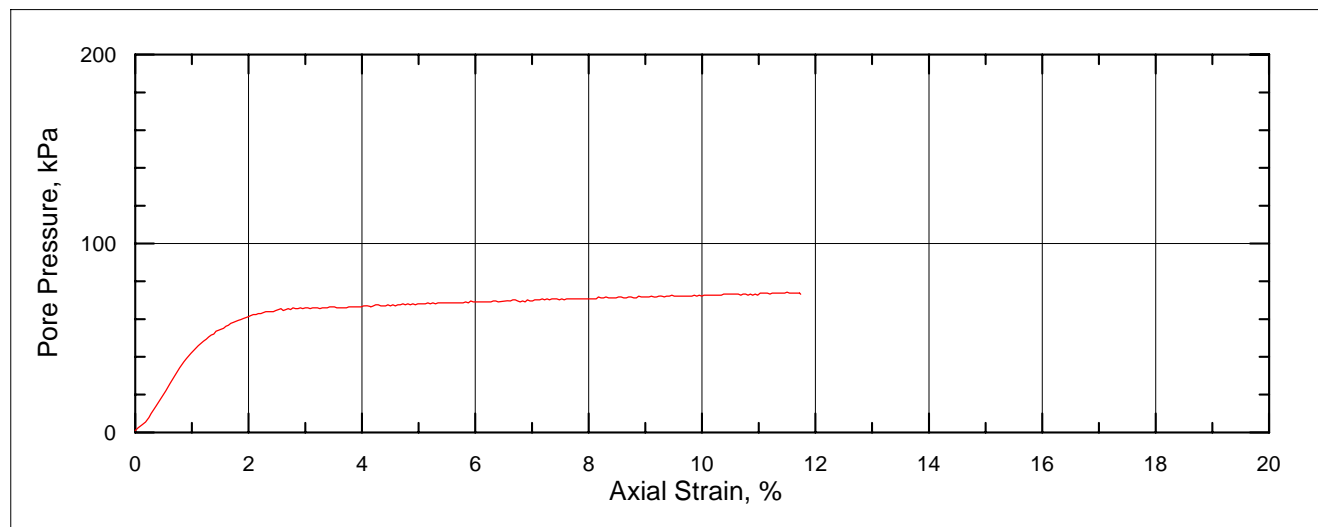
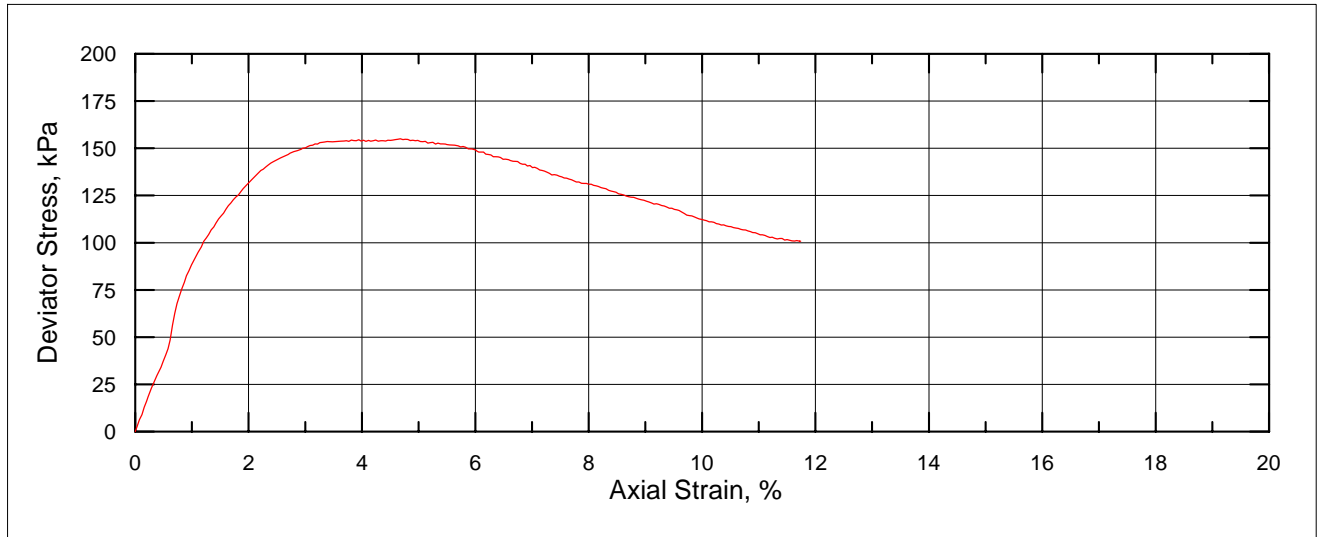
Sample ID: T5-3_Sa14

Project No.: SW8801
Date: 11-May-11
Depth(m): 15.2 to 16.0

Sample Description: Silty Clay trace sand and gravel

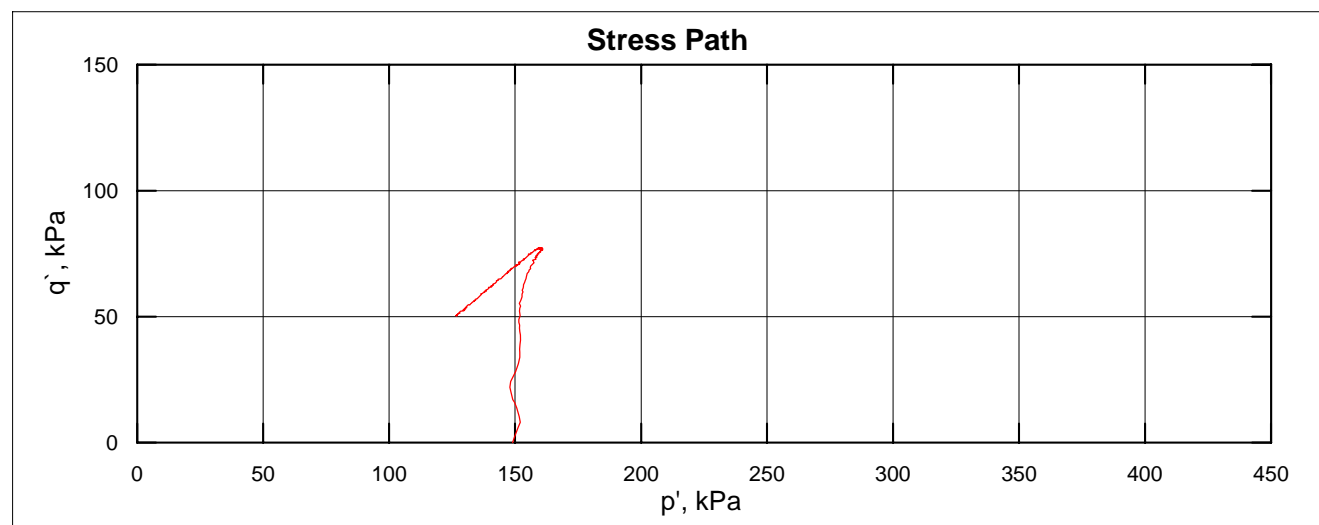
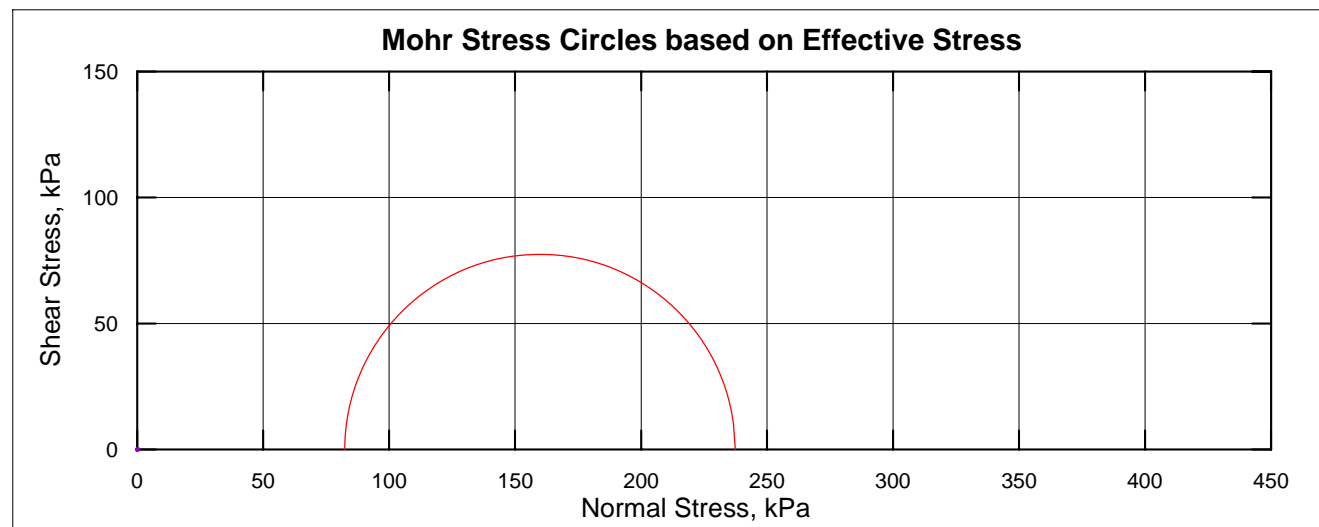
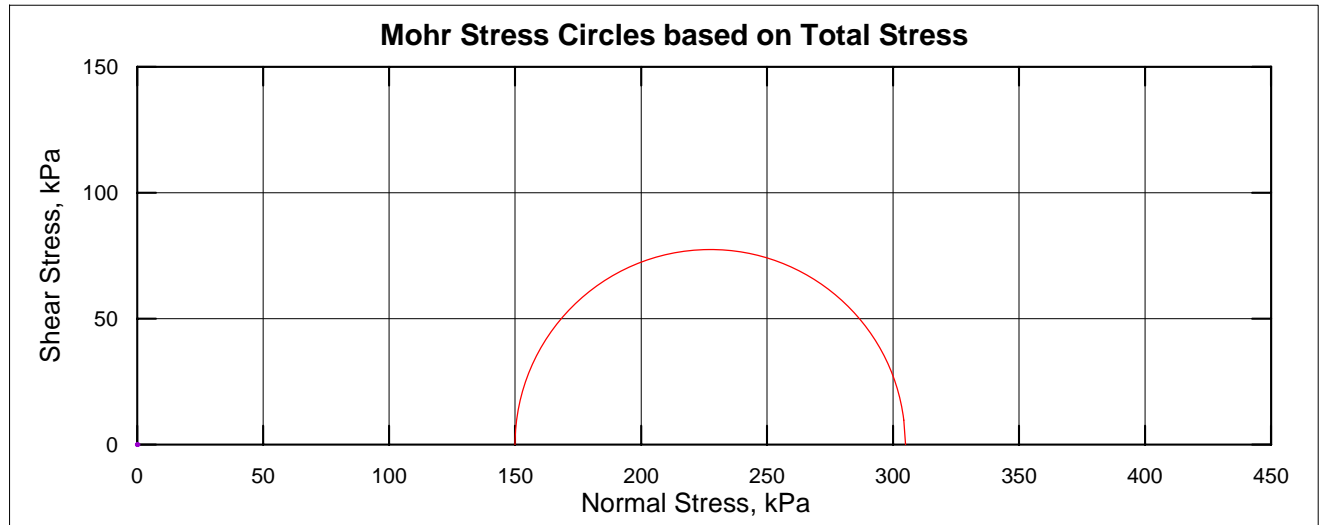
Sample Parameters				
Initial		Specimen 1	Specimen 2	Specimen 3
Diameter	cm	6.950		
Height	cm	14.101		
Volume	cm ³	534.945		
Wet Mass	g	1013.30		
Dry Density	kg/m ³	1462		
Water Content	%	29.6		
Specific Gravity	Actual	2.740		
Void Ratio		0.87		
Degree of Saturation		92.7		
Before Shear (after consolidation)				
Volume	cm ³	502.745		
B - Value		0.98		
After Shear				
Wet Mass	g	1003.33		
Dry Density	kg/m ³	1513		
Water Content	%	31.9		
Void Ratio		0.81		
Degree of Saturation		100.0		
Stress - Strain				
Cell Pressure	kPa	300.00		
Back Pressure	kPa	150.00		
Consolidation Stress	kPa	150.00		
Rate of Strain	mm/min	0.0200		
Vertical Strain at Failure	%	4.68		
Deviator Stress at Failure	kPa	154.95		
Pore Pressure at Failure	kPa	67.60		
Total Stress				
Minor Principal Stress, σ_3	kPa	150.00		
Major Principal Stress, σ_1	kPa	304.95		
Radius, $(\sigma_1 - \sigma_3)/2$	kPa	77.48		
Intersection Point, $(\sigma_1 + \sigma_3)/2$	kPa	227.48		
Effective Stress				
Minor Principal Stress, σ_3'	kPa	82.40		
Major Principal Stress, σ_1'	kPa	237.35		
Radius, $(\sigma_1' - \sigma_3')/2$	kPa	77.48		
Intersection Point, $(\sigma_1' + \sigma_3')/2$	kPa	159.88		

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST
FOR COHESIVE SOILS (ASTM D-4767)
(Multi specimen - Single stage)



— 150 kPa

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



Appendix D: Analytical Laboratory Test Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

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



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

Date Received: 14-MAY-11
Report Date: 24-MAY-11
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1005225
Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 092960

Gayle Braun
Senior Account Manager

[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 LABORATORY GROUP CHEMICAL ANALYSIS REPORT

Lab ID	Sample ID	Test Description	Result	Qualifier	D.L.	Units	Extracted	Analyzed	By
L1005225-2	BH: T5-1 SA#25 @ 100'								
Sample Date: JUSTIN PALMER on 13-MAY-11									
Matrix: SOIL									
soil:pH,SO4,S2,Redox,resistivity,%moist.									
pH									
	pH		7.83		0.1	pH units	18-MAY-11	18-MAY-11	AK1
Sulphide									
	Sulphide		<0.20		0.2	mg/kg	18-MAY-11	18-MAY-11	SC1
Sulphate									
	Sulphate		582		20	mg/kg	18-MAY-11	18-MAY-11	CY
Resistivity									
	Resistivity		1900		100	ohm cm	19-MAY-11	19-MAY-11	AK1
Redox Potential									
	Redox Potential		246		-1000	mV	19-MAY-11	19-MAY-11	AK1
	% Moisture		22.3		0.1	%	16-MAY-11	16-MAY-11	JB3
L1005225-3	BH: T5-3 SA#11 @ 35'								
Sample Date: JUSTIN PALMER on 13-MAY-11									
Matrix: SOIL									
	% Moisture		20.8		0.1	%	16-MAY-11	16-MAY-11	JB3
soil:pH,SO4,S2,Redox,resistivity,%moist.									
pH									
	pH		8.01		0.1	pH units	18-MAY-11	18-MAY-11	AK1
Sulphide									
	Sulphide		<0.20		0.2	mg/kg	18-MAY-11	18-MAY-11	SC1
Sulphate									
	Sulphate		520		20	mg/kg	18-MAY-11	18-MAY-11	CY
Resistivity									
	Resistivity		1830		100	ohm cm	19-MAY-11	19-MAY-11	AK1
Redox Potential									
	Redox Potential		234		-1000	mV	19-MAY-11	19-MAY-11	AK1
L1005225-8	BH: T5-2 SA#24 @ 90'								
Sample Date: JUSTIN PALMER on 13-MAY-11									
Matrix: SOIL									
soil:pH,SO4,S2,Redox,resistivity,%moist.									
pH									
	pH		7.89		0.1	pH units	18-MAY-11	18-MAY-11	AK1
Sulphide									
	Sulphide		<0.20		0.2	mg/kg	18-MAY-11	18-MAY-11	SC1
Sulphate									
	Sulphate		387		20	mg/kg	18-MAY-11	18-MAY-11	CY
Resistivity									
	Resistivity		1860		100	ohm cm	19-MAY-11	19-MAY-11	AK1
Redox Potential									
	Redox Potential		260		-1000	mV	19-MAY-11	19-MAY-11	AK1
	% Moisture		9.76		0.1	%	16-MAY-11	16-MAY-11	JB3

ALS LABORATORY GROUP CHEMICAL ANALYSIS REPORT

Lab ID	Sample ID	Test Description	Result	Qualifier	D.L.	Units	Extracted	Analyzed	By

Methodology Reference

ALS Test Code	Test Description	Methodology Reference (In-House Standard Operating Procedures which Generally Follow:)
SULPHIDE-WT	Sulphide	APHA 4500S2D
MOISTURE-WT	% Moisture	Gravimetric: Oven Dried
SO4-WT	Sulphate	EPA 300.0
PH-WT	pH	MOEE E3137A
REDOX-POTENTIAL-WT	Redox Potential	APHA 2580
RESISTIVITY-WT	Resistivity	MOEE E3137A

Appendix E: Slope Stability Analysis Results

Last Saved: 18/05/2012 - 4:30:09 PM
Analysis Method: Morgenstern-Price

Properties:

Name: Upper Clay 1	Unit Weight: 20 kN/m ³	C-Datum: 55 kPa	C-Rate of Change: -0.6 kPa/m	Limiting C: 50 kPa	Elevation: 175 m
Name: Lower Clay	Unit Weight: 20 kN/m ³	C-Datum: 50 kPa	C-Rate of Change: 6.7 kPa/m	Limiting C: 70 kPa	Elevation: 163 m
Name: Clay Crust	Unit Weight: 22 kN/m ³	Cohesion: 75 kPa	Phi: 0°		
Name: Clay Transition	Unit Weight: 21 kN/m ³	C-Datum: 75 kPa	C-Rate of Change: -10 kPa/m	Limiting C: 55 kPa	Elevation: 177 m
Name: Middle Clay	Unit Weight: 20 kN/m ³	C-Datum: 42 kPa	C-Rate of Change: 3.5 kPa/m	Limiting C: 50 kPa	Elevation: 165.5 m
Name: RSS	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 35°		
Name: RGM	Unit Weight: 21 kN/m ³	Cohesion: 40 kPa	Phi: 35°		
Name: Clay Backfill	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 0°		
Name: EPS	Unit Weight: 0.5 kN/m ³	Cohesion: 10 kPa	Phi: 0°		
Name: Clayey Silt	Unit Weight: 22 kN/m ³	Cohesion: 70 kPa	Phi: 0°		
Name: Slag	Unit Weight: 12 kN/m ³	Cohesion: 0 kPa	Phi: 35°		
Name: RSS (light)	Unit Weight: 12 kN/m ³	Cohesion: 50 kPa	Phi: 35°		
Name: Structure	Unit Weight: 0.5 kN/m ³	Cohesion: 100 kPa	Phi: 0°		
Name: Upper Clay 2	Unit Weight: 20 kN/m ³	Cohesion: 42 kPa	Phi: 0°		
Name: Base-Subbase (Equiv)	Unit Weight: 12 kN/m ³	Cohesion: 0 kPa	Phi: 35°		

