


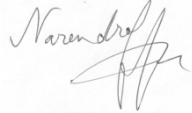

The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report – Bridge B-5

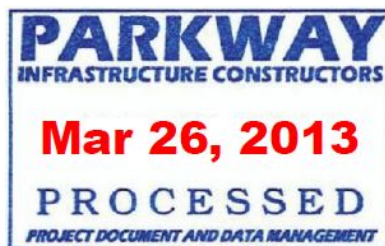
(Malden Rd. Overpass – Realigned E. C. Row EBL, Sta. 11+704 to 11+736, Windsor)

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1 Introduction

1.1 Preface

The Windsor-Essex Parkway (the Parkway) was conceived to strengthen transportation and trade links between Canada and the United States, reduce road congestion, and foster economic growth. The Parkway will connect Highway 401 to a new Canadian inspection plaza and a new international crossing over the Detroit River to Interstate 75 in Michigan, USA. It will be a six-lane highway, 11 km long with 15 bridges, 11 tunnels and a four-lane service road that will provide full access to schools, neighbourhoods, natural areas, and shopping. Other components of the project include community and environmental features, such as: 300+ acres of green space, 20 km of recreational trails, extensive landscaping throughout the corridor, as well as noise and environmental mitigation measures. The environmental mitigation measures were based upon Permit AY-D-001-09 which was approved in February 2010.

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

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

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

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

The lasting legacy of the Windsor Essex Parkway project will not only be its significant contribution as an international trade and transportation route, but rather include the establishment of a contiguous and

sustainable green space system that contributes to the quality of life in the community and supports the re-establishment of an ecologically rich Carolinian landscape.

On December 17, 2010 Infrastructure Ontario and 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 Parkway 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 Parkway 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 Bridge B-5 (E. C. Row East Bound Lane - Malden Road Overpass) located in the Windsor sector of the Windsor-Essex Parkway (Parkway) project. The report includes the results of the previous and additional geotechnical investigations carried out to support the design as well as the other relevant background information. This report is Issued For Construction (IFC) and includes the results of geotechnical investigation carried out to support the design and other relevant background information, and addresses review comments from peer reviews and MTO.

The proposed 32 m long one span Bridge B-5 structure will carry E. C. Row East Bound Lane (EBL) traffic over Malden Road between Sta. 11+704.039 and Sta. 11+736.039. The proposed structural solution incorporates integral abutments founded on deep end bearing piles.

The design presented in this report was generally advanced from the preliminary geotechnical design developed for the WEMG (Windsor Essex Mobility Group) proposal in June 2010 (ref. R-45)¹ which was recognized as 30% design. The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines as well as PIC.

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

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

¹ References are listed in Section 9.

2 Background Information

2.1 Geological Setting

The Parkway project site is located within the Essex Clay Plain (a part of the St. Clair Clay Plain physiographic region described in references R-18, R-20 and R-28). 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-28) 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 Parkway 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 consists of limestone, dolostone and shale comprises the Devonian Dundee Formation of the Hamilton Group of Formation and the underlying Devonian Lucas Formation of the Detroit River group of Formation.

The Windsor area, referred to as the Essex Domain (with respect to bedrock geology), is located in the Grenville Front Tectonic Zone (GFTZ) (ref. R-28). The bedrock geology within the Essex Domain was formed as part of the midcontinent rift south-eastern extension. The midcontinent rift south-eastern extension is composed of Paleozoic cover rocks which form the bedrock foundation of the Essex Domain. The bedrock was deposited in the Paleozoic Era during the Middle Devonian period. Within the Essex Domain the following strata were deposited the Hamilton Group, Dundee Formation, and Detroit River Group Onondaga Formation all consisting of Limestone, Dolostone, and Shale.

2.2 Site Seismic Background

Windsor-Tecumseh area is described in the Canadian Highway Bridge Design Code (CHBDC, ref. R-11) 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 Canadian Highway Bridge Design Code (CHBDC, ref. R-11) 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 were completed during the background investigation program (ref. R-23), 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 Bridge Layout

Bridge B-5 site is situated in the Windsor segment of the Parkway. The topography of the lands along the Bridge B-5 is generally flat with ground at about elevation 181.0² in the area of both west and east abutments. Land use north and south of the Bridge B-5 site is typically both residential and commercial (see Appendix E for selected site photos).

The bridge structure will be constructed under Parkway Phase III development and will be used to carry E. C. Row EBL traffic over Malden Road. E. C. Row EBL at this location will be constructed above the existing grade. A concrete wing wall flared at 73° to the E. C. Row EBL is indicated at each corner of the structure as shown on Drawing 285380-03-061-SEG3-0504 to 0506.

2.4 Frost Depth

In accordance with MTO-SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-40) and OPSD 3090.101³, the frost depth below the ground surface in Windsor area is estimated to 1.0 m. This estimate is considered applicable for natural soils and/or conventional pavement materials where the ground surface is usually cleaned from the snow cover.

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

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

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

3 Geotechnical Investigations

3.1 Scope and Procedures of Geotechnical Investigations

Geotechnical investigations involving a number of boreholes, cone penetration tests (CPT) and Nilcon vane tests had been carried out in 2007-09 by Golder Associates (ref. R-18 to R-25) to develop the conceptual design and serve as background information for development of the Parkway proposal designs. Additional geotechnical investigation was carried out in 2011 to supplement the available subsurface soil data, as required to support the detailed design development of the Parkway embankment and structures. The additional investigation program at and around the proposed location of Bridge B-5 comprised a total of 2 boreholes, 2 Nilcon vane test, 3 CPT and 1 DMT (flat blade dilatometer probes). Table 3-1 lists the test holes put down at or in close proximity of the bridge site during both the previous and the current geotechnical investigations.

Table 3-1: Test Holes At and Around Bridge B-5 Site

Reference	Boreholes	Nilcon Vane Tests	CPT	DMT
This Investigation (2011)	BH B4-1	NIL B4-1	CPT13-RW	DMT B4-1
	BH 6-RW	NIL 6-RW	CPT14-RW	
			CPT15-RW	
MTO Geocres Database (68-F-15-1)	BH-5			
	BH-6			
	BH-7			
	BH-8			
Previous Studies (2007-09)	BH-154		CPT-154	
	BH/CPT-155		BH/CPT-338	

A number of CPT and Nilcon tests were carried out in December 2012 to assess the ground condition improvement that is expected to occur in conjunction with the general construction of the high embankments. Details of the tests as well as results of the general ground response monitoring are provided under separate covers. These CPT and Nilcon tests did not suggest any change in subsurface stratigraphy or the design properties of the soils.

Drawing 285380-04-090-WIP3-0501 shows the locations of the test holes and an interpreted soil stratigraphic profile along the Parkway centreline for the general area from Sta. 11+500W to Sta. 12+200W. The test hole locations and stratigraphic sections at the bridge location and immediate vicinity are illustrated on Drawings 285380-04-090-WIP3-0502 and 285380-04-091-WIP3-0503.

3.2 Fieldwork for Additional Investigation

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

Soil sampling was generally carried out using a 50 mm diameter split spoon sampler. Thin-walled Shelby tube (70 mm diameter × 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 1.5 m long NQ (OD=75.7 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 elevation ranges where rock coring and Nilcon vane tests were carried out. The Nilcon vane tests listed in Table 3-1 were carried out typically adjacent the boreholes.

Table 3-2: Overburden Thickness and Instrumentation in Boreholes

Borehole	Location ⁶	Overburden Thickness, m	Test Name & Elevation, m				
			Rock Coring	Nilcon Vane	S-Piez.	VWP	MHSG
BH 6-RW	N4681950.3, E330198.8	32.2	148.6 to 147.3	177.8 to 154.3		174.3, 163.1, 148.8	174.4, 166.8
BH B4-1	N4681982.4, E330153.5	30.2	150.6 to 149.3	176.8 to 153.3		177.7, 166.5, 150.9	
BH-5	N4682005.8, E330193.3	>31.3 (BTWO)	149.4 to 147.8				
BH-6	N4682042.1, E330185.3	>12.6 (BTWO)					
BH-7	N4682056.5, E330145.1	30.9					
BH-8	N4682020.2, E330165.6	>13.1 (BTWO)					
BH-154	N4681959.9, E330200.6	31.2	149.6 to 144.2	175.3 to 166.3	147.5 to 150.5		

Legend: S-Piez. Standpipe Piezometer (Screen elevations)
VWP Vibrating Wire Piezometer (Sensor elevations)
MSG Spider Magnet Heave/Settlement Gauge
BTWO Borehole Terminated within the Overburden

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 Parkway length.

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

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

⁵ American Society for Testing and Materials.

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

The Nilcon vane tests and CPTs were carried out in cohesive soil strata after augering through the generally stiff/dense surficial materials. The Nilcon tests were carried out at selected depths 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). Pore pressure dissipation tests were carried out at selected depths at Test Holes CPT1-RW, CPT13-RW and CPT15-RW.

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

Test holes executed during pre-bid investigations and the recent 2011 investigation, as also the inferred soil profile along the Parkway alignment from Sta. 11+500W to Sta. 12+200W, are shown on 285380-04-090-WIP3-0401. The test hole locations in plan and soil stratigraphic section at the bridge location are shown on Drawings 285380-04-090-WIP3-0502 and 285380-04-091-WIP3-0503.

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.

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:

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 selected depths and electrical wires extended to the monitoring station at the ground surface (outside the parkway footprint area). The piezometer boreholes were filled with a bentonite-cement mixture designed to match, as near as practical, the permeability and strength-deformation characteristics of the native soils. Sensor elevation and details of installations are provided in Table 3-2 and applicable borehole logs.

Magnetic Heave/Settlement Gauges (MHSG): Spider magnets (RST, Model SSMM100 mechanical release spider target for 25 mm pipe) were installed in boreholes at selected locations and depths to permit future measurement of heave and settlement. Each magnetic torus was placed around a 25 mm diameter pipe, which was extended to above the ground surface. The spider legs of the magnetic torus grip into the surrounding soil, which enables the torus to move up or down on the pipe as the soil settles or heaves. The locations of the magnetic torus are determined by lowering a magnetic probe inside the pipe. Installation 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 AMEC geotechnical 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, triaxial shear tests and oedometer tests were carried out on samples selected from across the length of the Parkway and the interpreted parameters are discussed in Section 4.2⁷.

Selected samples of the silty clay 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 (further discussed in Section 6.4).

The results of geotechnical and geochemical (analytical) laboratory tests carried out on samples recovered from Bridge B-5 site are included in Appendices C and D, respectively. Some of the laboratory test results (e.g., geotechnical index properties) are indicated on the borehole logs.

3.5 Data Interpretation

Field Vane Test Data Correction: The chart (Figure 3-1⁸) 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-8 and R-33). 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-10, and Figure 3-2). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI.

Strength Profiles from Cone Penetration Tests: The undrained shear strength of the silty clay 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.

⁷ No advanced tests were carried out on samples recovered from Bridge B-5 site.

⁸ All figures are included at the end of the report text.

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 Parkway 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 15 and 13, respectively. In CPTs indicating pore pressures higher than cone tip resistance, the undrained shear strength was estimated from the excess pore pressures (using the N_u method).

Pre-Consolidation Pressures from Cone Penetration Tests: The approach used for estimating the pre-consolidation pressures from the estimated S_u profiles follows the Stress History and Normalized Soil Engineering Properties (SHANSEP) method developed at MIT (Ladd and Foott, 1974, ref. R-33). The following relationship was used to compute the pre-consolidation pressures:

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

Where:

S_u is the undrained shear strength;

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

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

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

OCR is the overconsolidation ratio; and

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

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

$$\sigma'_p = \sigma'_{vo} \times \left[\frac{S_{u,CPT}}{\sigma'_{vo}} \frac{1}{0.18} \right]^{1.05}$$

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-29), 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 curve is based on average plasticity index of the silty clay to clayey silt stratum and Chandler 1988 relationship (Figure 3-1) (ref. R-13).

The undrained shear strength (S_u), pre-consolidation pressure (σ'_p), natural water content (w_N) and compression index (C_c) profiles based on field and laboratory testing from boreholes, CPT and DMT carried out between Highway 401 Stations 11+500W to 12+200W are presented on Figure 3-3. Data specific to the Bridge B-5 site are presented in Figure 3-4. Also included on the figure are $0.18 \times \sigma'_{vo}$ curve (representing undrained strength profile for OCR=1 condition) and simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units.

4 Subsurface Conditions

The general soil stratigraphy at the borehole locations in the area of Bridge B-5 consists of the following successive strata: surficial layers of topsoil, occasional fills and upper granular deposit; an extensive cohesive clayey silt to silty clay stratum below about elevation 179.0 to 179.5; lower granular deposit below about elevation 151.3 to 152.3; and then limestone bedrock below about elevation 148.6 to 150.6. The thickness of the silty clay stratum varied between 28.4 and 30.8 m.

The bedrock was encountered at depths ranging from about 30.2 m to 32.2 m below the ground surface.

4.1 Surficial Fills, Topsoil and Upper Granular Deposit

Black topsoil was encountered at the ground surface at Boreholes BH6-RW and BH 154. The thickness of the encountered topsoil was 0.3 m, but is expected to vary in quality and thickness through the project area.

Granular deposits were also encountered surficially or immediately beneath the topsoil or fill in all boreholes. The upper granular deposit consisted of sandy silt to sand with trace gravel and the thickness at the borehole locations varied from 0.7 to 1.7 m.

Borehole BH B4-1 encountered 1.1 m thick surficial fill consisting of sand, clayey topsoil and gravel.

Test Holes DMT B4-1 and CPT13-RW, put down on the existing E. C. Row, encountered 6.1 to 6.9 m of silty clay fill with some sand and trace of gravel.

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 1.2 to 1.8 m. Based on the gradation, in-situ moisture content and strength characteristics, this stratum across the Parkway project may be divided into 4 layers as follows: brown to grey desiccated stiff to firm 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). Based on the investigation results the thickness of crust generally decreases from east to west. 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 (Figures C.1 to C.2 in Appendix C) suggest the silty clay deposit to be a low to medium plasticity material.

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

Property	Clay Crust	Clay Transition	Upper Silty Clay	Lower Clayey Silt
Elevation Range, m	180 ^(*) – 177	177 – 175	175 – 160	160 – 151
Natural Water Content, w_N , %	13.0 – 27.6	18.2 – 26.0	16.7 – 29.0	14.5 – 36.1
Liquid Limit, w_L	24.0 – 40.0	28.4 – 38.0	25.0 – 42.0	20.3 – 33.0
Plastic Limit, w_P	15.0 – 21.0	12.0 – 19.0	14.0 – 20.0	12.7 – 18.0
Plasticity Index, PI	6.5 – 21.6	13.7 – 19.0	10.0 – 24.5	7.6 – 16.0
Liquidity Index, LI	0.8 – 0.1	0.2 – 0.4	0.0 – 0.6	0.0 – 0.9
Unit Weight, γ , kN/m ³	20.2 – 21.4	19.9 – 20.9	20.0 – 21.9	20.0 – 21.9

(*) Elevation varies

As illustrated on Figure 3-4, the undrained shear strength of the silty clay stratum varied with depth generally as follows:

- Crust layer: $> 80 \pm 20$ kPa
- Transition layer: 80 ± 20 kPa to 70 ± 15 kPa
- Upper silty clay: 70 ± 15 kPa to 45 ± 10 kPa & then 55 ± 10 kPa
- Lower clayey silt: 55 ± 10 kPa to 65 ± 20 kPa

The stress-strain properties and the effective shear strength properties of the silty clay to clayey silt soils were based on published correlations (Kulhawy and Mayne, 1990, ref. R-32, Leroueil et al, 2001, R-37 and Terzaghi et al.1990, ref. R-44) as well as on the tests reported in Golder's Subsurface Condition Interpretation Report (ref. R-21) and the tests performed during the additional geotechnical investigation carried out as part of the detailed design development for the entire Parkway length.

The stress-strain relationships have been 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 initial compression indexes derived from the above equations were later calibrated based on the settlement and pore pressure measurements observed in 2012 under embankment fill load at Bridge B-5 site (BH06-RW-MSG/PZ). The average values used for the clay substrata for the Bridge B-5 site are summarized in Table 4-2.

Table 4-2: Summary of Interpreted Compressibility Properties

Property	Clay Crust	Clay Transition	Upper Silty Clay (*)	Lower Clayey Silt
Average Natural Water Content, w_N , %	21	20	25 (21)	21
Virgin Compression Index, C_c	0.17	0.16	0.21 (0.17)	0.17
Recompression Index, C_r	0.019	0.018	0.023 (0.018)	0.018
Swelling Index, C_s	0.04	0.04	0.05 (0.04)	0.04
Secondary Compression Index, C_α	0.005	0.005	0.006 (0.005)	0.005
(*) Values in brackets represent the initial estimates before calibrations from monitoring				

The effective shear strength properties applicable to the silty clay to clayey silt stratum were determined from triaxial tests performed during the pre-bid geotechnical investigation (Figure 4-3) and supported also by published PI versus ϕ' relationships (ref. R-31, R-36 and R-44, Figure 4-4), and are summarized as follows:

Apparent cohesion, c'	0 kPa
Angle of internal friction, ϕ	30°
Friction angle at critical state, Φ_c	25° to 26°(*)

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

The modulus of elasticity has been correlated using the empirical relationships listed below with the undrained shear strength of the material in accordance with published information (ref. R-32) and local experience (ref. R-21):

$$E_u = 300S_u$$

$$E' = 0.9E_u$$

The results for the unweathered portion of the silty clay stratum using the average shear strength profiles for the material are shown in Table 4-3.

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

Soils Stratigraphy	Undrained Elastic Modulus, MPa	Undrained Poisson's Ratio (*)	Drained Elastic Modulus (E), MPa	Drained Poisson's Ratio (*)
Clay Crust	30	0.49	27	0.35
Clay Transition	23		21	
Upper Silty Clay	16		14	
Lower Clayey Silt	23		21	

(*) assumed values consistent with theory and standard practice (ref. R-32)

The hydraulic conductivity of the silty clay to clayey silt stratum was interpreted from pore pressure dissipation tests carried out in the CPT probes as well as the laboratory oedometer tests. The hydraulic conductivity values obtained from previous (2007-09) and additional (2011) investigations are plotted on Figure 4-5.

4.3 Lower Granular Deposit

Two of the four boreholes advanced to bedrock (historical tests holes BH-5 and BH-7) encountered a heterogeneous non-cohesive deposit underlying the silty clay stratum at elevations between 151.3 and 152.3. The gradation of the material varied from sandy silt to silty sandy with some gravel. No Standard Penetration Test (SPT) was available for this layer. This layer, where present, was approximately 1.7 to 2.9 m thick but will vary significantly throughout the project area.

4.4 Bedrock

A grey to brown, limestone bedrock was encountered in Boreholes BH B4-1, BH 6-RW, BH-5, BH-7, and BH-154). The bedrock was medium to coarse grained, occasionally pitted, faintly to strongly porous and fractured. Bedrock was encountered at elevations ranging from 148.6 to 150.6 in the vicinity of Bridge B-5. The Rock Quality Designation (RQD) of the recovered rock varied from 36 to 97 per cent, indicating a poor to excellent quality. Based on this core logging the rock mass classification was estimated to range from 2.8 to 5.5 for the Q-System (Barton *et. al.*, 1974, ref. R-5) and 53 to 58 for the Rock Mass Rating (RMR) based on Bieniawski (1976, ref. R-7) and indicates that the rock mass can be considered as a Fair quality rock mass based on the later system. Rock quality generally increases with depth. Photographs of rock cores recovered from the additional investigation are provided in Appendix E.

It was found during the preliminary investigations reported in Golder's Subsurface Condition Interpretation Report (ref. R-21) that little variation in the strength of the rock mass conditions was identified from site to site. For this reason in order to obtain a reasonable statistical sample, the density, unit weight and uniaxial compressive strength of the samples from all of the key sites have been grouped and are summarised in Table 4-4. The average strength of the limestone is determined to be 85.5 MPa and is 'strong rock' based on the ISRM (1978), ref. R-30. Additionally, based on the coefficient of variation, enough tests have been performed to characterise the compressive strength.

Rock core samples from BH B6-2 and BH 152 were tested and had unconfined compressive strengths of 122.7 and 66.4 MPa, respectively. The results of the compressive strength testing indicate that the limestone rock may be described as "strong" to "very strong" rock.

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

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-46).

4.5 Groundwater Conditions

Shallow and deep standpipe (S-Piez) and vibrating wire piezometers (VWP) were installed in selected boreholes to measure the water levels within overburden and bedrock (Table 3-2). The latest piezometric levels within the overburden and the bedrock were recorded at about elevations 178.7 to 180.5 and 180.5, respectively (Table 4-5). These observations suggest an essentially hydrostatic condition or a slight upward gradient between the overburden and the bedrock. In consideration of the slight short-term artesian condition at BH B4-1 and 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	Piezometer Type	Screen / Sensor El, m	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	El, m
BH 6-RW	180.8	VWP	174.4	Clayey Silt	July 22, 2011	178.6
					Oct 1, 2011	178.7
			163.1	Clayey Silt	July 22, 2011	179.7
					Oct 1, 2011	179.6
			151.2	Bedrock	July 22, 2011	180.5
					Oct 1, 2011	180.7
BH B4-1	180.8	VWP	178.1	Clayey Silt	July 22, 2011	179.2
					Oct 1, 2011	179.2
			166.8	Clayey Silt	July 22, 2011	180.5
					Oct 1, 2011	179.8
			151.0	Bedrock	July 22, 2011	180.5
					Oct 1, 2011	180.3
BH-154	180.9	S-Piez	147.5 to 150.5	Bedrock	Jan 28, 2009	178.3

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 (H₂S) and methane (CH₄) gases that are liberated from the water on exposure to atmospheric pressure.

The H₂S 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 to 3 mg/L in the groundwater. The gas odour was not detected during the drilling at the Bridge B-5 site.

Although the presence of the H_2S and CH_4 gases was not observed during the 2011 geotechnical investigation at Bridge B-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.

The understanding of the engineering behaviour (related to the impact on design and construction) of the gassy soils is rather limited. In the case of low permeability cohesive soils it is known that these soils may experience rapid drop in undrained shear strength during unloading. Due to the relatively high compressibility of the pore water fluid in gassy soils, the immediate pore water pressure response (ΔU) to total stress changes can be very low. In the cases of excavations this phenomena leads to reduction in effective stress and hence shear strength (ref. R-26 and R-43). While no significant excavations below the original ground are anticipated at this structure, it is recommended that the design and construction methodologies should be developed in consideration of the potential presence of these gases (ref. R-16).

Table 4-6: Summary of 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

5 Development of Geotechnical Designs

5.1 Bridge Configuration

Bridge B-5 (Malden Road Overpass) will be constructed along the proposed E. C. Row East Bound Lane between Sta. 11+704.039 and Sta. 11+736.039 (Drawing 285380-03-060-SEG3-0501). As shown, the 32 m long and 12.05 m wide Bridge B-5 is a one-span deck on girder structure incorporating integral abutments founded on deep end-bearing HP 310×110 steel vertical piles (see Drawings 285380-03-061-SEG3-0504 to 0506). Cantilever wing walls flared at approximate 73° to the E. C. Row EBL centerline are indicated at each corner of the structure. A false RSS abutment is used to retain the approachway backfill.

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

Table 5-1: Summary of Interpreted Design Elevations at Abutments

Location	Existing Ground Surface Elevation ⁽¹⁾	Top of Pavement Elevation ⁽²⁾	Pile Cap Underside Elevation ⁽³⁾	Malden Road Subgrade Elevation ⁽¹⁾
West Abutment (WP#1)	180.8	188.2	183.1	179.5
East Abutment (WP#5)	180.8	188.2	183.1	179.5

Notes:

- (1) Interpreted from highways cross-section drawings.
- (2) Interpreted from WP elevations shown on Drawing 285380-03-060-SEG3-0501.
- (3) Interpreted from elevations shown on Drawing 285380-03-061-SEG3-0504.

Based on the available information, it is considered that Bridge B-5 construction (including wick drains and embankment construction) is expected to involve the following sequence of earthwork, design elements and loading stages:

- Ground stripping , installation of drainage blanket and wick drains at west and east approachway embankments on both sides of the bridge;
- Construction of approachway embankments on west and east sides of the bridge, including temporary preloading embankment extensions that will extend to the limit of the existing pavement edges along Malden road;
- Waiting time for ground consolidation under the preloading embankments;
- Removal of excess embankment fill and temporary excavation to about 1.60 m depths below existing grade to accommodate construction of RSS;
- Installation of piles (HP310×110) and 600 mm diameter Corrugated Steel Pipe (CSP) around the abutment pile stickups; filling of the CSP casing with loose sand;
- Construction of the RSS structures and associated permanent subdrainage works, and approved backfill behind the RSS structure;
- Construction of the structural abutments including pile cap and bridge girder;

- Completion of backfill and concrete slope paving at front (toe) of abutments of the bridge; and
- Completion of pavements over Malden Road and E. C. Row East Bound Lane.

The embankments for the E. C. Row EBL on west and east sides of Bridge B-5 will be built with compacted silty clay fill. The side slopes of the embankments are generally 3H:1V. The maximum height of these embankments above original ground surface will be about 8 m.

The design and construction requirements for the high embankments at this area are complex due to poor soil condition, limited duration of time available to achieve consolidation and strength gain in the silty clay deposit, space limitation (preventing slope flattening and surcharging for preloading) and settlement constraint. These conditions necessitated use of Perforated Vertical Drains (PVD) or wick drains to expedite consolidation of the foundation stratum and strength improvement, multi-stage construction (2 stages at this bridge site) and surcharge loading to minimize future long-term settlement.

Presently, PVD or wick drains (100 mm wide with 2 mm core thickness) are being installed in triangular pattern at 2 m spacing at the site. The bottom of wick drains was set at about elevation 160 at this site.

The wick drain design and construction requirements were specified to be as per the requirements of the OPSS 220, "Construction Specification for Wick Drain Installation". Details of site preloading, staging and consolidation with wick drains are provided in "Design Report - High Embankments, 2850380-04-119-0003" (ref. R-2).

Details and recommendations for instrumentation, monitoring program, as well as guidelines for alert levels, interpretation and contingencies are provided in a separate report 285380-04-118-0001 (ref. R-3).

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 Parkway Project. The foundations' designs have been developed as per the principles of Limit States Design (LS Method) based on Load and Resistance Factors (CFEM, ref. R-10 and CHBDC, R-11).

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 false abutments was checked for all potential surfaces of sliding. WS Method was also used for the external design of the RSS false abutments.

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. Shear strength design values obtained from the profiles are summarized in Table 5-2 for in situ/initial ground conditions. To adequately represent the variability with depth of the undrained shear strength and pre-consolidation properties, the upper silty clay and the lower clayey silt strata have been further sub-divided. The properties of backfill materials assumed in the geotechnical analyses are described in Table 5-3.

The design values of the coefficient of horizontal permeability (k_h) and the hydraulic conductivity anisotropy ratio ($A=k_h/k_v$) provided in Table 5-4 were interpreted from in-situ and laboratory tests. 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 5-2: Summary of Interpreted In Situ Design Clay Strength and Consolidation History

Clay Substratum	Elevation Range, m	Undrained Shear Strength (S_u), kPa	Effective Strength Parameters	Preconsolidation Pressure (σ_p'), kPa	OCR
Clay Crust	181* to 177	75**	Peak friction angle, $\phi_{max} = 30^\circ$ $= 0$	500	> 6
Clay Transition	177 to 175	75 to 67		500 to 350	4
Upper Grey Silty Clay - 1	175 to 163	67 to 42		350 to 230	2
Upper Grey Silty Clay - 2	163 to 160	42 to 49		230 to 260	1.1
Lower Grey Clayey Silt -1	160 to 159	49 to 75		260 to 500	1.1 to 1.5
Lower Grey Clayey Silt -2	< 159	75		500	1.5

(*) Crust top elevation varies

(**) Applicable for global stability verifications

OCR = Over-Consolidation Ratio

Note: The in-situ undrained shear strength and pre-consolidation pressure profiles applicable to Bridge B-5 site are shown on Figure 3-4, and the effective shear strength parameters are based on the relationship presented in Section 4.2.

Table 5-3: Assumed Backfill Material Properties

Backfill Material	Unit weight, kN/m^3	Undrained Shear Strength, kPa	Drained Angle of Internal Friction, deg	Modulus of Elasticity, E, MPa	Poisson's ratio, μ
Compacted Clay Fill	21	50	30	22.5	0.35
Compacted Granular Fill	21	N/A	32	50	0.35

Table 5-4: Design In Situ Hydraulic Conductivity Parameters for Silty Clay Stratum

Clay Substratum	Horizontal Permeability, cm/sec	Anisotropy Ratio, k_h/k_v	Initial Void Ratio, e_0
Clay Crust	6.8×10^{-7}	1	0.58
Clay Transition	3.9×10^{-7}	2	0.55
Upper Grey Silty Clay - 1	1.1×10^{-7}		0.63
Upper Grey Silty Clay - 2			0.50
Lower Grey Clayey Silt - 1			0.50
Lower Grey Clayey Silt - 2			0.63

For design purposes the long-term groundwater level in the overburden was considered to be at elevation 179.5 (i.e., Malden Rd subgrade level).

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 PA criteria and the needs to accommodate the construction of the structure as per design.

Excavations for the bridge abutments are expected to encounter surficial fills/pavement (preloading fill, Malden Road pavement materials), topsoil and likely water bearing granular soils, and will be extended 1.6 m below existing grade to about elevation 179.2 into the native granular soils or silty clay. Particular attention to work staging and temporary excavations is necessary due to the close proximity of Bridge B-5 and B-4 sites, and existing Malden Rd overpass (ref. R-4).

5.5 Pile Foundations

5.5.1 ULS and SLS Resistance to Axial Loads

It is understood that HP310×110 steel piles will be used at this project. The pile driving equipment and installation procedure should be established in the field by the Contractor with approval of the Engineer. A number of static load tests should be carried out at key locations along the alignment of Parkway 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 an Ultimate Limit States (ULS) axial geotechnical resistance in excess of 4,000 kN is expected to be mobilised. A factored ULS resistance of at least 2,000 kN is anticipated for piles driven to bedrock.

The Serviceability Limit State (SLS) reaction of the HP310×110 piles, based on the conventional 25 mm settlement, is estimated to exceed the ULS resistance due to the practically unyielding nature of the bearing surface. Hence, the SLS resistance does not govern the design.

Based on the available borehole data at this structure, the elevation of bedrock surface (where the tips of piles are anticipated to be set) varies between 148.6 and 150.6. In the unlikely 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:

Project:	Windsor-Essex Parkway	Date:	December / 2012
Document:	Geotechnical Investigation and Design Report - Bridge B-5 (Malden Rd. Overpass – Realigned E.C. Row EBL, Sta. 11+704 to 11+736, Windsor)	Rev:	0
Doc No.:	285380-04-119-0115 (Geocres No. 40J6-48)	Page No.:	20

- 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.
- Considering the general geologic conditions in vicinity of this site, 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.
- Adequate hammers should be used to ensure the mobilization of the design ultimate geotechnical resistance and prevent damages to the piles during driving.
- 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 on Piles

The ULS and SLS geotechnical resistances to lateral loads on piles should be determined on the basis of field load tests. Both the ULS and SLS lateral load resistances are strongly dependent on the soil properties, structural configuration of the pile and pile foundation, load configuration and deformations.

The SLS geotechnical resistance to lateral loads is dependent on the acceptable levels of the lateral pile deflections under the design loads and should be obtained on the basis of field load tests. In the absence of field tests, the preliminary design may be based on a conventional SLS resistance of 105 kN along the strong axis, and 75 kN along the weak axis of the HP310×110 (for a free-head loading condition). 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 unstabilized pile displacements due to soil failure or pile structural failure. In the absence of field tests, the ULS lateral resistance may be assumed as 160 kN, and 80 kN along the strong axis and weak axis, respectively. 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 (125 and 384 kN-m for weak and strong axes).

The above resistances were estimated using the “p-y” model (LPile 5.0 model Ensoft 2010, ref. R-17). The above estimates were based on a conventional pile model assumed to be embedded within stiff to firm silty clay below elevation 179.5. The “p-y” curves were generated using the Reese method described in the Technical manual for LPile, using the Reese “Stiff-Clay without free water” and Matlock “Soft Clay” models in conjunction with the soil parameters defined in Table 5-5.