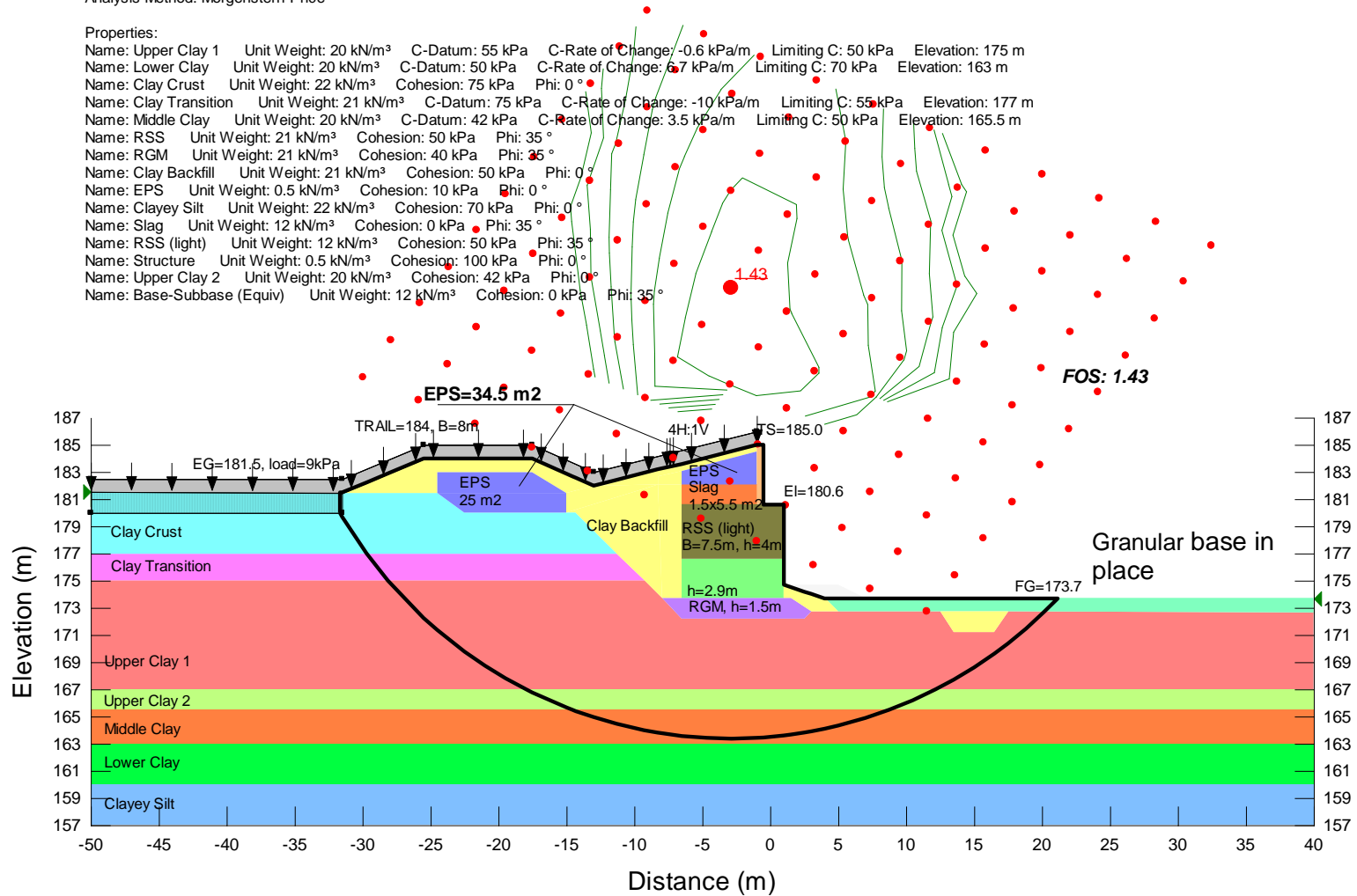


Figure E-2: Slope Stability Result – North Abutment – End of Construction (Undrained) Loading

File Name: Slope_T5N_Sta14+513W_20120516.gsz
Name: End of Construction

Last Saved: 18/05/2012 - 4:36:40 PM
Analysis Method: Morgenstern-Price

Properties:

Name: Upper Clay 1	Unit Weight: 20 kN/m ³	C-Datum: 55 kPa	C-Rate of Change: -0.6 kPa/m	Limiting C: 50 kPa	Elevation: 175 m
Name: Lower Clay	Unit Weight: 20 kN/m ³	C-Datum: 50 kPa	C-Rate of Change: 6.7 kPa/m	Limiting C: 70 kPa	Elevation: 163 m
Name: Clay Crust	Unit Weight: 22 kN/m ³	Cohesion: 75 kPa	Phi: 0 °		
Name: Clay Transition	Unit Weight: 21 kN/m ³	C-Datum: 75 kPa	C-Rate of Change: -10 kPa/m	Limiting C: 55 kPa	Elevation: 177 m
Name: Middle Clay	Unit Weight: 20 kN/m ³	C-Datum: 42 kPa	C-Rate of Change: 3.6 kPa/m	Limiting C: 50 kPa	Elevation: 165.5 m
Name: RSS	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 35 °		
Name: RGM	Unit Weight: 21 kN/m ³	Cohesion: 40 kPa	Phi: 35 °		
Name: Clay Backfill	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 0 °		
Name: EPS	Unit Weight: 0.5 kN/m ³	Cohesion: 10 kPa	Phi: 0 °		
Name: Clayey Silt	Unit Weight: 22 kN/m ³	Cohesion: 70 kPa	Phi: 0 °		
Name: Slag	Unit Weight: 12 kN/m ³	Cohesion: 0 kPa	Phi: 35 °		
Name: RSS (light)	Unit Weight: 12 kN/m ³	Cohesion: 50 kPa	Phi: 35 °		
Name: Structure	Unit Weight: 0.5 kN/m ³	Cohesion: 100 kPa	Phi: 0 °		
Name: Pavement	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 35 °		
Name: Upper Clay 2	Unit Weight: 20 kN/m ³	Cohesion: 42 kPa	Phi: 0 °		

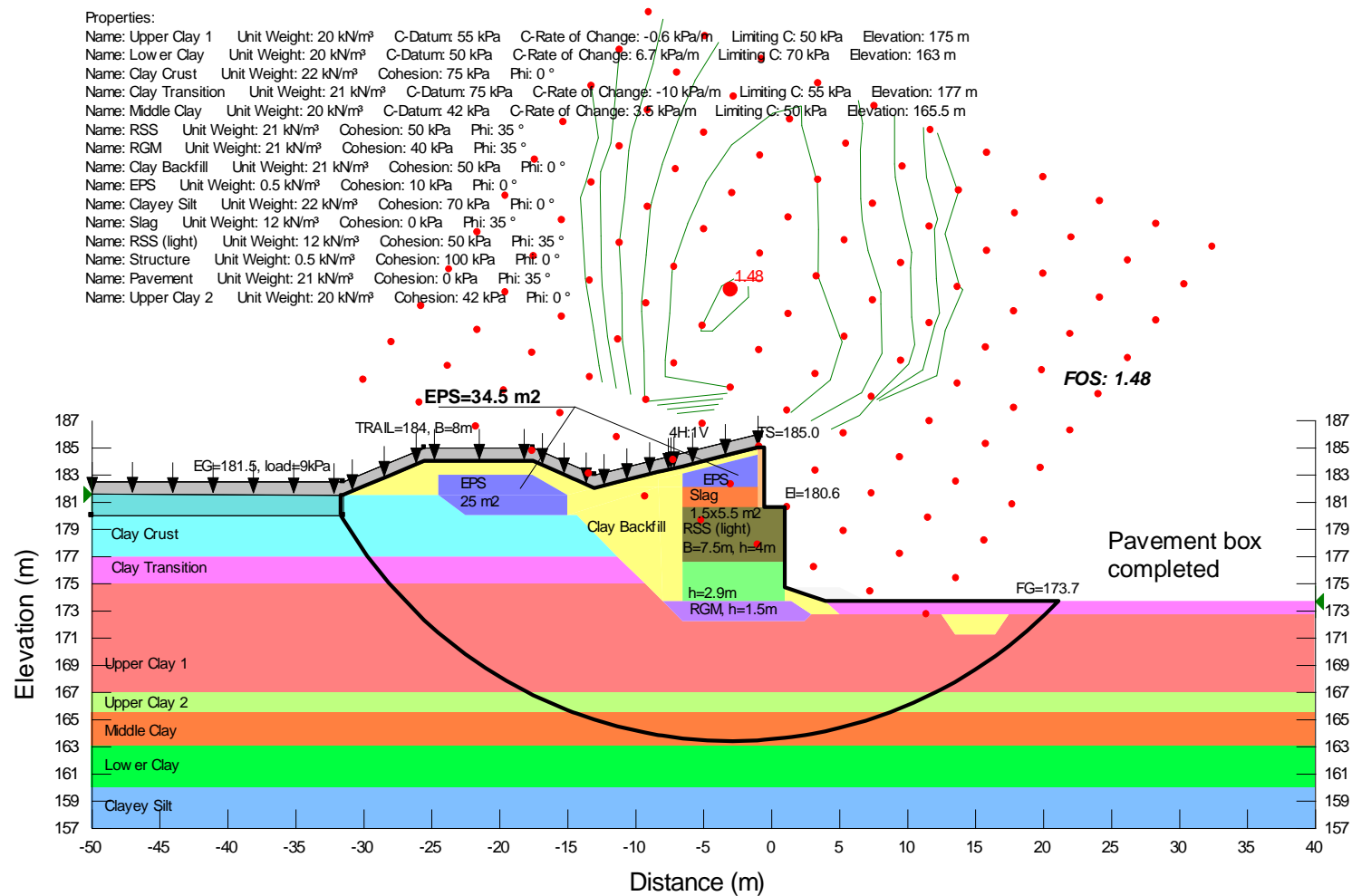


Figure E-3: Slope Stability Result – North Abutment – Long-term (Drained) Loading

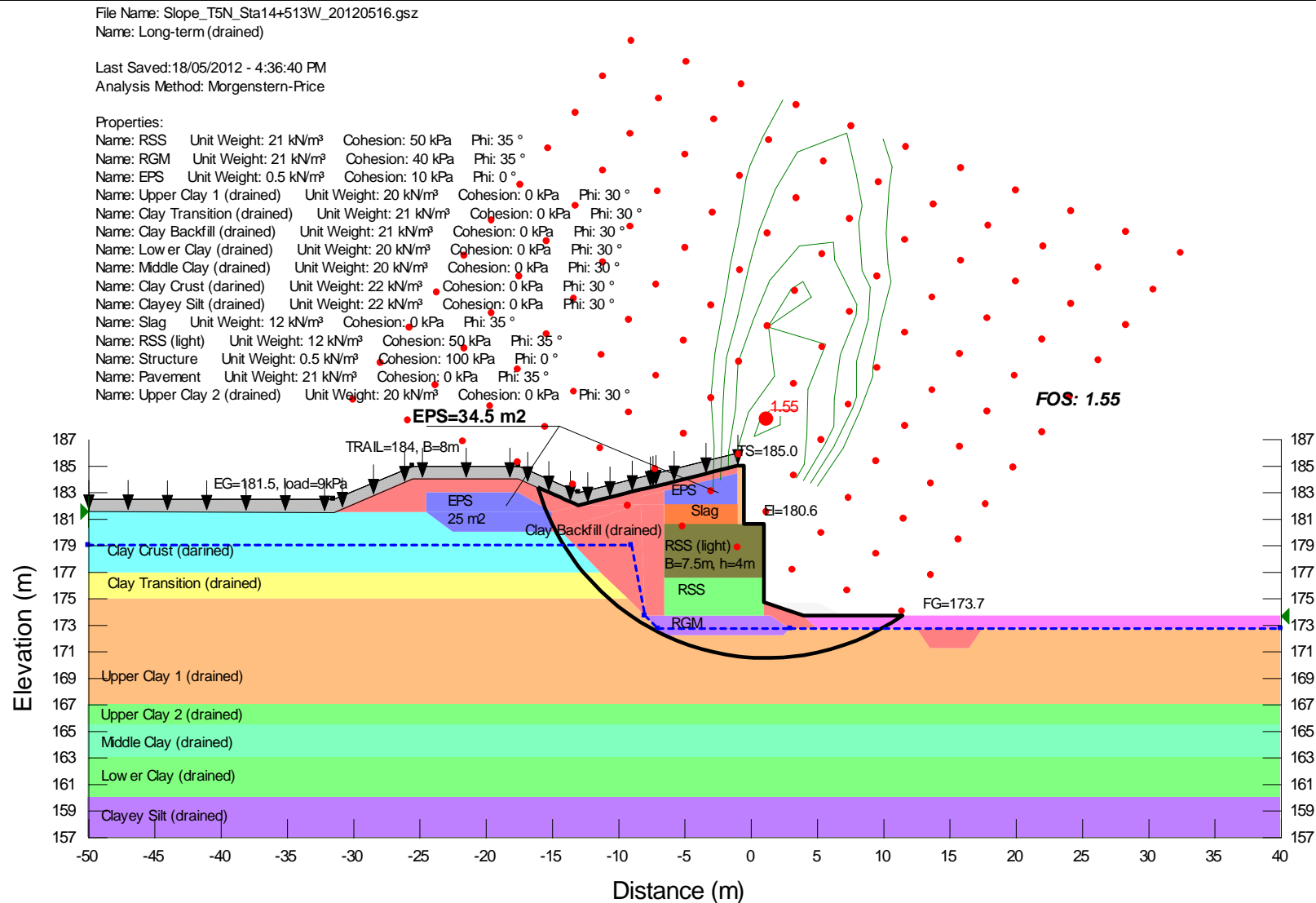


Figure E-5: Slope Stability Result – South Abutment (East & Central Segments) – End of Construction (Undrained) Loading

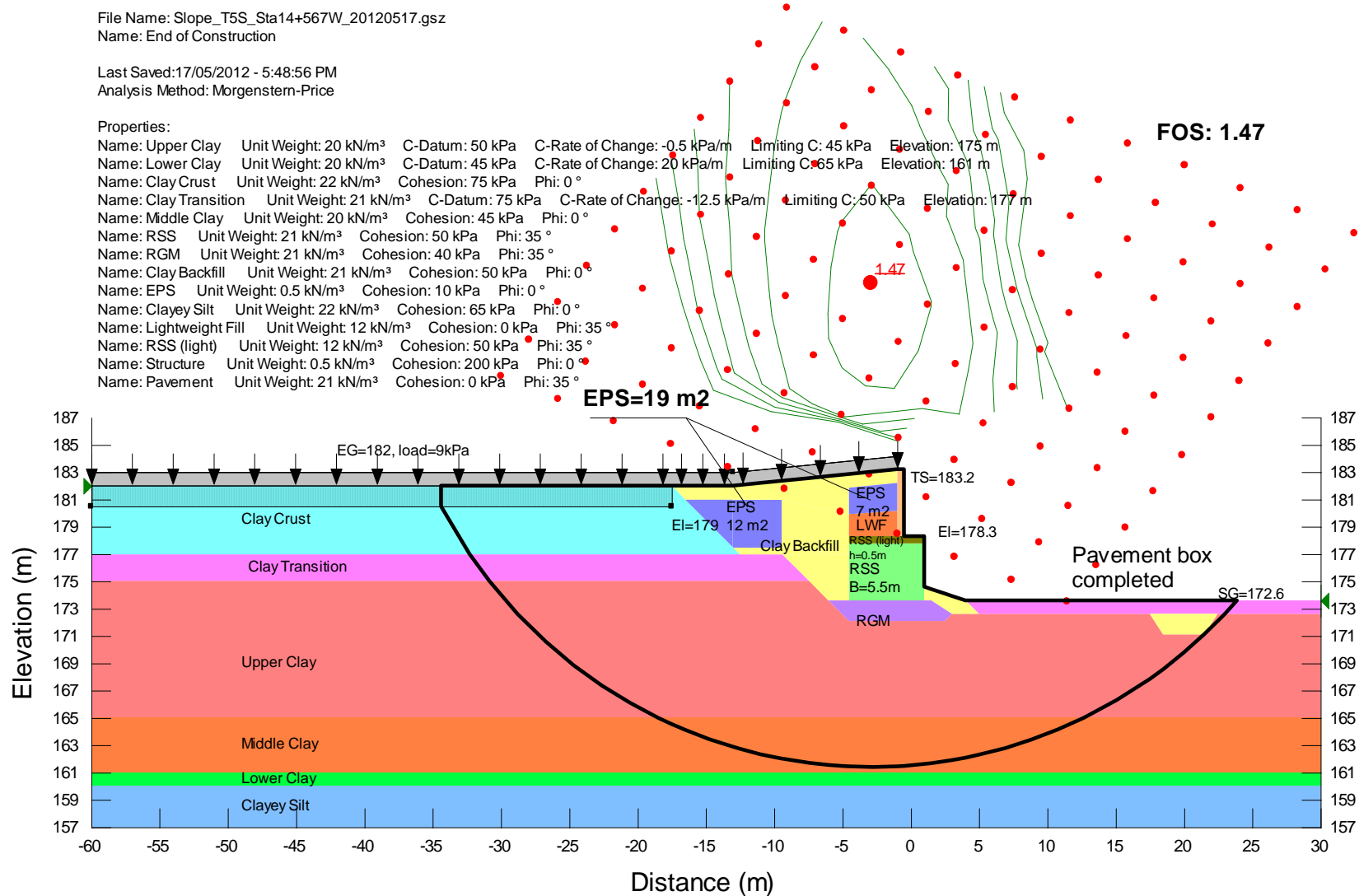


Figure E-6: Slope Stability Result – South Abutment (East & Central Segments) – Long-term (Drained) Loading

File Name: Slope_T5S_Sta14+567W_20120517.gsz
Name: Long-term (drained)

Last Saved: 18/05/2012 - 11:15:10 AM
Analysis Method: Morgenstern-Price

Properties:

Name: RSS Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 35 °
Name: RGM Unit Weight: 21 kN/m³ Cohesion: 40 kPa Phi: 35 °
Name: EPS Unit Weight: 0.5 kN/m³ Cohesion: 10 kPa Phi: 0 °
Name: Upper Clay (drained) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clay Transition (drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clay Backfill (drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Lower Clay (drained) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Middle Clay (drained) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clay Crust (drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clayey Silt (drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Lightweight Fill Unit Weight: 12 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS (light) Unit Weight: 12 kN/m³ Cohesion: 50 kPa Phi: 35 °
Name: Structure Unit Weight: 0.5 kN/m³ Cohesion: 200 kPa Phi: 0 °
Name: Pavement Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °

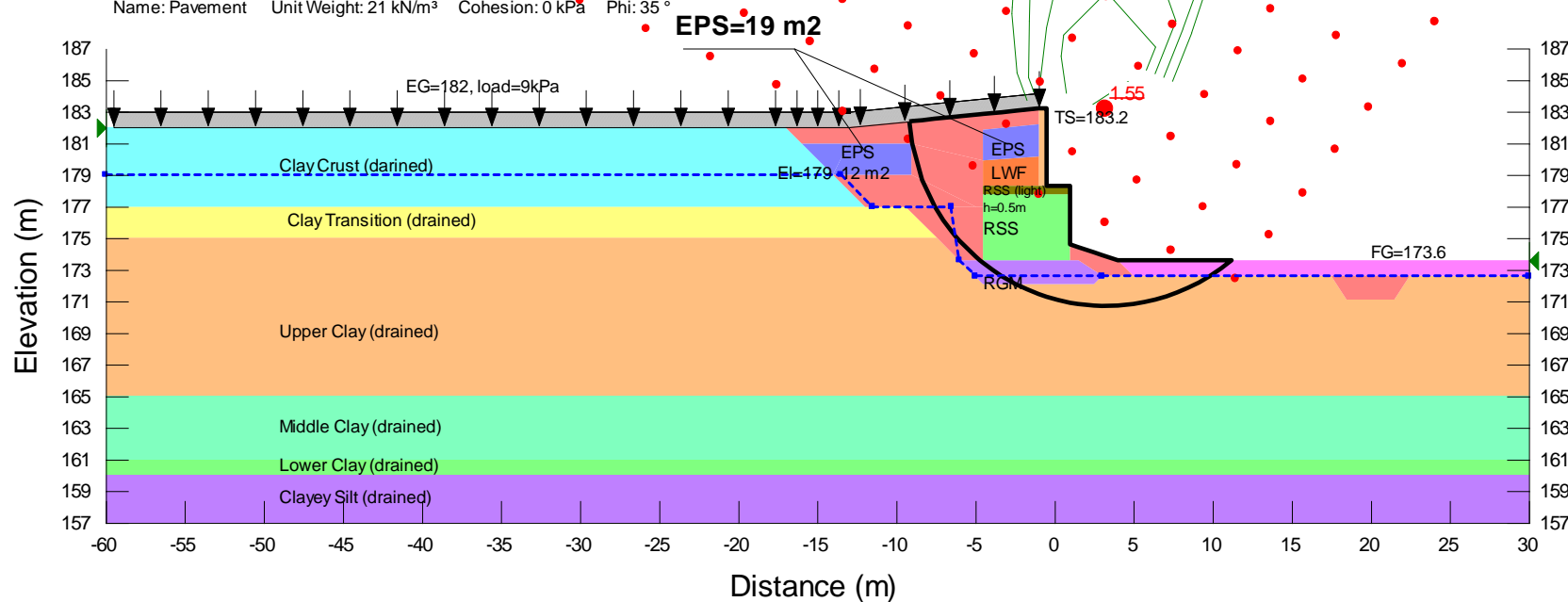


Figure E-7: Slope Stability Result – South Abutment (West Segment) – Short-term (Undrained) Loading

File Name: Slope_T5S_Sta14+540W_20120516.gsz
Name: Short-term (restricted)

Last Saved: 17/05/2012 - 2:47:53 PM
Analysis Method: Morgenstern-Price

Properties:

Name: Upper Clay	Unit Weight: 20 kN/m ³	Cohesion: 40 kPa	Phi: 0°		
Name: Lower Clay	Unit Weight: 20 kN/m ³	C-Datum: 45 kPa	C-Rate of Change: 20 kPa/m	Limiting C: 65 kPa	Elevation: 161 m
Name: Clay Crust	Unit Weight: 22 kN/m ³	Cohesion: 75 kPa	Phi: 0°		
Name: Clay Transition	Unit Weight: 21 kN/m ³	C-Datum: 75 kPa	C-Rate of Change: -17.5 kPa/m	Limiting C: 40 kPa	Elevation: 177 m
Name: Middle Clay	Unit Weight: 20 kN/m ³	C-Datum: 40 kPa	C-Rate of Change: 1.3 kPa/m	Limiting C: 45 kPa	Elevation: 165 m
Name: RGM	Unit Weight: 21 kN/m ³	Cohesion: 40 kPa	Phi: 35°		
Name: Clay Backfill	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 0°		
Name: EPS	Unit Weight: 0.5 kN/m ³	Cohesion: 10 kPa	Phi: 0°		
Name: Clayey Silt	Unit Weight: 22 kN/m ³	Cohesion: 65 kPa	Phi: 0°		
Name: RSS (light)	Unit Weight: 12 kN/m ³	Cohesion: 50 kPa	Phi: 35°		
Name: Structure	Unit Weight: 0.5 kN/m ³	Cohesion: 200 kPa	Phi: 0°		
Name: Base-Subbase (Equiv)	Unit Weight: 12 kN/m ³	Cohesion: 0 kPa	Phi: 35°		

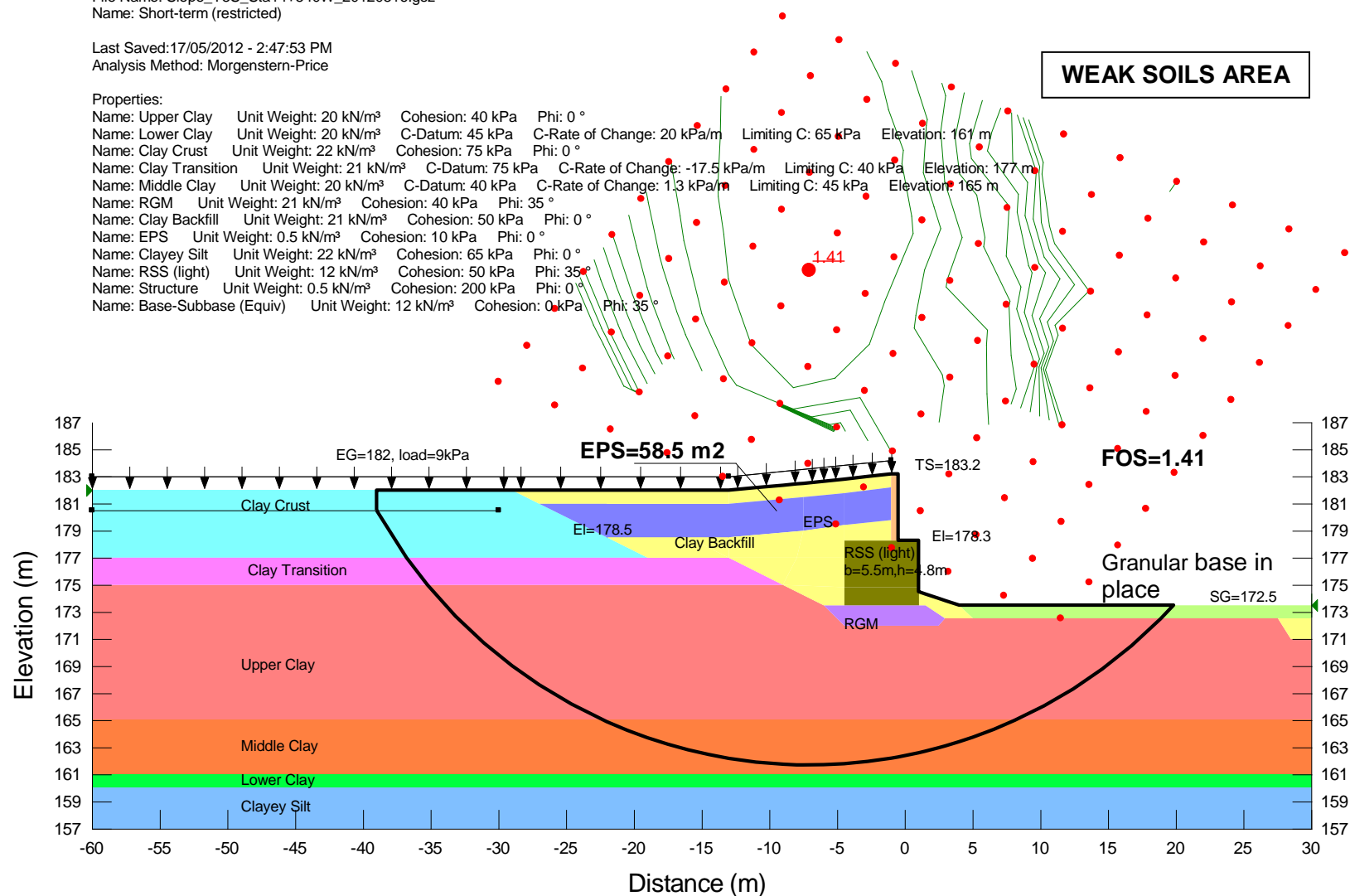


Figure E-8: Slope Stability Result – South Abutment (West Segment) – End of Construction (Undrained) Loading

File Name: Slope_T5S_Sta14+540W_20120516.gsz
Name: End of Construction

Last Saved: 17/05/2012 - 2:47:53 PM
Analysis Method: Morgenstern-Price

Properties:

Name: Upper Clay	Unit Weight: 20 kN/m ³	Cohesion: 40 kPa	Phi: 0°	
Name: Lower Clay	Unit Weight: 20 kN/m ³	C-Datum: 45 kPa	C-Rate of Change: 20 kPa/m	Limiting C: 65 kPa
Name: Clay Crust	Unit Weight: 22 kN/m ³	Cohesion: 75 kPa	Phi: 0°	Elevation: 161 m
Name: Clay Transition	Unit Weight: 21 kN/m ³	C-Datum: 75 kPa	C-Rate of Change: -17.5 kPa/m	Limiting C: 40 kPa
Name: Middle Clay	Unit Weight: 20 kN/m ³	C-Datum: 40 kPa	C-Rate of Change: 1.3 kPa/m	Limiting C: 45 kPa
Name: RGM	Unit Weight: 21 kN/m ³	Cohesion: 40 kPa	Phi: 35°	Elevation: 165 m
Name: Clay Backfill	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 0°	
Name: EPS	Unit Weight: 0.5 kN/m ³	Cohesion: 10 kPa	Phi: 0°	
Name: Clayey Silt	Unit Weight: 22 kN/m ³	Cohesion: 65 kPa	Phi: 0°	
Name: RSS (light)	Unit Weight: 12 kN/m ³	Cohesion: 50 kPa	Phi: 35°	
Name: Structure	Unit Weight: 0.5 kN/m ³	Cohesion: 200 kPa	Phi: 0°	
Name: Pavement	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 35°	

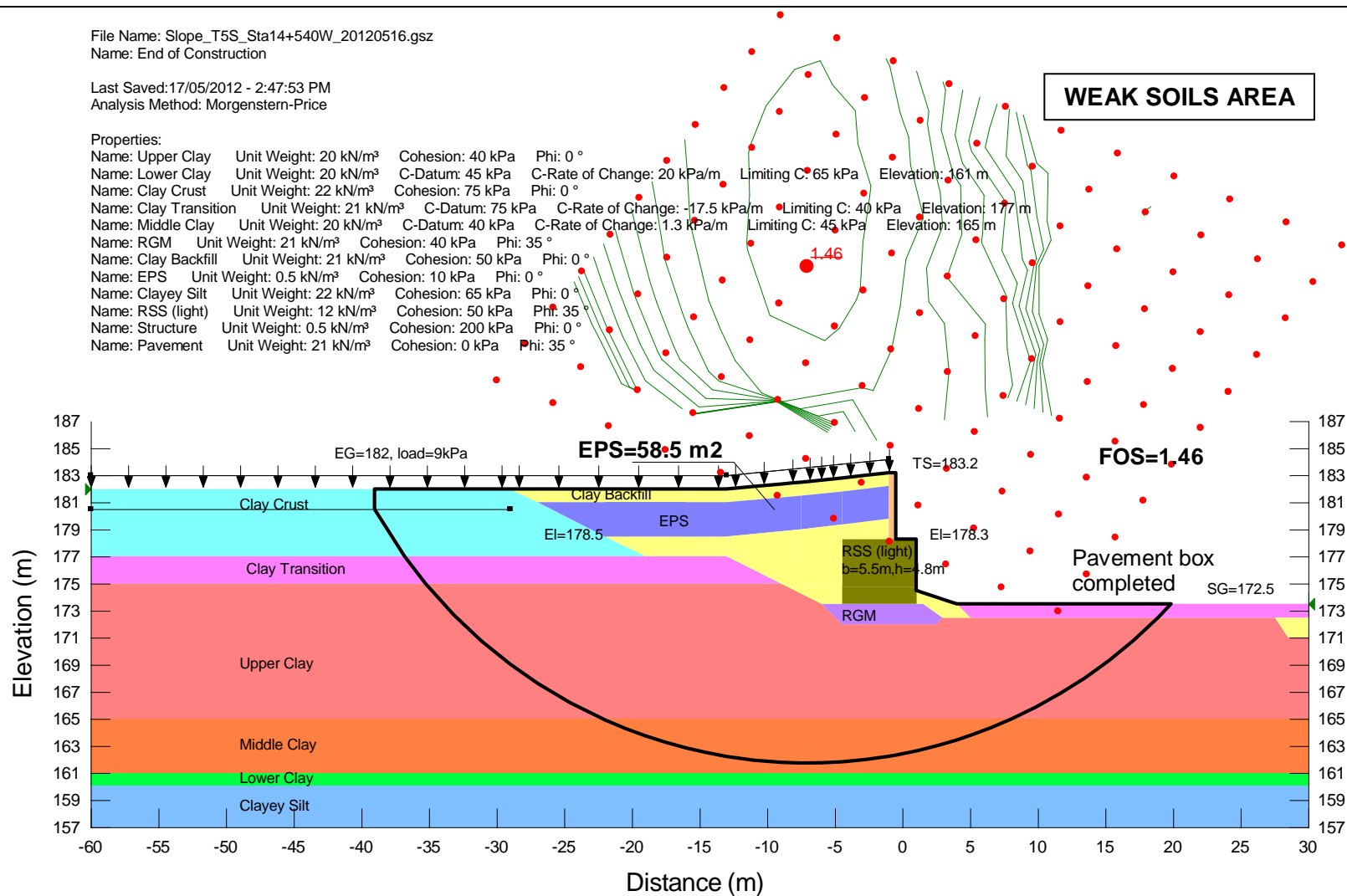


Figure E-9: Slope Stability Result – South Abutment (West Segment) – Long-term (Drained) Loading

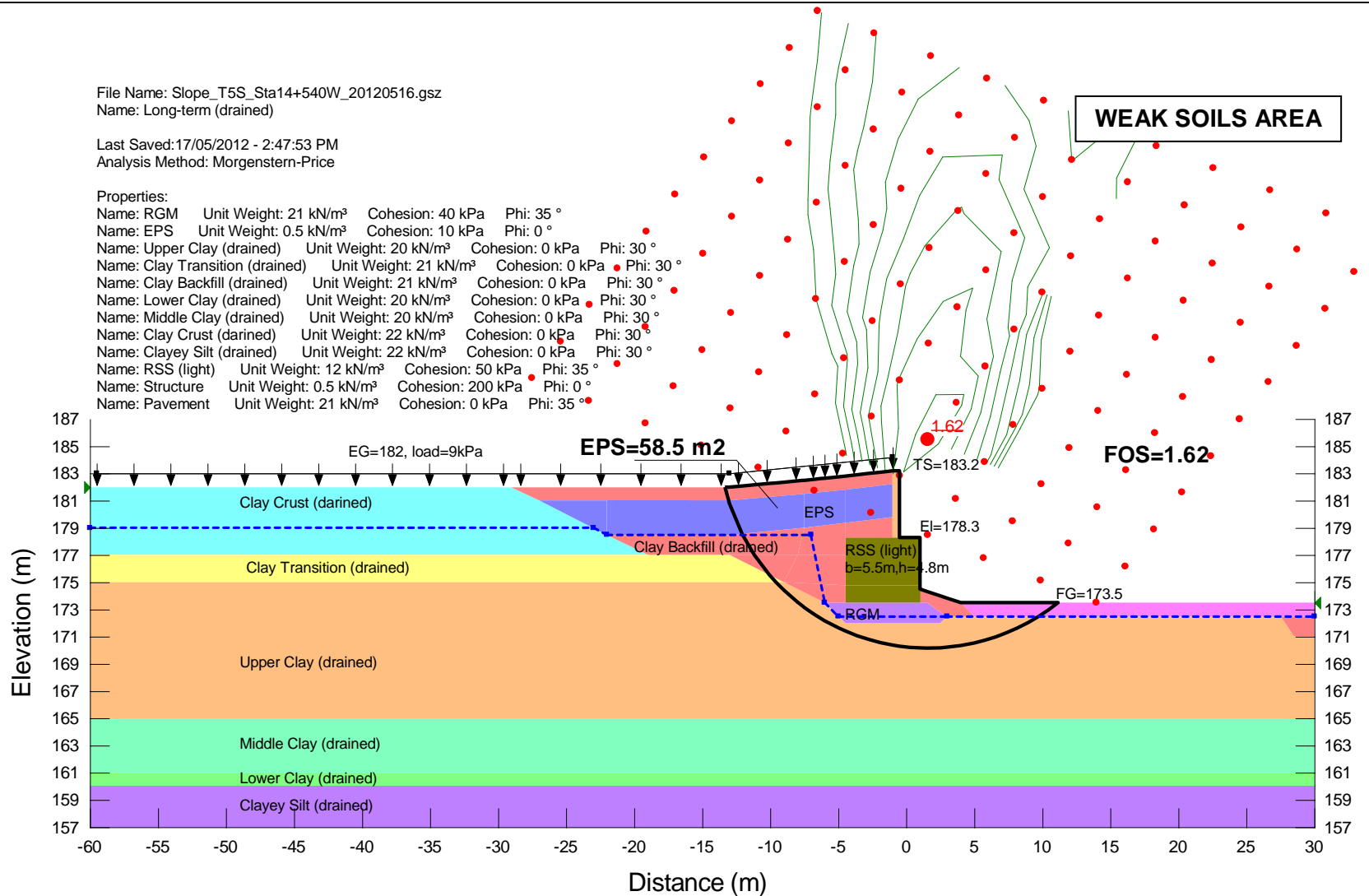


Figure E-10: Slope Stability Result – North West Tapered Abutment Wall – Short-term (Undrained) Loading

File Name: Slope_Tapered-T5NW_long_20120327.gsz

Name: Short-term (restricted)

Last Saved: 18/05/2012 - 4:18:29 PM

Analysis Method: Morgenstern-Price

Properties:

Name: Upper Clay 1	Unit Weight: 20 kN/m ³	C-Datum: 55 kPa	C-Rate of Change: -0.6 kPa/m	Limiting C: 50 kPa	Elevation: 175 m
Name: Lower Clay	Unit Weight: 20 kN/m ³	C-Datum: 50 kPa	C-Rate of Change: 6.7 kPa/m	Limiting C: 70 kPa	Elevation: 163 m
Name: Clay Crust	Unit Weight: 22 kN/m ³	Cohesion: 75 kPa	Phi: 0°		
Name: Clay Transition	Unit Weight: 21 kN/m ³	C-Datum: 75 kPa	C-Rate of Change: -10 kPa/m	Limiting C: 55 kPa	Elevation: 177 m
Name: Middle Clay	Unit Weight: 20 kN/m ³	C-Datum: 42 kPa	C-Rate of Change: 3.5 kPa/m	Limiting C: 50 kPa	Elevation: 165.5 m
Name: RSS	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 35°		
Name: RGM	Unit Weight: 21 kN/m ³	Cohesion: 40 kPa	Phi: 35°		
Name: Clay Backfill	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 0°		
Name: Clayey Silt	Unit Weight: 22 kN/m ³	Cohesion: 70 kPa	Phi: 0°		
Name: RSS (light)	Unit Weight: 12 kN/m ³	Cohesion: 50 kPa	Phi: 35°		
Name: Upper Clay 2	Unit Weight: 20 kN/m ³	Cohesion: 42 kPa	Phi: 0°		
Name: Base-Subbase (Equiv)	Unit Weight: 12 kN/m ³	Cohesion: 0 kPa	Phi: 35°		

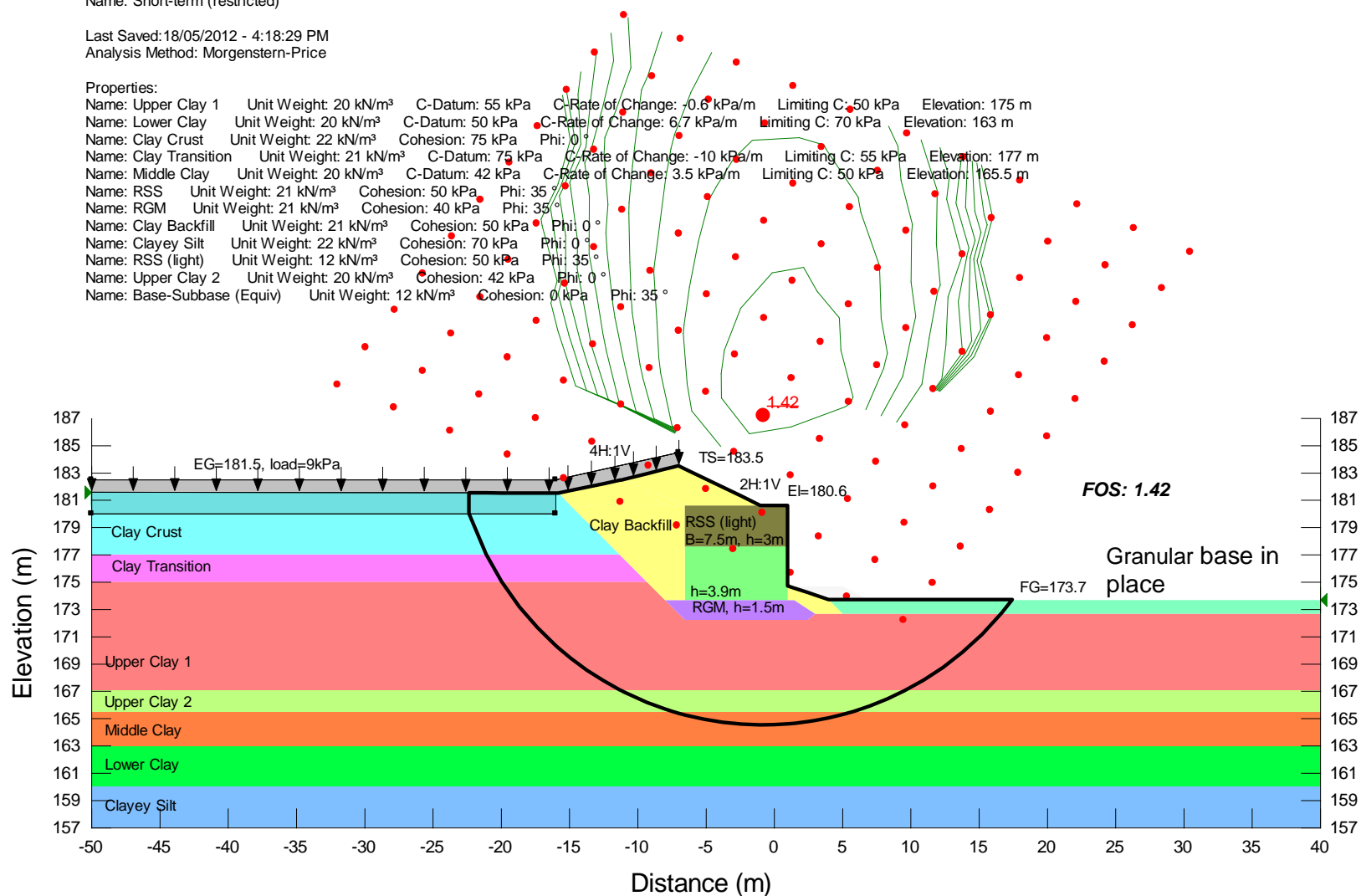


Figure E-11: Slope Stability Result – North West Tapered Abutment Wall – Long-term (Drained) Loading