Table 5-5: Soil Parameters for P-Y Curve Calculation

Soils Around the Piles	Elevation	Soil Model in L-Pile	Bulk Unit Weight, kN/m^3	Undrained Shear Strength (S_u), kPa	ϵ_{50}	ϕ , °	n_h , MPa/m
Loose Sand Fill within CSP	183.4 to 179.5	Sand (Reese)	19	-	-	32	2
Clay Crust	179.5 to 177	Stiff Clay without Free Water (Reese)	22	75	0.005	-	-
Clay Transition	177 to 175	Stiff Clay without Free Water (Reese)	22	75 to 67	0.005 to 0.007	-	-
Upper Grey Silty Clay - 1	175 to 163	Stiff Clay without Free Water (Reese)	21	67 to 42	0.007 to 0.010	-	-
Upper Grey Silty Clay -2	163 to 160	Soft Clay (Matlock)	21	42 to 49	0.010	-	-
Lower Grey Clayey Silt -1	160 to 159	Stiff Clay without Free Water (Reese)	21	49 to 75	0.010 to 0.007	-	-
Lower Grey Clayey Silt -2	159 to 151	Stiff Clay without Free Water (Reese)	21	75	0.007	-	-

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

The actual SLS and ULS lateral resistances may differ from the values described above for the conventional model depending on the structural restraints at the pile head due to embedment within the pile caps as well as in consideration to the length of pile shaft within the loose sand filled CSP. Both the ULS and SLS to lateral loads resistances are also strongly dependent on the structural and load configuration and on the acceptable deformations.

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

Significant lateral loads in excess of the 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 limit the loss of hammer energy for pile driving, batters are usually limited to no steeper than 1H:5V. However, greater batter may be considered if required.

The stress-deformation analysis of the piles to lateral loads, including the theoretical determination of the associated design ULS and SLS resistances may be carried out using one of the following methods:

Horizontal Subgrade Reaction Method

This method is in general limited to initial design stages for preliminary estimates of the foundation response. The coefficient of horizontal subgrade reaction, k_h , may be based on the following equations:

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

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

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

d (m) = Pile diameter/width

The recommended undrained shear strength ranges of various soil layers are tabulated in Table 5-5.

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

Table 5-6: Lateral Load Capacity Reduction Factors for Pile Groups using the Subgrade Reaction Method

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

d = pile diameter

Reference: Foundations and Earth Structures – Design Manual, NAVFAC DM-7.2 (ref. R-15).

Alternative Nonlinear 'p-y' Curve Method

Alternative pile design methods can be considered using more elaborated methods such as the nonlinear "p-y" interaction method and elastic continuum theory as discussed in the Canadian Foundation Engineering Manual (ref. R-10).

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-10). 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-17). 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 Table 5-5. "Stiff and Soft Clay" p-y curves, as given in the LPILE manual, should be developed appropriate for either static or cyclic loading conditions in absence of free water. For p-y curves below the water table, effective unit weights in the soil mass shall be used.

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

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

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

Where:

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

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

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

The space between the piles under the abutments is 1,850 mm (Drawing 285380-03-061-SEG3-0504). Accordingly, group reduction factors will not apply for lateral pile loadings along the bridge and will be negligible for loads parallel to the abutment.

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): Nominal potential for downdrag loads on piles was considered in conjunction with the anticipated ground movements that are assumed to occur following completion of bridge constructions.

Soil stress-deformation analyses described later in Section 5.6.2 were conducted using the SIGMA/W software. The estimated ground vertical movement (settlement/heave) are presented in Figure G-12 in Appendix G. The estimated vertical movements correspond to the following representative stages: after embankment constructions, after completion of the true abutments with associated backfills, after backfilling at the front of abutments (End of Construction - EC) and in long-term (LT). The analyses indicate the following:

- No significant amount of ground consolidation settlements are expected to occur along the pile shaft during construction of the abutments that are to be built after completion of embankments and substantial consolidation of the foundation soils.
- A potential post construction settlement due to secondary consolidation (creep) of up to 90 mm is expected to occur over a period of time following the completion of the primary consolidation (ref. R-2).

Considering the construction staging and the anticipated settlement of the soils described above, a residual (long-term) downdrag of about 380 kN is estimated to potentially develop for the abutment piles.

In accordance with the Canadian Foundation Engineering Manual (ref. R-10), 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 as follows:

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

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

- A nominal amount of ground lateral movement (Figure G-13) along the pile shaft anticipated to occur after the installation of the piles was estimated using the stress-deformation analysis described below in Section 5.6.2.
- Pile head was modeled with both fixed-head and free-head boundary conditions.

- The above soil deformation field was imposed as “loads” along the pile shaft. The calculation was conducted using the “p-y” model (LPile 5.0 model Ensoft 2010, ref. R-17). The “p-y” curves were generated using the Reese method described in the Technical manual for LPile and soil parameters indicated in Table 5-5.

Based on the above approach and anticipated lateral ground displacement, the estimated maximum unfactored bending moments in the shaft were 20 kN-m for the strong axis pile loadings for a free-head condition and 30 kN-m for a fixed-head condition. These results should be considered in the structural design of the piles and in the design of RSS structural components. These bending moments, shear forces and deflections are in addition to those caused by bridge loads applied to the piles.

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

5.6 RSS False Abutment Walls

The conceptual configuration developed for Bridge B-5 that meets the geotechnical requirements are shown on Figure 5-1. These configurations and preliminary dimensions were developed at representative sections based on (a) the global stability of the soil mass containing the structure, (b) the anticipated deformations, and (c) the foundation soil bearing resistances.

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

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

The properties of the proprietary products used in the geotechnical analyses are summarized in Table 5-8.

Table 5-8: Assumed Proprietary Product Properties

Material	Unit Weight, kN/m ³	Limit Equilibrium (SLOPE Models)		Stress Deformation (SIGMA Models)	
		Friction Angle, °	Apparent Cohesion, kPa	E, MPa	PR
RSS (Granular Fill)	21	35	Min. 50 ^(*)	50	0.35
LWF	12	35	50	40	0.35

(*) Used only for slope stability modeling purpose to prevent slip surface through RSS

The RSS foundation is to be installed on a Granular Mat (GM) over prepared subgrade (intact native stratum with no disturbance due to construction activities, groundwater inflow, etc., and appropriately protected immediately after excavation to final grade).

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.

Due to the similarity of design elevations between west and east abutment, the analyses have been carried out for a typical cross section illustrated in Figure F1- to F-3. The global stability analyses have been carried out for short-term (ST - undrained soil properties), end of construction (EOC - undrained soil properties) and long-term steady state (LT - drained soil properties) loading conditions. The short-term loading condition refers to a temporary stage during construction and before the construction of the pavement box material over Malden or when it was removed during the future operation stages (the approachway embankment was assumed completed). The EOC loading condition refers to the transient/undrained situation, shortly after the works are entirely completed and fully operational. The long-term loading condition refers to steady state (drained) condition after completion of the project. As earlier discussed in Section 5.4, the stability of temporary slopes for construction is part of the Contractor's responsibilities.

The reinforcement due to presence of piles and potential strength gain due to wick drain installation was not considered in the stability models (conservative approach). Live Loads of 12 kPa for short-term and long-term model were applied at the top of ground surface, while tension crack was assumed for short-term only. The calculated factors of safety (FS) listed in Table 5-9 for the abutments meet the PA requirements. LWF placed at top of the RSS wall was required to satisfy the external stability of the RSS wall (i.e., bearing capacity).

Table 5-9: Summary of the Abutment Slope Stability Analyses

Abutment	Calculate Factors of Safety for Loading Conditions			Figure
	Short-term ⁽¹⁾	End of Construction ⁽²⁾	Long-term ⁽³⁾	
North & South Walls	1.51 (1.34)	1.68 (1.52)	2.26 (2.07)	E-1 to E-3

(*) 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 subgrade

(2) Undrained response with pavement box over subgrade

(3) Drained response with all design components present

The above calculated factors of safety (FS) satisfy the design criteria for all loading conditions. The FS for the EOC and LT conditions surpass the PA requirements by significant margins since the design configurations are governed by bearing capacity and basal stability.

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 bridge structure. As such, the structural elements (deck, box structures and piles) were not included in the model, albeit their presence was simulated with boundary restraints.

The configuration of the calculation model is presented in Figures G-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 (consistent with published data – ref. R-32, and confirmed by DMT at the site) for the soil deposit (0 days);
- b) Installation of wick drains and construction of the first stage of approachway embankment to elevation 185 (120 days duration – day 1 to 120);
- c) Construction of final stage of approachway embankment to elevation 188 (60 days duration – day 120 to 180);
- d) Construction of surcharge buttress to the edge Malden Rd shoulder to elevation 188 (90 days duration – day 180 to 270);
- e) Removal of the excess preload fill to accommodate construction of the abutment (5 days duration – day 270 to 275);
- f) Pile installation followed by construction of RSS abutment, bridge abutment and the associated backfill (25 days duration – day 275 to 300);
- g) Completion of the backfill / grade restoration at front of abutment – end of construction (1 days duration – day 300 to 301); and
- h) Dissipation of excess pore pressure leading to long-term steady state condition (30 years=11,000 days duration – day 301 to 11,301).

The stratigraphy and selection of the soil properties (except for the concrete abutment) was based on the design soil properties discussed in Section 5.3. The abutment was modeled as homogeneous elastic material.

The SDA were carried out using an effective stress-based model. The phreatic surface was assumed to correspond to the initial groundwater level at elevation 180.0 and then follow the excavation and subgrade surfaces. Elastic-plastic Mohr-Coulomb models were used for all soil layers except the unweathered firm to stiff silty clay, which was simulated by the Modified Cam-Clay model. Hydraulic conductivity properties described in Table 5-4 were assigned to the different soil layers.

The stress-deformation model suggests dissipation of most of the excess pore water pressures shortly after embankment loading sequence - described above - due to effective operation of wick drains. After the completion of the entire construction, the model is allowed to dissipate the remaining excess pore-pressures over a period of time until a steady-state pore pressure condition is achieved.

Figures G-1 to G-4 show the cumulative settlement/heave for the end of pre-loading (“270 days”), end approachway embankment construction (“275 days”), end of abutment construction (“301 days”) and the long-term (“11,301 days”) loading conditions. Figures G-5 to G-8 show the cumulative lateral deformations for the above named loading condition. Figure G-9 illustrates the stabilized pore water pressure contours at the end of dissipation (long-term) period.

As the SDA models used are two-dimensional (2-D), the calculated deformations in the longitudinal direction (along the bridge) are considered to be over-estimate by 15% to 25% compared to results from a 3-D model.

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 5-10.

Figures G-14 to G-16 show soil settlement, lateral displacement and vertical effective stress profiles along the pile line within the abutment embankment zone. These deformations were estimated from SDA, which were also used in pile calculation in Section 5.5.

SDA was also carried out to assess the impact of the construction of new embankment on the existing E. C. Row, results of which have been presented in the design report for Bridge B-4 (ref. R-4).

The ground movements generated by the construction loads are anticipated to stabilize within approximately 8 years following completion of construction.

Table 5-10: Summary of Calculated Cumulative Deformations

Parameter	End of Embankment Construction	End of Abutment Construction	Long-term (Drained)	Remarks
Settlements on Top of Ground at Distances (m) from the Edge of Bridge Deck of (†):				Figure G-10
0 m	-155 mm	-180 mm (*)	-185 mm	
5 m	-164 mm	-181 mm (*)	-185 mm	
10 m	-169 mm	-182 mm (*)	-185 mm	
20 m	-170 mm	-180 mm (*)	-182 mm	
30 m	-168 mm	-175 mm (*)	-176 mm	
50 m	-162 mm	-167 mm (*)	-168 mm	
> 70 m	-160 mm	-165 mm (*)	-167 mm	
Settlement at the top of RSS facing	N/A	45 mm	50 mm	Figure G-11
Lateral displacement of RSS facing	N/A	4 mm	5 mm	Figure G-12
Rotation of the RSS facing	N/A	0.001	0.002	-
Maximum Heave at Malden Rd	22 mm	16 mm	~5 mm	Figure G-13

(-) Denotes settlements

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

(†) Distances measured perpendicular to the bridge abutment.

(*) Indicates calculated settlement at top of wall/abutment backfill compensated/corrected during constructions.

The above results indicate that long-term post construction settlements should in general be nominal (less than 10 mm) meeting the PA requirements. The calculated post-construction differential rotation displacements, measured as the difference between the position of the wall top and lowest exposed part of the wall face are significantly less than 50% of the required minimum batter.

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

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

5.6.4 RSS Wall External Stability

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

Bearing Capacity: The design of the RSS may be based on the following bearing capacity values (q_{uls}) determined for the native subgrade soils at the abutments (Table 5-11).

Table 5-11: Subgrade Bearing Capacity

Abutment	Assumed Lowest Subgrade Elevation	Loading Condition	q_{uls} (kPa)
North & South Walls	179.5 ⁽¹⁾	Short-Term (Undrained)	280 ⁽²⁾
		Long-Term (Drained)	510 ⁽³⁾

(1) Below RSS wall

(2) Based on an average initial (no strength gain) cohesion of 55 kPa within the assumed zone of influence of the RSS wall foundation

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

Sliding Resistance: The geotechnical resistance against sliding (H_{ri}) can be determined using the following expression:

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

Where:

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

c' = cohesion/adhesion at sliding interface

δ = friction angle at sliding interface

V = vertical force (kN)

H_f = design horizontal load (kN)

Allowance for buoyancy where applicable should be made. The following interfaces properties can be used in the design of RSS walls (Table 5-12).

Table 5-12: Soil Properties for use in Base Sliding

Interface	Undrained (Short-Term)		Drained (Long-Term)	
	δ , degrees	C' , kPa	δ , degrees	C' , kPa
RSS to Silty Clay	0	60	30	0

Based on geotechnical analyses discussed in Sections 5.6.1 to 5.6.4, preliminary abutment configurations and dimensions were determined (Table 5-13). As noted previously in Sections 5.6, the abutment configurations and dimensions indicated in these analyses are the minimum required, and are to be finalized by proprietary suppliers. The final design of the abutments is to be developed in consultation with the suppliers of the proprietary components. A 0.3 m thick granular mat is proposed to be placed prior to construction of RSS, as shown in the proposed abutment configurations in Figure 5-1. Alternatively, the abutments can be built with a 1.5 m thick Reinforced Granular Mat (RGM) as presented in the below table.

Table 5-13: Tentative Design Dimensions of the Abutment (North and South)

Abutment Option	Assumed Total Height ⁽¹⁾ , m	RSS Structure Size (Width \times Height) ⁽²⁾ , m	LWF Volume ⁽³⁾ , m ³ /m
Without RGM ⁴	8.7	6.0 \times 3.9	12.0
With 1.5 m thick	8.7	6.0 \times 3.9	None

- (1) Maximum height, measured from top of finished grade at abutment centreline to the base of the RSS structure
- (2) Height measured between underside of the stem (pile cap) and bottom of RSS. The RSS supplier may require wider structures to meet the internal design requirement. The effects of a wider structure on bearing capacity will need to be assessed.
- (3) RGM – Geogrid Reinforced Granular Mat

5.7 False Abutment Wingwalls

The wingwalls for the false RSS abutment consist of a high RSS wall retaining the backfill between bridge B-4 and B-5, and tapered RSS walls located north of and outside the bridge footprint (Drawing 285380-03-61-SEG3-0511).

Global stability analyses have been carried out for the highest section of the walls. 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 5-14.

Table 5-14: Calculated Factors of Safety for Wing Wall Global Instability

Maximum Wall Height, m	RSS Width, m	Minimum Calculated Factor of Safety		
		Undrained		Drained
		Short-Term	End-of-Construction	Long-Term
5.6	7	1.52 (1.35) ¹	1.69 (1.53)	2.28 (2.10)

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

In case of tapered wing walls at north, bearing capacity checks have been carried out for two configurations, namely long and short panels. The long panel refers to the long RSS wall immediately outside the bridge footprint, while the short panel (assumed 1.5 m lower) represent the remainder of tapered wall. Table 5-15 shows the recommended RGM and LWF requirements based on the external stability and bearing capacity of the RSS walls.

The wingwall configurations and dimensions indicated in these analyses and shown in Figure 5-1 indicate 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 5-15: Tentative Wing Wall Dimensions

Wing Wall Location		Maximum Wall Height, m	RGM Thickness, m	RSS Width, m	LWF required within RSS
South Tapered Wall	Long	5.6	1.5	7.0	14 m ³ /m
	Short	4.1	GM*	6.0	No
North Wall (between Bridge B-4 and B-5)		5.6	1.5	7.0	14 m ³ /m

(*) Minimum 300 mm Granular Mat (GM) required to be placed above approved subgrade prior to RSS construction.

Reinforced concrete wing wall flared at 730 to the highway 401 centerline is indicated at each corner of the structure (Drawings 285380-03-061-SEG3-0408 and 0409). The earth pressures on these structural elements are being provided in the next section.

5.8 Backfilling and Earth Pressures on Walls

Behind the concrete abutment and wing walls, non-frost susceptible free draining granular fill should be placed in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC). The backfill should be compacted in maximum 200 mm thick loose lifts in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the backfill. Other aspects of the abutment backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150.

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

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

Earth pressures on abutments and wing walls may be calculated on the basis of the following parameters listed in Table 5-14). Group III soils can be used as general backfill in 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.

Table 5-16: Soil Parameters for Earth Pressure Calculations

Soil Parameter	Group I Soils	Group II Soils	Group III Soils	LWF
Fill Unit Weight, kN/m ³	22	21	20.5	12.5
Friction angle, (degrees)	33 to 35	29 to 32	22 to 30	35
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	0.27
'At Rest' or Restrained, $K_o^{(*)}$	0.43 to 0.46	0.47 to 0.52	0.50 to 0.62	0.43
'Passive', $K_p^{(*)}$	3.3 to 3.7	2.9 to 3.2	2.2 to 3.0	3.7

(*) 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.

5.9 Permanent Subdrainage System

A permanent subdrainage system should be provided behind the abutments and connected to the roadway drainage system. The permanent subdrainage system should accommodate seepage from external sources, like runoff from ground surface, perched groundwater, or accidental water main breaks and groundwater recharge from infiltrations from ground surface sources.

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

Depending on the gradation of the imported granular soils in contact with native silty clay, silty sands and silts, a filter material layer may be required at the interface between native and imported soils. The LWF material must be encapsulated within a filter fabric.

6 Construction Requirements

6.1 General Construction Requirements

Drawings 285380-04-094-WIP3-0572 and 0573 provide the required specifications for placement of regular backfill and lightweight fill behind abutment walls.

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 bridge. 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. Construction requirements related to gas main along Malden Road, wick drains, embankment and bridge approachways are described in “Design Report - High Embankments” (ref. R-2).

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. Undewatered native sand / silty sand may be classified as Type 4 Soils. 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 layer above the design elevation should not be less than 500 mm and should be carried out only when the Contractor is ready to prepare and cover the subgrade with the materials specified in the design same day the final excavation is exposed and approved. No construction traffic should be permitted over subgrade without approved protective covers.
- The final excavation lift above the design subgrade should be carried out using buckets equipped with smooth lips. Once exposed, the subgrade must be immediately inspected. Upon approval, the subgrade should be immediately protected; depending on the type of construction, geofabrics, granular mats, a skim coat (minimum 75 mm thick) of lean concrete protection (mud mat), etc. should be used.
- Regular monitoring and inspections of the condition of the temporary slopes for signs of instability, deterioration, sloughing, etc should be carried out by qualified personnel. Appropriate mitigation measures should be implemented.

- Excavations in this area should be limited in size in the area and appropriate monitoring of the residence should take place. Monitoring should consist of a precondition survey along with regular surveying conducted of the nearby utilities, residences, etc.
- Air quality and subgrade pore pressure monitoring should be carried out during construction. The equipment operating in confined spaces should be selected to safely operate in a potentially soil gas environment.

6.2 Construction Dewatering

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

Excavation within the upper granular deposits may encounter moderate to significant groundwater inflow. Runoff and seepage into the excavations from perched groundwater, old farm tiles, utility trenches, and upper granular layers are also likely to occur. In adverse conditions, the runoff and seepage from perched groundwater and sand/silt pockets can be significant and accompanied by piping and wash-outs of the fines causing sloughing of the slopes.

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

Effective drainage is an important aspect in the life expectancy and performance of any abutment wall, wing wall, or pavement structure associated with the bridge. 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 E. C. Row East Bound Lane.

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

6.3 Instrumentation and Monitoring during Construction

As mentioned earlier in Section 3.3, a program of site instrumentation and monitoring of the temporary works during construction should be implemented by the Contractor in addition to the limited instrumentation already installed during the geotechnical investigation.

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 (ref. R-3).

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

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

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

6.4 Corrosion Potential

Analytical testing was carried out on samples of the silty clay stratum obtained in Boreholes BH B4-1 (Sample 2A). Table 6-1 summarizes the results of various analyses carried out on the soil samples to assess the potential for corrosion on concrete and metallic elements.

Table 6-1: Results of Analytical Testing on Soils

Location of Soil Samples	Elevation of Soil Sample	pH	Redox Potential, mV	Resistivity, ohm.cm	Sulphide, mg/kg	Sulphate, mg/kg
Borehole BH B4-1 (Sample 2A)	179.3	7.59	161	2390	< 0.20	227

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-12).

Based on the measured electrical resistivity, pH, redox potential, sulphide contents etc., the tested soil would be considered noncorrosive to buried metallic elements (ref. R-4).

The above results should be further 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 construction quality control program, including geotechnical inspection, instrumentation, testing and instrument monitoring, should be developed and implemented throughout the construction phase. In addition, related laboratory testing should be carried out in conjunction with the fieldwork to monitor compliance with the various materials and project specifications.

7 Limitations of Report

This report presents the subsurface soil and groundwater conditions inferred from geotechnical investigation and geotechnical design of the embankment and bridge structures mentioned in the report. The report was prepared with the condition that the structural and other designs of the Parkway 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 Parkway project site to confirm that the subsurface conditions do not deviate materially from those encountered in test holes, and that any material deviations, if encountered, do not adversely affect the geotechnical design.

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

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

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

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

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

8 Closure

The geotechnical report for Bridge B-5 was prepared 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., who also provided the senior review of the report. Mr. Matt Oldewening, P.Eng., managed the geotechnical investigation and Mr. Brian Lapos, P.Eng., is the project manager.

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

Yours truly,

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



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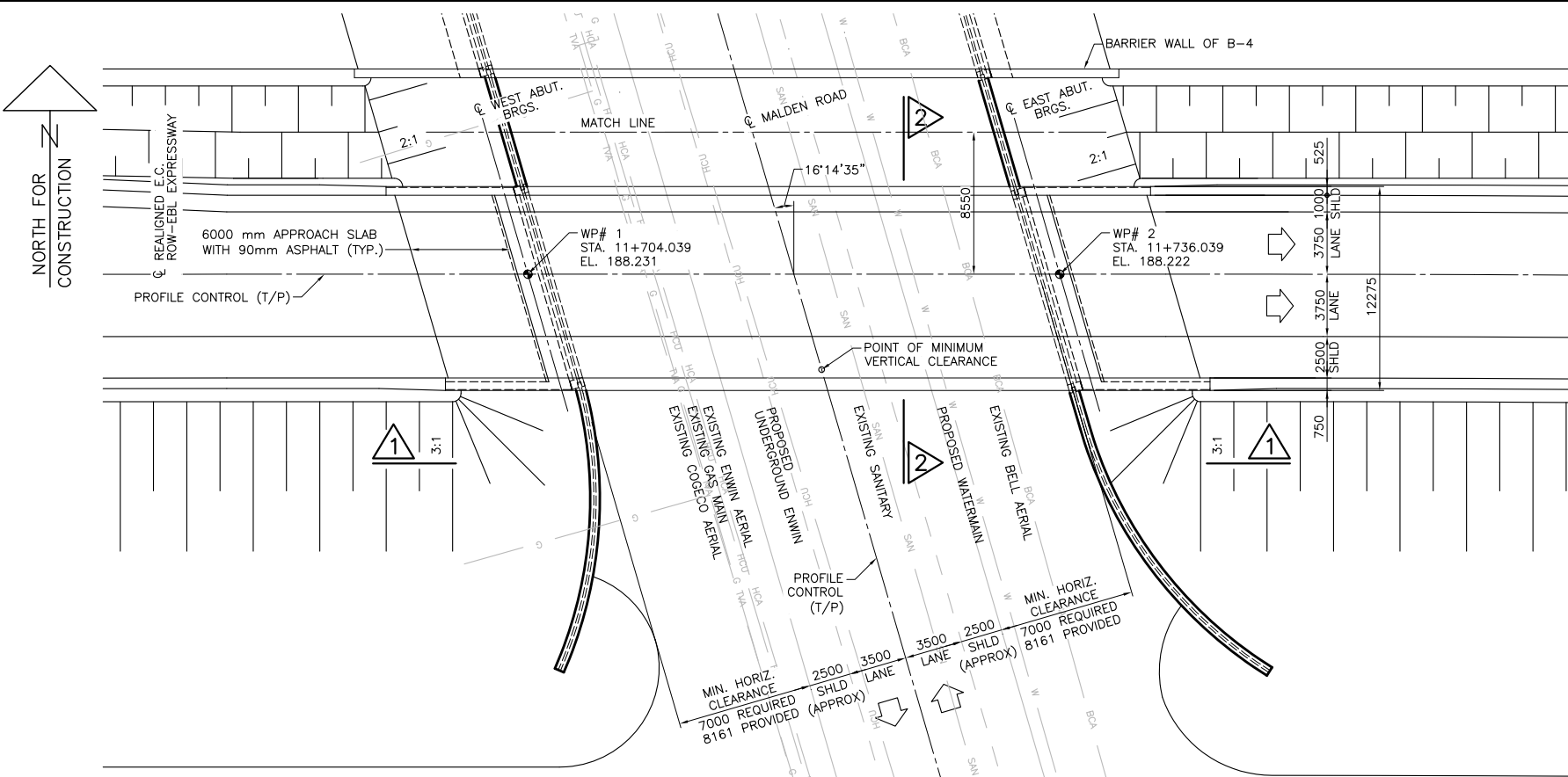
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Drawings

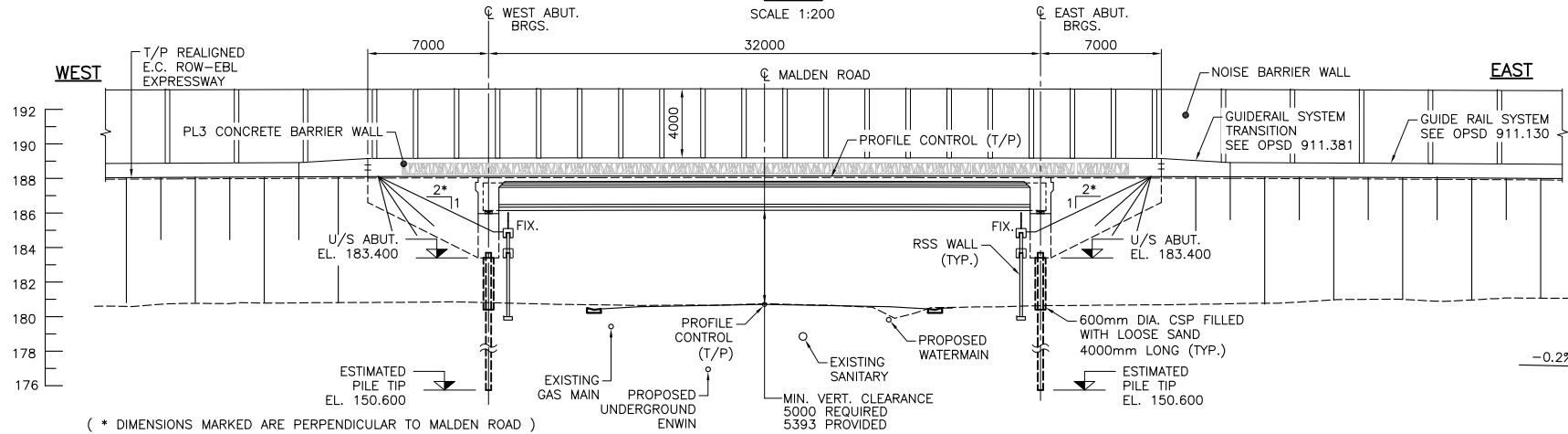
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Document: Geotechnical Investigation and Design Report - Bridge B-5 (Malden Rd.
Overpass – Realigned E.C. Row EBL, Sta. 11+704 to 11+736, Windsor)
Doc No.: 285380-04-119-0115 (Geocres No. 40J6-48)

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



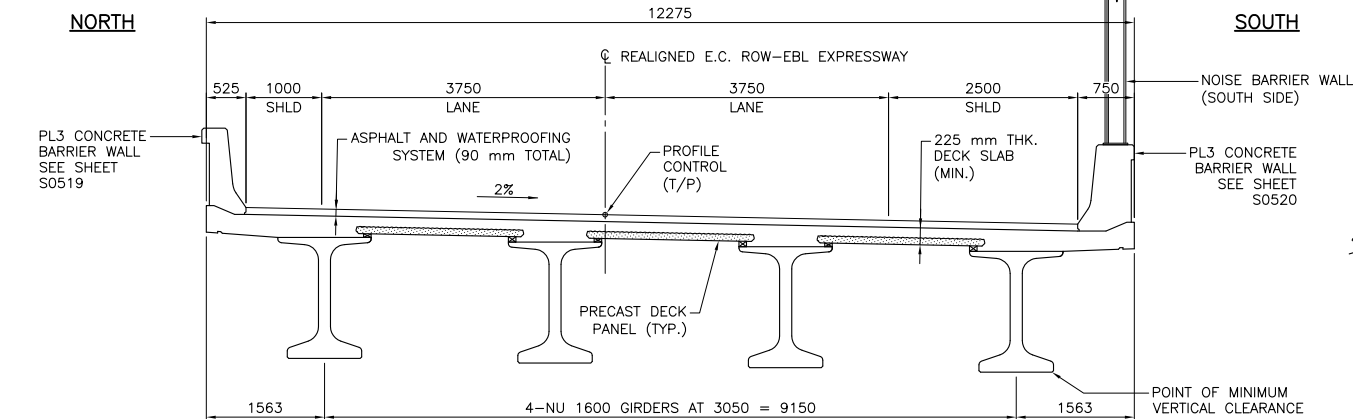
PLAN

SCALE 1:200



(* DIMENSIONS MARKED ARE PERPENDICULAR TO MALDEN ROAD)

SCALE 1:200



SCALE 1:50

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

**Parkway
Infrastructure
Engineers**



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



NEW CONSTRUCTION
BRIDGE B-5
REALIGNED E.C. ROW EBL - MALDEN ROAD OVERPASS
GENERAL ARRANGEMENT

SHEET
S0501

Phase 3
IFC

GENERAL NOTES

1. CLASS OF CONCRETE:

- PRECAST GIRDERS: 60 MPa
- PRECAST DECK PANELS: 40 MPa
- CAST-IN-PLACE DECK OVERLAY: 40 MPa
- REMAINDER: 30 MPa

2. CLEAR COVER TO REINFORCING STEEL:

- FOOTINGS: TOP 100 ± 25
BOTTOM 70 ± 20
- DECK: TOP 70 ± 20
BOTTOM 40 ± 10
- REMAINDER U.N.O: 70 ± 20

3. REINFORCING STEEL:

- REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
- BAR MARKS WITH PREFIX 'C' DENOTE COATED BARS.
- BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
- BAR MARKS WITH PREFIX 'G1' DENOTE GFRP GRADE 1 BARS
- STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 OR TYPE XM-28 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa, UNLESS OTHERWISE SPECIFIED.
- UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.
- BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12-1 AND SS12-2 UNLESS SHOWN OTHERWISE.