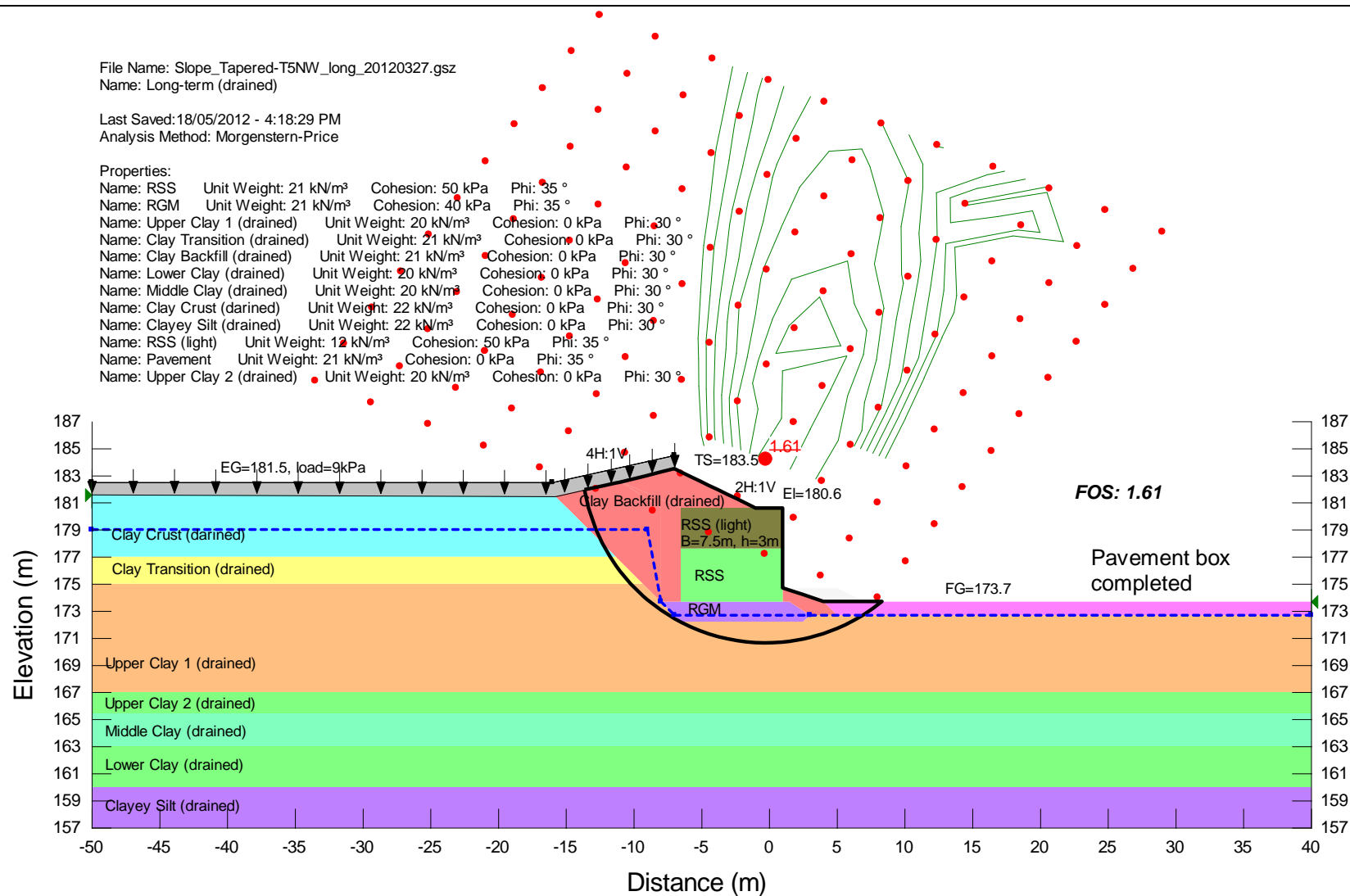


Figure E-12: Slope Stability Result – North East Tapered Abutment Wall – Short-term (Undrained) Loading

File Name: Slope_Tapered-T5NE-long_20120313.gsz
Name: Short-term

Last Saved: 18/05/2012 - 4:09:46 PM
Analysis Method: Morgenstern-Price

Properties:

Name: Upper Clay 1	Unit Weight: 20 kN/m ³	C-Datum: 55 kPa	C-Rate of Change: -0.6 kPa/m	Limiting C: 50 kPa	Elevation: 175 m
Name: Lower Clay	Unit Weight: 20 kN/m ³	C-Datum: 50 kPa	C-Rate of Change: 6.7 kPa/m	Limiting C: 70 kPa	Elevation: 163 m
Name: Clay Crust	Unit Weight: 22 kN/m ³	Cohesion: 75 kPa	Phi: 0°		
Name: Clay Transition	Unit Weight: 21 kN/m ³	C-Datum: 75 kPa	C-Rate of Change: -10 kPa/m	Limiting C: 55 kPa	Elevation: 177 m
Name: Middle Clay	Unit Weight: 20 kN/m ³	C-Datum: 42 kPa	C-Rate of Change: 3.5 kPa/m	Limiting C: 50 kPa	Elevation: 165.5 m
Name: RSS	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 85°		
Name: RGM	Unit Weight: 21 kN/m ³	Cohesion: 40 kPa	Phi: 35°		
Name: Clay Backfill	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 0°		
Name: Clayey Silt	Unit Weight: 22 kN/m ³	Cohesion: 70 kPa	Phi: 0°		
Name: Upper Clay 2	Unit Weight: 20 kN/m ³	Cohesion: 42 kPa	Phi: 0°		

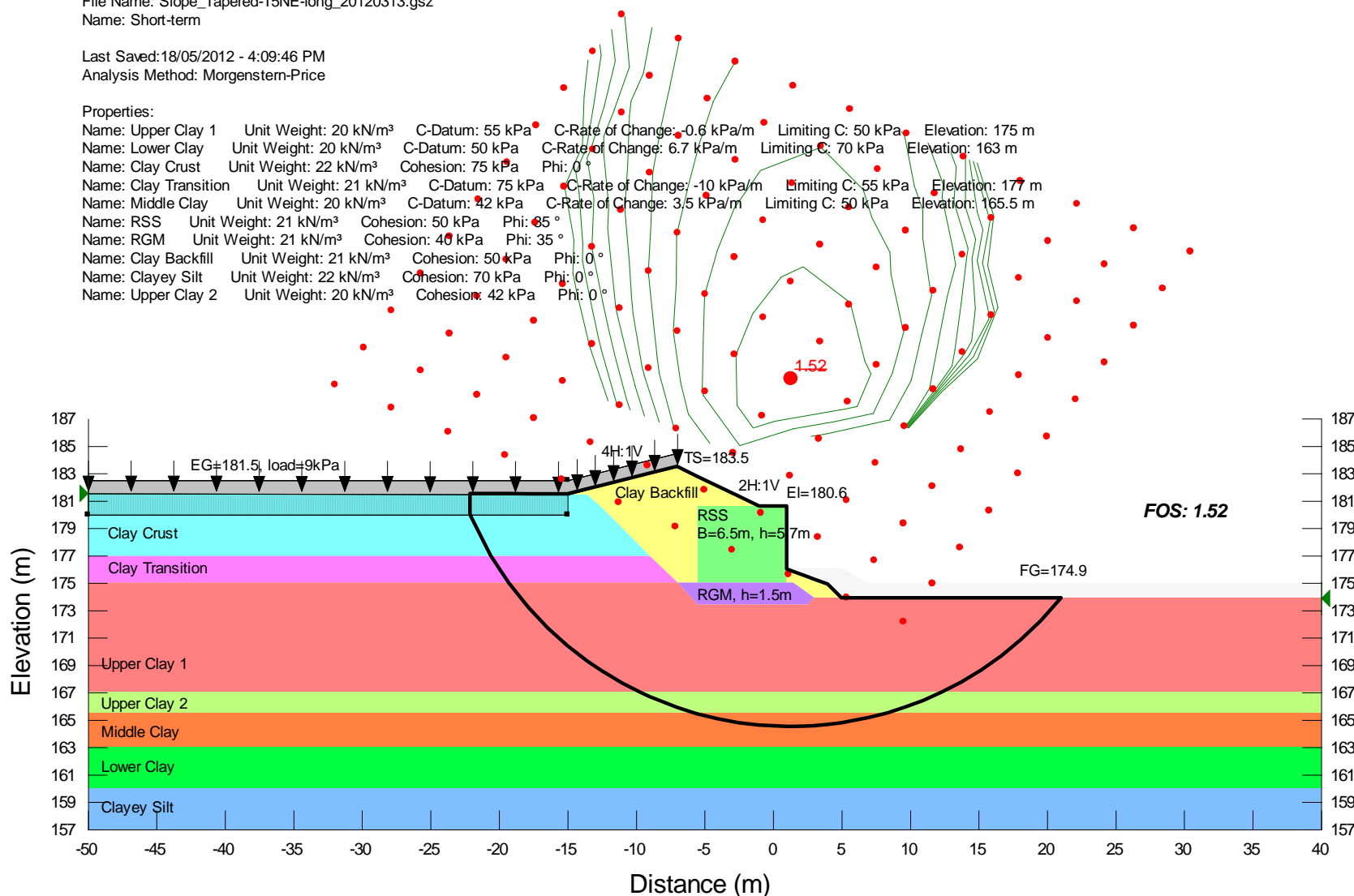


Figure E-13: Slope Stability Result – North East Tapered Abutment Wall – Long-term (Drained) Loading

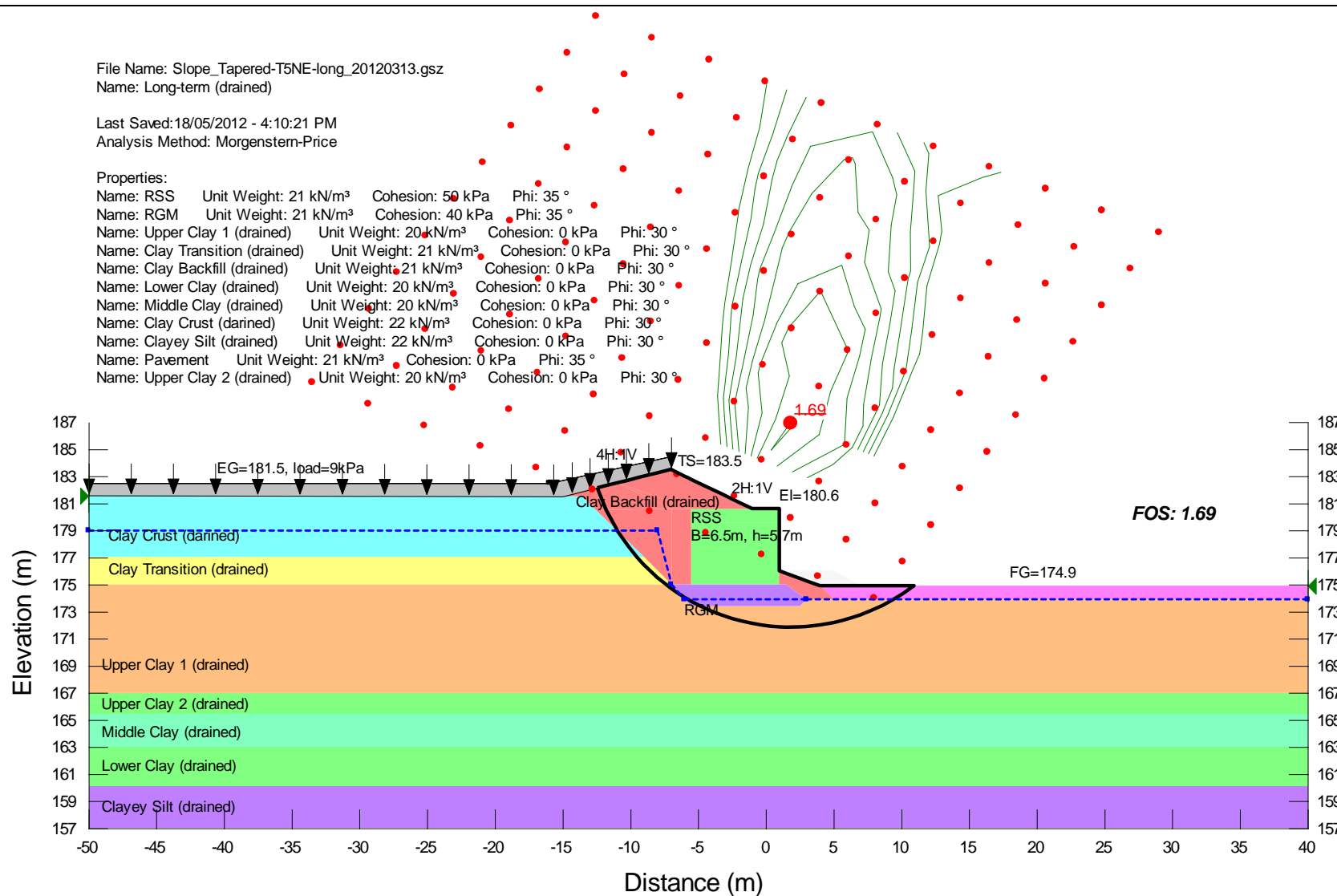


Figure E-14: Slope Stability Result – South West Tapered Abutment Wall – Short-term (Undrained) Loading

File Name: Slope_Tapered-T5SW_long_20120225.gsz
Name: Short-term

Last Saved: 18/05/2012 - 4:02:24 PM
Analysis Method: Morgenstern-Price

Properties:

Name: Upper Clay	Unit Weight: 20 kN/m ³	Cohesion: 40 kPa	Phi: 0°
Name: Lower Clay	Unit Weight: 20 kN/m ³	C-Datum: 45 kPa	C-Rate of Change: 20 kPa/m
Name: Clay Crust	Unit Weight: 22 kN/m ³	Cohesion: 75 kPa	Phi: 0°
Name: Clay Transition	Unit Weight: 21 kN/m ³	C-Datum: 75 kPa	C-Rate of Change: -17.5 kPa/m
Name: Middle Clay	Unit Weight: 20 kN/m ³	C-Datum: 40 kPa	C-Rate of Change: 1.3 kPa/m
Name: RSS	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 35°
Name: RGM	Unit Weight: 21 kN/m ³	Cohesion: 40 kPa	Phi: 35°
Name: Clay Backfill	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 0°
Name: EPS	Unit Weight: 0.5 kN/m ³	Cohesion: 10 kPa	Phi: 0°
Name: Clayey Silt	Unit Weight: 22 kN/m ³	Cohesion: 65 kPa	Phi: 0°
Name: RSS (light)	Unit Weight: 12 kN/m ³	Cohesion: 50 kPa	Phi: 35°

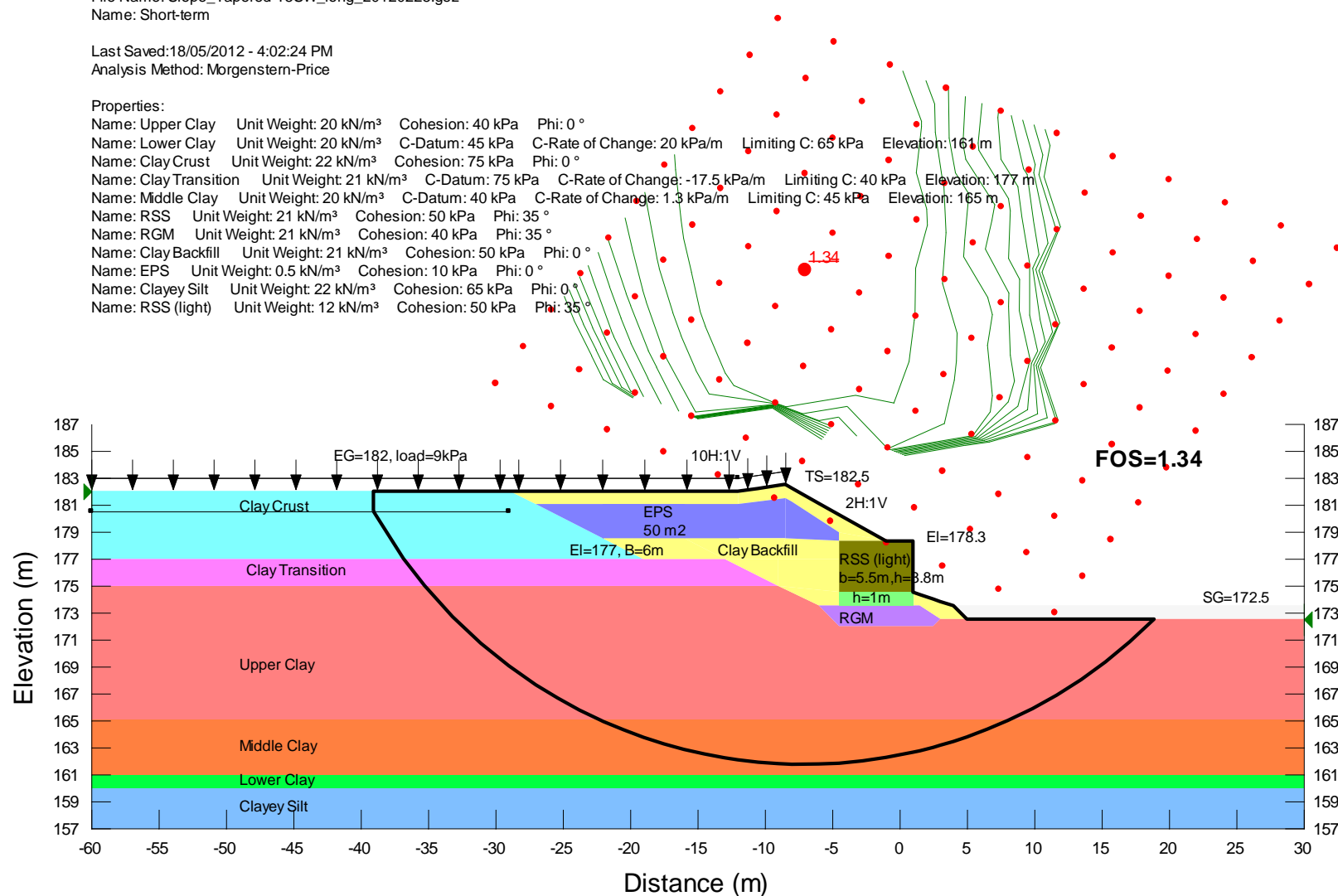


Figure E-15: Slope Stability Result – South West Tapered Abutment Wall – Long-term (Drained) Loading

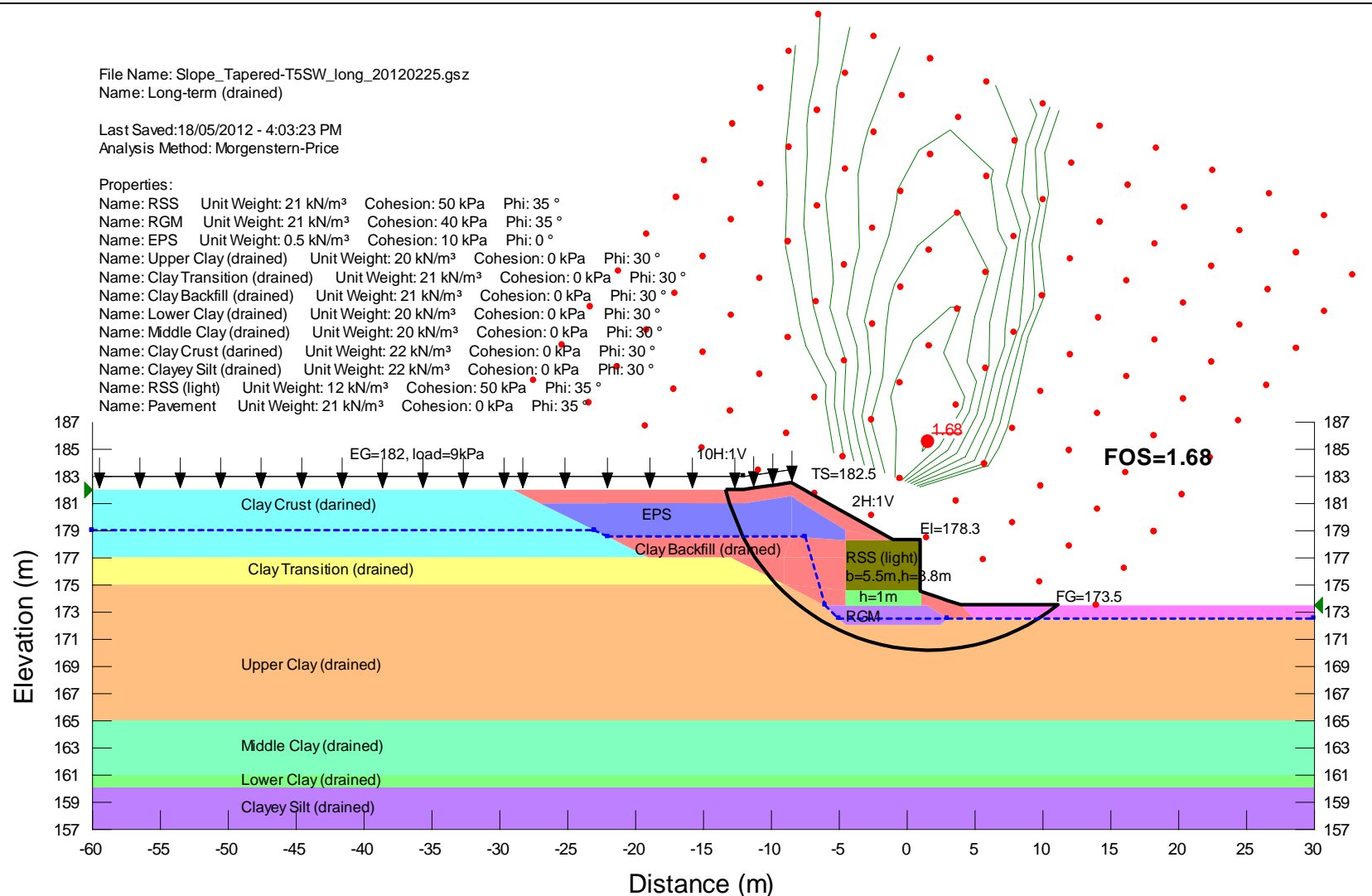


Figure E-16: Slope Stability Result – South East Tapered Abutment Wall – Short-term (Undrained) Loading