CONSTRUCTION NOTES

- THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESS FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT THAN THOSE SPECIFIED IN THE BEARING DESIGN DATA TABLE, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL AND CONCRETE FORMWORK TO SUIT.
- BACKFILL SHALL NOT BE PLACED AGAINST ANY PORTION OF THE ABUTMENTS OR WINGWALLS UNTIL THE CONCRETE FOR THE DECK HAS BEEN PLACED AND ITS COMPRESSIVE STRENGTH HAS REACHED 30 MPa. BACKFILL SHALL BE PLACED BEHIND BOTH ABUTMENTS SIMULTANEOUSLY KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME AT ALL TIMES. DURING PLACEMENT, THE HEIGHT OF BACKFILL AGAINST ONE ABUTMENT SHALL NEVER EXCEED THE HEIGHT OF BACKFILL AGAINST THE OTHER ABUTMENT BY MORE THAN 500 mm. THE CONTRACTOR MUST ENSURE THE STABILITY OF THE ABUTMENTS DURING CONSTRUCTION.
- ALL EXISTING UTILITIES SHALL BE ACCURATELY LOCATED PRIOR TO ANY CONSTRUCTION BEING CARRIED OUT. UNLESS NOTED OTHERWISE ON STRUCTURAL AND UTILITIES DRAWINGS, ALL EXISTING UTILITIES ARE TO REMAIN IN PLACE AND SHALL BE PROTECTED FROM DAMAGE DURING CONSTRUCTION OF THE BRIDGE AND EMBANKMENTS.
- TEMPORARY EXCAVATION, SUBGRADE EXPOSURE AND PROTECTION, AND BACKFILLING SHALL CONFORM TO OPSS 902.
- SETTLEMENTS AND GROUND DEFORMATIONS SHALL BE MONITORED DURING AND AFTER CONSTRUCTION.
- VIBRATIONS SHALL BE MONITORED AT STRATEGIC LOCATIONS DURING PILING AND CONSTRUCTION ON TEMPORARY SLOPES AND ADJACENT TO UTILITIES.
- FOR ALL HIGHWAY WORKS REFER TO HIGHWAY NEW CONSTRUCTION DRAWINGS.
- FOR ALL ELECTRICAL AND ATMS WORKS REFER TO ELECTRICAL AND ATMS NEW CONSTRUCTION DRAWINGS.
- FOR ALL UTILITY WORKS REFER TO UTILITY NEW CONSTRUCTION DRAWINGS.
- FOR RSS NOTES SEE DRAWING S0510.
- FOR INFORMATION ON EXISTING PAVEMENT AND INFRASTRUCTURE REFER TO HIGHWAYS REMOVAL DRAWINGS AND GENERAL NOTES PROVIDED WITHIN HIGHWAY REMOVALS DRAWING PACKAGE.

LIST OF ABBREVIATIONS

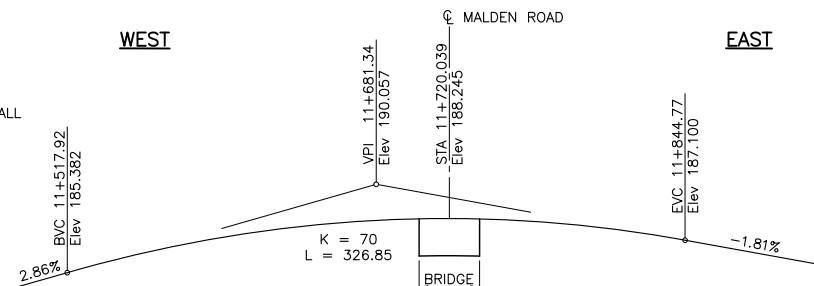
ABUT.	ABUTMENT	EXP	EXPANSION	THK.	THICK
BRGS.	BEARINGS	FIX	FIXED	T.O.	TOP OF
BVC	BEGINNING OF	HORIZ.	HORIZONTAL	T/P	TOP OF PAVEMENT
	VERTICAL CURVE	HWY	HIGHWAY	TYP.	TYPICAL
CL	CENTER LINE	MAX.	MAXIMUM	U.N.O.	UNLESS NOTED
CSP	CORRUGATED	MIN.	MINIMUM		OTHERWISE
	STEEL PIPE	N.B.	NORTHBOUND	U/S	UNDERSIDE
DIA.	DIAMETER	N.T.S.	NOT TO SCALE	VERT.	VERTICAL
EB	EASTBOUND	RSS	RETAINED SOIL SYSTEM	VPI	VERTICAL POINT OF
EL.	ELEVATION	SB	SOUTHBOUND		INTERSECTION
EVC	END OF VERTICAL	SHLD	SHOULDER	WB	WESTBOUND
	CURVE	STA.	STATION	WP	WORKING POINT

APPLICABLE STANDARD DRAWINGS

OPSD 911.130	GUIDE RAIL SYSTEM, CONCRETE BARRIER CAST-IN-PLACE, TYPE A INSTALLATION
OPSD 911.381	GUIDE RAIL SYSTEM, CONCRETE BARRIER PERMANENT TRANSITION INSTALLATION CONCRETE BARRIER TO STRUCTURE
OPSD 980.101	PEDESTRIAN BARRICADE INSTALLATION
OPSD 3000.100	FOUNDATION, PILES, STEEL H-PILE DRIVING SHOE
OPSD 3000.150	FOUNDATION, PILES, STEEL H-PILE SPLICE
OPSD 3101.150	WALLS, ABUTMENT, BACKFILL, MINIMUM GRANULAR REQUIREMENT
OPSD 3121.150	WALLS, RETAINING, BACKFILL, MINIMUM GRANULAR REQUIREMENT
OPSD 3190.100	WALLS, RETAINING AND ABUTMENT, WALL DRAIN
OPSD 3360.100	DECK LIGHT POLE BASES, STRUCTURES WITH BARRIER WALLS
OPSD 3370.100	DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD 3370.101	DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2 mm WIDE AND CONSTRUCTION JOINTS
OPSD 3390.100	DECK DRIP CHANNEL
OPSD 3419.100	BARRIERS AND RAILINGS, STEEL GUARDRAIL AND CHANNEL ANCHORAGE
OPSD 3941.200	FIGURES IN CONCRETE, SITE NUMBER AND DATE, LAYOUT
OPSD 3950.100	JOINTS, CONCRETE EXPANSION AND CONSTRUCTION, ON STRUCTURE

NOT FOR
CONSTRUCTION

PROFILE OF REALIGNED E.C. ROW - EBL EXPRESSWAY
N.T.S.



DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS	DATE	REV.	BY	DESCRIPTION
07-DEC-12	0	LM		ISSUED FOR CONSTRUCTION
DESIGN	LM	CHK	KA	CODE CAN/CSA S6-06/LOAD CL 625-ONT
DRAWN	CW	CHK	LM	SITE 6-605 DATE 03-MAR-11

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

**Parkway
Infrastructure
Engineers**



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



NEW CONSTRUCTION
BRIDGE B–5
REALIGNED E.C. ROW EBL – MALDEN ROAD OVERPASS
FOUNDATION LAYOUT

SHEET
S0504

Phase 3
IFC

NOTES

- FOR GENERAL NOTES, SEE DRAWING SHEET S0501.
- PRIOR TO COMMENCING INSTALLATION OF PILES, SEE NOTES ON DRAWING SHEET S0526.
- ALL PILES SHALL BE HP 310 X 110 WITH STEEL CONFORMING TO CSA G40.20–04/G40.21–04, GRADE 350W.
- PILE SPACING SHALL BE MEASURED AT THE UNDERSIDE OF ABUTMENT SEAT.
- EACH PILE LENGTH SHOWN IN THE PILE DATA TABLE IS THE THEORETICAL LENGTH FROM THE SPECIFIED CUT–OFF ELEVATION TO THE ESTIMATED TIP ELEVATION.
- ALL PILES SHALL HAVE TYPE I DRIVING SHOES IN ACCORDANCE WITH OPSP 3000.100 OR APPROVED EQUIVALENT.
- ALL PILE SPLICES SHALL BE BUTT WELDED IN ACCORDANCE WITH STANDARD DRAWING SS103–12, OPSP 3000.150, AND OPSS 903. SPLICE PLATES ARE NOT PERMITTED.
- SUPPLY PILE LENGTHS TO AVOID HAVING FIELD SPLICES WITHIN 6 METRES OF THE UNDERSIDE OF ABUTMENT SEATS.
- PILE DRIVING EQUIPMENT SHALL BE APPROPRIATE TO THE SUBSURFACE AND DRIVING CONDITIONS TO DEVELOP THE ULTIMATE GEOTECHNICAL RESISTANCE AND PREVENT DAMAGE TO THE PILES DURING DRIVING. CONSIDERATION SHALL BE GIVEN TO POTENTIAL DRIVING DIFFICULTIES DUE TO THE PRESENCE OF COBBLES AND/OR BOULDERS.
- HAMMER DETAILS (HAMMER TYPE AND MODEL, RATED ENERGY, HELMET AND CUSHION DETAILS) SHALL BE SUBMITTED TO THE ENGINEER FOR REVIEW 10 DAYS PRIOR TO EQUIPMENT MOBILIZATION ON SITE.
- EACH PILE SHALL BE DRIVEN IN ACCORDANCE WITH OPSS 903 TO BEDROCK OR REFUSAL IN THE VERY DENSE COHESIONLESS DEPOSIT OVERLYING THE BEDROCK TO DEVELOP AN ULTIMATE GEOTECHNICAL RESISTANCE OF 4000 kN GIVING A DESIGN FACTORED ULS RESISTANCE OF 2000 kN.
- THE ULTIMATE GEOTECHNICAL RESISTANCE OF A PILE AND REFUSAL CRITERIA SHALL BE CONFIRMED ON AT LEAST 3% OF THE PILES BY THE PDA METHOD SUPPLEMENTED WITH STATIC LOAD TESTS IN THE AREA OF THE STRUCTURE.
- SURVEY ALL PILE HEAD ELEVATIONS AT THE END OF DRIVING AND JUST PRIOR TO FORMING OF PILE CAP. RE–TAP PILES WHERE UPLIFT IS GREATER THAN 5 mm, OR AS DIRECTED BY THE ENGINEER.
- THE 600 mm DIAMETER CSP DENOTES GALVANIZED CORRUGATED STEEL PIPES WITH SQUARE ENDS AND A WALL THICKNESS OF 1.6 mm.
- DURING PILE DRIVING THE CONTRACTOR SHALL IMPLEMENT APPROPRIATE MITIGATION MEASURES AGAINST ANY SEEPAGE OF NATURAL GAS AND GROUNDWATER THAT MIGHT CAUSE LOSS OF BEARING RESISTANCE.
- THE CONTRACTOR SHALL MONITOR VIBRATIONS AT STRATEGIC LOCATIONS (e.g. TEMPORARY SLOPES, UTILITIES, AND STRUCTURES) AND ESTABLISH APPROPRIATE FREQUENCY BASED UNITS ON PEAK PARTICLE VELOCITIES IN ORDER TO PREVENT DAMAGE CAUSED BY PILE DRIVING.

APPLICABLE STANDARD DRAWINGS

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

WORKING POINT DATA			
WORK POINT No.	STATION	CO–ORDINATES	
		NORTHING	EASTING
WP #1	11 + 704.039	4681967.549	330153.470
WP #2	11 + 736.039	4681956.524	330183.511

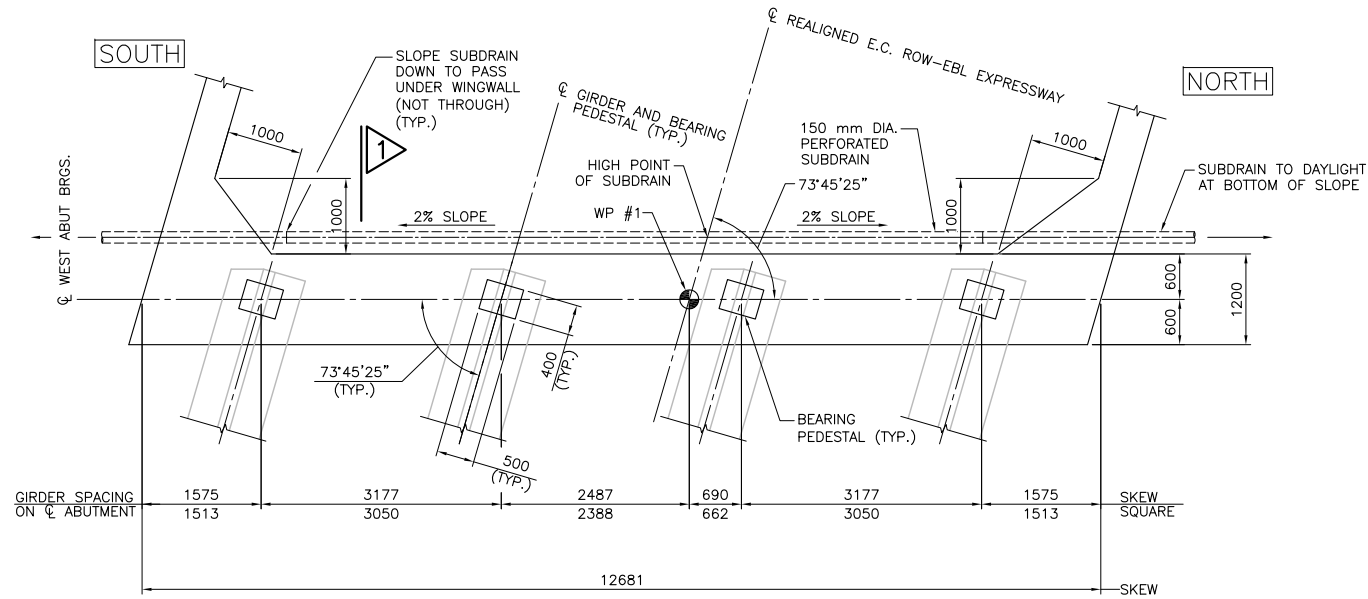
PILE DATA			
LOCATION	NUMBER OF PILES REQUIRED	LENGTH (m)	BATTER
WEST ABUTMENT	7	32.8	VERTICAL
EAST ABUTMENT	7	32.8	VERTICAL

NOT FOR
CONSTRUCTION

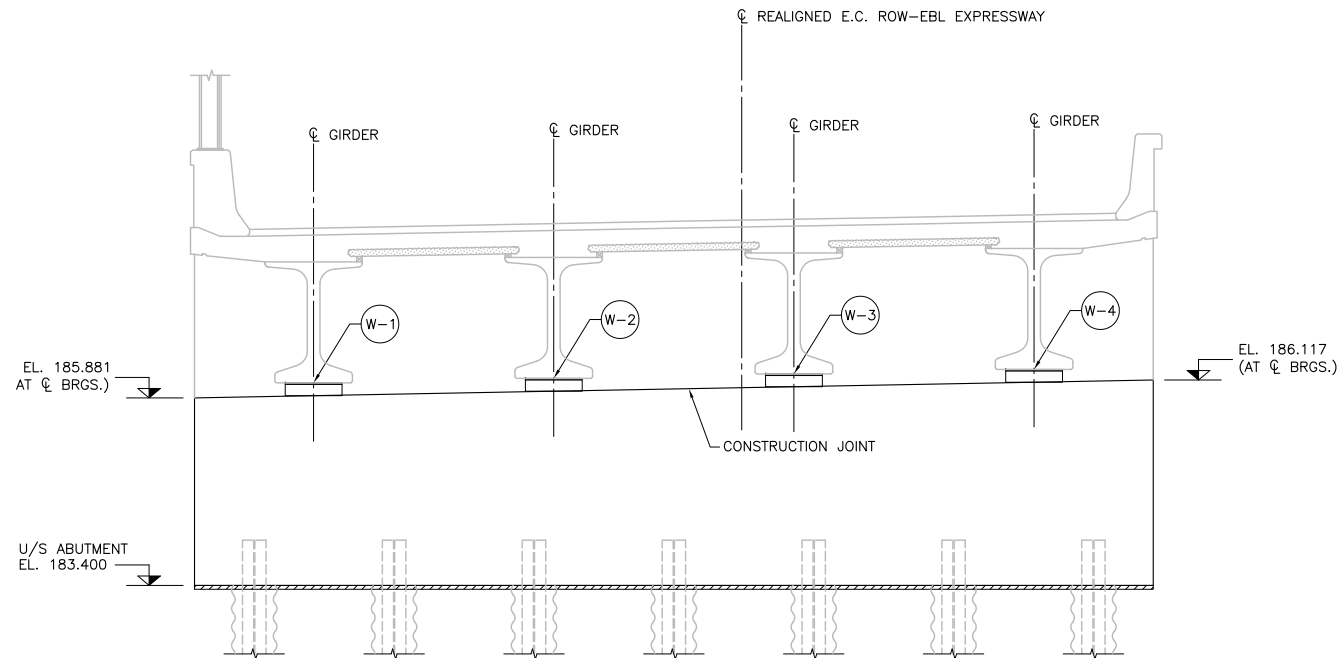
DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS		DATE	REV.	BY	DESCRIPTION
		07–DEC–12	0	LM	ISSUED FOR CONSTRUCTION
DESIGN	LM	CHK	KA	CODE CAN/CSA S6–06	LOAD CL 625–ONT
DRAWN	CW	CHK	LM	SITE 6–605	DATE 08–MAR–11

DOC: 285380–03–061–WP3–0504



PLAN AT TOP OF WEST ABUTMENT
BEARING SEAT
1:50



WEST ELEVATION
1:50

TOP OF BEARING PAD ELEVATIONS*				
LOCATION	W-1	W-2	W-3	W-4
WEST ABUT.	186.080	186.139	186.198	186.258

* SEE CONSTRUCTION NOTES ON DWG. S0501

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

**Parkway
Infrastructure
Engineers**



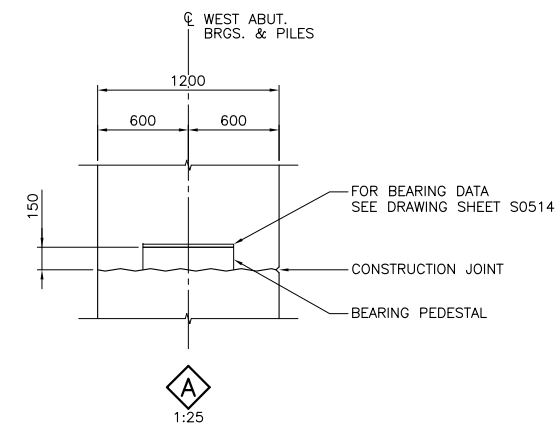
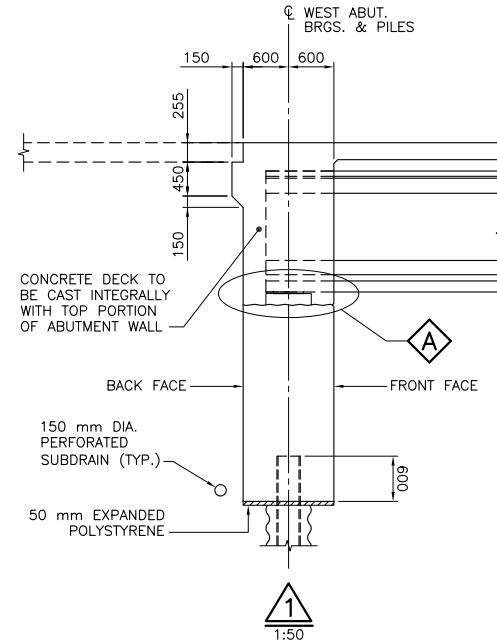
Windsor-Essex
Parkway Project
RFP No. 09-54-1007

NEW CONSTRUCTION
BRIDGE B-5
REALIGNED E.C. ROW EBL - MALDEN ROAD OVERPASS
WEST ABUTMENT LAYOUT



SHEET
S0505

Phase 3
IFC



BRIDGE CONSTRUCTION SEQUENCE

- EXCAVATE AS REQUIRED TO ELEVATION AT THE UNDERSIDE OF THE PROPOSED RSS WALLS IN FRONT OF THE ABUTMENTS.
- PLACE AND DRIVE STEEL H-PILES FOR ABUTMENTS AS SHOWN ON DRAWING SHEET S0504.
- PLACE THE 600 mm DIAMETER x 4000 LONG CORRUGATED STEEL PIPES (CSP) FOR THE PILES AT THE ABUTMENTS.
- PLACE LOOSE DRY FINE SAND (GRADATION AS PER MTO REPORT SO-96-1) AROUND THE PILE IN EACH CSP AT EACH ABUTMENT.
- CONSTRUCT THE RSS STRUCTURES AND ASSOCIATED PERMANENT SUBRAIN WORKS, AND APPROVED BACKFILL BEHIND THE RSS STRUCTURE.
- PLACE EXPANDED POLYSTYRENE AND CONSTRUCT THE ABUTMENT SEATS AND LOWER PORTIONS OF WINGWALLS UP TO THE UNDERSIDE OF THE GIRDER BEARING PEDESTALS.
- DESIGN AND INSTALL TEMPORARY LATERAL BRACING TO KEEP THE ABUTMENT BEARING SEATS PLUMB, STABLE, AND FROM MOVING HORIZONTALLY DURING CONSTRUCTION OF THE SUPERSTRUCTURE. THE MINIMUM SAFE LATERAL RESISTANCE OF THE BRACING SHALL BE 15 kN / m. BRACING SHALL REMAIN IN PLACE UNTIL THE CONCRETE FOR THE DECK AND ABUTMENT DIAPHRAGMS HAS BEEN CAST AND HAS ATTAINED A COMPRESSIVE STRENGTH OF 30 MPa.
- CONSTRUCT GIRDER BEARING PEDESTALS (PLINTHS).
- PLACE GIRDER BEARING PADS AT ABUTMENTS.
- ERECT PRESTRESSED CONCRETE GIRDERS AND INSTALL TEMPORARY BRACING TO ENSURE STABILITY UNTIL THE CONCRETE FOR THE DECK AND PERMANENT DIAPHRAGMS HAS BEEN CAST AND ATTAINED A COMPRESSIVE STRENGTH OF 35 MPa.
- PLACE PRECAST CONCRETE DECK PANELS AND CAST THE CONCRETE FOR THE DECK, DIAPHRAGMS, AND UPPER PORTIONS OF WINGWALLS MONOLITHICALLY. THE ABUTMENT DIAPHRAGMS SHALL BE CONSTRUCTED INTEGRALLY WITH THE ABUTMENT SEATS TO FORM A COMPLETE FRAME.
- INSTALL PERFORATED SUBDRAINS.
- PLACE AND COMPACT AS PER OPSS 501 GRANULAR B TYPE 1 AND LIGHTWEIGHT FILL BACKFILL SIMULTANEOUSLY AT THE NORTH AND SOUTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME AT ALL TIMES. DURING PLACEMENT, THE DIFFERENCE IN HEIGHT OF BACKFILL FROM ONE ABUTMENT TO THE OTHER SHALL NEVER BE GREATER THAN 500 mm. BACKFILL SHALL NOT BE PLACED AGAINST ABUTMENT WALLS UNTIL THE CONCRETE FOR THE DECK AND DIAPHRAGMS HAS BEEN PLACED AND ITS COMPRESSIVE STRENGTH HAS REACHED 30 MPa.
- CONSTRUCT APPROACH SLABS AND BARRIER WALLS.
- PLACE WATERPROOFING SYSTEM AND ASPHALTIC CONCRETE PAVEMENT (ACP).

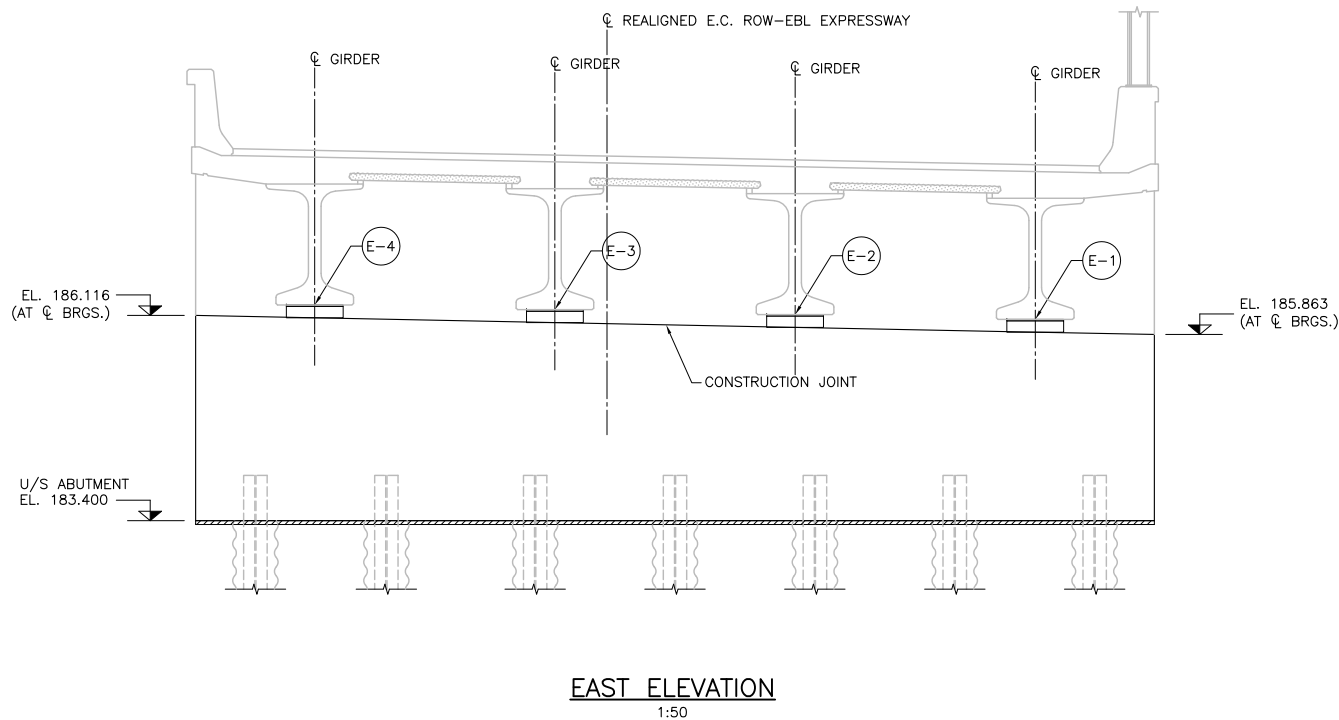
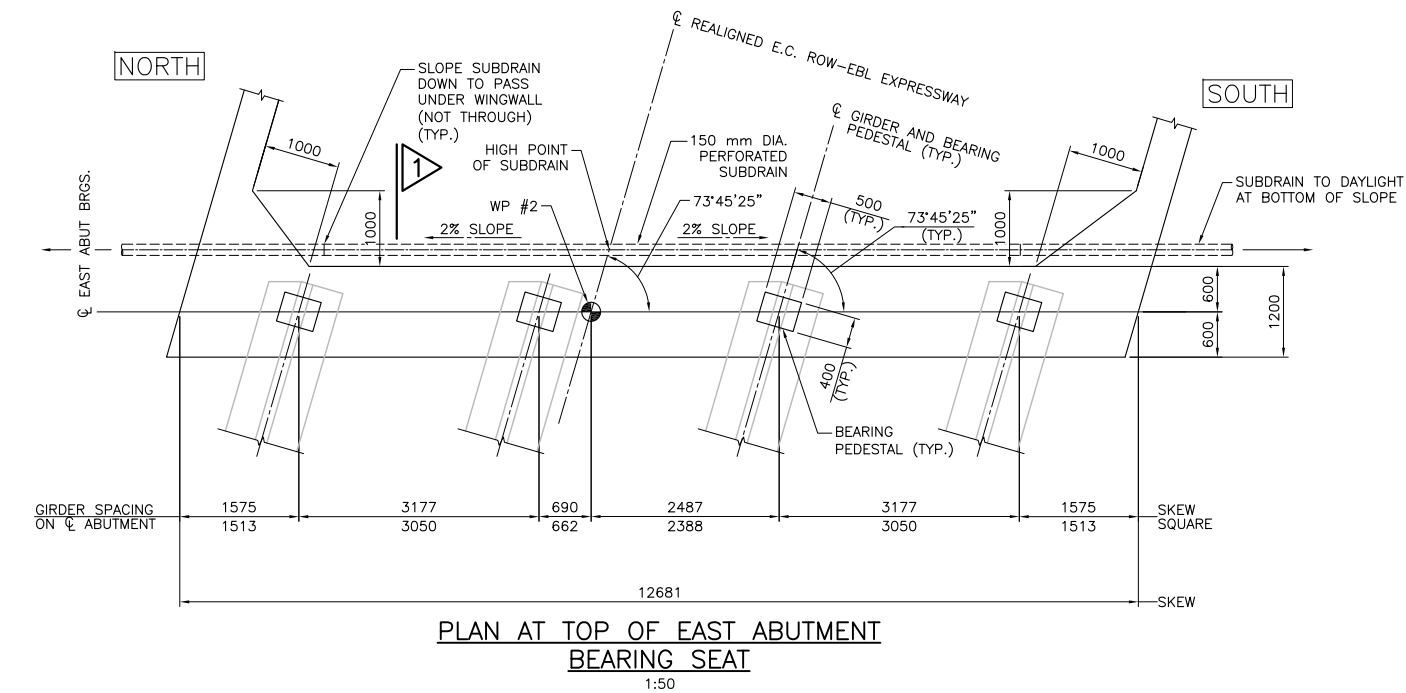
APPLICABLE STANDARD DRAWINGS

OPSD 3190.100	WALLS, RETAINING AND ABUTMENT, WALL DRAIN
OPSD 3950.100	JOINTS, CONCRETE EXPANSION AND CONSTRUCTION, ON STRUCTURE

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR
CONSTRUCTION

REVISIONS		DATE	REV.	BY	DESCRIPTION
07-DEC-12	0	LM			ISSUED FOR CONSTRUCTION
DESIGN	LM	CHK	KA	CODE CAN/CSA S6-06	LOAD CL 625-ONT
DRAWN	CW	CHK	LM	SITE 6-605	DATE 06-SEP-11



TOP OF BEARING PAD ELEVATIONS*				
LOCATION	E-1	E-2	E-3	E-4
EAST ABUT.	186.064	186.127	186.191	186.254

* SEE CONSTRUCTION NOTES ON DWG. S0501

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

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Parkway Project
RFP No. 09-54-1007



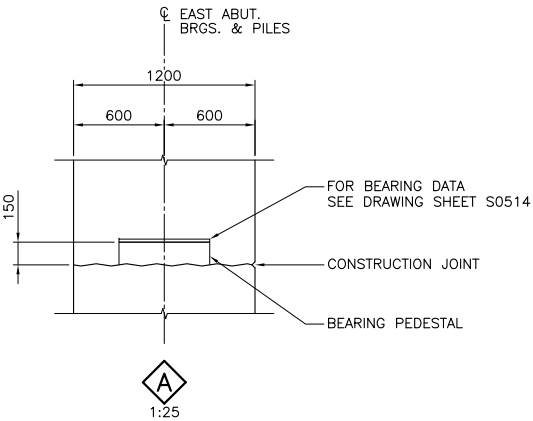
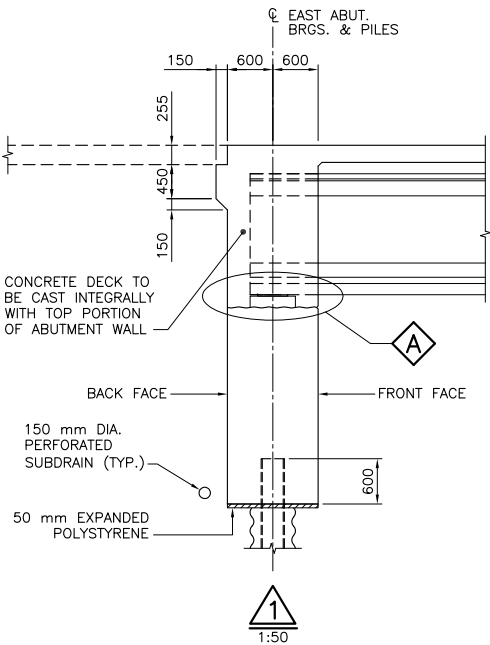
NEW CONSTRUCTION
BRIDGE B-5
REALIGNED E.C. ROW EBL - MALDEN ROAD OVERPASS
EAST ABUTMENT LAYOUT

SHEET
S0506

Phase 3
IFC

NOTES

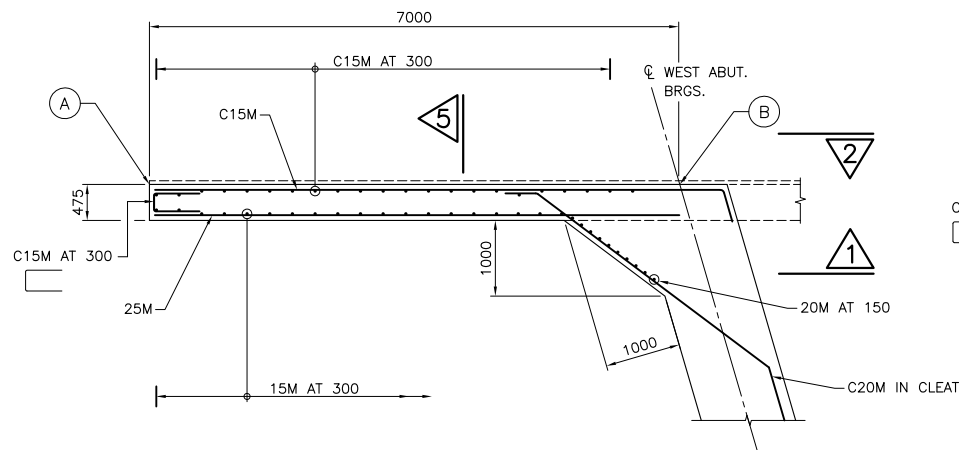
1. SEE DRAWING SHEET S0505 FOR NOTES.



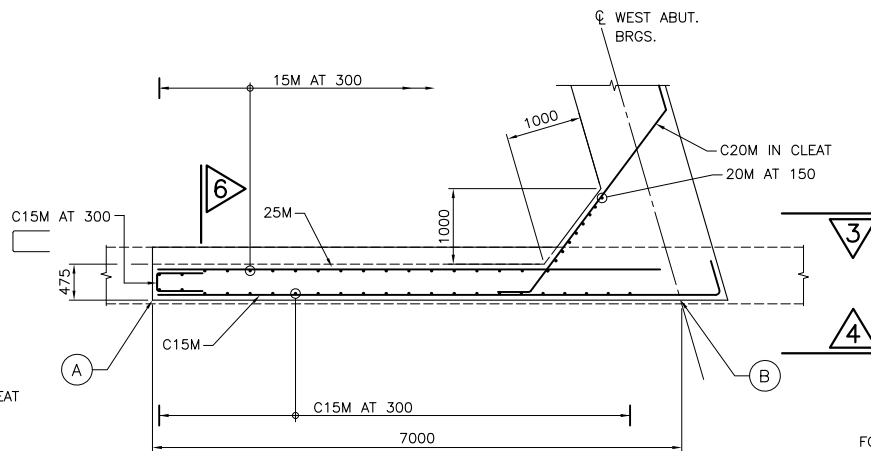
DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR
CONSTRUCTION

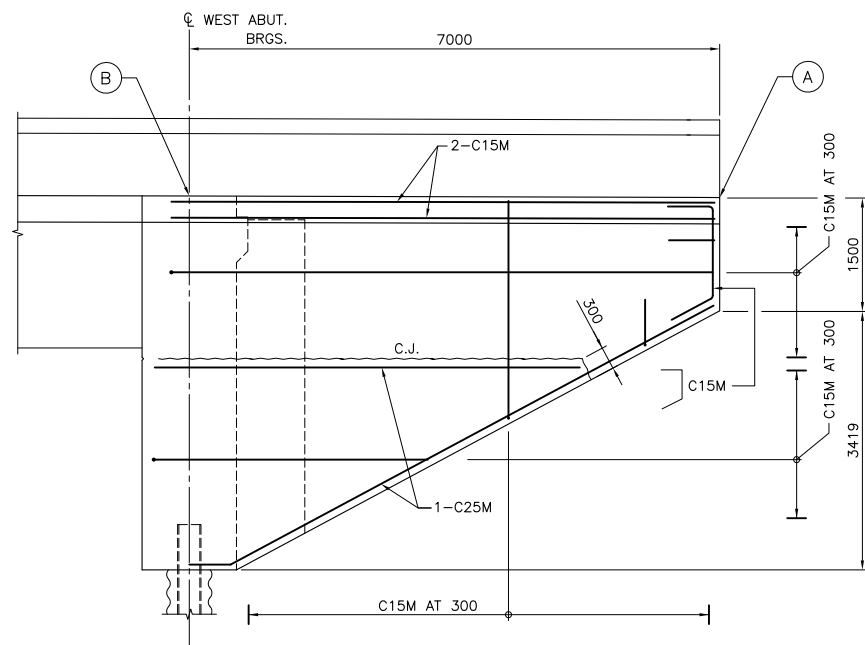
REVISIONS		DATE	REV.	BY	DESCRIPTION
07-DEC-12		0	LM	ISSUED FOR CONSTRUCTION	
DESIGN	LM	CHK	KA	CODE CAN/CSA S6-06	LOAD CL 625-ONT
DRAWN	CW	CHK	LM	SITE 6-605	DATE 04-APR-11



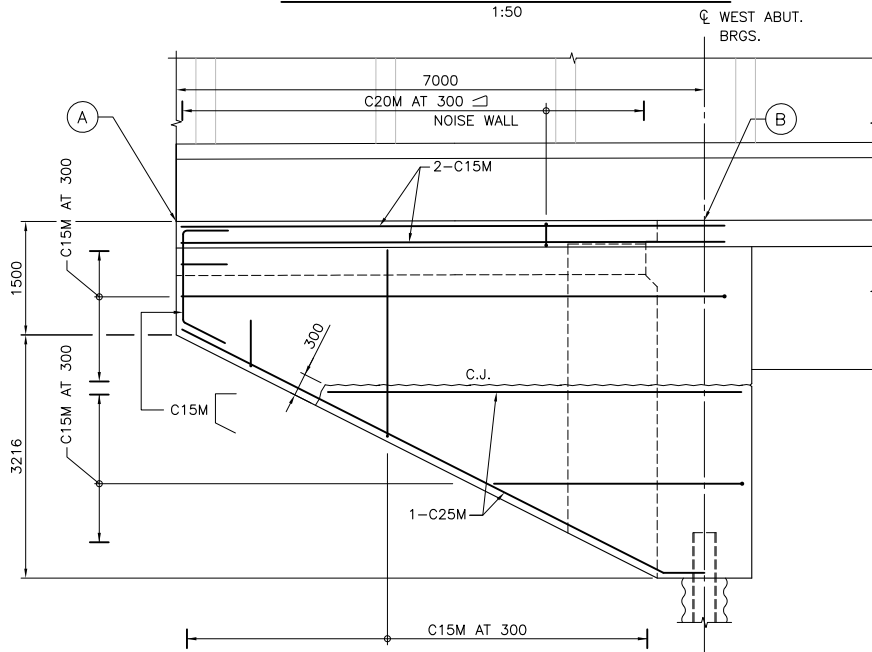
PLAN - NORTHWEST WINGWALL
1:50



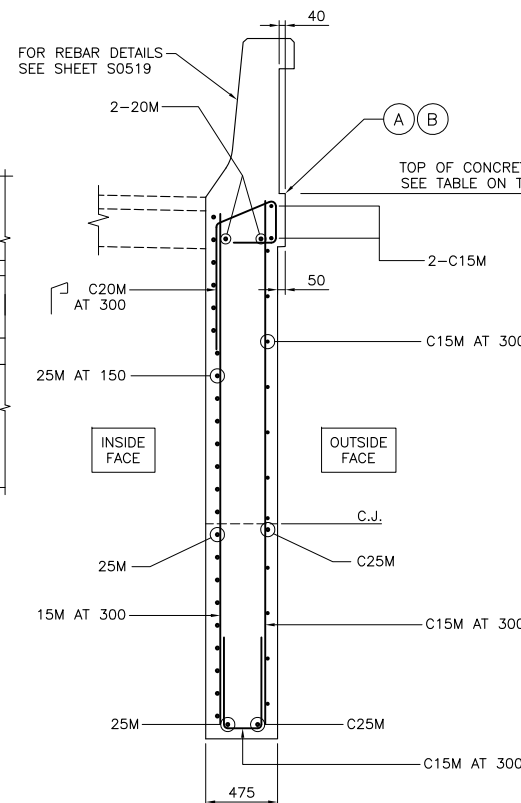
PLAN - SOUTHWEST WINGWALL
1:50



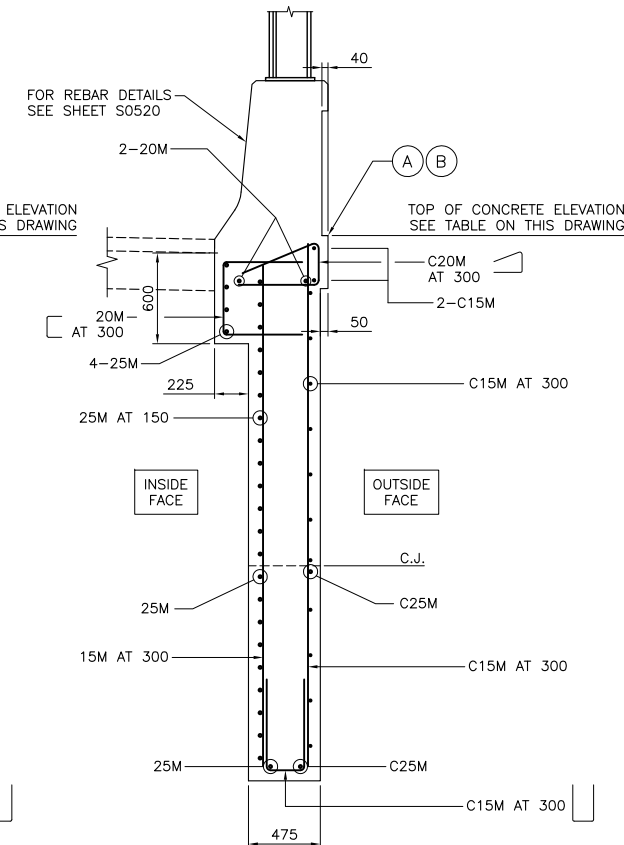
2 (NORTHWEST WINGWALL - OUTSIDE FACE)
1:50



4 (SOUTHWEST WINGWALL - OUTSIDE FACE)
1:50



5 (NORTHWEST WINGWALL - INSIDE FACE)
1:25



6 (SOUTHWEST WINGWALL - INSIDE FACE)
1:25

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

Parkway
Infrastructure
Engineers



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

NEW CONSTRUCTION
BRIDGE B-5
REALIGNED E.C. ROW EBL - MALDEN ROAD OVERPASS
WEST WINGWALL DETAILS

SHEET
S0508
Phase 3
IFC

NOTE:

1. FOR EMBEDDED ELECTRICAL WORK SEE SHEET S0525

WINGWALL ELEVATIONS

WINGWALL	POINT	
	A	B
NORTHWEST	188.329	188.348
SOUTHWEST	188.119	188.134

NOT FOR
CONSTRUCTION

REVISIONS	DATE	REV.	BY	DESCRIPTION
07-DEC-12	0	LM		ISSUED FOR CONSTRUCTION
DESIGN	LM	CHK	KA	CODE CAN/CSA S6-06/LOAD CL-625-ONT
DRAWN	CW	CHK	LM	SITE 6-605 DATE 17-APR-12

METRIC

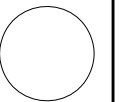
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

**Parkway
Infrastructure
Engineers**



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

NEW CONSTRUCTION
BRIDGE B-5
REALIGNED E.C. ROW EBL - MALDEN ROAD OVERPASS
EAST WINGWALL DETAILS

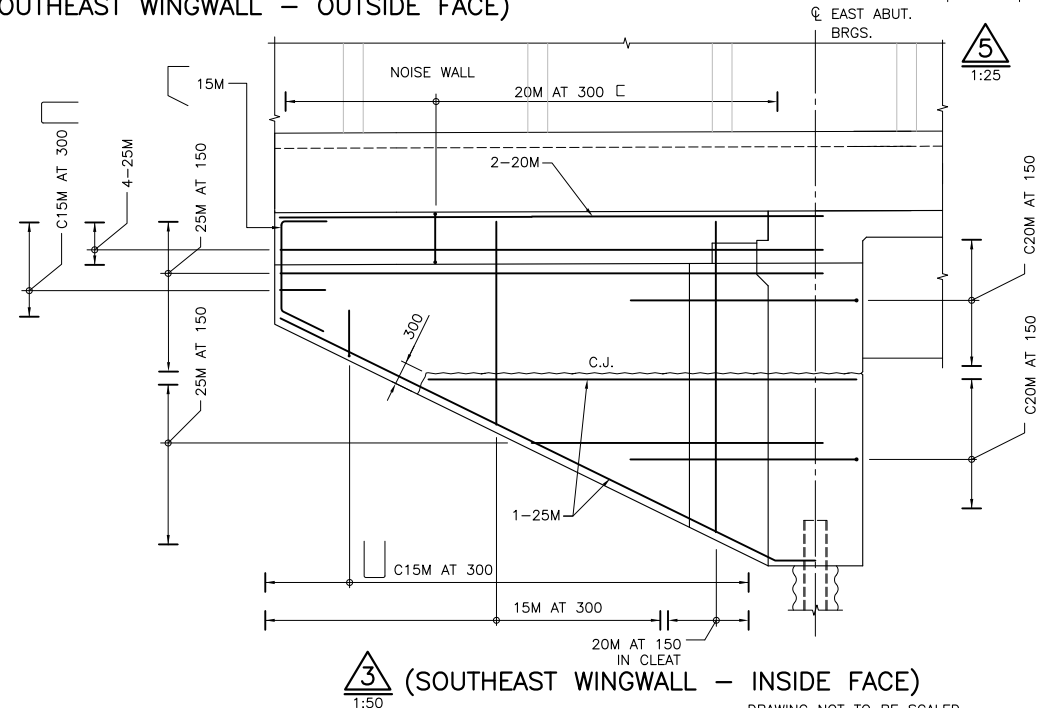
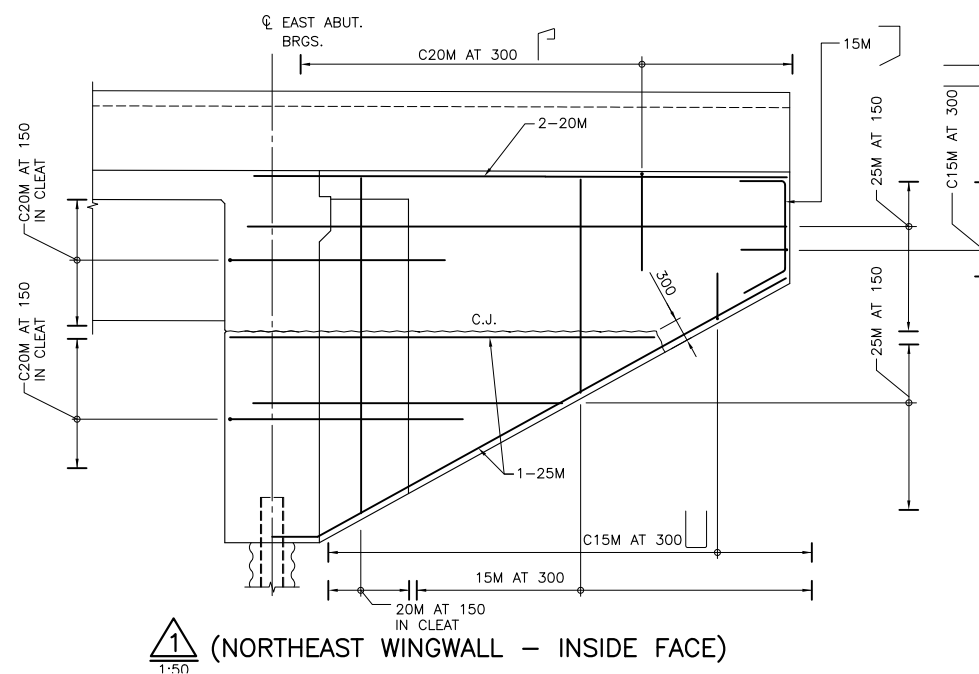
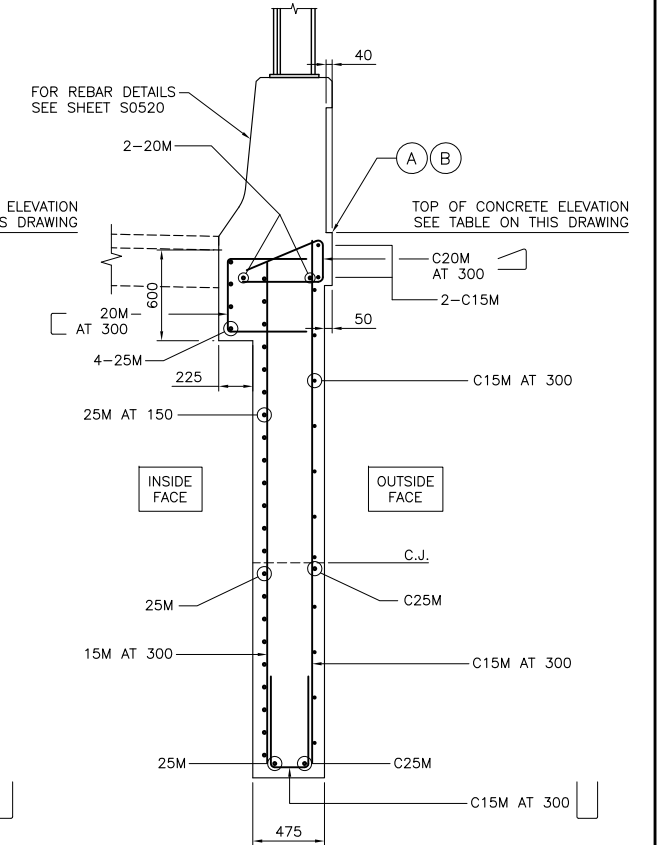
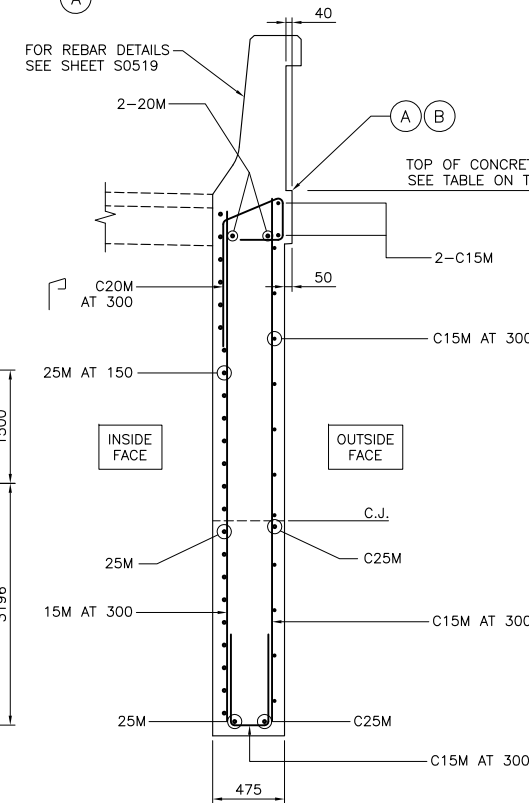
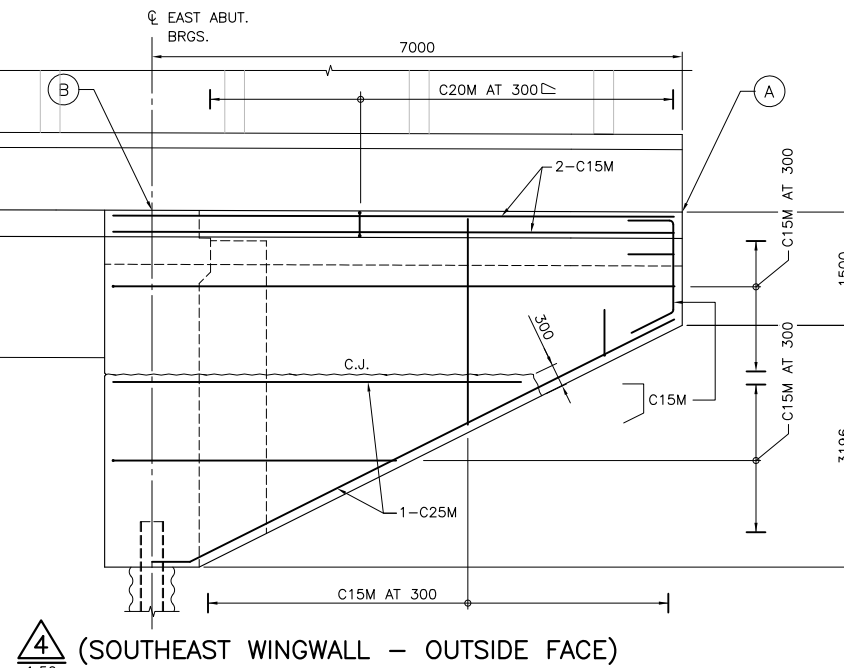
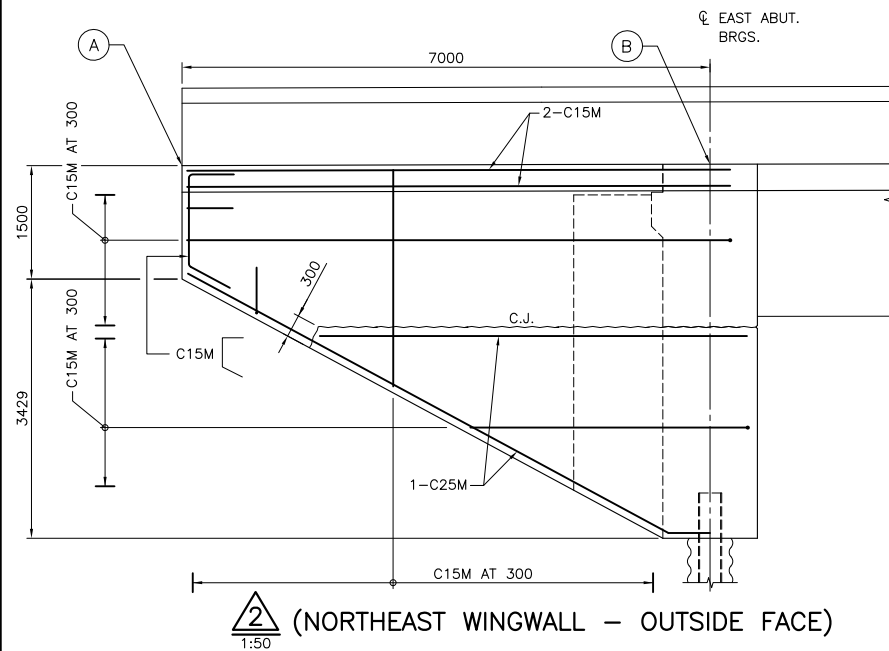
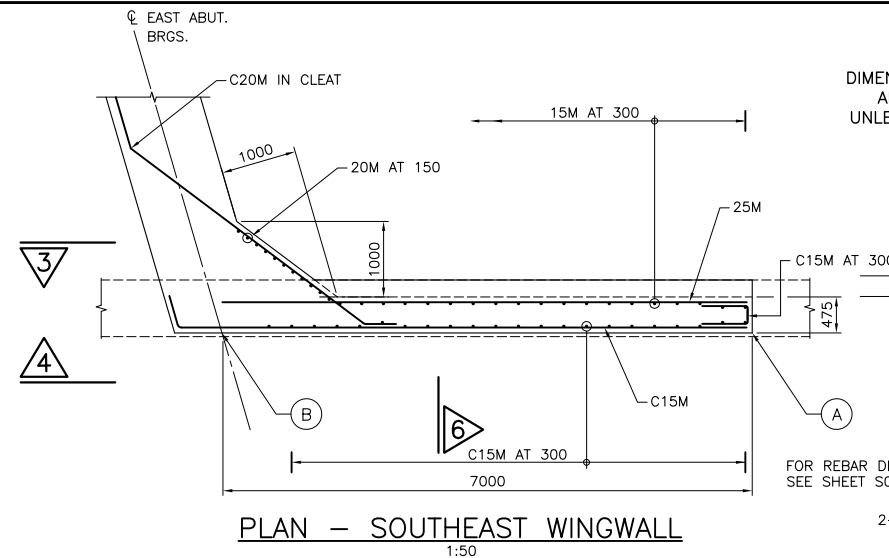
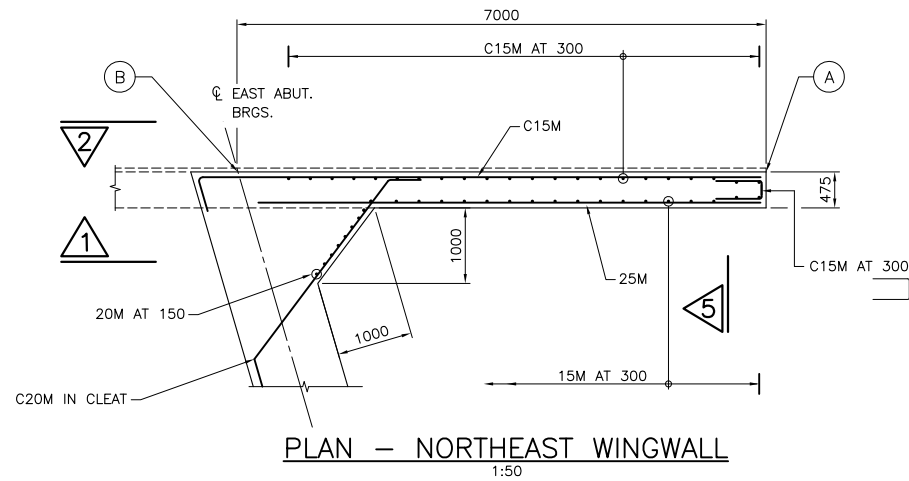


SHEET
S0509

Phase 3
IFC

NOTE:

- FOR EMBEDDED ELECTRICAL WORK SEE SHEET S0525



WINGWALL ELEVATIONS

WINGWALL	POINT	
	A	B
NORTHEAST	188.326	188.346
SOUTHEAST	188.093	188.116

**NOT FOR
CONSTRUCTION**

REVISIONS	DATE	REV.	BY	DESCRIPTION
07-DEC-12	0	LM	ISSUED FOR CONSTRUCTION	
DESIGN	LM	CHK	KA	CODE CAN/CSA S6-06/LOAD CL-625-ONT
DRAWN	CW	CHK	LM	SITE 6-605 DATE 17-APR-12

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers

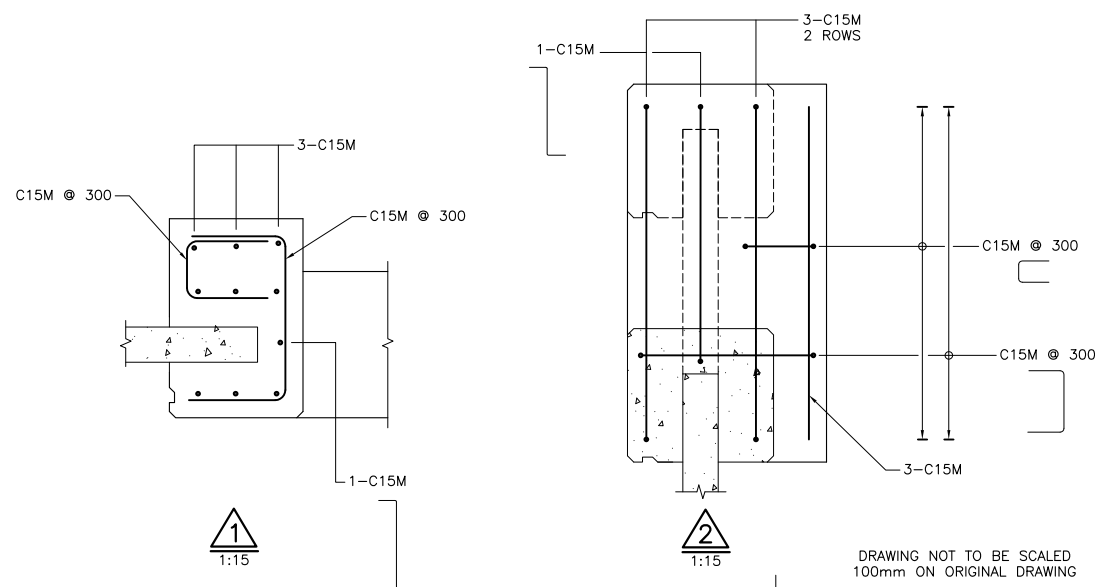
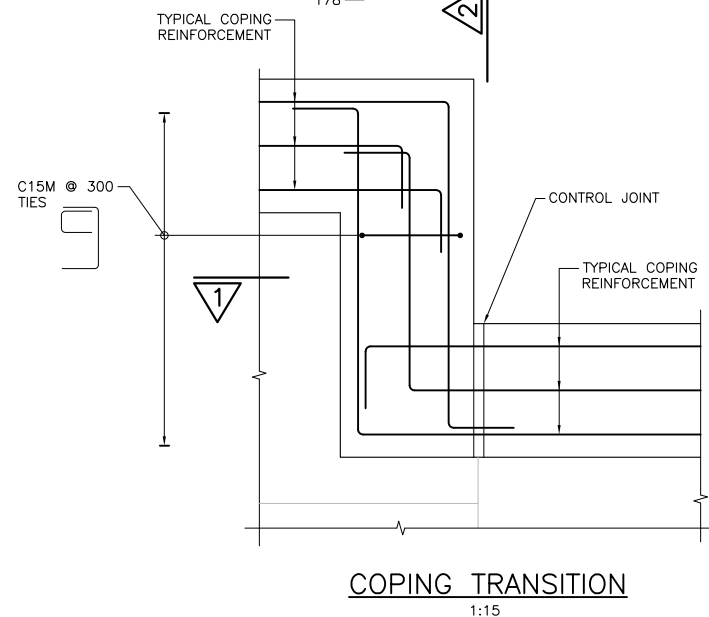
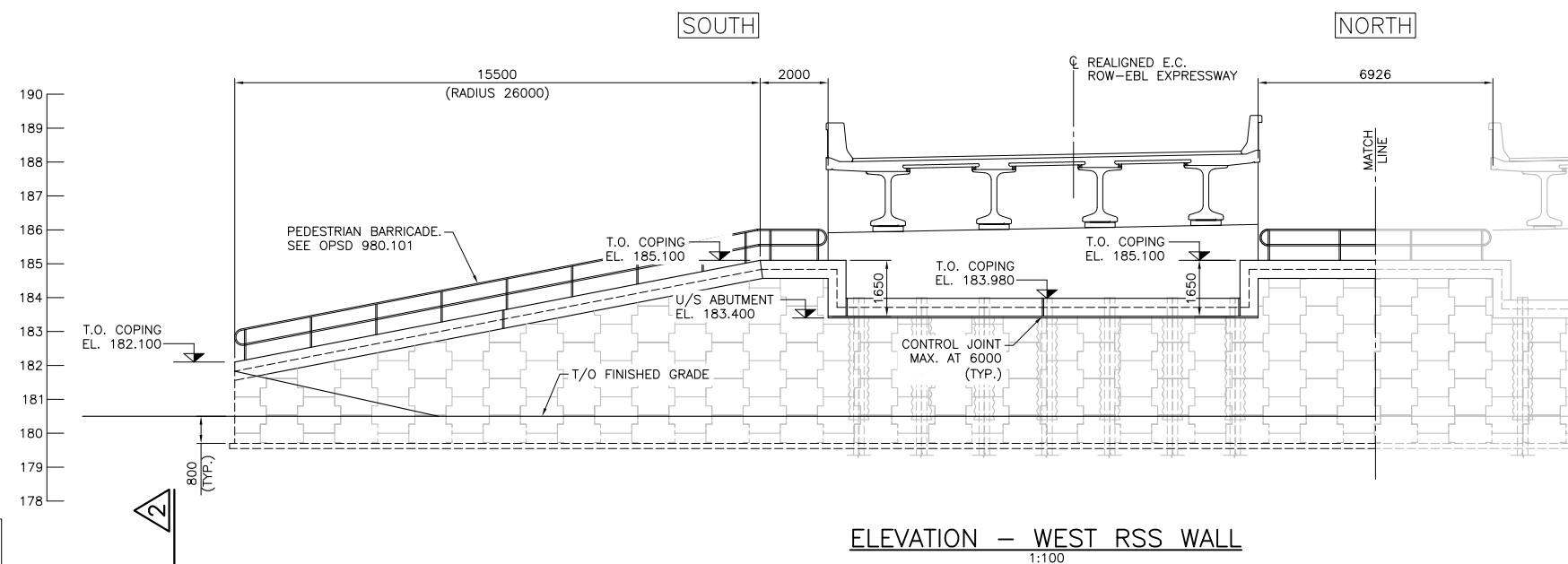
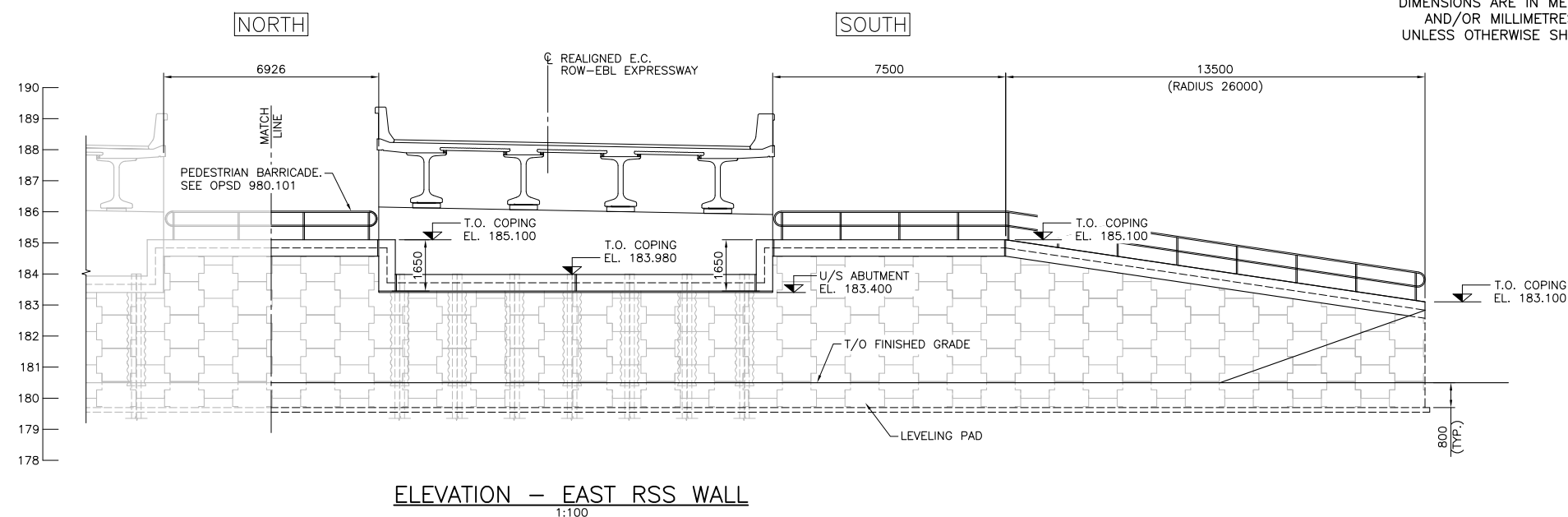