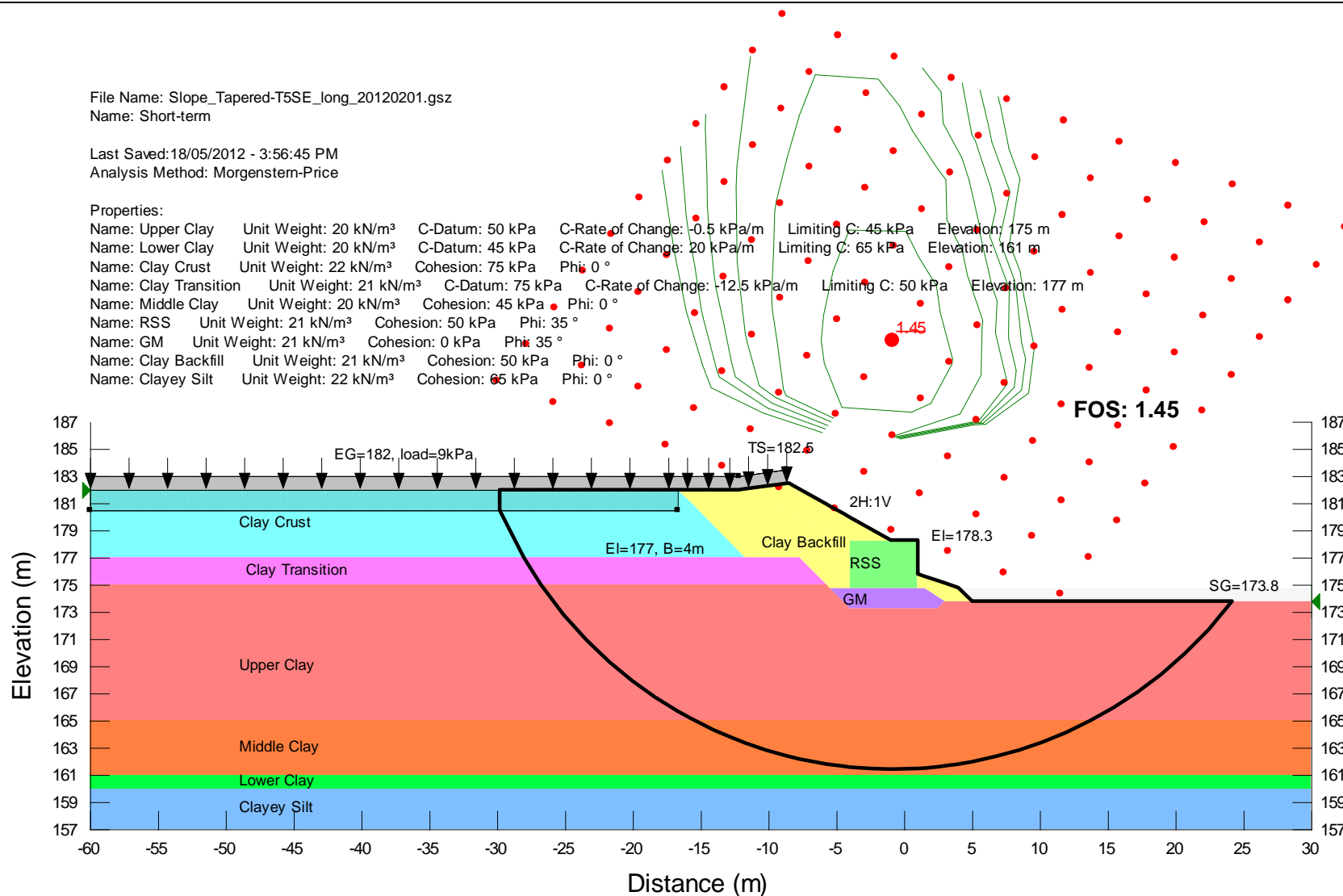


Figure E-17: Slope Stability Result – South East Tapered Abutment Wall – Long-term (Drained) Loading

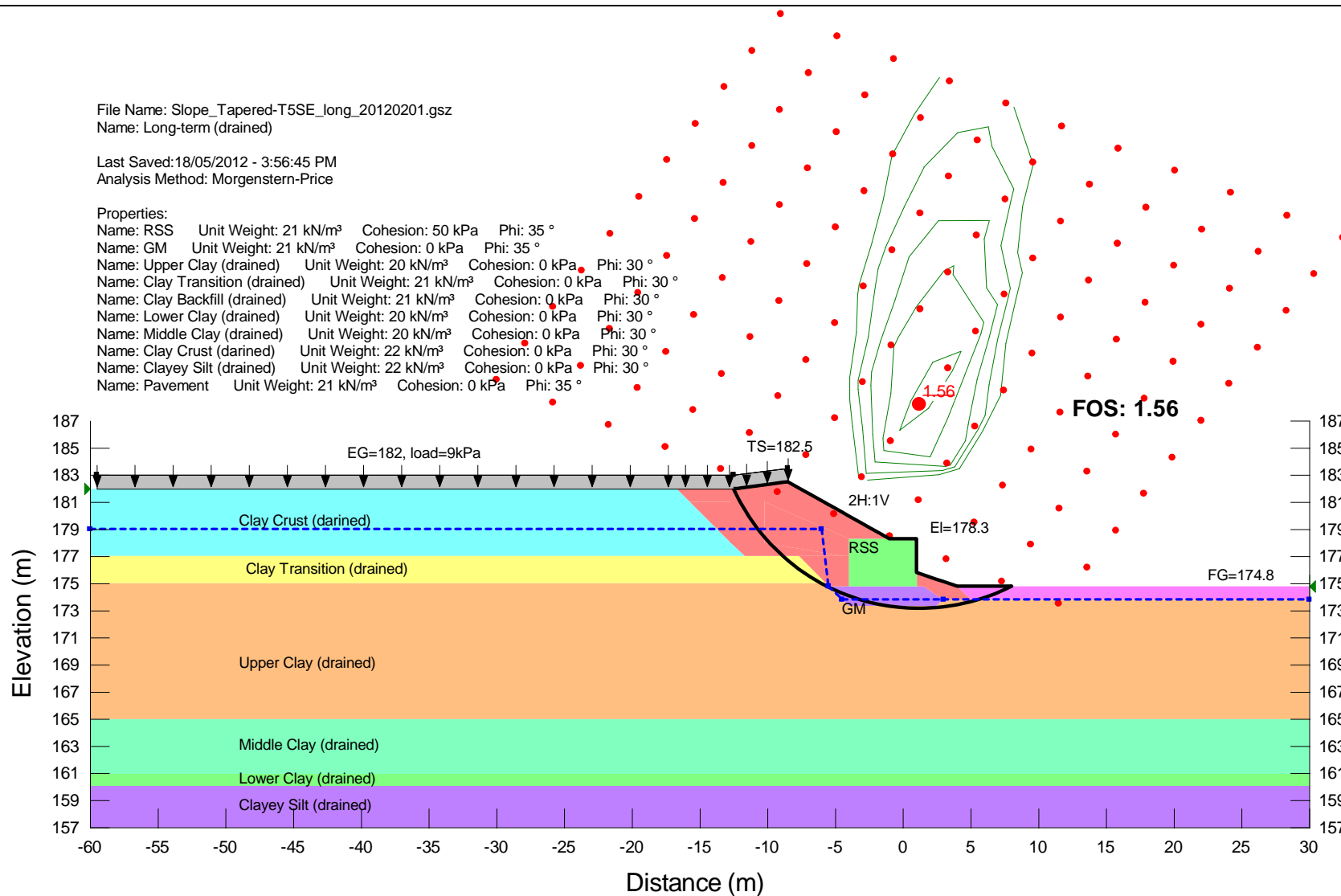
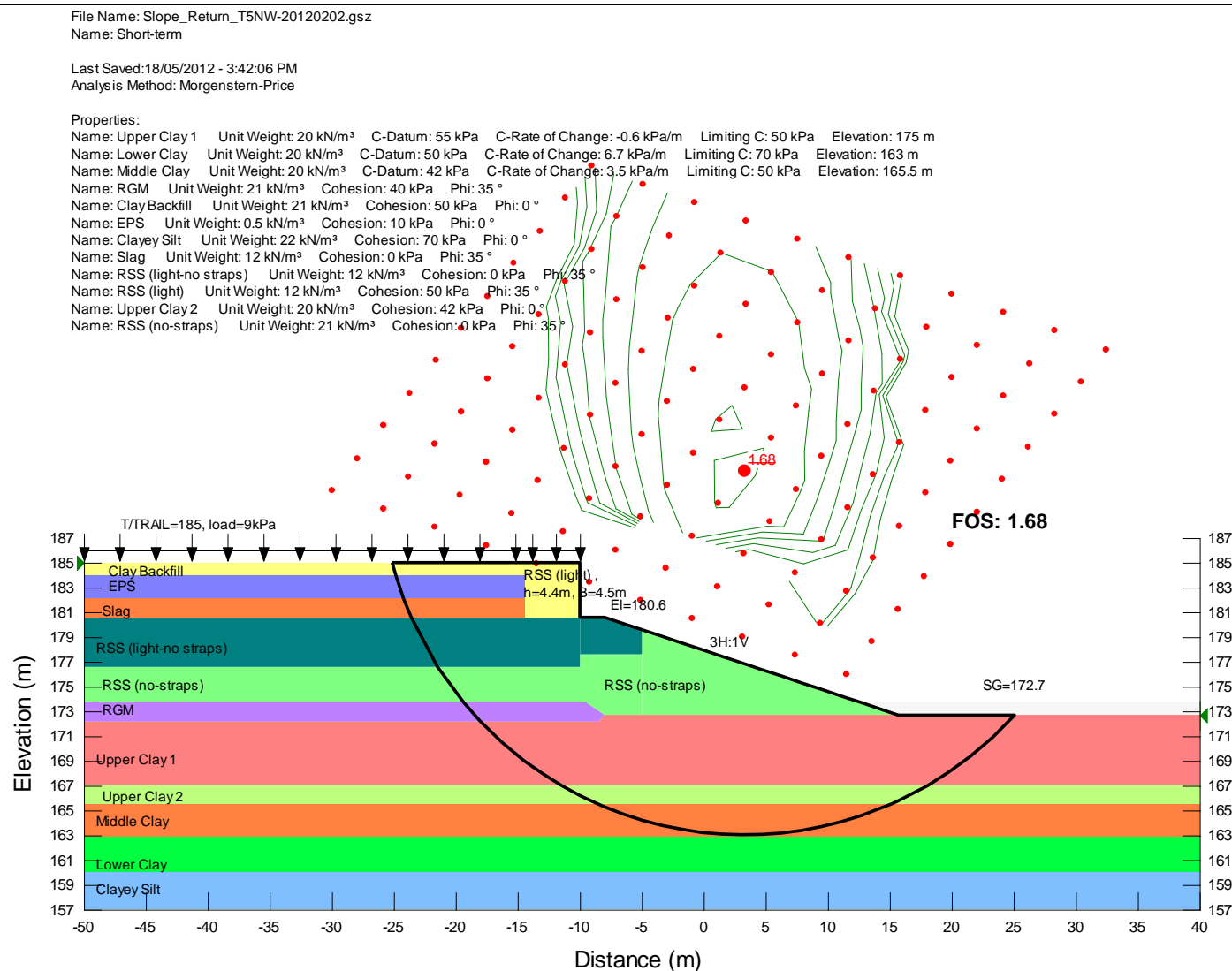


Figure E-18: Slope Stability Result – North Abutment Return Wall – Short-term (Undrained) Loading



Note: RSS (no-strap) was used to simulate unreinforced material due to one-dimensional behaviour of straps in the RSS block.

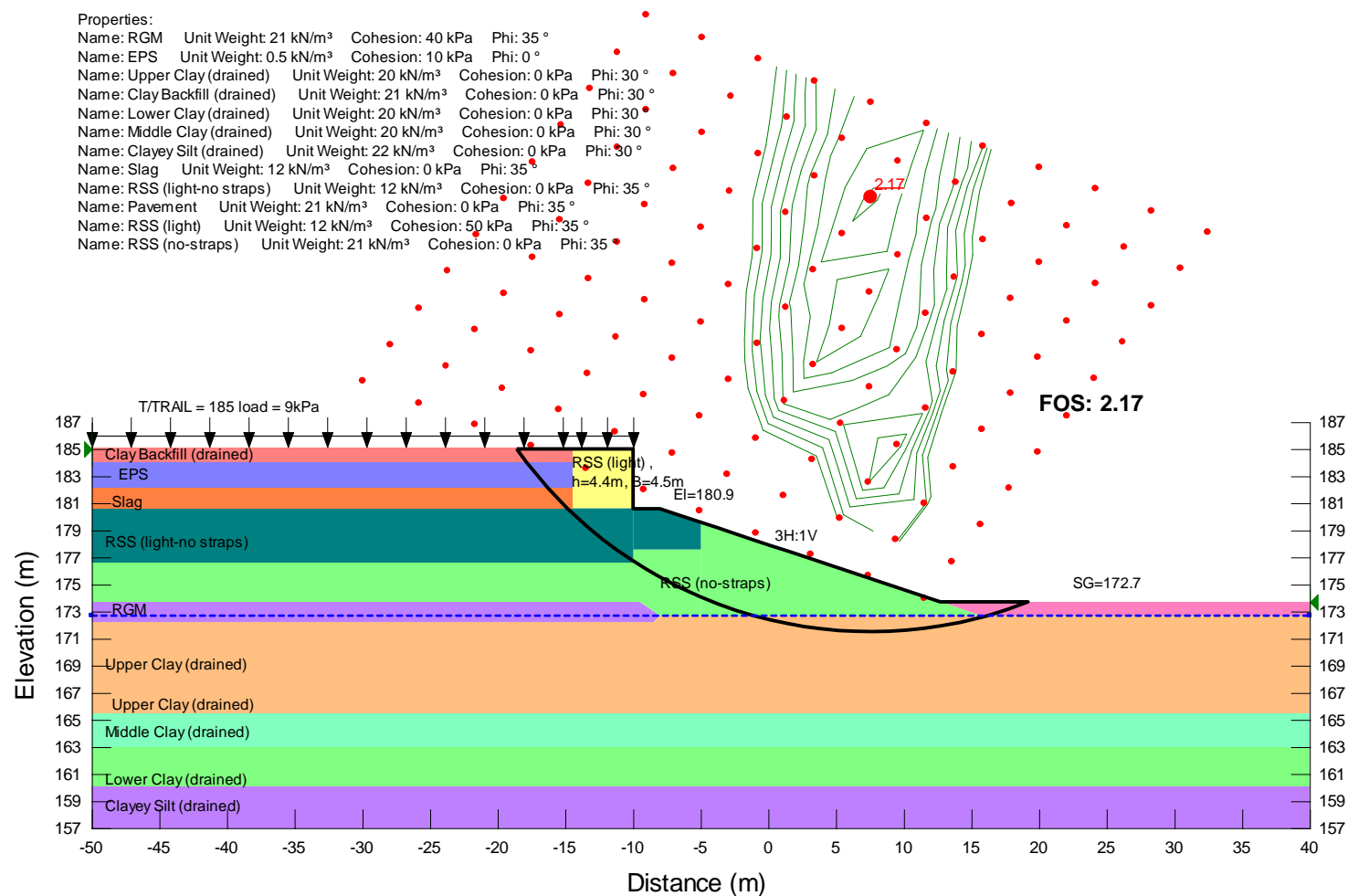
Figure E-19: Slope Stability Result – North Abutment Return Wall – Long-term (Drained) Loading

File Name: Slope_Return_T5NW-20120202.gsz
Name: Long-term (drained)

Last Saved: 18/05/2012 - 3:42:06 PM
Analysis Method: Morgenstern-Price

Properties:

Name: RGM Unit Weight: 21 kN/m³ Cohesion: 40 kPa Phi: 35 °
Name: EPS Unit Weight: 0.5 kN/m³ Cohesion: 10 kPa Phi: 0 °
Name: Upper Clay (drained) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clay Backfill (drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Lower Clay (drained) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Middle Clay (drained) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clayey Silt (drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Slag Unit Weight: 12 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS (light-no straps) Unit Weight: 12 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Pavement Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS (light) Unit Weight: 12 kN/m³ Cohesion: 50 kPa Phi: 35 °
Name: RSS (no-strap) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °



For RSS (no-strap) see Note on Figure E-18

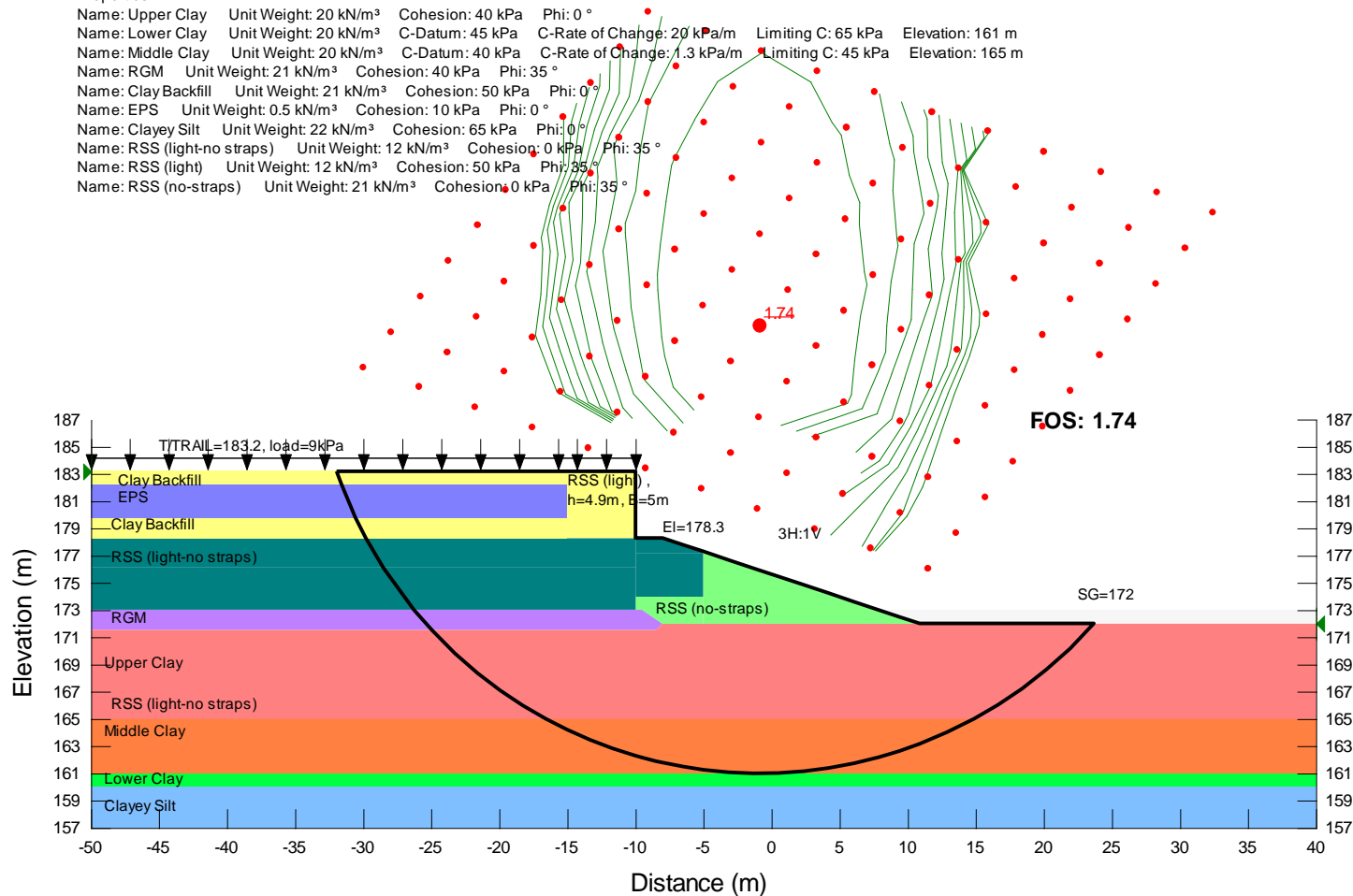
Figure E-20: Slope Stability Result – South Abutment Return Wall – Short-term (Undrained) Loading