Windsor-Essex
Parkway Project
RFP No. 09-54-1007

NEW CONSTRUCTION
BRIDGE B-5
REALIGNED E.C. ROW EBL - MALDEN ROAD OVERPASS
RSS WALL ELEVATIONS

SHEET
S0511

Phase 3
IFC



NOT FOR
CONSTRUCTION

REVISIONS							
	07-DEC-12	0	LM	ISSUED FOR CONSTRUCTION			
	DATE	REV.	BY	DESCRIPTION			
DESIGN	LM	CHK	KA	CODE	CAN/CSA	S6-06	LOAD CL-625-ONT
DRAWN	SC	CHK	LM	SITE	6-604		DATE 22-JUN-12

METRIC



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

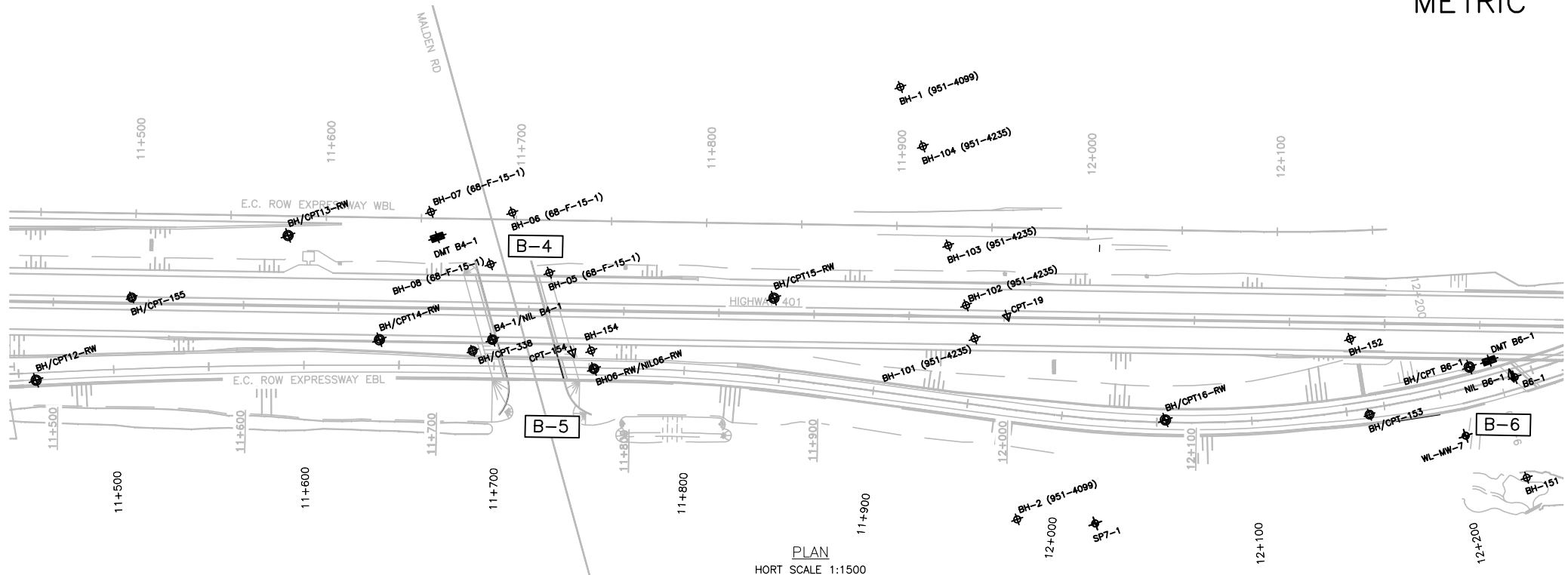


REVISIONS	DATE	REV. BY	DESCRIPTION
19-DEC-12	0	SF	ISSUED FOR CONSTRUCTION
DESIGN	SF	APR	NSV
DATE	30-MAY-11		

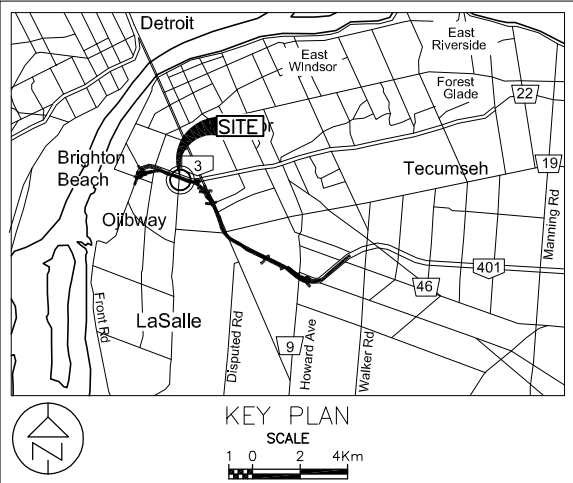
LOCATION PLAN & INTERPRETED
STRATIGRAPHIC PROFILE
STA 11+500W TO STA 12+200W

SHEET
G0501

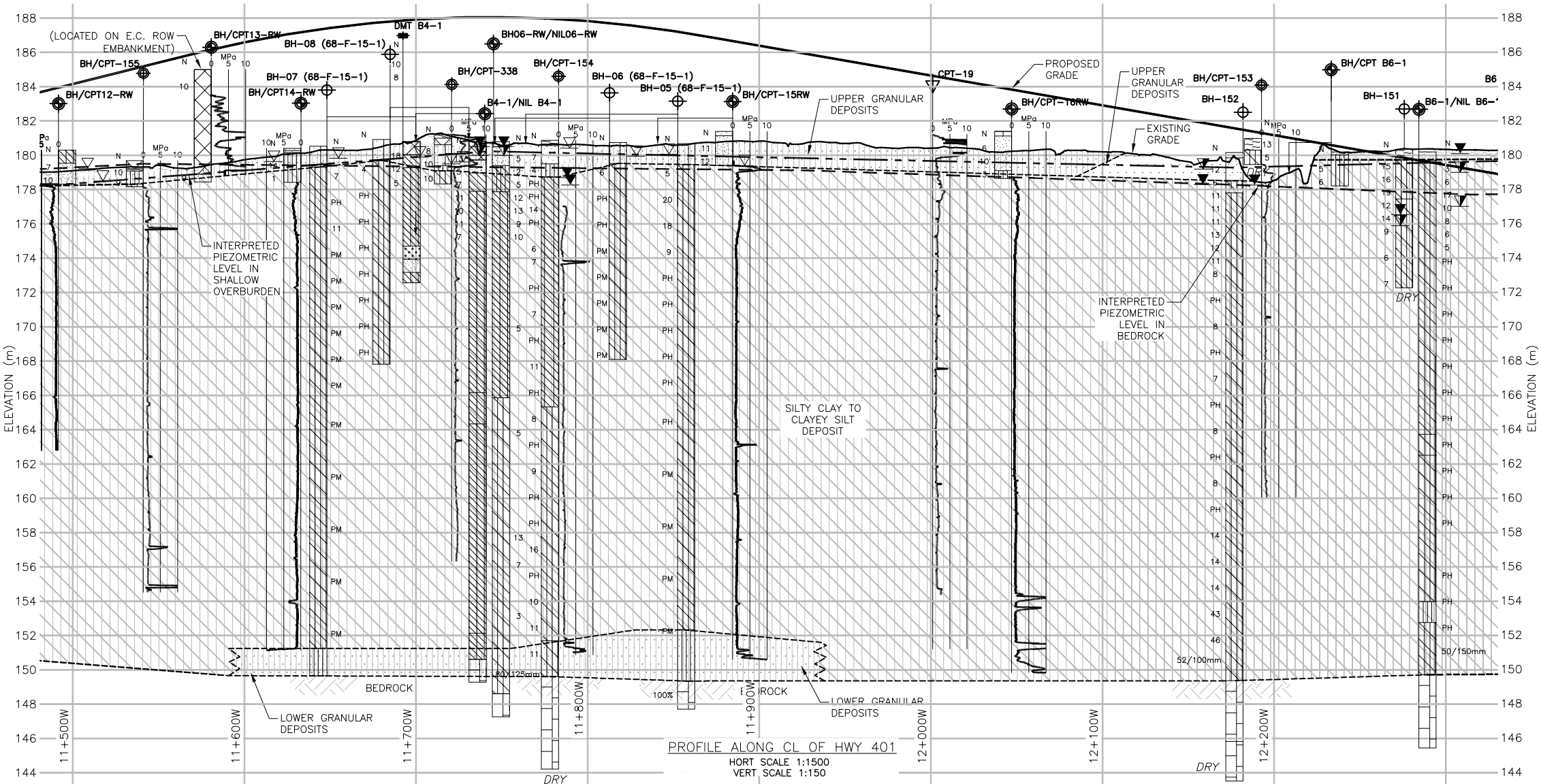
Phase 3
IFC



PLAN
HORIZONTAL SCALE 1:1500



KEY PLAN
SCALE
1:0, 2:4km



PROFILE ALONG CL OF HWY 401
HORIZONTAL SCALE 1:1500
VERTICAL SCALE 1:150

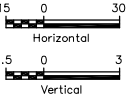
LEGEND

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

NOTES

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

SCALES



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



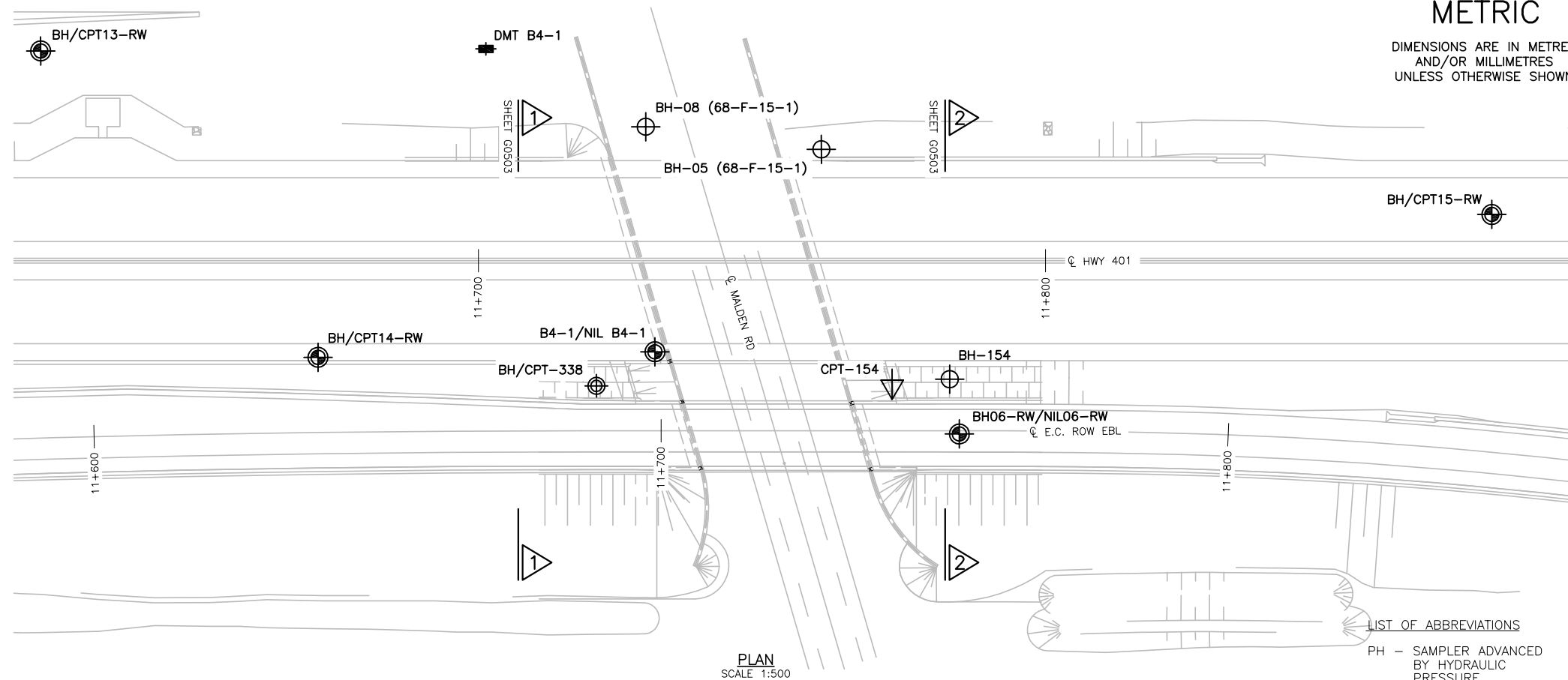
Windsor-Essex
Parkway Project
RFP No. 09-54-1007

NEW CONSTRUCTION
BRIDGE B-5
REALIGNED E.C. ROW EBL - MALDEN ROAD OVERPASS
BOREHOLE LOCATIONS & SOIL STRATA



SHEET
G0502

Phase 3
IFC



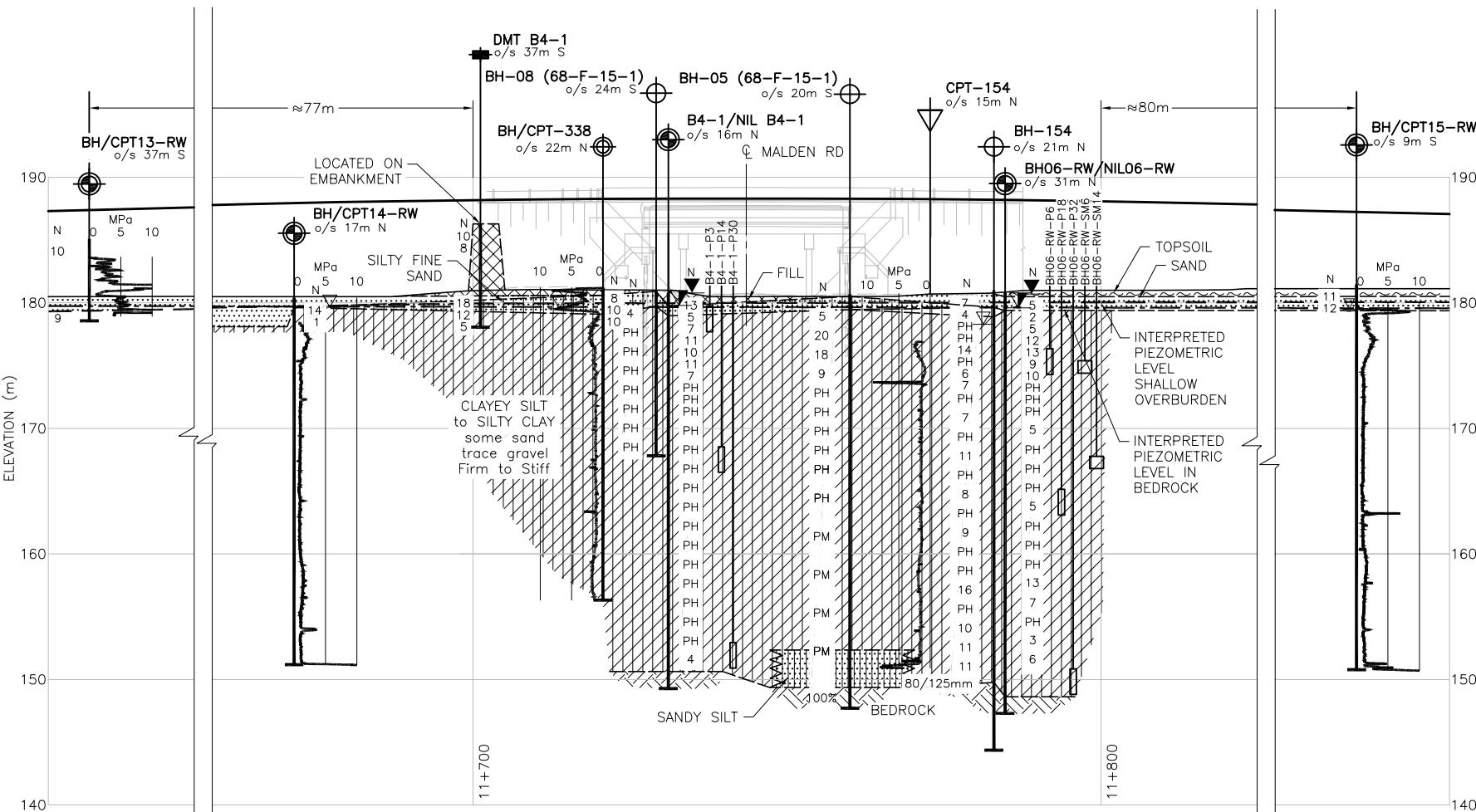
LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

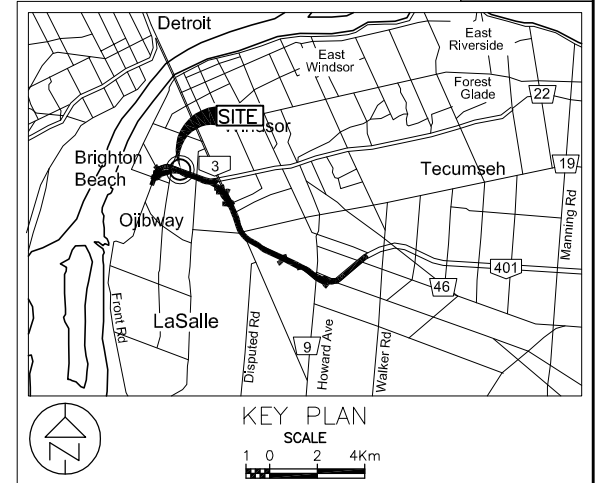
ELEVATION	No.	ELEVATION	CO-ORDINATES (UTM, NAD 83 ZONE 17)	
			NORTHING	EASTING
	AMEC BOREHOLES			
	B4-1	180.8	4681982.4	330153.5
	BH06-RW	180.8	4681950.3	330198.8
	BH/CPT13-RW	185.1	4682069.5	330070.1
	BH/CPT14-RW	180.4	4682001.9	330097.3
	BH/CPT15-RW	181.5	4681954.3	330300.3
	DMT B4-1	186.3	4682042.8	330143.9
	NIL B4-1	180.9	4681982.5	330151.6
NIL06-RW	181.0	4681948.0	330200.9	
PREVIOUS BOREHOLES				
BH-05 (68-F-15-1)	180.7	4682005.8	330193.3	
BH-08 (68-F-15-1)	180.9	4682020.2	330165.6	
BH-154	180.9	4681959.9	330200.6	
CPT-154	180.8	4681963.3	330191.0	
BH/CPT-338	181.2	4681980.3	330141.6	



PROFILE ALONG REALIGNED E.C. ROW EBL THROUGH BRIDGE B-5

HORT SCALE 1:500
VERT SCALE 1:250

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING



LEGEND

-
-
-
-
-
-
-
-
-
- N SPT N-VALUE
- 16 BLOWS/0.3m UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
- MHSG - MAGNETIC HEAVE/SETTLEMENT GAUGE (SM)
- P - VIBRATING WIRE PIEZOMETER (VWP)
- DRY BOREHOLE DRY DURING DRILLING
-
-
-

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	19-DEC-12				ISSUED FOR CONSTRUCTION			
	DATE	REV.	BY	DESCRIPTION	DATE	REV.	BY	DESCRIPTION
DESIGN	SF	CHK	SF	CODE CAN/CSA S6-06	LOAD	CL-625-ONT		
DRAWN	MM	CHK	NSV	SITE	6-605	DATE	02-DEC-11	

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



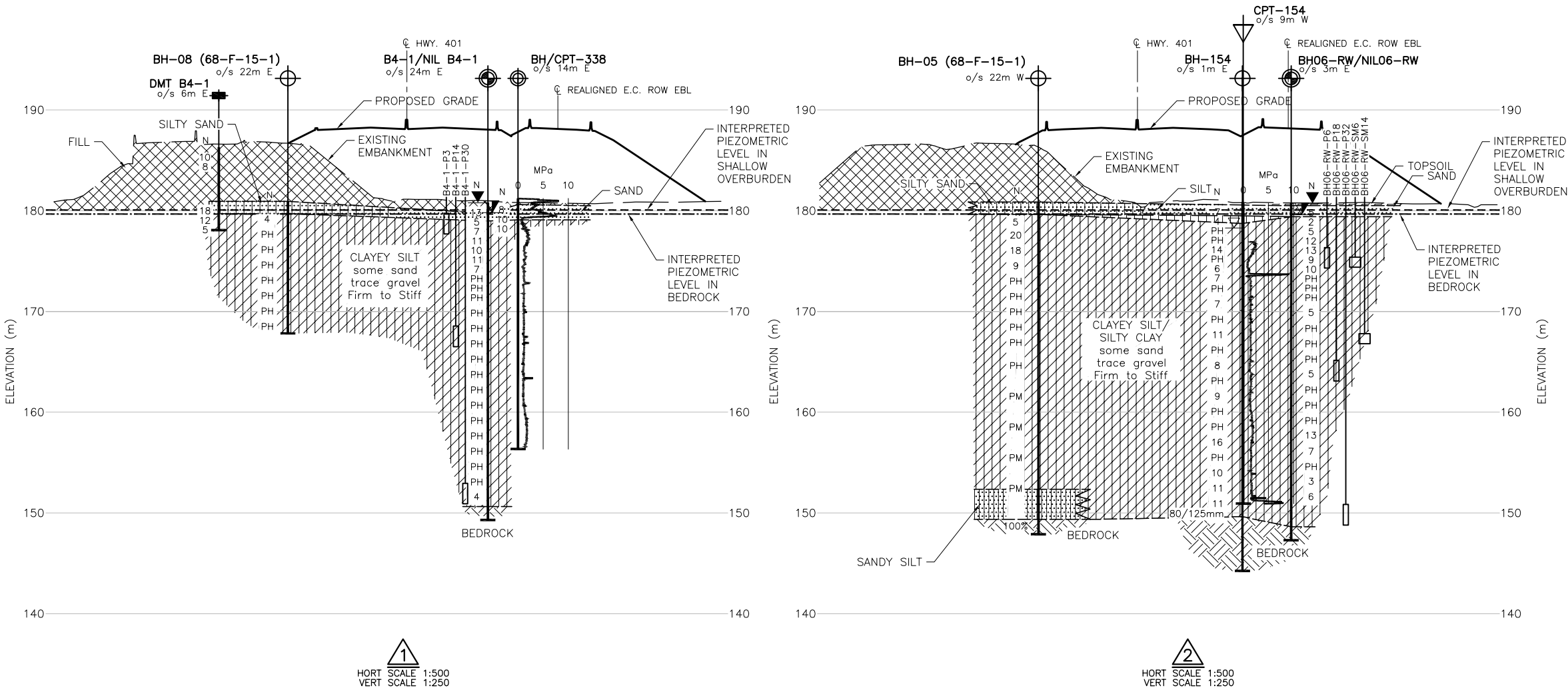
Windsor-Essex
Parkway Project
RFP No. 09-54-1007

NEW CONSTRUCTION
BRIDGE B-5
REALIGNED E.C. ROW EBL - MALDEN ROAD OVERPASS
SOIL STRATIGRAPHY



SHEET
G0503

Phase 3
IFC



LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

LEGEND

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

NOTES

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

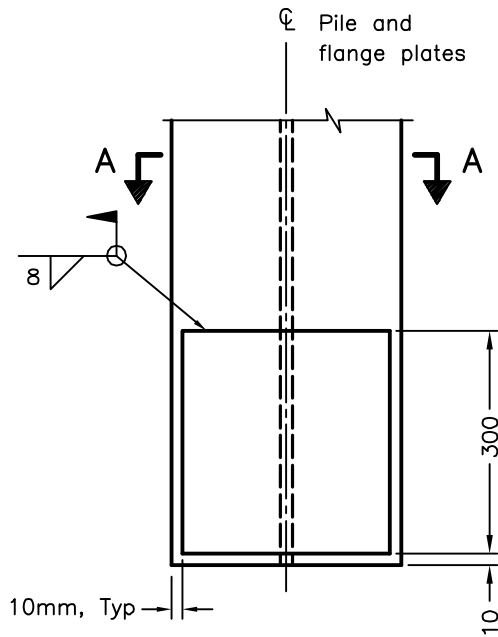
DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS	19-DEC-12				0				SF				ISSUED FOR CONSTRUCTION			
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DESIGN	SF	CHK	SF	CODE CAN/CSA S6-06	LOAD	CL-625-ONT										
DRAWN	MM	CHK	NSV	SITE	6-605	DATE	02-DEC-11									

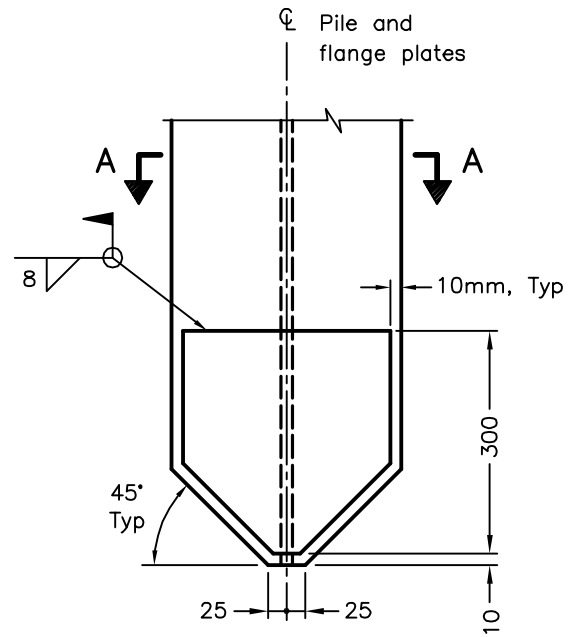
Applicable OPSDs

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report - Bridge B-5 (Malden Rd.
Overpass – Realigned E.C. Row EBL, Sta. 11+704 to 11+736, Windsor)
Doc No.: 285380-04-119-0115 (Geocres No. 40J6-48)

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

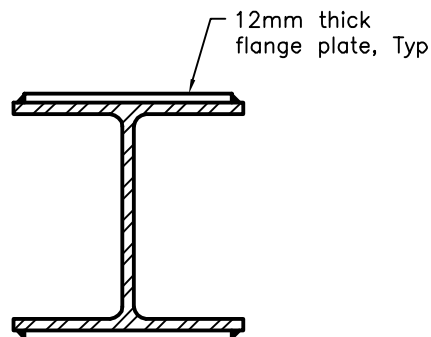


TYPE I



TYPE II

ELEVATION



PILE DRIVING SHOE
SECTION A-A

NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

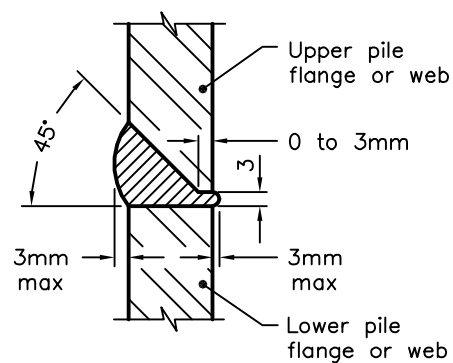
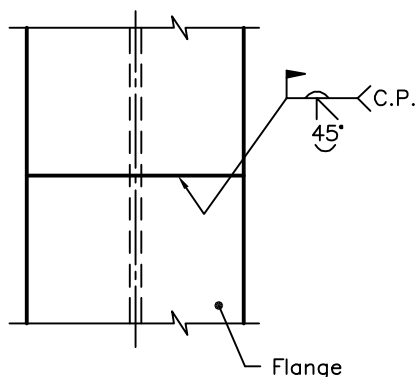
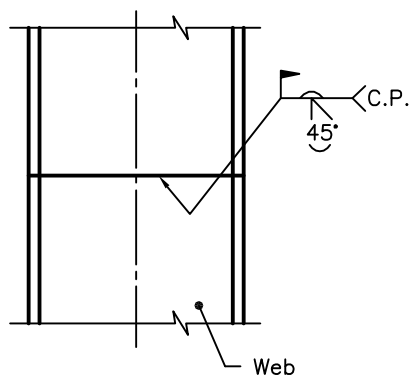
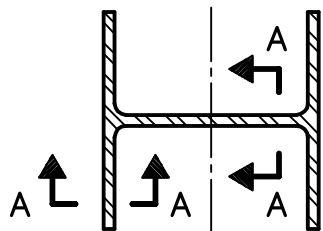
Rev 2

FOUNDATION
PILES

STEEL H-PILE DRIVING SHOE

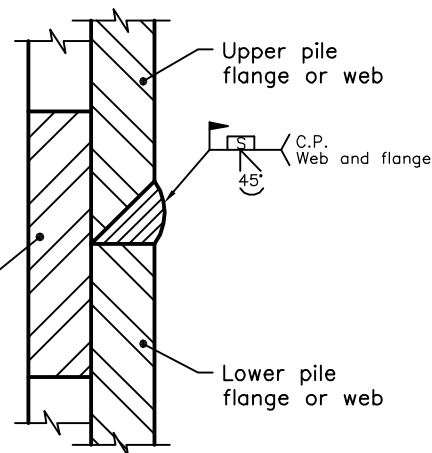
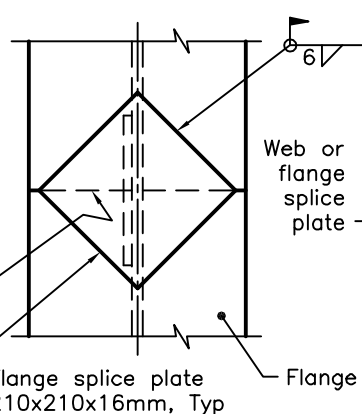
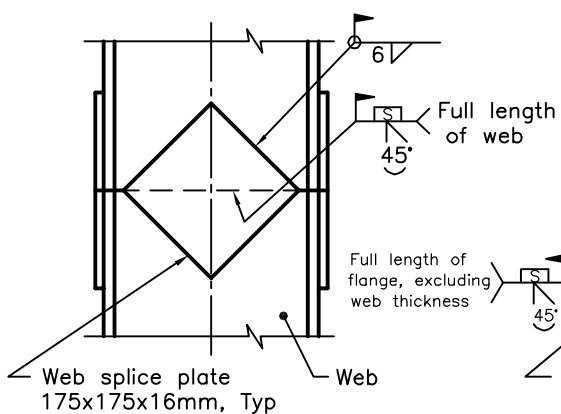
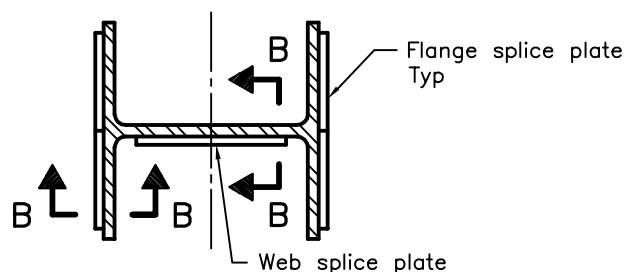
OPSD 3000.100





BUTT WELD

SECTION A-A



BUTT WELD WITH SPLICE PLATES

SECTION B-B

NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

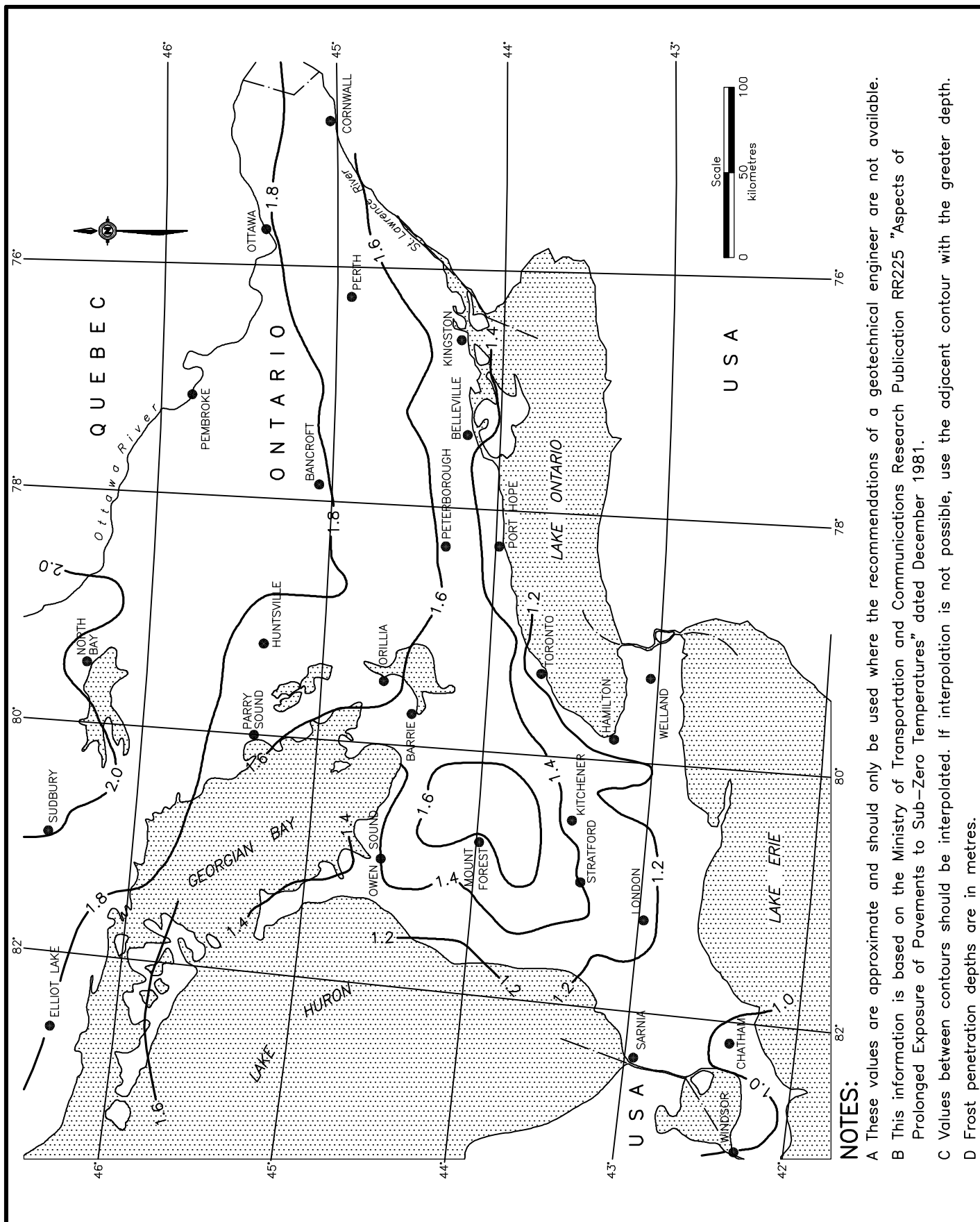
Rev

1

**FOUNDATION
PILES
STEEL H-PILE SPLICE**

OPSD 3000.150





NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

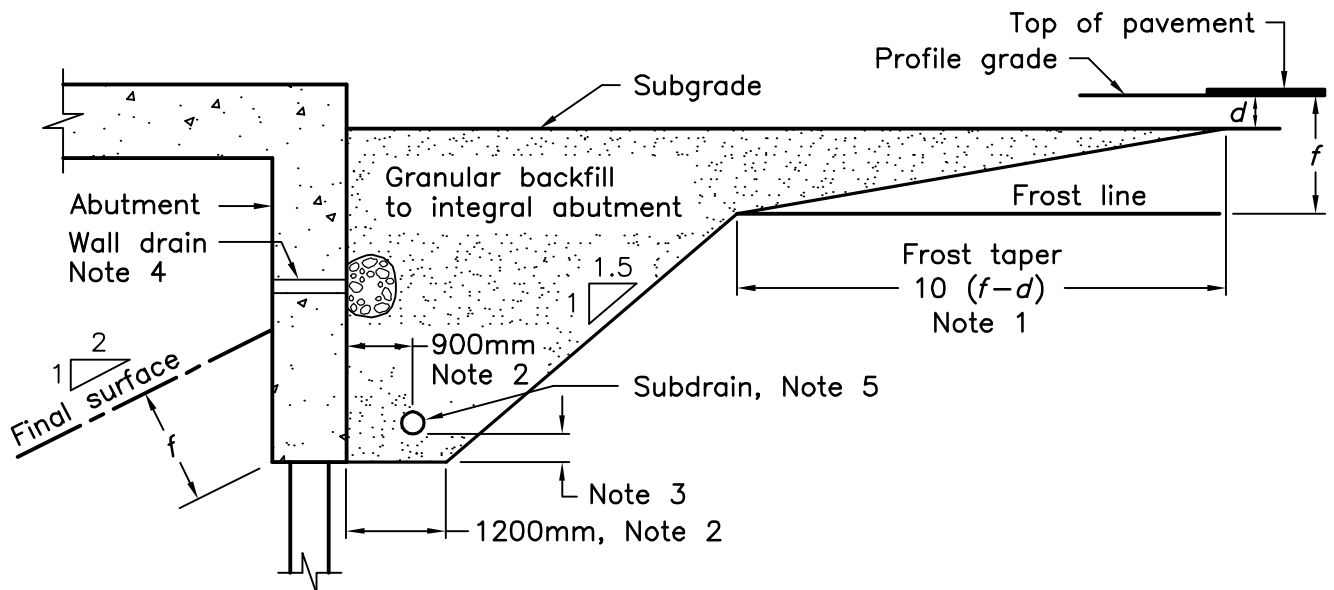
Nov 2010

Rev 1

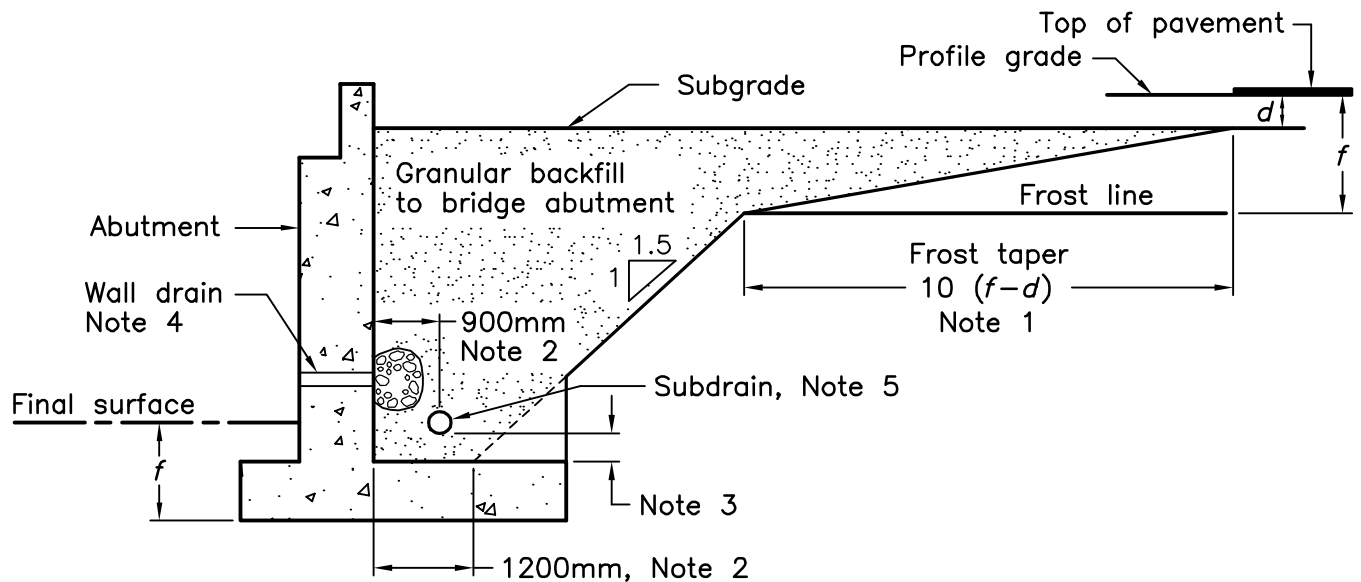
**FOUNDATION
FROST PENETRATION DEPTHS
FOR SOUTHERN ONTARIO**



OPSD 3090.101



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2010

Rev 1



WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD 3101.150

Figures

Figure 3-1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures

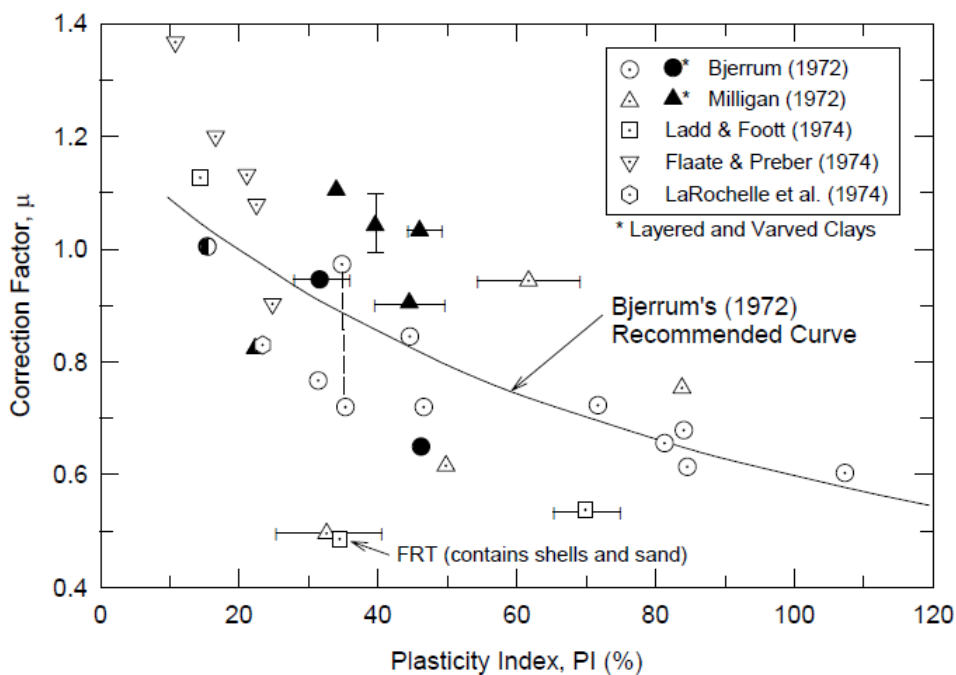
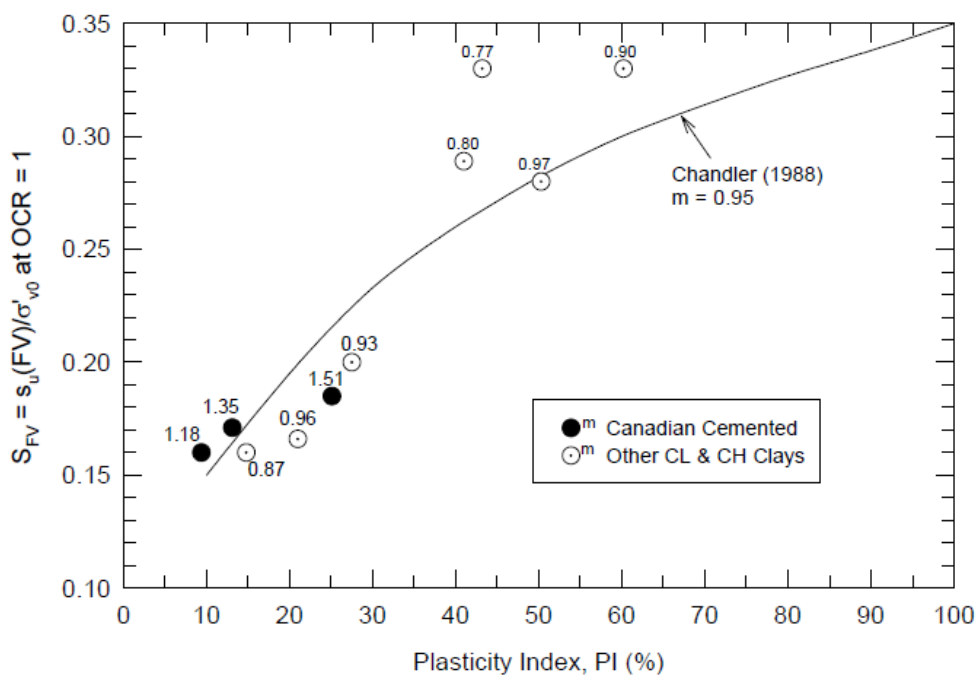
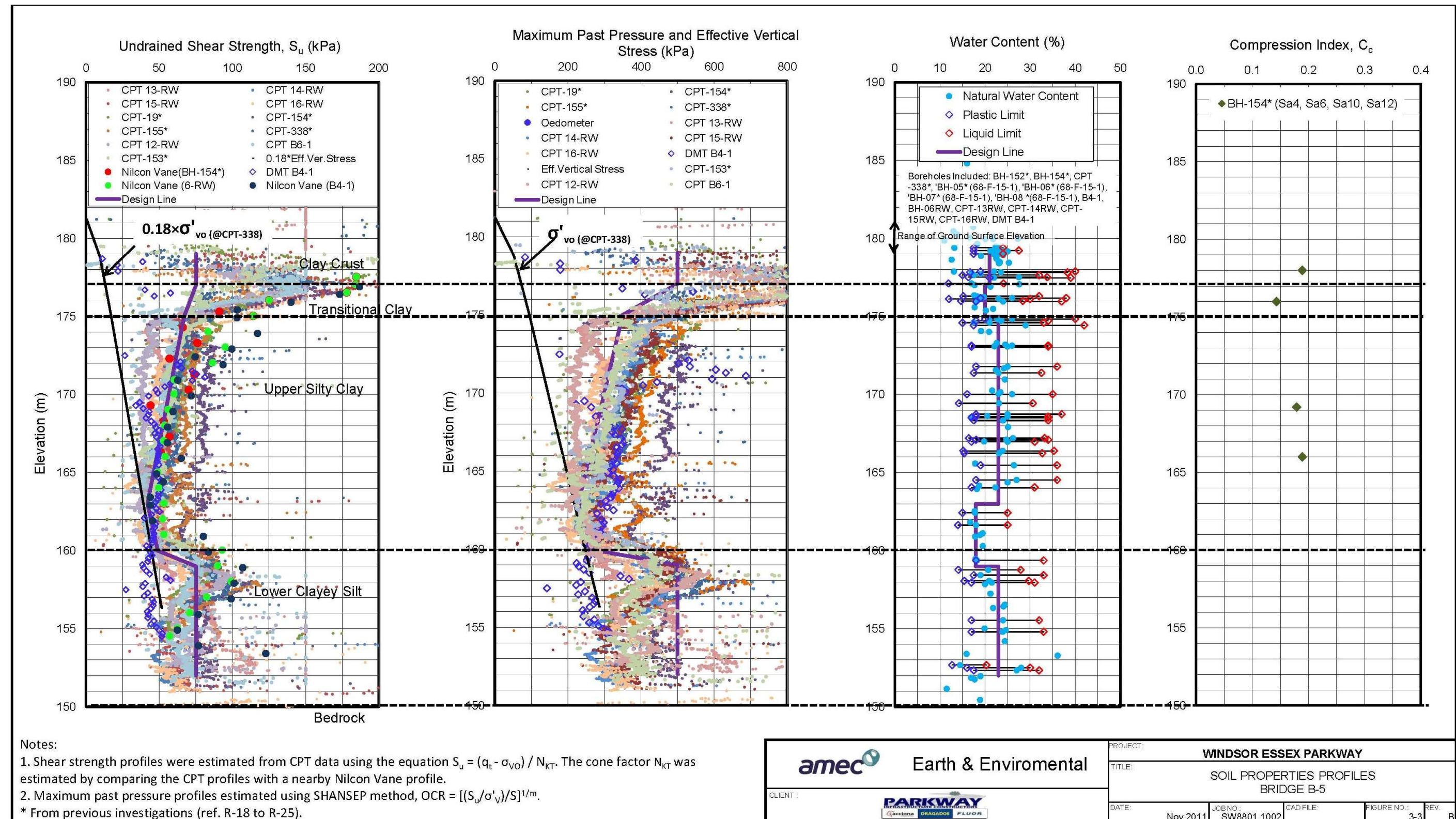
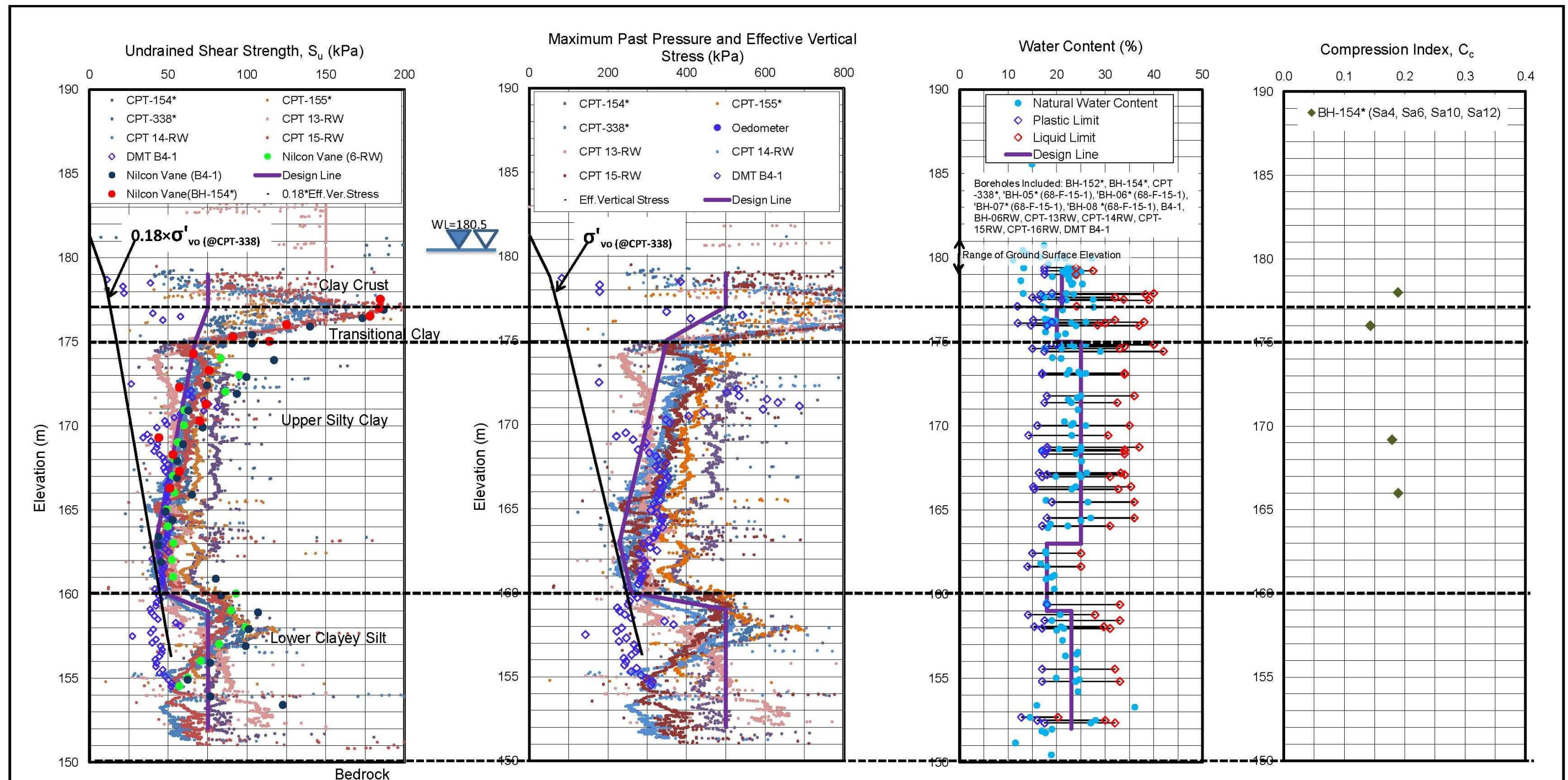


Figure 3-2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays







amec Earth & Environmental <small>acciona DRAGADOS FLUOR</small>		PROJECT: WINDSOR ESSEX PARKWAY			
<small>CLIENT:</small>		TITLE: SOIL PROPERTIES PROFILES BRIDGE B-5			
<small>DATE:</small>		<small>JOB NO.:</small>	<small>CAD FILE:</small>	<small>FIGURE NO.:</small>	<small>REV.:</small>
Nov 2011		SW8801.1002		3-4	B

Figure 4-1: Compressibility Parameters at WEP

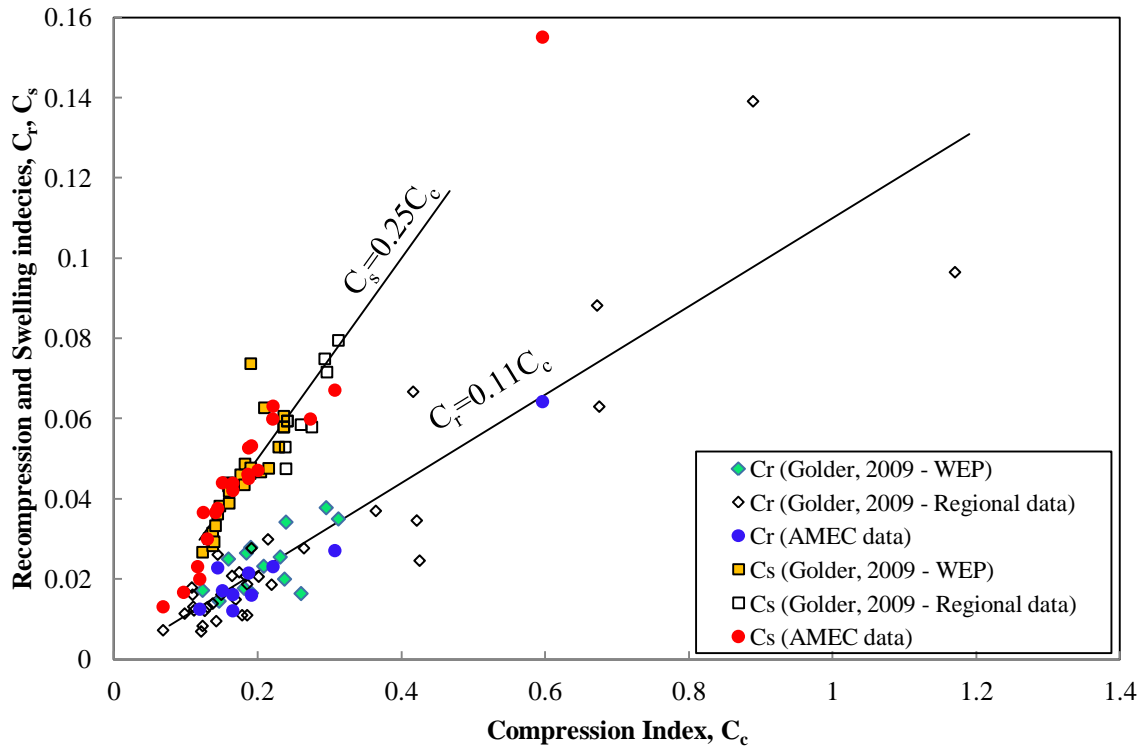
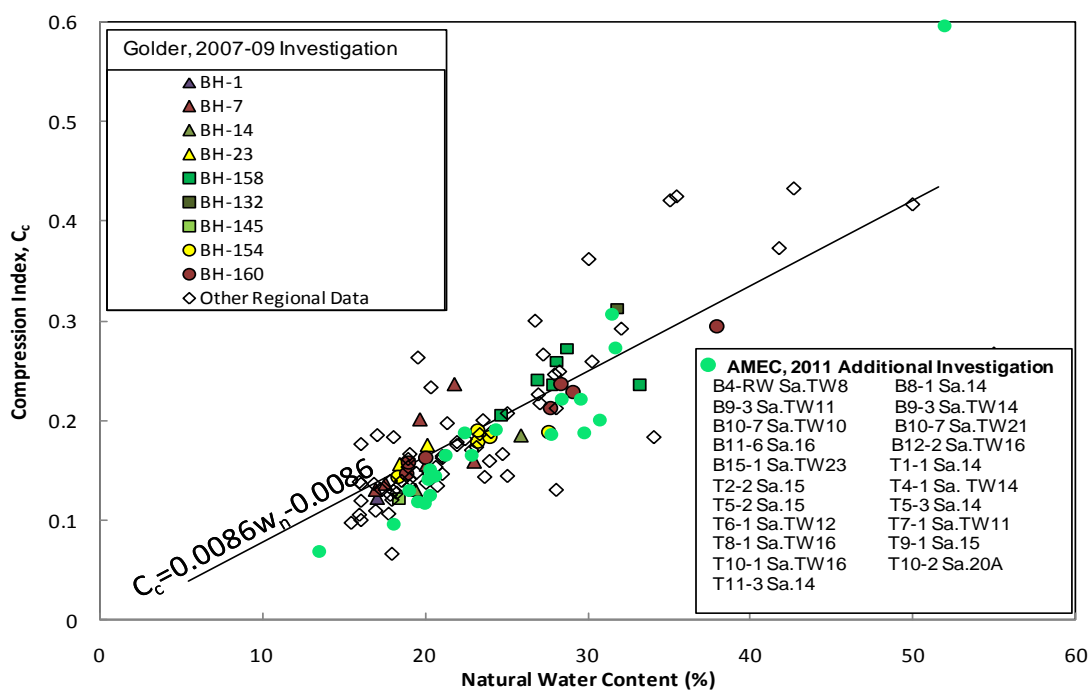