File Name: Slope_Return_T5SW-20120202.gsz
Name: Short-term

Last Saved: 18/05/2012 - 3:42:04 PM
Analysis Method: Morgenstern-Price

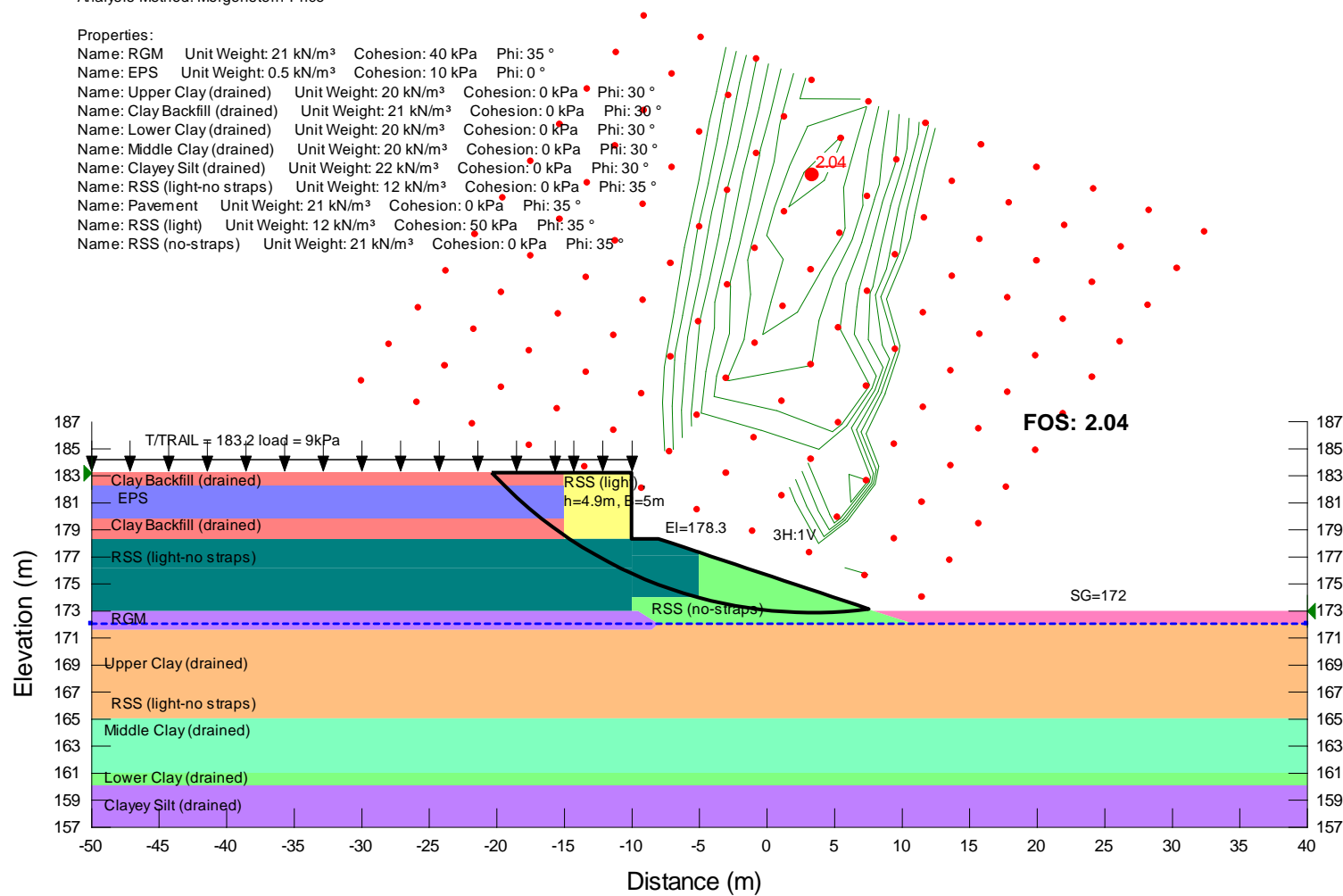
Properties:

Name: Upper Clay	Unit Weight: 20 kN/m ³	Cohesion: 40 kPa	Phi: 0°
Name: Lower Clay	Unit Weight: 20 kN/m ³	C-Datum: 45 kPa	C-Rate of Change: 20 kPa/m
Name: Middle Clay	Unit Weight: 20 kN/m ³	C-Datum: 40 kPa	C-Rate of Change: 1.3 kPa/m
Name: RGM	Unit Weight: 21 kN/m ³	Cohesion: 40 kPa	Phi: 35°
Name: Clay Backfill	Unit Weight: 21 kN/m ³	Cohesion: 50 kPa	Phi: 0°
Name: EPS	Unit Weight: 0.5 kN/m ³	Cohesion: 10 kPa	Phi: 0°
Name: Clayey Silt	Unit Weight: 22 kN/m ³	Cohesion: 65 kPa	Phi: 0°
Name: RSS (light-no straps)	Unit Weight: 12 kN/m ³	Cohesion: 0 kPa	Phi: 35°
Name: RSS (light)	Unit Weight: 12 kN/m ³	Cohesion: 50 kPa	Phi: 35°
Name: RSS (no-strap)	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 35°



For RSS (no-strap) see Note on Figure E-18

Name: RGM	Unit Weight: 21 kN/m ³	Cohesion: 40 kPa	Phi: 35 °	
Name: EPS	Unit Weight: 0.5 kN/m ³	Cohesion: 10 kPa	Phi: 0 °	
Name: Upper Clay (drained)	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	
Name: Clay Backfill (drained)	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	
Name: Lower Clay (drained)	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	
Name: Middle Clay (drained)	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	
Name: Clayey Silt (drained)	Unit Weight: 22 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	
Name: RSS (light no straps)	Unit Weight: 12 kN/m ³	Cohesion: 0 kPa	Phi: 35 °	
Name: Pavement	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 35 °	
Name: RSS (light)	Unit Weight: 12 kN/m ³	Cohesion: 50 kPa	Phi: 35 °	
Name: RSS (no-straps)	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 35 °	



For RSS (no-strip) see Note on Figure E-18

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

Date: May / 2012
Rev: 0
Page No.: Appendix E 21 of 21

Appendix F: Stress-Deformation Analysis Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

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

Figure F-1: Cumulative Heave/Settlement – End of Construction

Roadway - Elastic Plastic and MCC
Tunnel T-5 Typical Abutment
Last Solved Date: 8/9/2011

End of Construction

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Sand Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 25000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Upper Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Lower Clay (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 20 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: RGM Backfill Model: Linear Elastic Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: RSS Backfill (light) Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 14 kN/m³ Poisson's Ratio: 0.35
Name: Backfill (light) Model: Elastic-Plastic Young's Modulus (E): 30000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 14 kN/m³
Name: Backfill (Drained) Model: Elastic-Plastic Young's Modulus (E): 20000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³

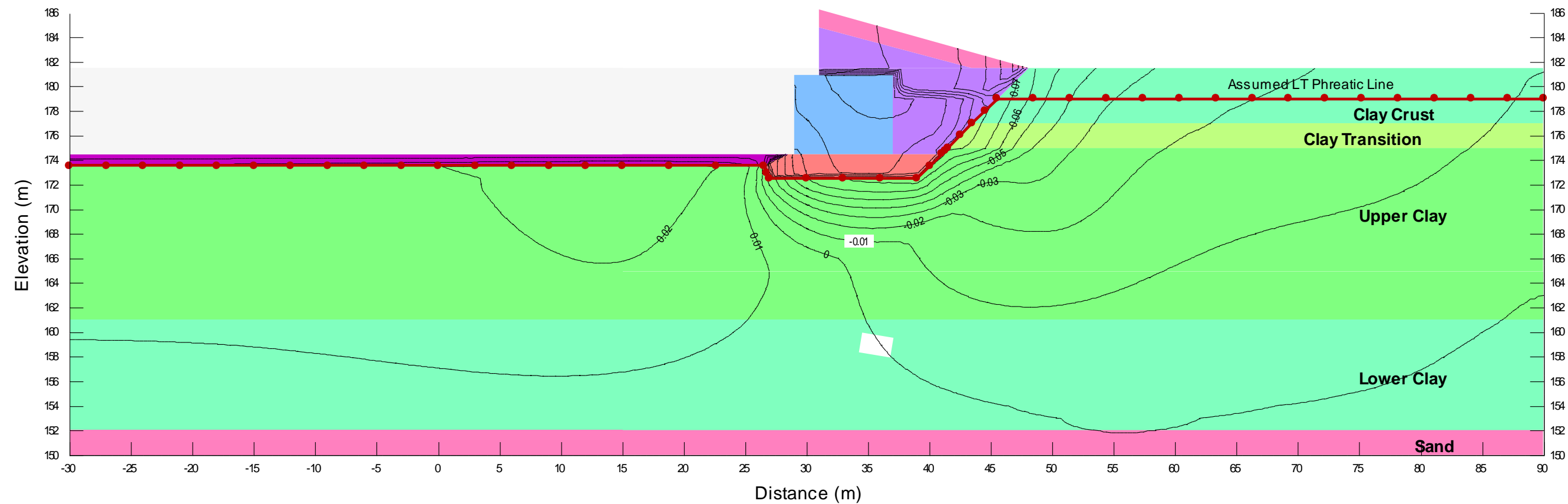


Figure F-2: Cumulative Heave/Settlement – Long-term (Drained)

Dissipation- Elastic Plastic and MCC
Tunnel T-5 Typical Abutment
Last Solved Date: 8/9/2011

Long-term (Drained)

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 22 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Sand Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 25000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Upper Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi: 26 ° K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Lower Clay (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 20 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: RGM Backfill Model: Linear Elastic Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: RSS Backfill (light) Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 14 kN/m³ Poisson's Ratio: 0.35
Name: Backfill (light) Model: Elastic-Plastic Young's Modulus (E): 30000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 14 kN/m³
Name: Backfill (Drained) Model: Elastic-Plastic Young's Modulus (E): 20000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³

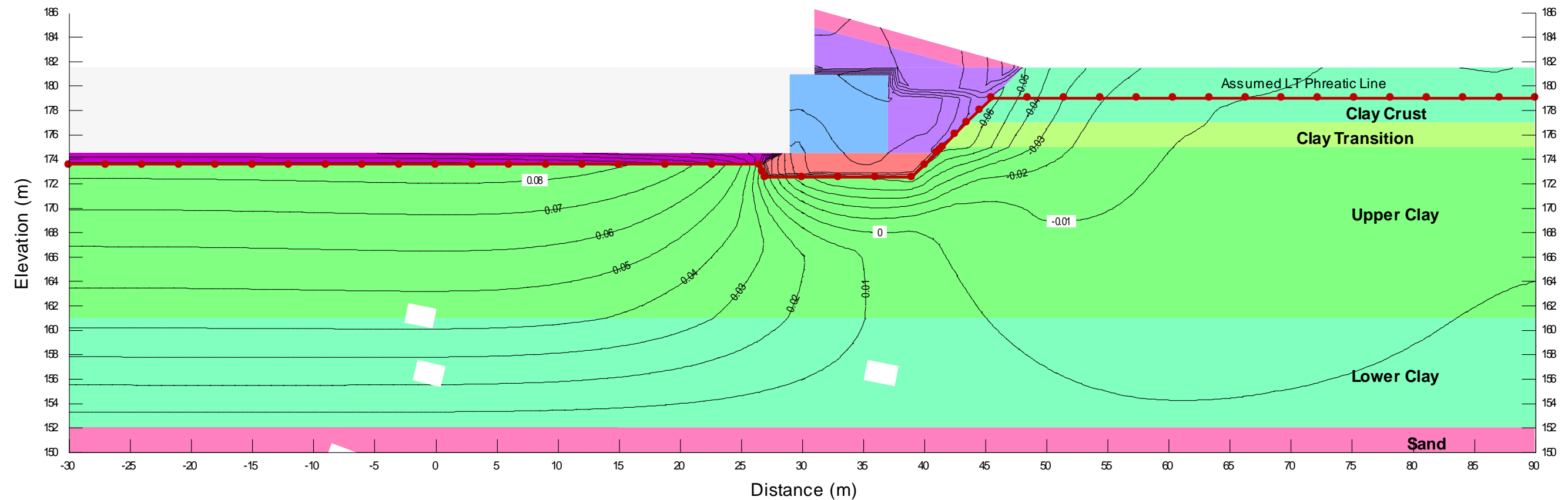


Figure F-3: Stabilized Porewater Pressure Contours – Long-term (Drained)

Dissipation- Elastic Plastic and MCC
Tunnel T-5 Typical Abutment
Last Solved Date: 8/9/2011

Long-term (Drained)

Name: Pavement Model: Linear Elastic Young's Modulus (E): 50000 kPa Unit Weight: 22 kN/m³ Poisson's Ratio: 0.25
Name: Clay Crust (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³ K-Ratio: 1 K-Function: Conductivity_Crust
Name: Sand Model: Elastic-Plastic Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Clay Transition (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 25000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Upper Clay (Drained) Model: Soft Clay (MCC) O.C. Ratio: 1.2 Poisson's Ratio: 0.35 Lambda: 0.082 Kappa: 0.009 Initial Void Ratio: 0.6 Unit Weight: 21 kN/m³ Phi': 26 ° K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: Lower Clay (Drained) Model: Elastic-Plastic Effective Young's Modulus (E'): 18000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 20 kN/m³ K-Ratio: 0.5 K-Function: Conductivity_Unweathered
Name: RGM Backfill Model: Linear Elastic Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: RSS Backfill (light) Model: Linear Elastic Young's Modulus (E): 40000 kPa Unit Weight: 14 kN/m³ Poisson's Ratio: 0.35
Name: Backfill (light) Model: Elastic-Plastic Young's Modulus (E): 30000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 14 kN/m³
Name: Backfill (Drained) Model: Elastic-Plastic Young's Modulus (E): 20000 kPa Poisson's Ratio: 0.35 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³

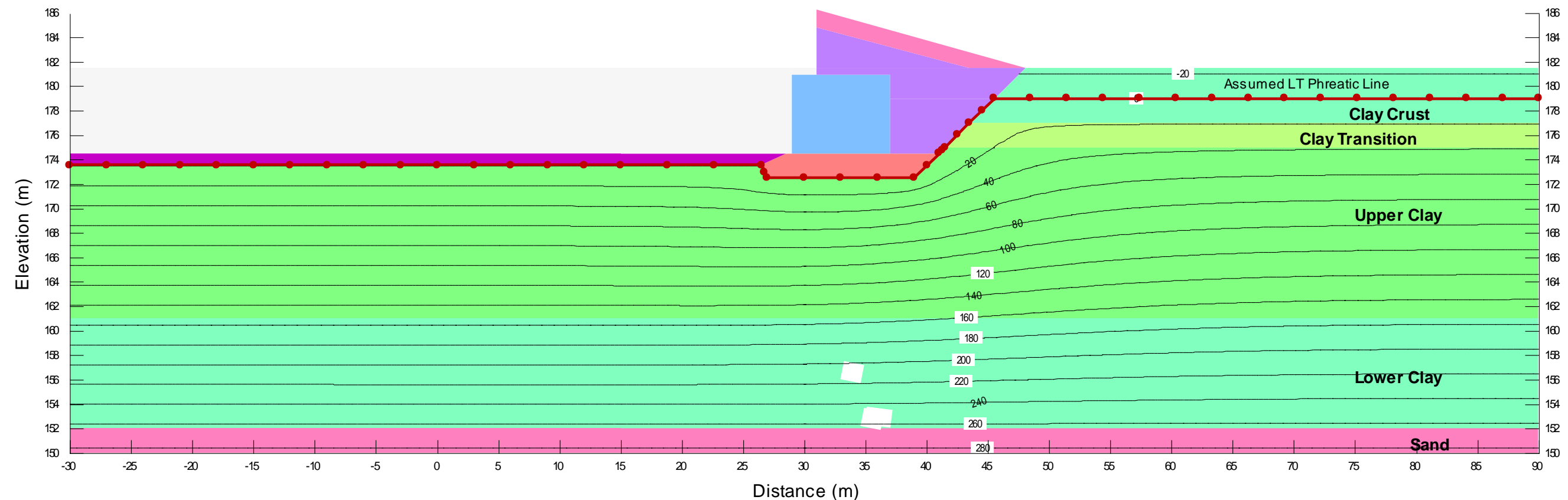
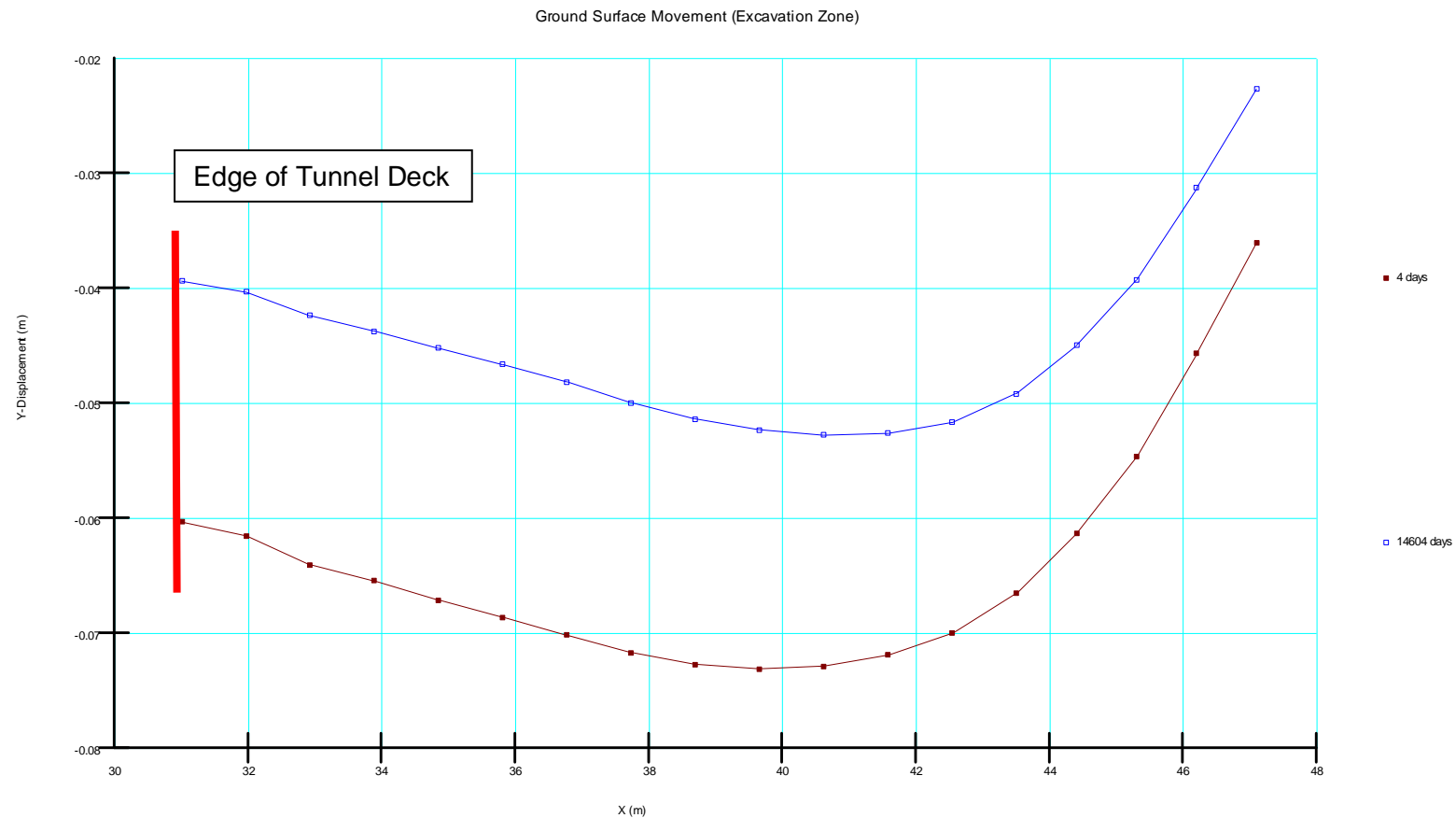
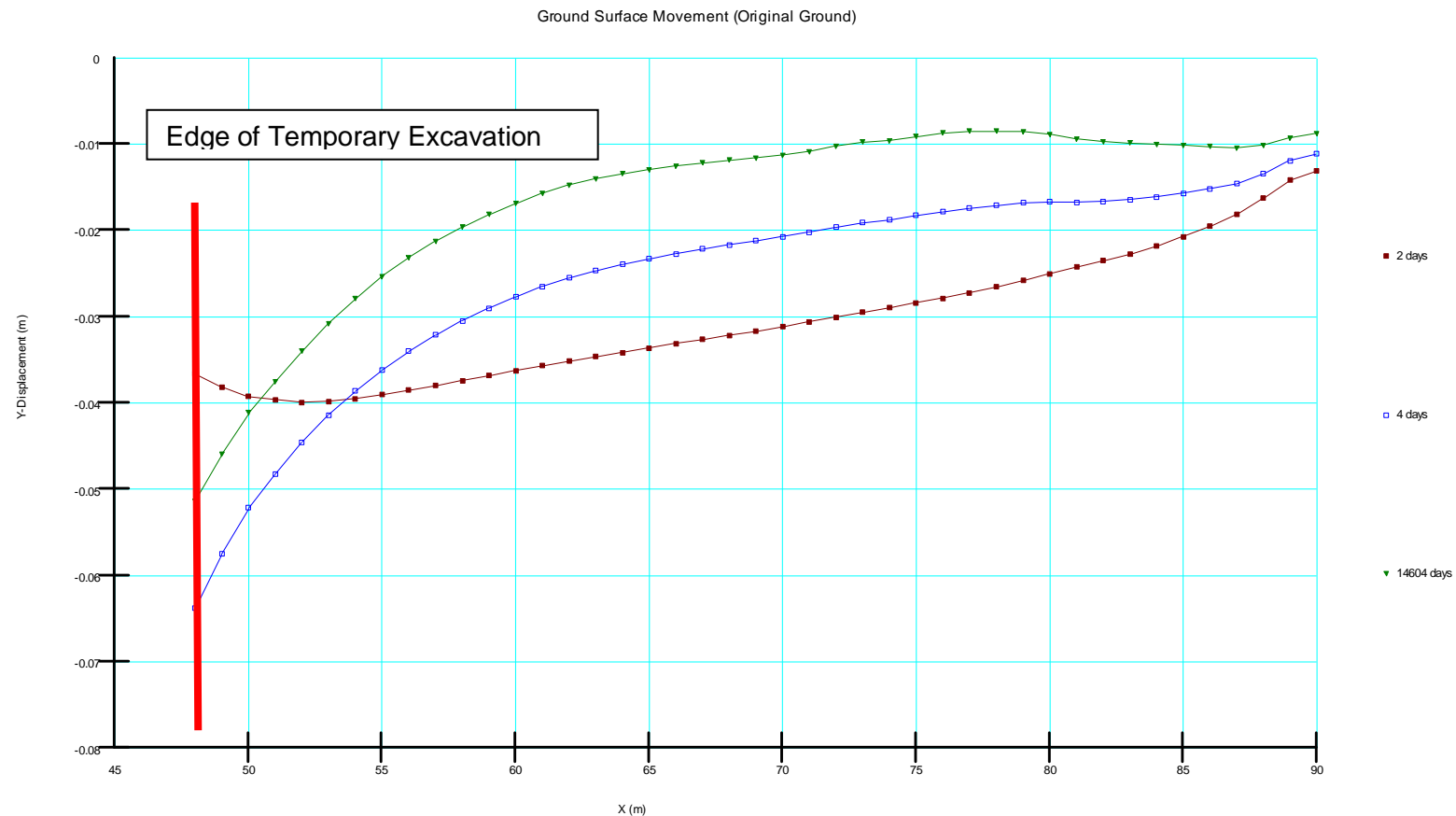