Figure 4-2: C_c versus C_α Relationship at WEP

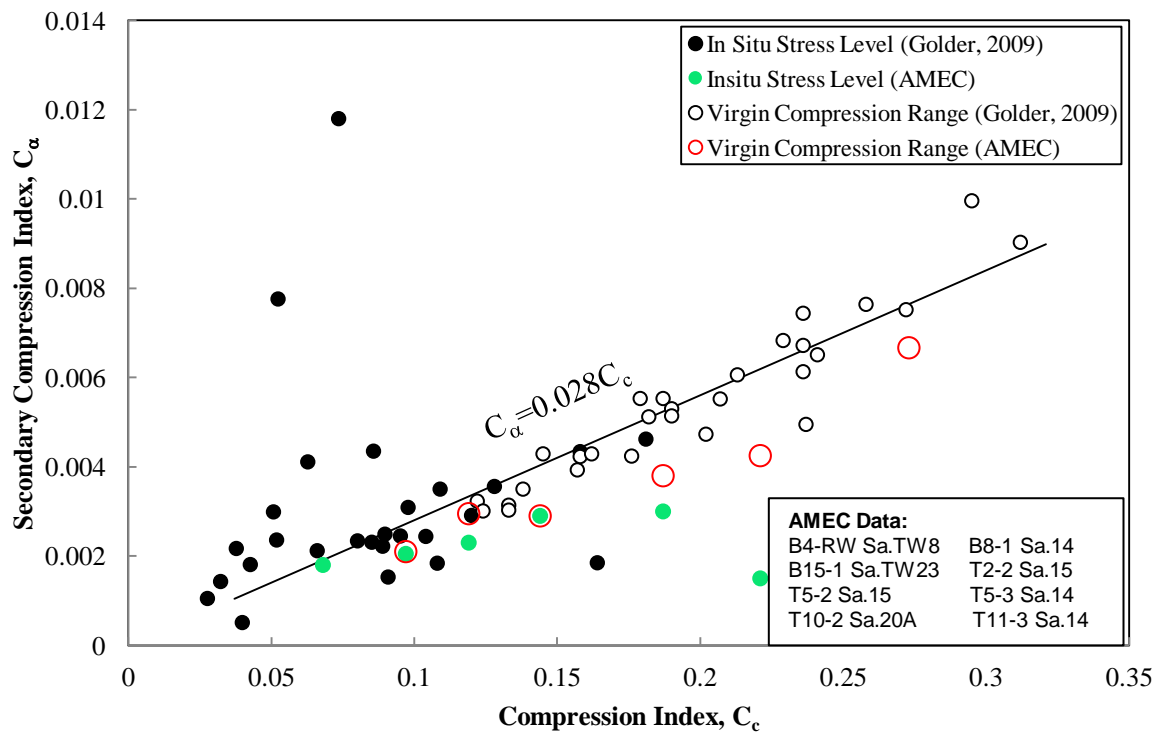


Figure 4-3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP

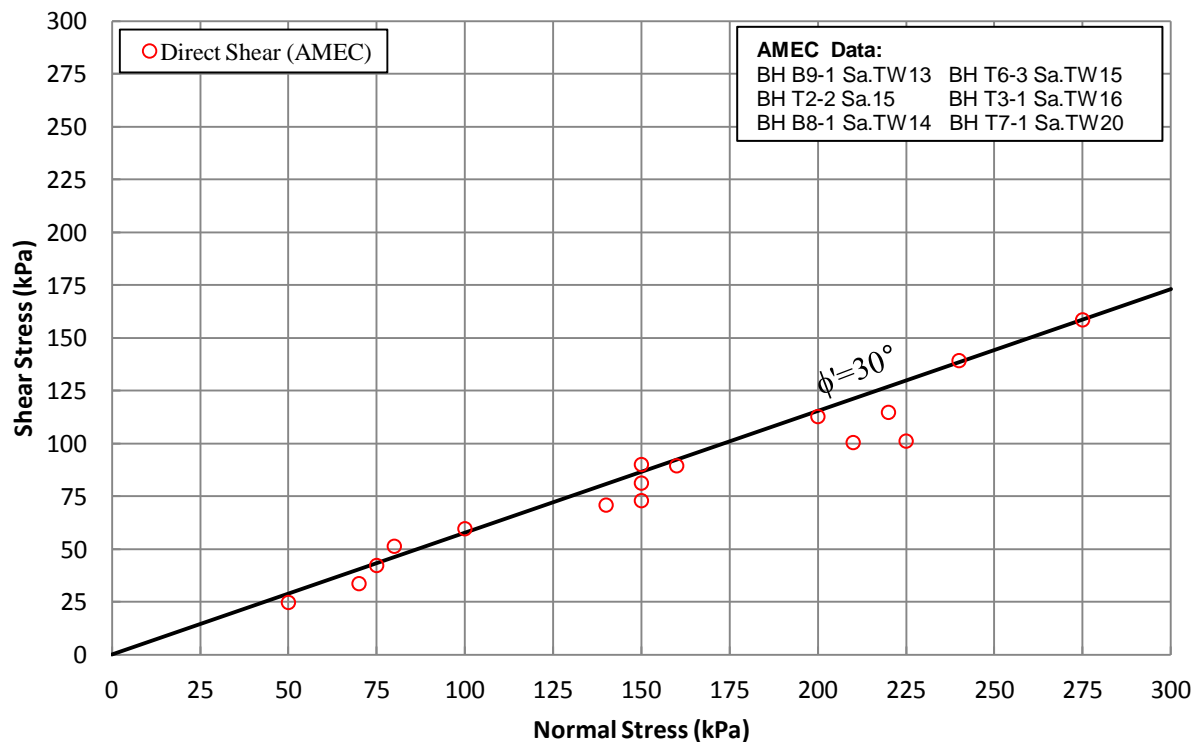
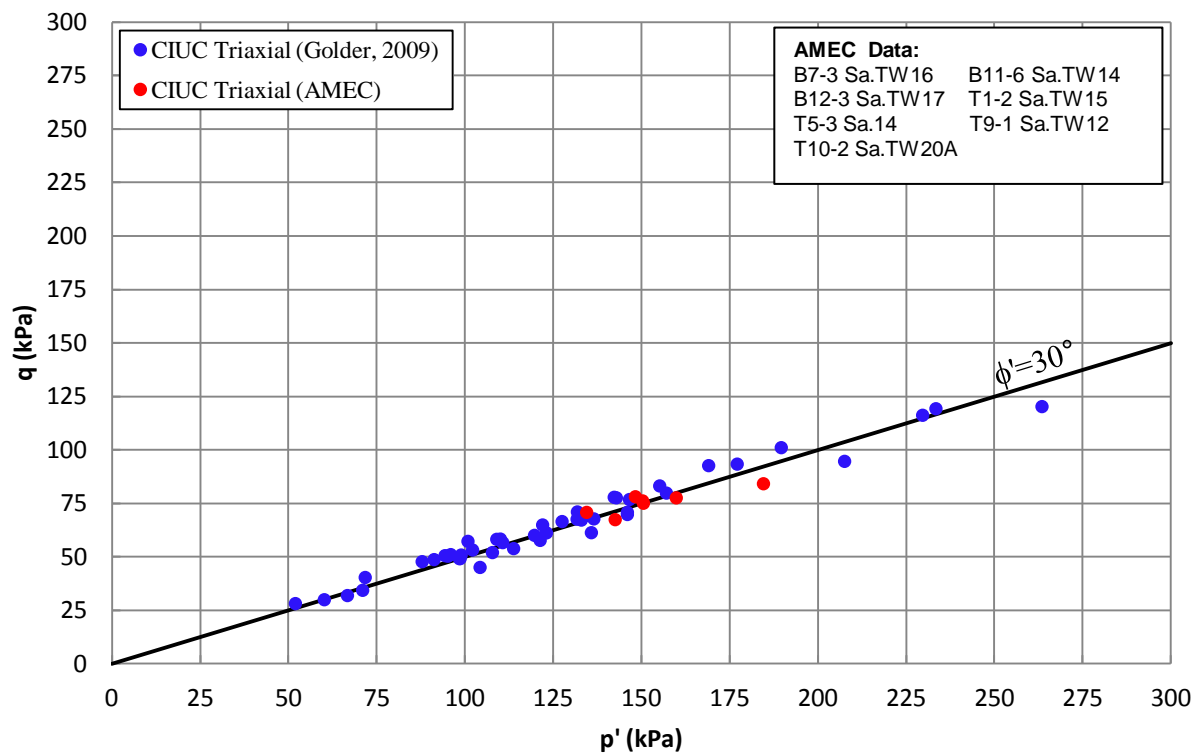


Figure 4-4: Relationship between $\sin \phi'$ and Plasticity Index for Normally Consolidated Soils

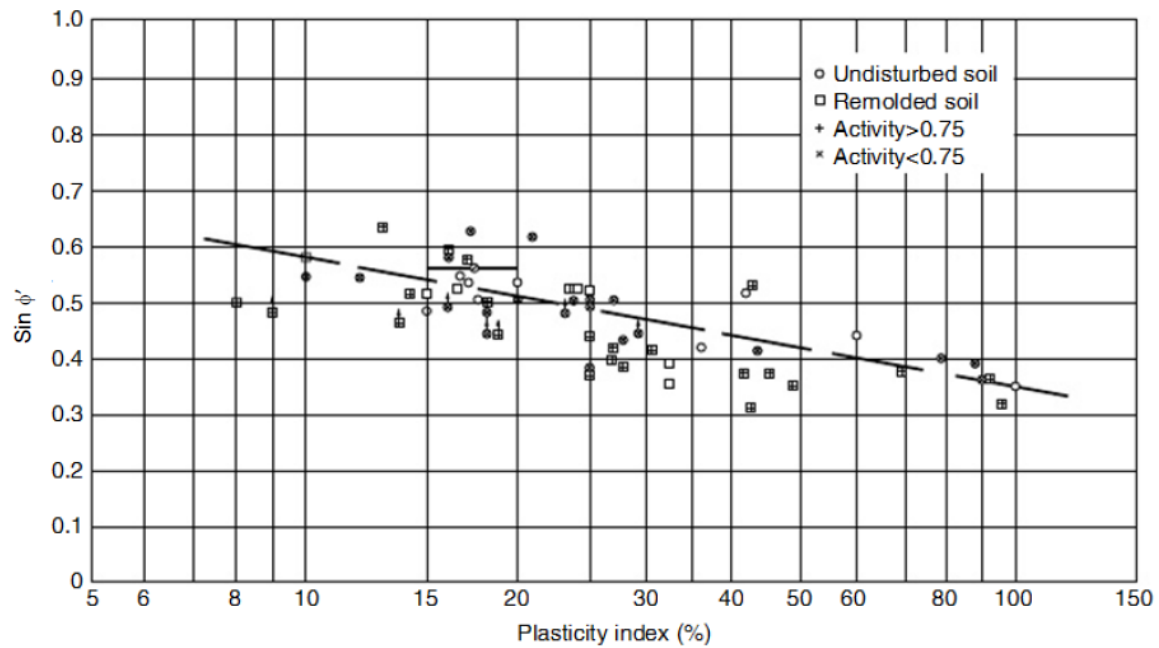
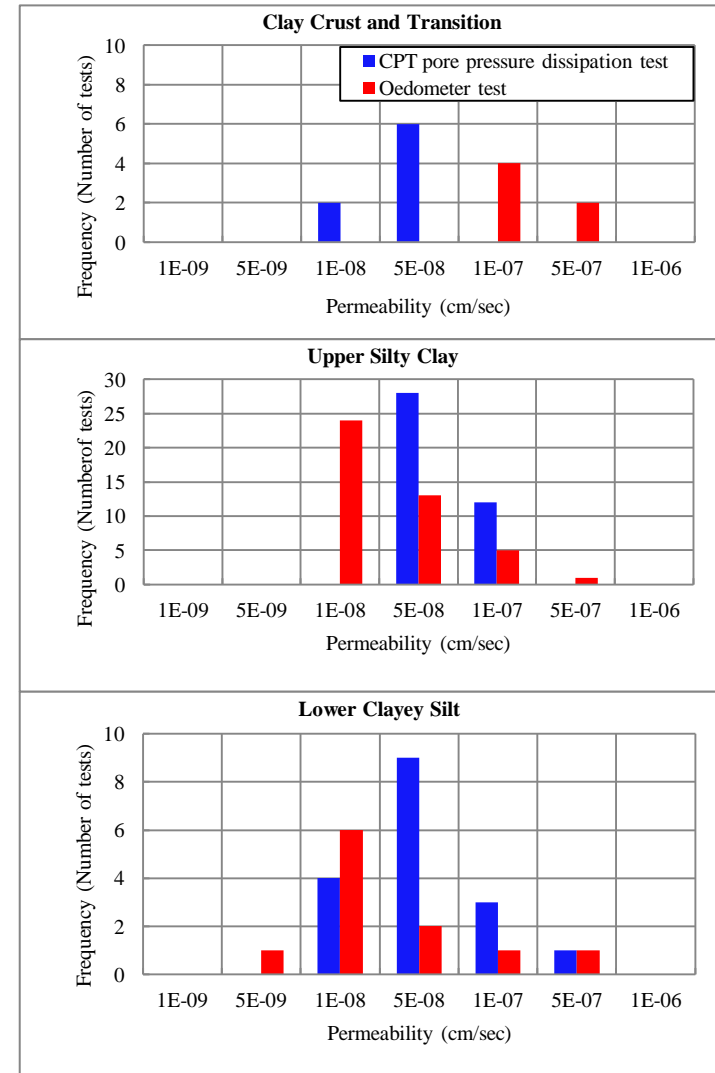
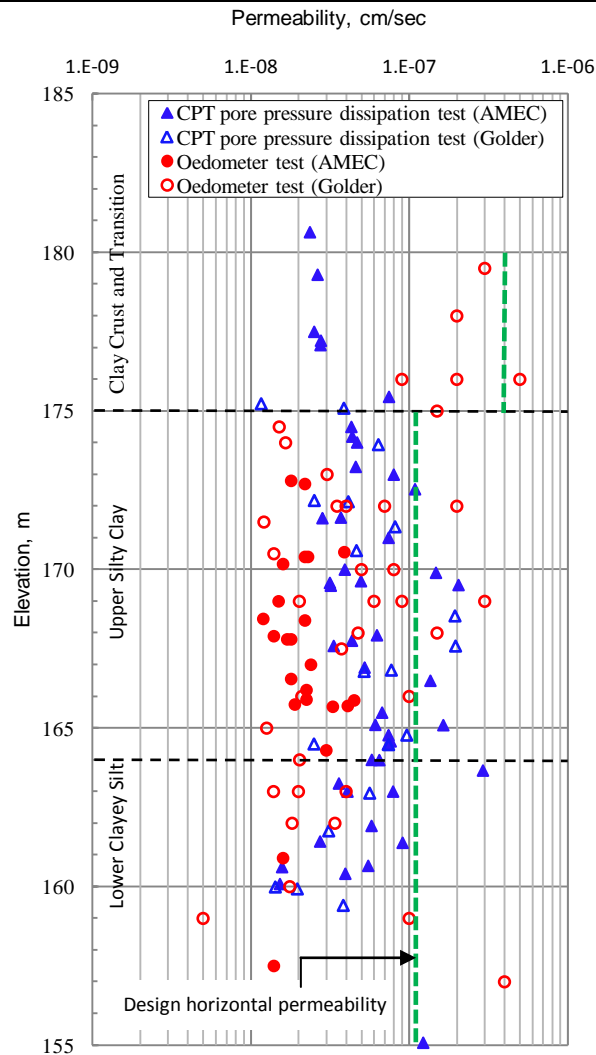
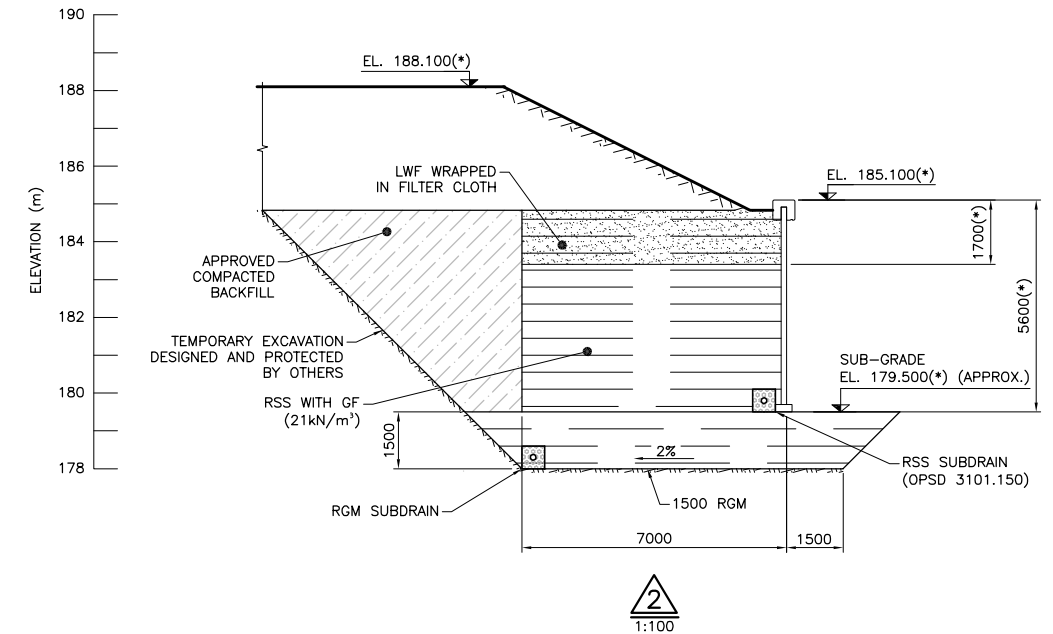
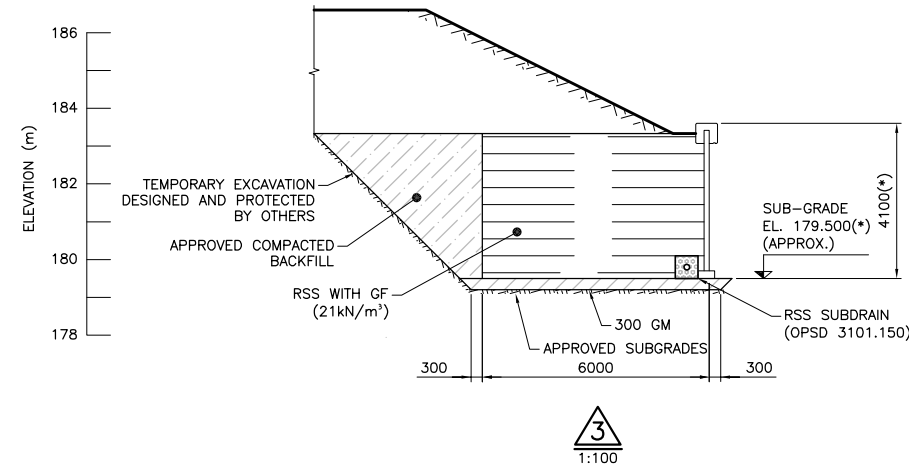
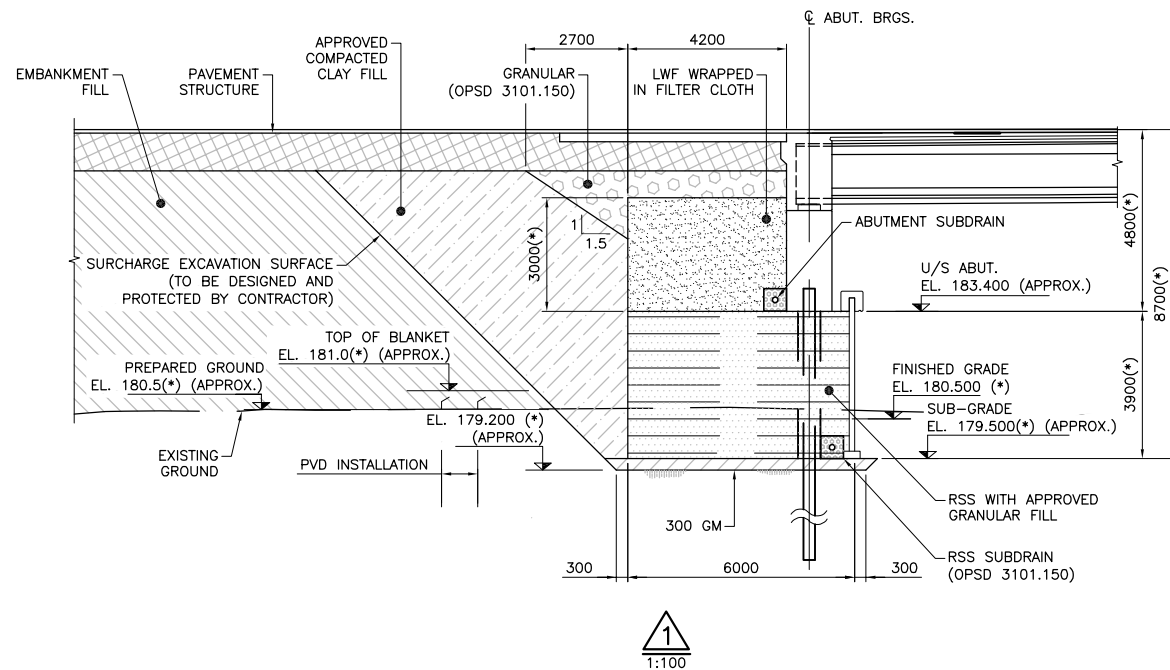
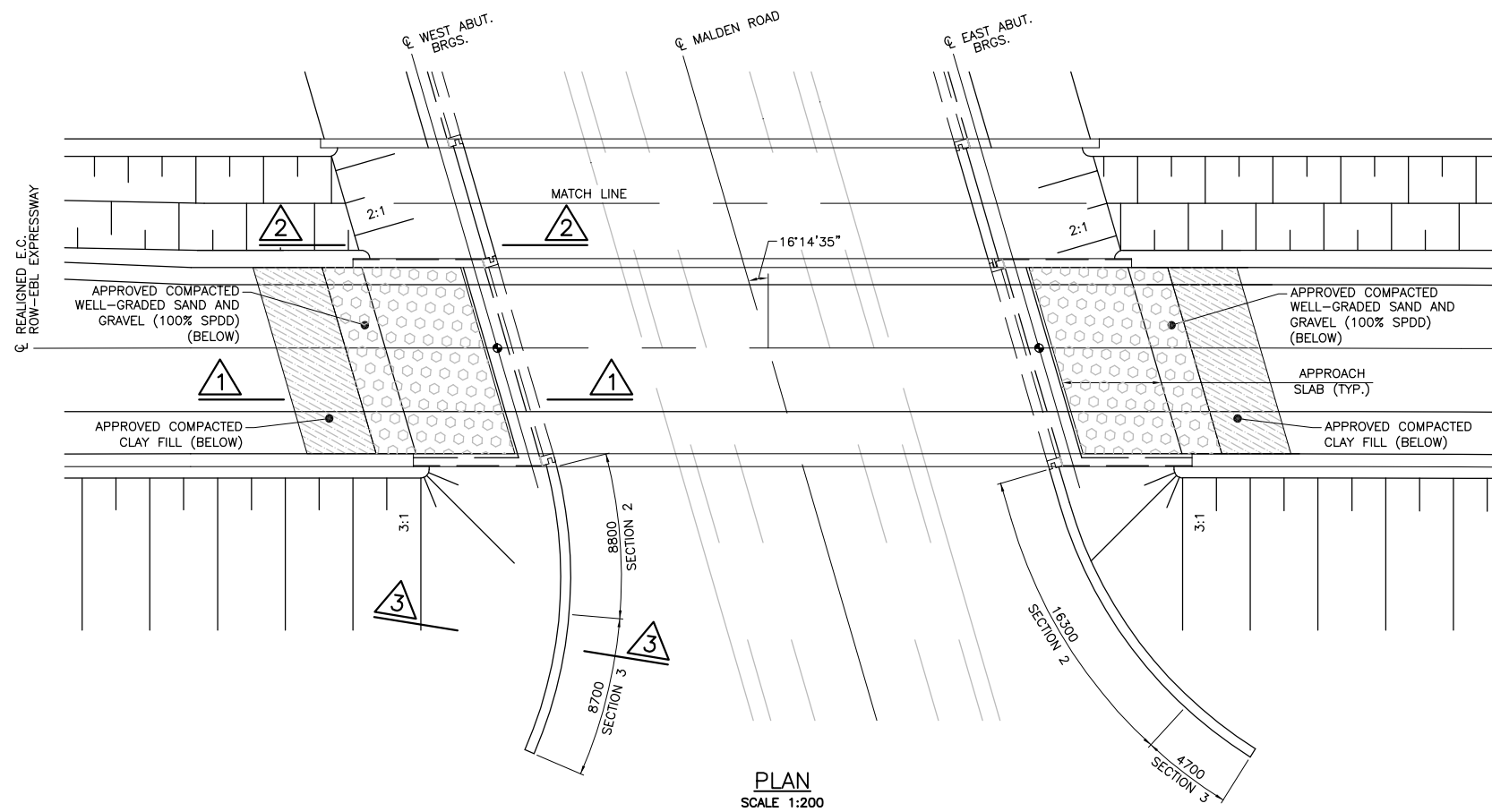


Figure 4-5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests





LEGEND

LWF - LIGHTWEIGHT FILL
GM - GRANULAR MAT
RGM - REINFORCED GRANULAR MAT
RSS - REINFORCED SOIL STRUCTURE
(*) - VARIES

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOTES

- THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
- THIS DRAWING ILLUSTRATES THE GENERAL BACKFILL ARRANGEMENT AT SELECTED REPRESENTATIVE LOCATIONS OF THE WEST ABUTMENT OF BRIDGE B-5 BASED ON GEOTECHNICAL DESIGN ANALYSES.
- THE EAST ABUTMENT BACKFILL ARRANGEMENT IS SIMILAR TO THE WEST ABUTMENT.
- ABUTMENT AND APPROACH EMBANKMENT ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGN WERE OBTAINED FROM STRUCTURAL DRAWINGS. ABUTMENT ELEVATIONS VARY.
- ABUTMENT CONSTRUCTION TO START AFTER COMPLETION OF GROUND IMPROVEMENT FOR APPROACH EMBANKMENTS (SEE REPORT 285380-04-119-0003).
- COMMENCE PILE INSTALLATION ONLY AFTER COMPLETION OF THE GROUND IMPROVEMENT WITH WICK DRAINS AND SURCHARGING.
- 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 IMPLEMENTED.
- CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS. EXCAVATED SLOPES ARE SUSCEPTIBLE TO DETERIORATION AND MAY EXPERIENCE DEFORMATIONS AND INSTABILITY. THE TEMPORARY SLOPES ARE TO BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED, AND TREATED AS REQUIRED.
- RSS STRAP LENGTHS SHOWN ARE PRELIMINARY. FINAL RSS WALLS WILL BE DESIGNED BY THE SUPPLIER. FINAL RGM WILL BE DESIGNED BY THE SUPPLIER.
- SEE ACCOMPANYING DRAWINGS FOR APPLICABLE CONSTRUCTION NOTES.

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

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report - Bridge B-5 (Malden Rd.
Overpass – Realigned E.C. Row EBL, Sta. 11+704 to 11+736, Windsor)
Doc No.: 285380-04-119-0115 (Geocres No. 40J6-48)

Date: December / 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 Cohesionless Soils	SPT N-Value*
Very loose	0 to 5
Loose	5 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

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

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

Soil Sampling

Sample types are abbreviated as follows:

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

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

Field and Laboratory Testing

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

Instrumentation Installation

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

Comments

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

BEDROCK DESCRIPTION

STRENGTH CLASSIFICATION

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

JOINT SPACING CLASSIFICATION

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

ROCK QUALITY CLASSIFICATION

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

Reference: Deere et al, 1967

WEATHERING CLASSIFICATION

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

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

TERMINOLOGY

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

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

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

MTC SOIL CLASSIFICATION

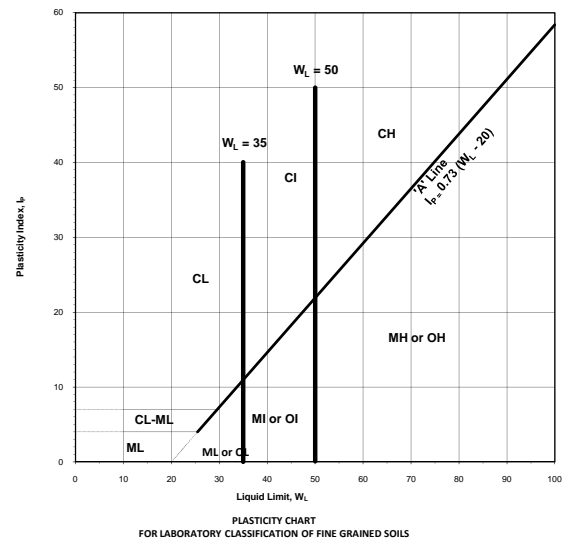
Based on MTC Soil Classification Manual



MAJOR DIVISION					GROUP SYMBOL	TYPICAL DESCRIPTION	INFORMATION REQUIRED FOR DESCRIBING SOILS	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					
		LIQUID LIMIT LESS THAN 35 AND 50	DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)				FOR UNDISTURBED SOILS ADD INFORMATION ON STRUCTURE, STRATIFICATION, CONSISTANCY IN UNDISTURBED AND REMOLDED STATES, MOISTURE & DRAINAGE CONDITION.	NOT MEETING ALL GRADATION FOR SW			
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							
		HIGH ORGANIC SOILS				Pt					PEAT AND OTHER HIGHLY ORGANIC SOILS		
READILY IDENTIFIED BY COLOUR, ODOUR, SPONGY FEEL & FREQUENTLY BY FIBROUS TEXTURE											ATTENBERG LIMITS ABOVE A- LINE WITH Ip GREATER THAN 7	ABOVE A-LINE WITH Ip BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS	

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

FRACTION	U.S STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS		
GRAVEL	COARSE	PASSING	RETAINED	PERCENT	DESCRIPTOR
		75 mm	26.5 mm		
SAND	FINE	26.5 mm	4.75 mm	40-50	AND
		COARSE	4.75 mm	2.00 mm	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



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**MTC SOIL CLASSIFICATION MANUAL
ENGINEERING PROPERTIES OF SOIL**



TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	PERMEABILITY WHEN COMPACTED	STRENGTH WHEN COMPACTED	COMPRESSIBILITY WHEN COMPACTED	WORKABILITY AS A CONSTRUCTION MATERIAL	SCOUR RESISTANCE	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 BH06-RW

1 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4681950.3, E330198.8 ORIGINATED BY SD
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 13 Jun 11 - 14 Jun 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									WATER CONTENT (%)
								○ UNCONFINED	● POCKET PEN.	+ FIELD VANE	× LAB VANE						
180.8	Ground Surface															GR SA SI CL	
0.0	Black TOPSOIL															-vibrating wire piezometers (VWP) installed at shallow and mid-depth in sampled borehole; deep VWP installed in adjacent boring at (N4681948.0, E330200.9); Shallow Spider Magnet (MG) installed in adjacent boring at (N4681949.5, E330195.5); Mid-depth MG installed in adjacent boring at (N4681946.6, E330195.5)	
180.5	Brown FINE SAND Some silt Wet, loose		1	SS	5												
0.3																	
179.4	Grey CLAYEY SILT Very soft, wet		2	SS	2												
1.4																	

Continued Next Page

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

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2 OF 3

METRIC

[illegible]

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Continued Next Page

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

METRIC

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

METRIC

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N° VALUES	SHEAR STRENGTH kPa			WATER CONTENT (%)							
						○ UNCONFINED ● POCKET PEN.			+ FIELD VANE × LAB VANE		W _P	W	W _L			
180.8 0.0	Fill Surface						20	40	60	80	100					GR SA SI CL
	Heterogeneous FILL Including sand, silt, clay, topsoil, and gravel															-slope indicator casing installed in sampled borehole; vibrating wire piezometers (VWP) installed in adjacent boring at (4681985N, 330152E) Spider Magnets (SM) installed in adjacent boring at (4681987N, 330153E)
179.7 1.1	Brown-Grey FINE SAND Trace to some silt		1A, B	SS	13											
179.0 1.8	Saturated Loose		2A, B	SS	5											
	Grey CLAY															
178.1 2.7	Grey SILT Laminated, saturated Loose, firm		3	SS	7											
	Grey CLAYEY SILT Stiff		4	SS	11											-VWP #P3 installed at 3.05m below surface
	-Soft, wet clay seams and hairline sand/silt seams to approx. 4.5m		5	SS	10											
			6	SS	11											-SM installed at 4.01m below surface
	Firm		7	SS	7											8 29 41 22
			8	TW	PH											
			9	TW	PH											
			10	TW	PH											
				VT												
			11	TW	PH											
			12	SS	PH											
				VT												
			13	TW	PH											
166.2 74.6																-VWP #P12 installed at 14.33m below surface

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

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

RECORD OF BOREHOLE No B4-1

3 OF 3

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4681982.4, E330153.5 ORIGINATED BY LC
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 9 Jun 11 - 11 Jun 11 CHECKED BY MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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						
								20 40 60 80 100						
150.6							<div><div></div><div></div><div></div><div></div><div></div></div> <div>20 40 60 80 100</div>					<div>PLASTIC LIMIT</div> <div>NATURAL MOISTURE CONTENT</div> <div>LIQUID LIMIT</div>		
30.2	Grey Colour LIMESTONE Medium to Coarse Grained Laminated, stylolites present, fractures present at location		24	RC			<div><div></div><div></div><div></div><div></div><div></div></div> <div>20 40 60 80 100</div>					<div>W_p</div> <div>W</div> <div>W_L</div>		-VWP #P31 installed at 29.9m below ground surface -slightly artesian groundwater condition at bedrock interface ROD = 78%
149.3			25	RC										TCR = 92%
31.5	END OF BOREHOLE													SCR = 79%
	Piezometric Levels in VWP #P3: June 25, 2011: EL. 179.3m July 9, 2011: EL. 179.4m July 22, 2011: EL. 179.2m						149							
	Piezometric Levels in VWP #P12: June 25, 2011: EL. 179.5m July 9, 2011: EL. 181.3m July 22, 2011: EL. 180.5m						148							
	Piezometric Levels in VWP #P31: July 29, 2011: EL. 180.5m July 22, 2011: EL. 180.5m						147							
							146							
							145							
							144							
							143							
							142							
							141							
							140							
							139							
							138							
							137							
							136							

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

RECORD OF BOREHOLE No CPT13-RW

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4682069.5, E330070.1 ORIGINATED BY TR
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 11 Jun 11 - 11 Jun 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											
185.1	Fill Surface							○ UNCONFINED	+ FIELD VANE	● POCKET PEN.	× LAB VANE						WATER CONTENT (%)		
0.0	FILL Alternating layers of brown Silty Clay Some sand, trace gravel And grey Silty Clay, some sand, trace gravel							20 40 60 80 100									10 20 30		
183.9			1	SS	10														
1.2	Continued with CPT to 6.1m																		
										</									

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

RECORD OF BOREHOLE No CPT14-RW

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4682001.9, E330097.3 ORIGINATED BY TA
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 7 Jun 11 - 7 Jun 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								
								20	40	60	80					
180.4	Ground Surface															
0.0	TOPSOIL															
180.0																
0.4	SAND															
179.6	Poorly Graded (Fine) Brown															
0.8	Trace silt, moist		1	SS	14											
	CLAYEY SILT															
	Trace sand															
	Very soft		2	SS	1											
	Grey															
178.4	END OF SAMPLED BOREHOLE (Continued with CPT to refusal)															
2.0																
						</										

RECORD OF BOREHOLE No CPT15-RW

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4681954.3, E330300.3 ORIGINATED BY TA
DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 8 Jun 11 - 8 Jun 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	×						LAB VANE		
								● POCKET PEN.											
181.5	Ground Surface																		
0.0																			
181.2	250mm TOPSOIL																		
0.3	SAND, Poorly Graded Trace silt Compact Brown		1	SS	11														
	Saturated		2	SS	12														
179.5	END OF SAMPLED BOREHOLE (Continued with CPT to refusal)																		
2.0																			