Figure F-4: Cumulative Ground Settlement at Approachway - Excavation Zone



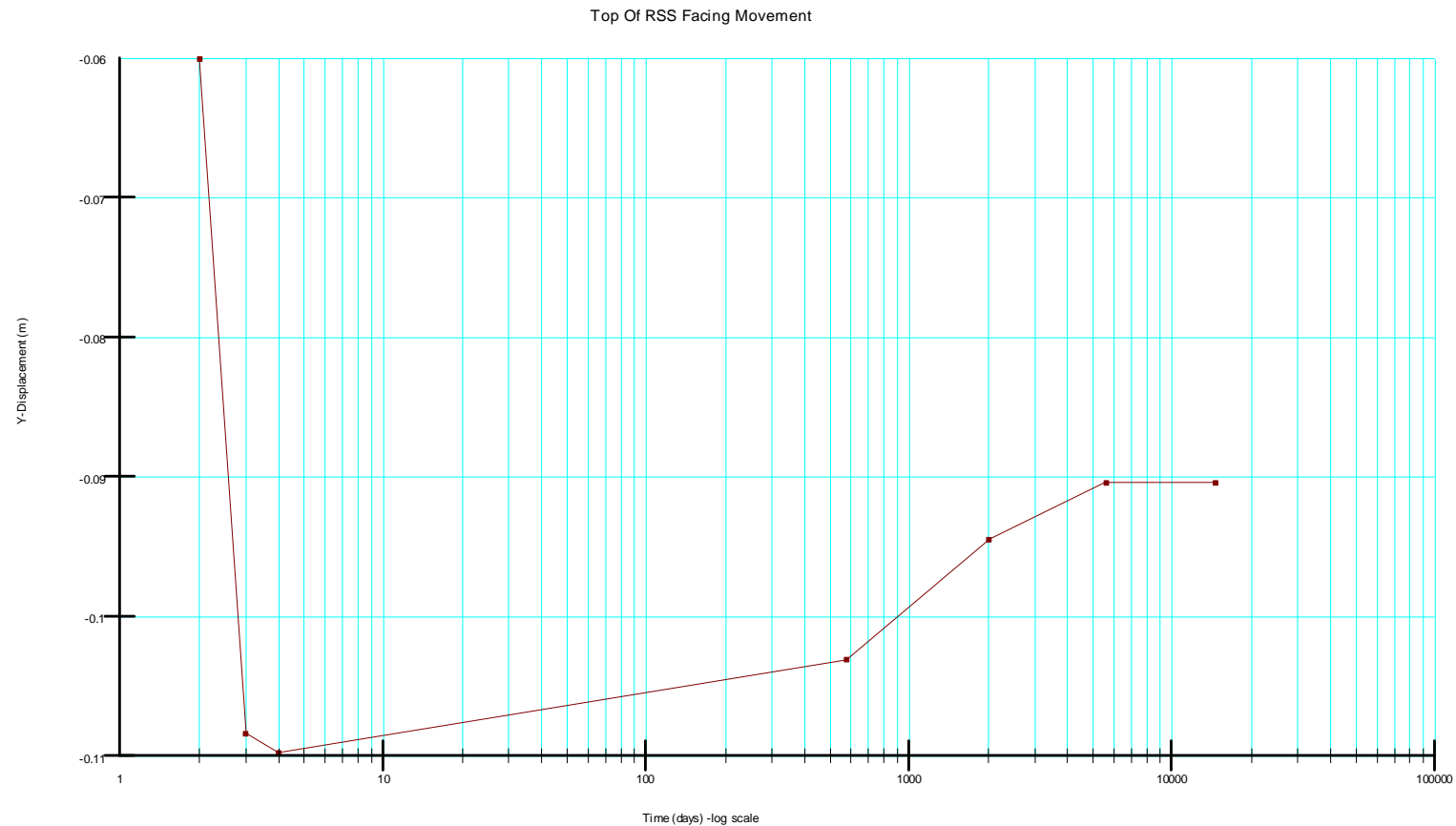
4 days = End of Construction
14604 days = Long-term Condition

Figure F-5: Cumulative Ground Settlement at Approachway – Original Ground



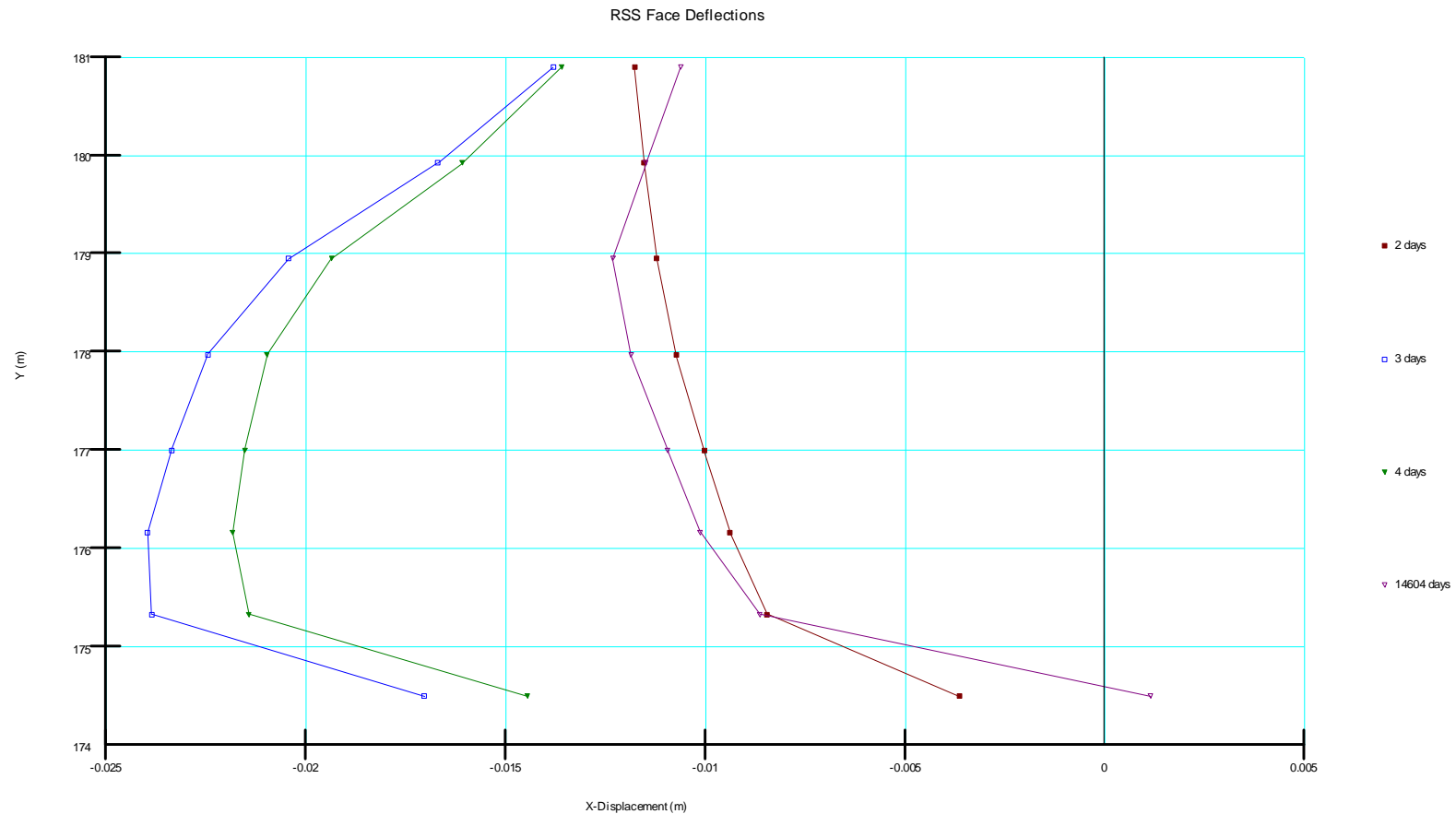
2 days = RSS Completion
 4 days = End of Construction
 14604 days = Long-term Condition

Figure F-6: Cumulative Settlement at Top of RSS Wall Facing



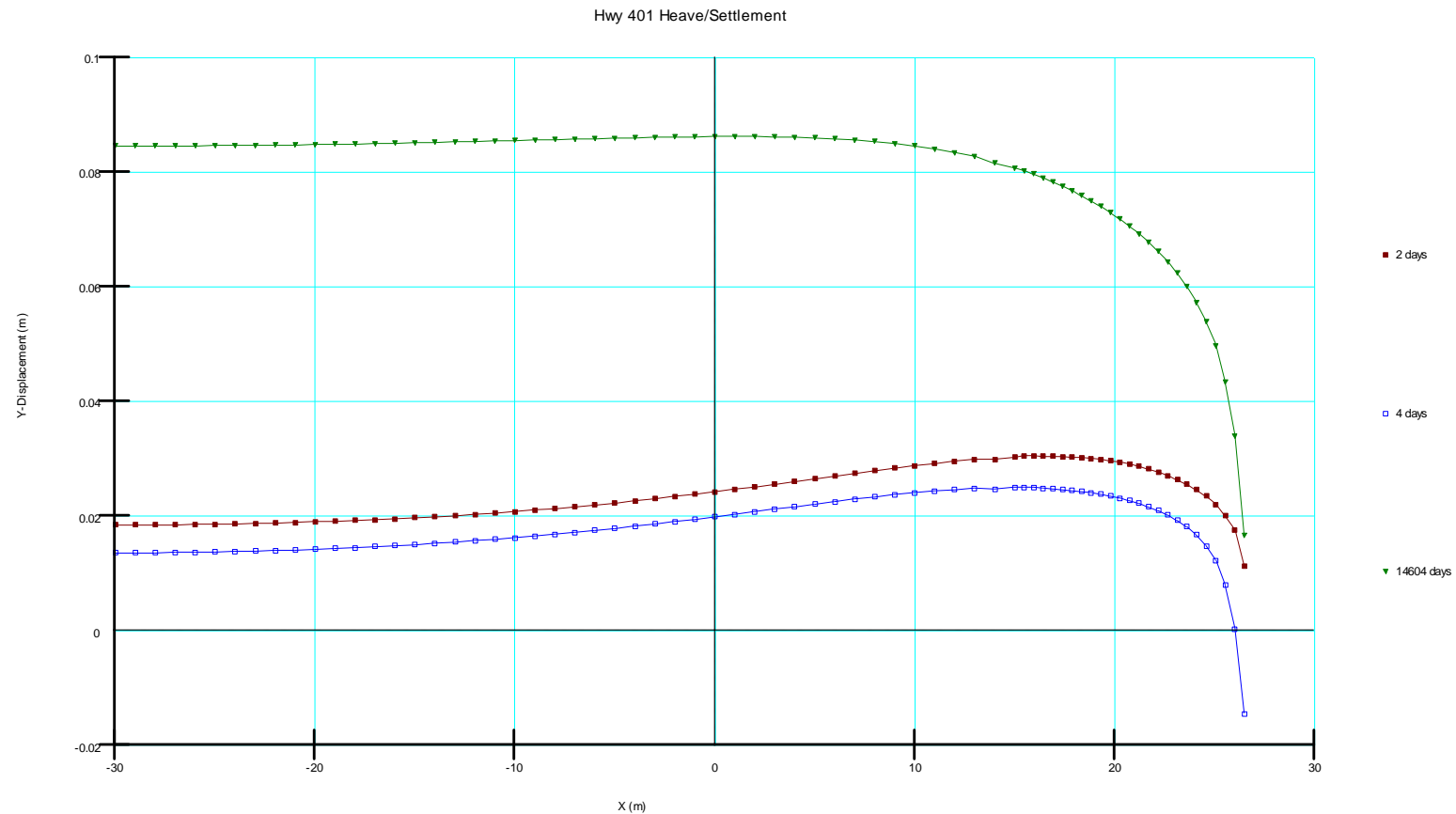
2 days = RSS Completion
4 days = End of Construction
14604 days = Long-term Condition

Figure F-7: Cumulative Lateral Deflection of RSS Wall



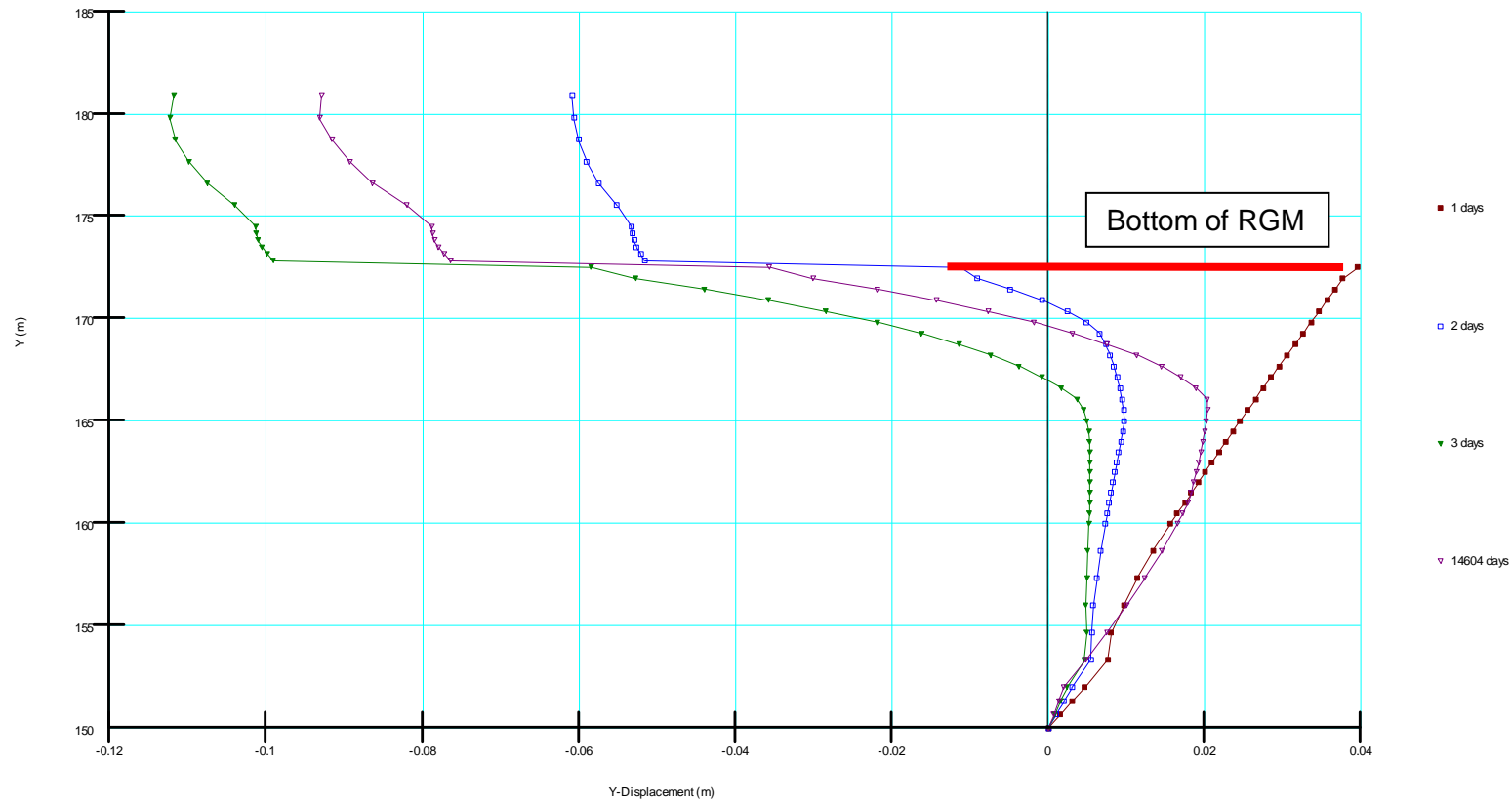
2 days = RSS Completion
 3 days = Abutment Completion
 4 days = End of Construction
 14604 days = Long-term Condition

Figure F-8: Cumulative Highway 401 Settlement/Heave



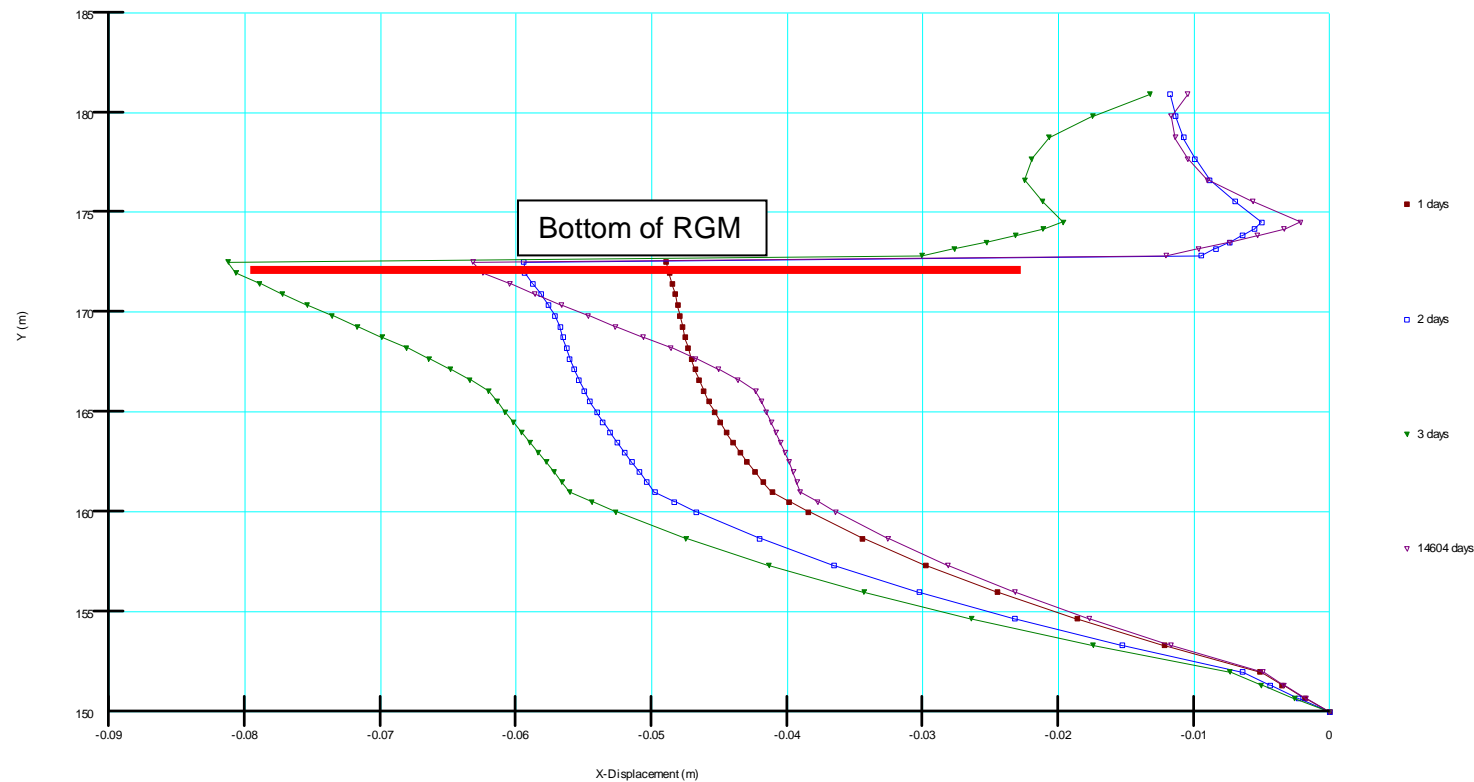
2 days = RSS Completion
 4 days = End of Construction
 14604 days = Long-term Condition

Figure F-9: Cumulative Soil Settlement Profile along Pile Line



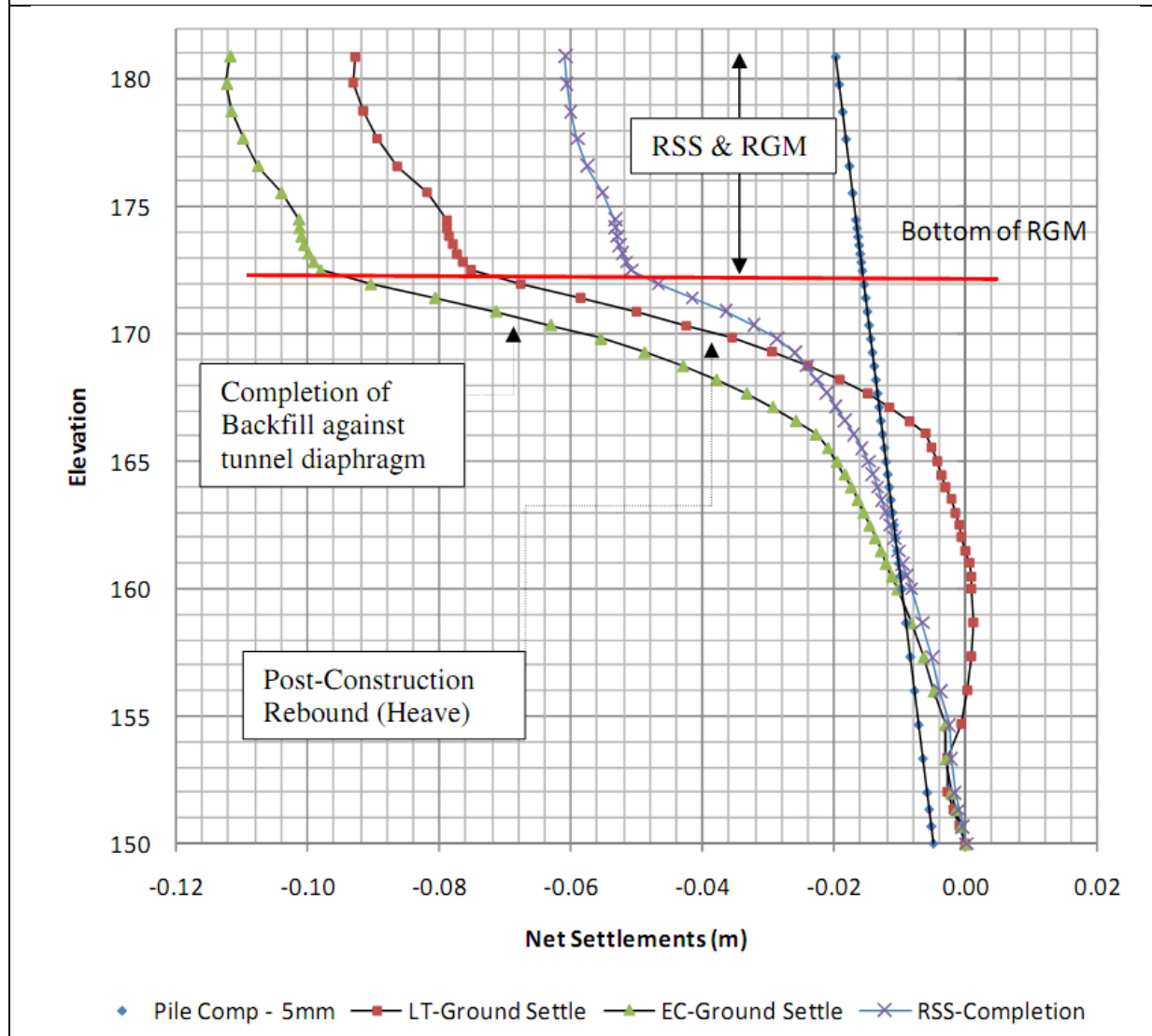
1 days = End of Excavation
2 days = RSS Completion
3 days = Abutment Completion
14604 days = Long-term Condition

Figure F-10: Cumulative Lateral Soil Displacement Profile along Pile Line



1 days = End of Excavation
 2 days = RSS Completion
 3 days = Abutment Completion
 14604 days = Long-term Condition

Figure F-11: Net Soil Settlement Profile along Pile Line



Appendix G: Pile Analysis Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

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

Figure G-1: Soil Movement along Pile (Calculated from SIGMA/W)

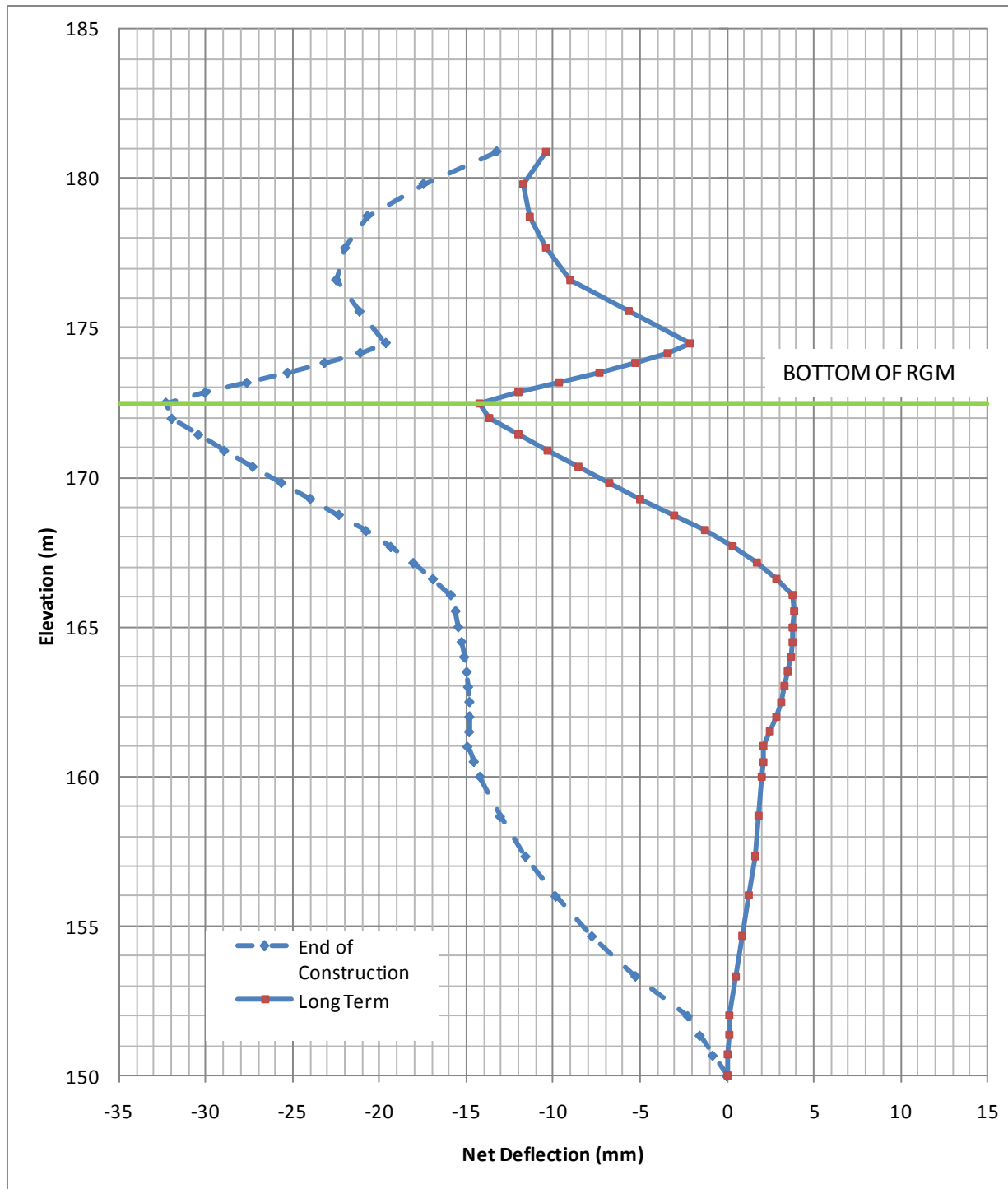
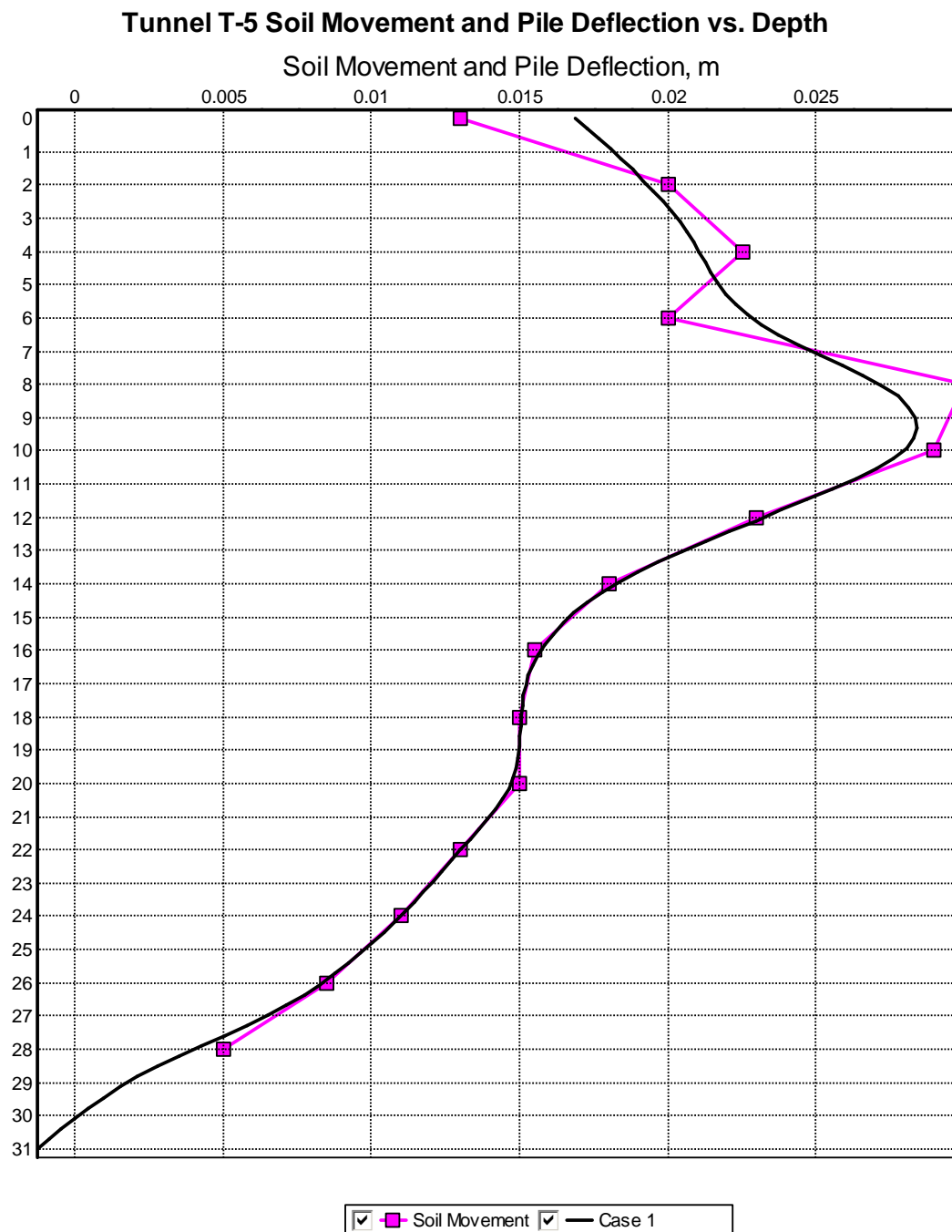


Figure G-2: Input Soil Movement and Calculated Pile Deflection



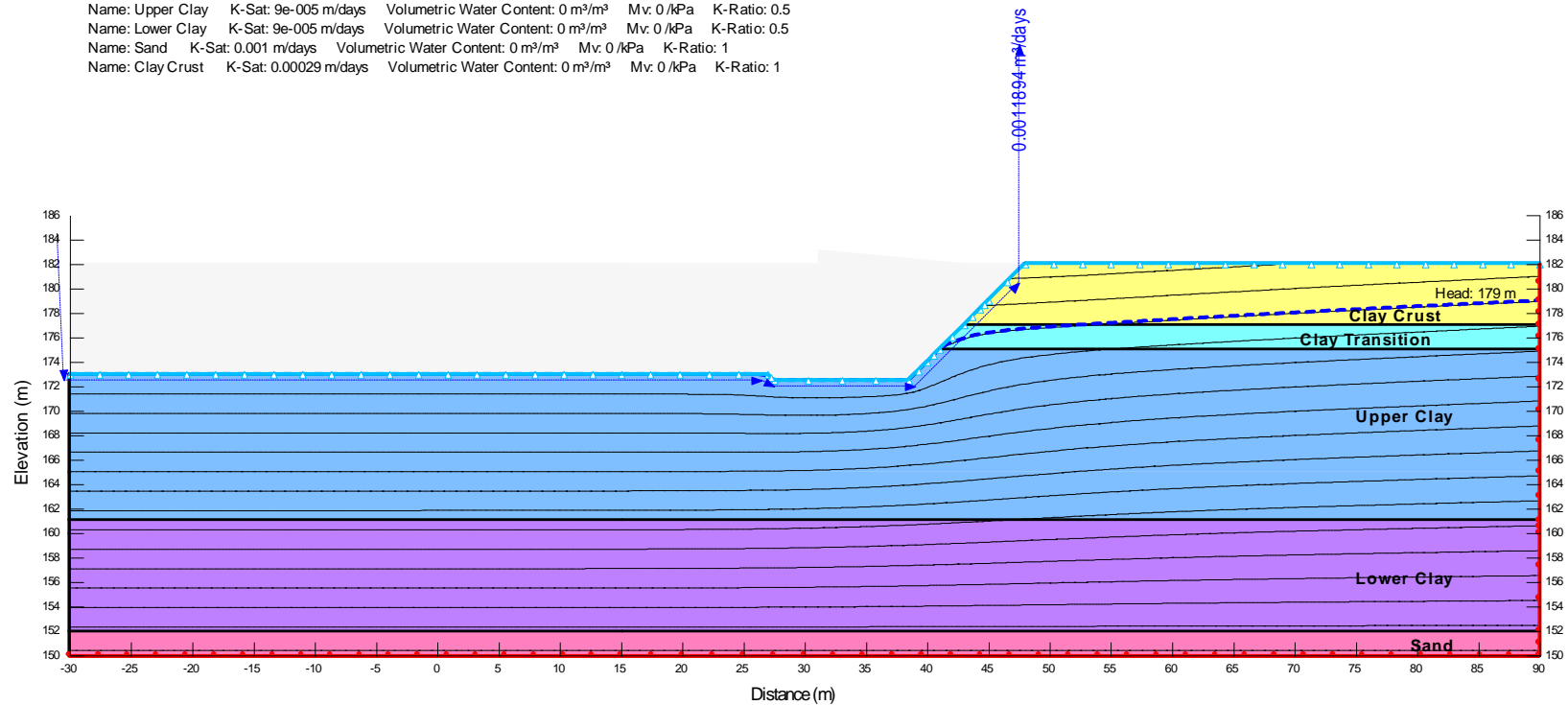
LPILE Plus 5.0, (c) 2008 by Ensoft, Inc.

Appendix H: Seepage Analysis Results

Figure H-1: Tunnel T-5 (Typical Abutment Section) Steady-State Seepage Analysis

Steady-State Seepage
Last Solved Date: 20/09/2011

Name: Clay Transition K-Sat: 9e-005 m/days Volumetric Water Content: 0 m³/m³ Mv: 0 /kPa K-Ratio: 0.5
Name: Upper Clay K-Sat: 9e-005 m/days Volumetric Water Content: 0 m³/m³ Mv: 0 /kPa K-Ratio: 0.5
Name: Lower Clay K-Sat: 9e-005 m/days Volumetric Water Content: 0 m³/m³ Mv: 0 /kPa K-Ratio: 0.5
Name: Sand K-Sat: 0.001 m/days Volumetric Water Content: 0 m³/m³ Mv: 0 /kPa K-Ratio: 1
Name: Clay Crust K-Sat: 0.00029 m/days Volumetric Water Content: 0 m³/m³ Mv: 0 /kPa K-Ratio: 1



Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report
Tunnel T-5 (Sta. 14+510W to 14+630W)
Doc No.: 285380-04-119-0010 (Geocres No. 40J6-36)

Date: May / 2012
Rev: 0
Page No.: App H - 1 of 1

Appendix I: Selected Site Photographs

Photograph I-1: Huron Church NBL at Reddock St- looking North



Photograph I-2: Huron Church SBL- looking North



Appendix J: Selected Core Photographs

Photograph J-1: Borehole T5-1 – Rock Core Elevation 148.5 to 145.1 m



Photograph J-2: Borehole T5-2 – Rock Core Elevation 149.0 to 147.2 m



Photograph J-3: Borehole T5-3 – Rock Core Elevation 148.9 to 146.8 m