RECORD OF BOREHOLE No DMT B4-1

1 OF 1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4682042.8, E330143.9 ORIGINATED BY LC
 DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
 DATUM Geodetic DATE 22 Jun 11 - 22 Jun 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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _P	W	W _L			
								○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE	WATER CONTENT (%)						
							20 40 60 80 100								
186.3	Fill Surface														
186.0	FILL Clay and Topsoil														
0.2	Brown to Grey FILL Clay														
	Some sand, trace gravel -Trace to some organic roots (topsoil) Trace roots		1	SS	10										
184.5			2	SS	8										
1.8	Continue with DMT readings to 6.0m														
180.2															
6.1	FILL Brown Fine Sand with concrete pieces to ash/cinders in tip		3A, B	SS	18	▽	180								
179.4	-Wet to saturated														
6.9	Brown to Grey SAND		4	SS	12		179								
178.7	-Trace to some silt, trace gravel Loose to compact														
7.6	Grey SILT		5	SS	5										
178.1	Trace clay, with numerous hairline sand/silt lenses Firm														
8.2	END OF BOREHOLE (continue with DMT to refusal)						178								
							177								
							176								
							175								
							174								
							173								
							172								

-refusal to hydraulic push of DMT probe at approx. 6.0m; attempted to drive probe, but met with refusal and damaged probe; augered to 6.1m and sampled by SPT to 8.2m, then continued with replacement probe

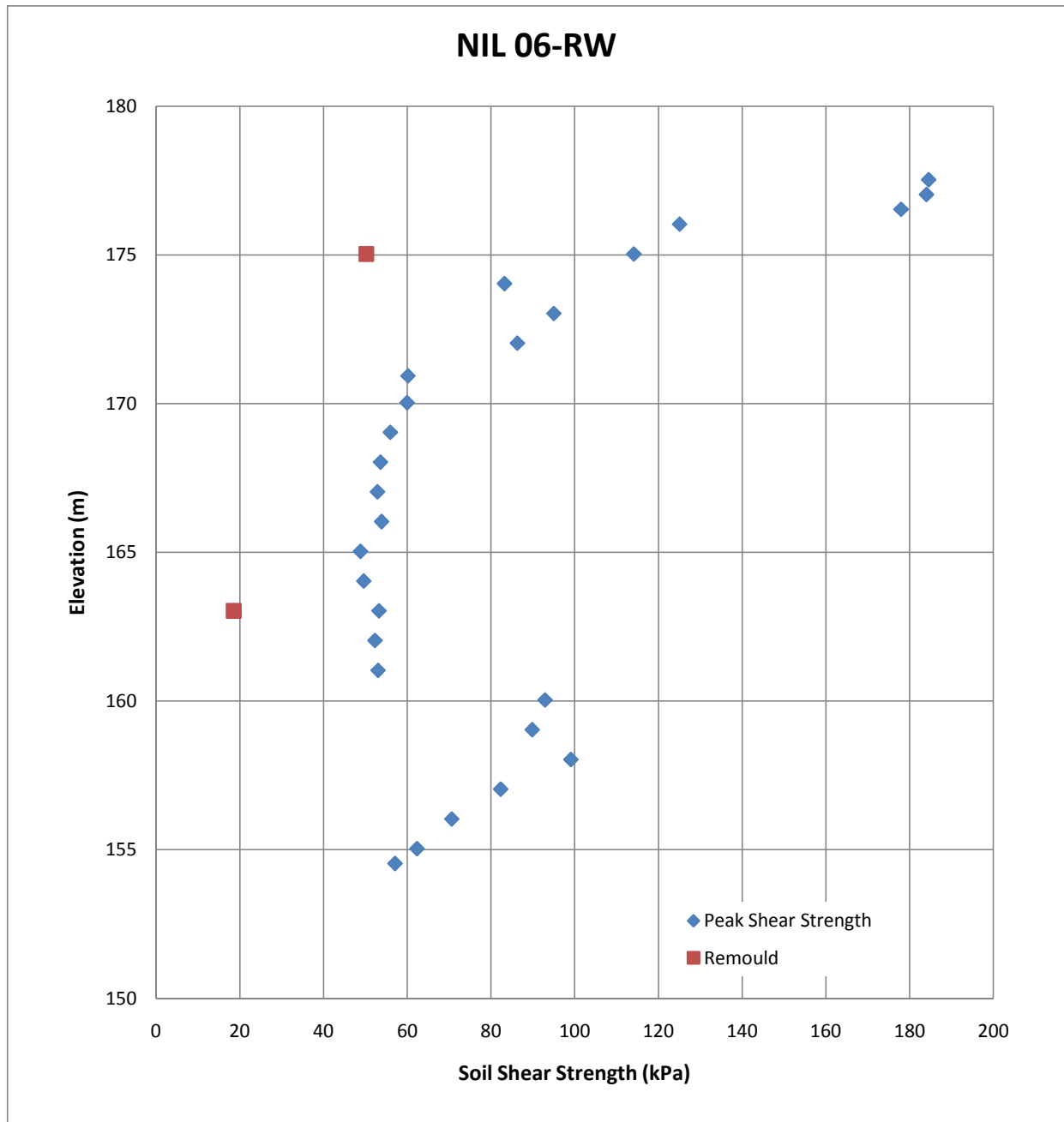
-refusal to hydraulic push of DMT probe at approx. 6.0m; attempted to drive probe, but met with refusal and damaged probe; augered to 6.1m and sampled by SPT to 8.2m, then continued with replacement probe

RECORD OF NILCON VANE TEST NIL 06-RW

Project : Windsor-Essex Parkway
 Location: 4681948.0 N; 330200.9 E
 Ground Surface Elevation: 181.0 m

Test Date: 6/27/2011
 Predrill Depth : 3.0 m

Sheet 1 of 1
 Datum Geodetic



Operator: SD

Checked: DD

RECORD OF NILCON VANE TEST NIL B4-1

Project : Windsor-Essex Parkway

Test Date: 6/26/2011 - 6/27/2011

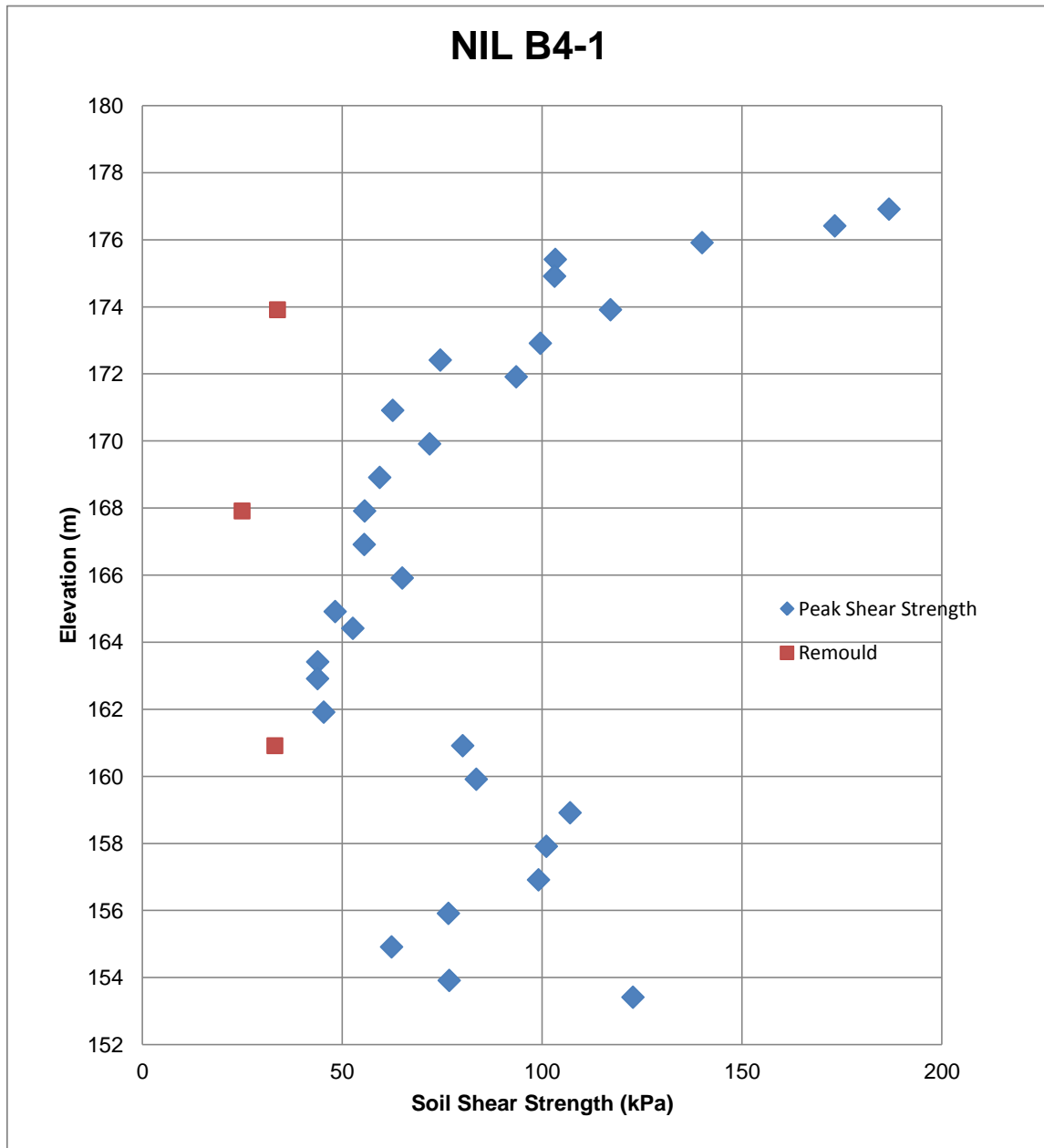
Sheet 1 of 1

Location: N4681982.5; E330151.6

Predrill Depth : 3.5 m

Datum Geodetic

Ground Surface Elevation: 180.9 m



Operator: SD

Checked: DD

RECORD OF DILATOMETER TEST DMT B4-1 SHALLOW

Project : Windsor-Essex Parkway
Location: N 4682042.8; E 330143.9

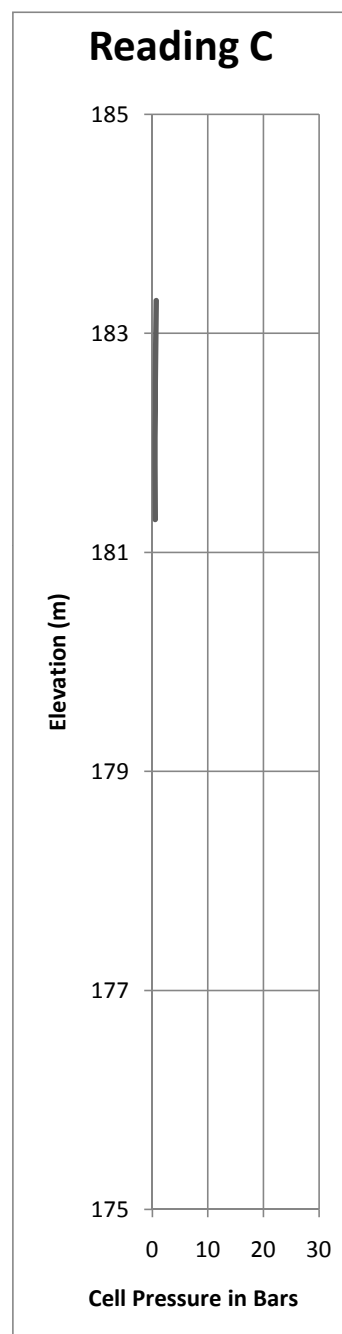
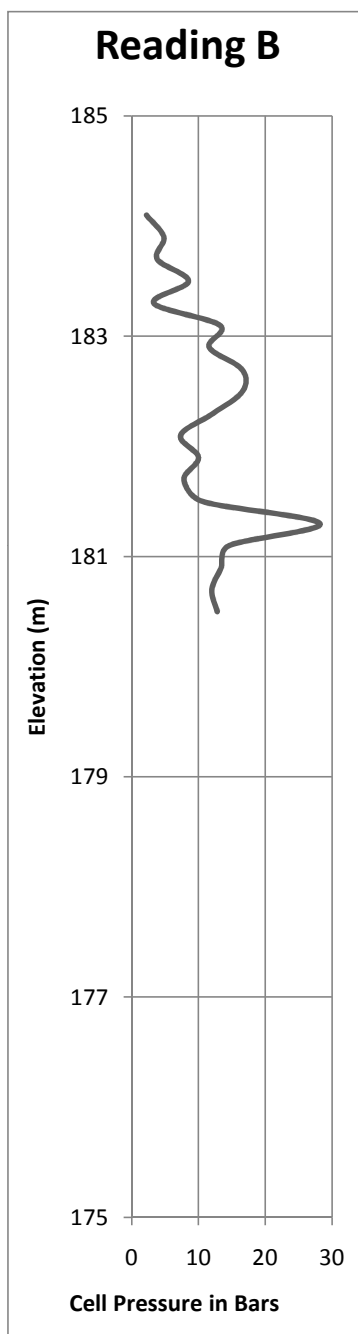
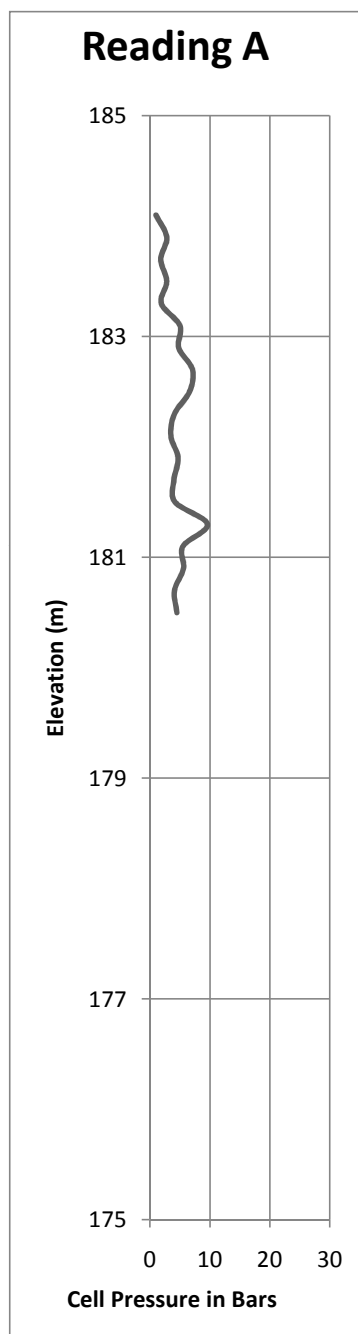
Test Date: 6/22/2011
Predrill Depth : 2.0 m

Sheet 1 of 1
Datum Geodetic

Ground Surface Elevation : 186.3

Delta A: 0.21 Bar

Delta B: 0.28 Bar



Note DMT refusal at elevation 180.3 m. Redrilled to elevation 178.1 m.
Continued with DMT to elevation 160.1 m

Operator: LC

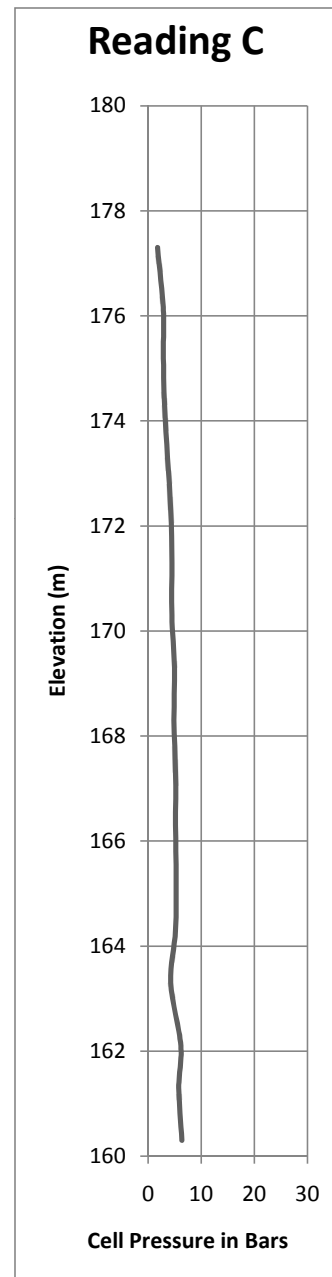
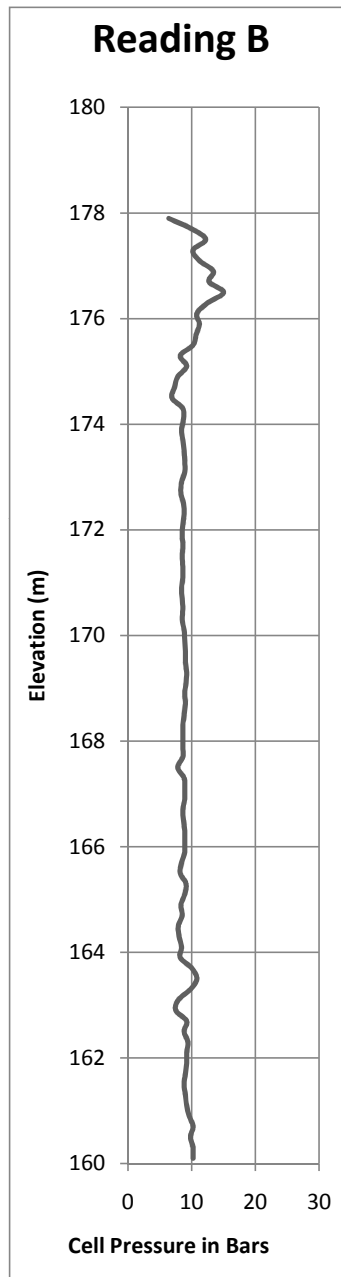
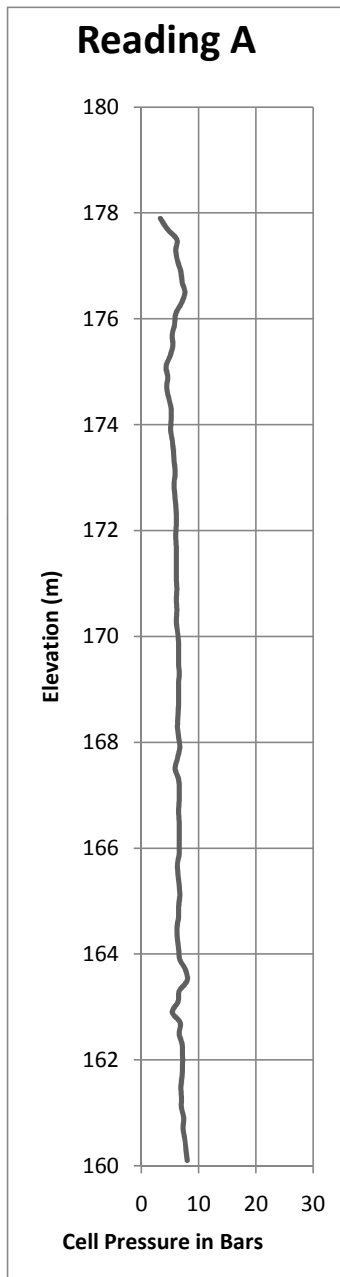
Checked: DD

RECORD OF DILATOMETER TEST DMT B4-1 DEEP

Project : Windsor-Essex Parkway
Location: N 4682042.8; E 330143.9
Ground Surface Elevation : 186.3

Test Date: 6/22/2011
Predrill Depth : 8.2 m
Delta A: 0.19 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.52 Bar



Note Restarted DMT at elevation 172.5m.

Operator: LC

Checked: DD

RECORD OF CONE PENETRATION TEST CPT 13-RW

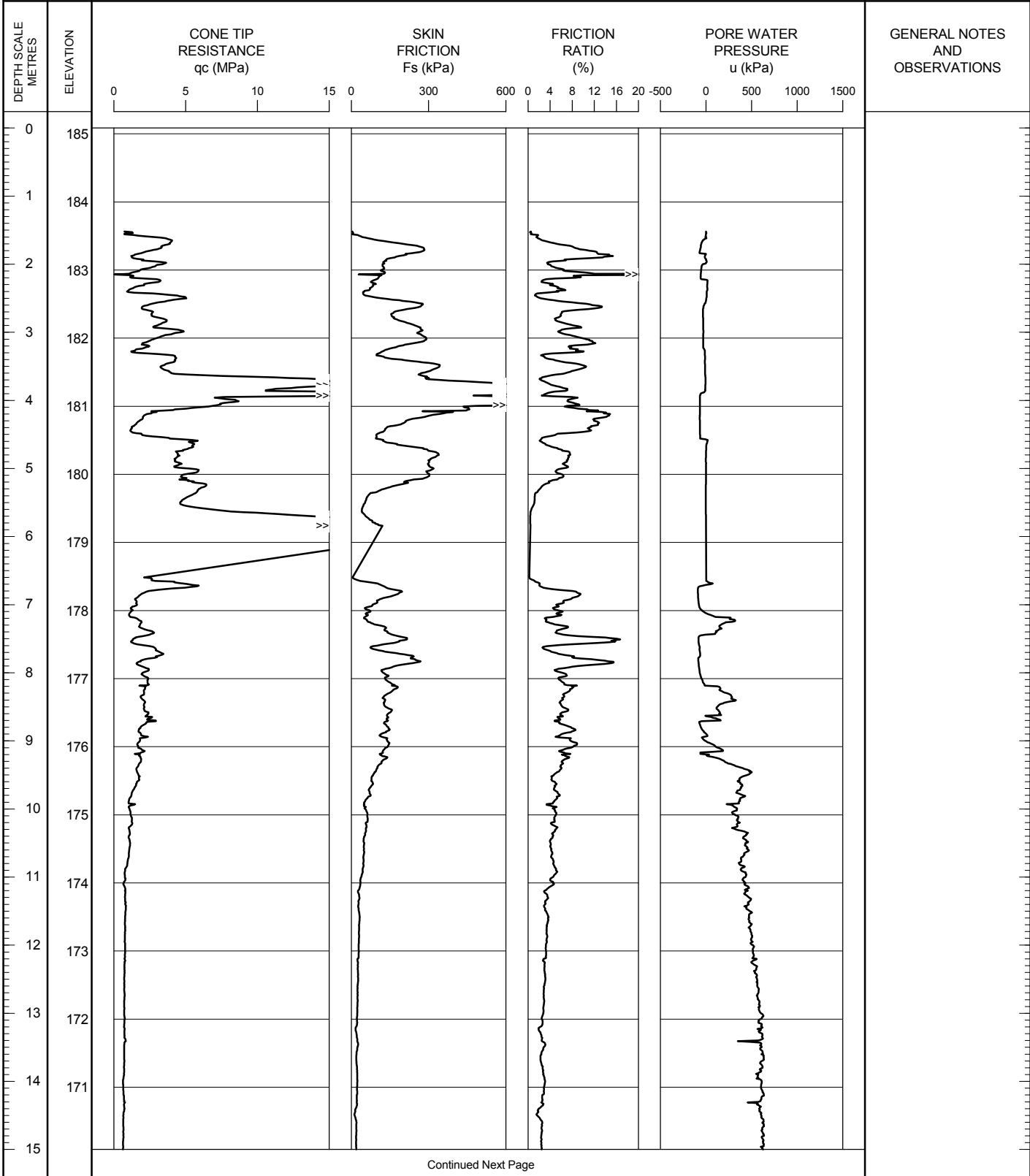
METRIC

PROJECT Windsor-Essex Parkway
LOCATION N4682069.5; E330070.1

TEST DATE 6/8/2011 - 6/11/2011

SHEET 1 OF 3
DATUM Geodetic

GROUND SURFACE ELEVATION: 185.1 PREDRILL DEPTH: 1.5 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



WEP CPT LOG CPT-RW/GPJ ONTARIO MOT.GDT 14/12/12

OPERATOR: TA
CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT 13-RW

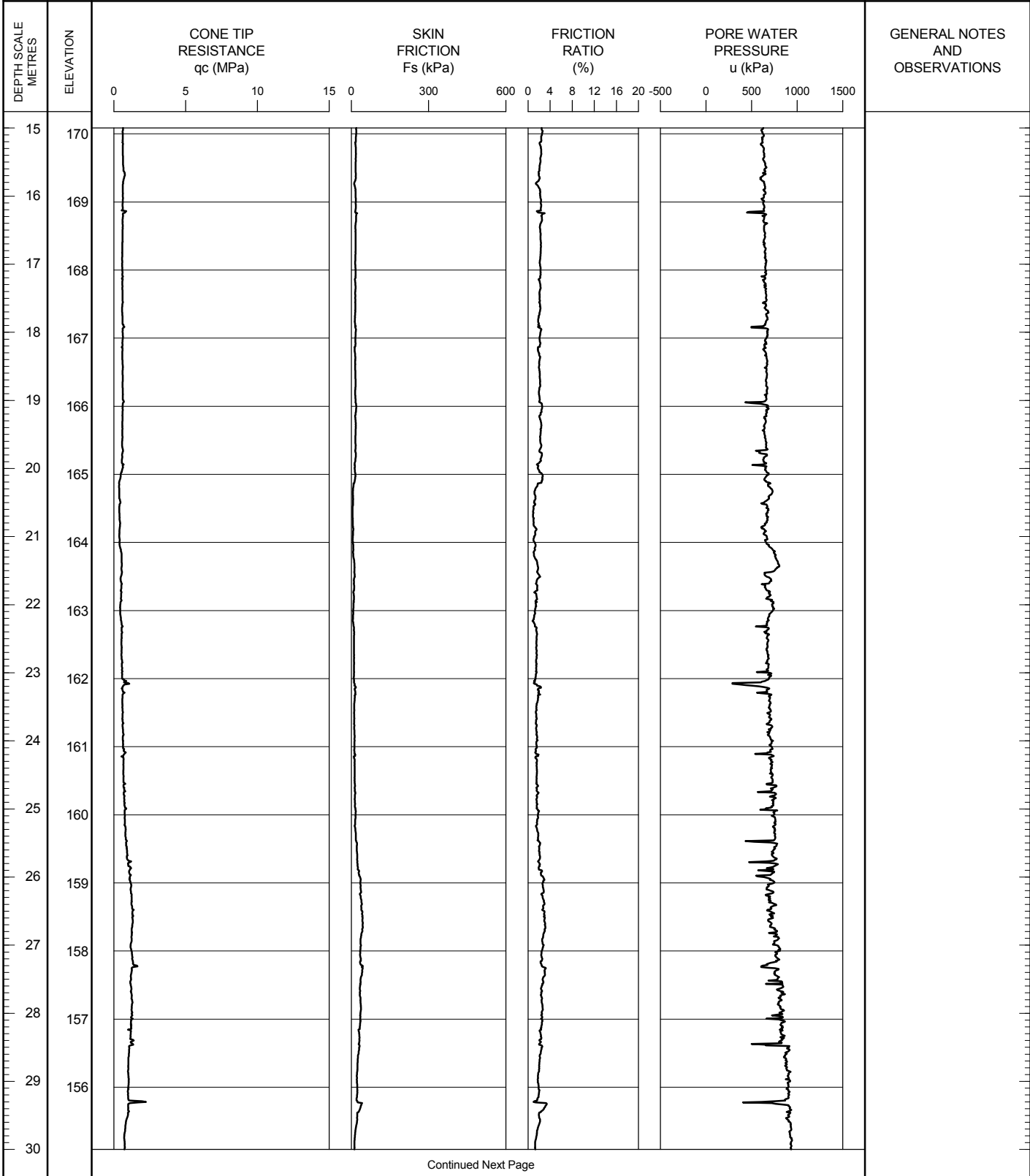
METRIC

PROJECT Windsor-Essex Parkway
LOCATION N4682069.5; E330070.1

TEST DATE 6/8/2011 - 6/11/2011

SHEET 2 OF 3
DATUM Geodetic

GROUND SURFACE ELEVATION: 185.1 PREDRILL DEPTH: 1.5 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA
CHECKED: DD

WEF CPT LOG CPT-RW.GPJ ONTARIO MOT.GDT 14/12/12

RECORD OF CONE PENETRATION TEST CPT 13-RW

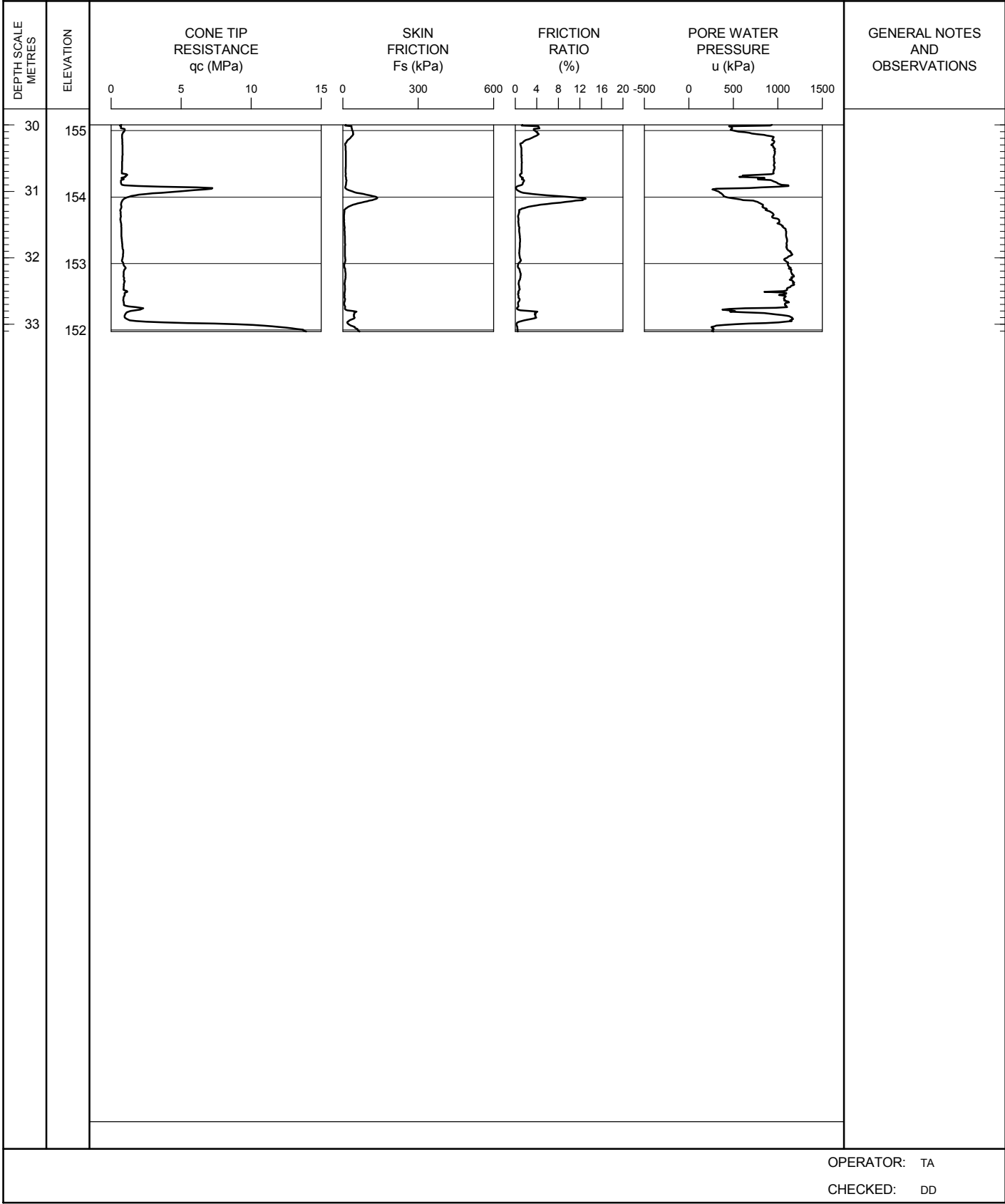
METRIC

PROJECT Windsor-Essex Parkway
LOCATION N4682069.5; E330070.1

TEST DATE 6/8/2011 - 6/11/2011

SHEET 3 OF 3
DATUM Geodetic

GROUND SURFACE ELEVATION: 185.1 PREDRILL DEPTH: 1.5 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



RECORD OF CONE PENETRATION TEST CPT 14-RW

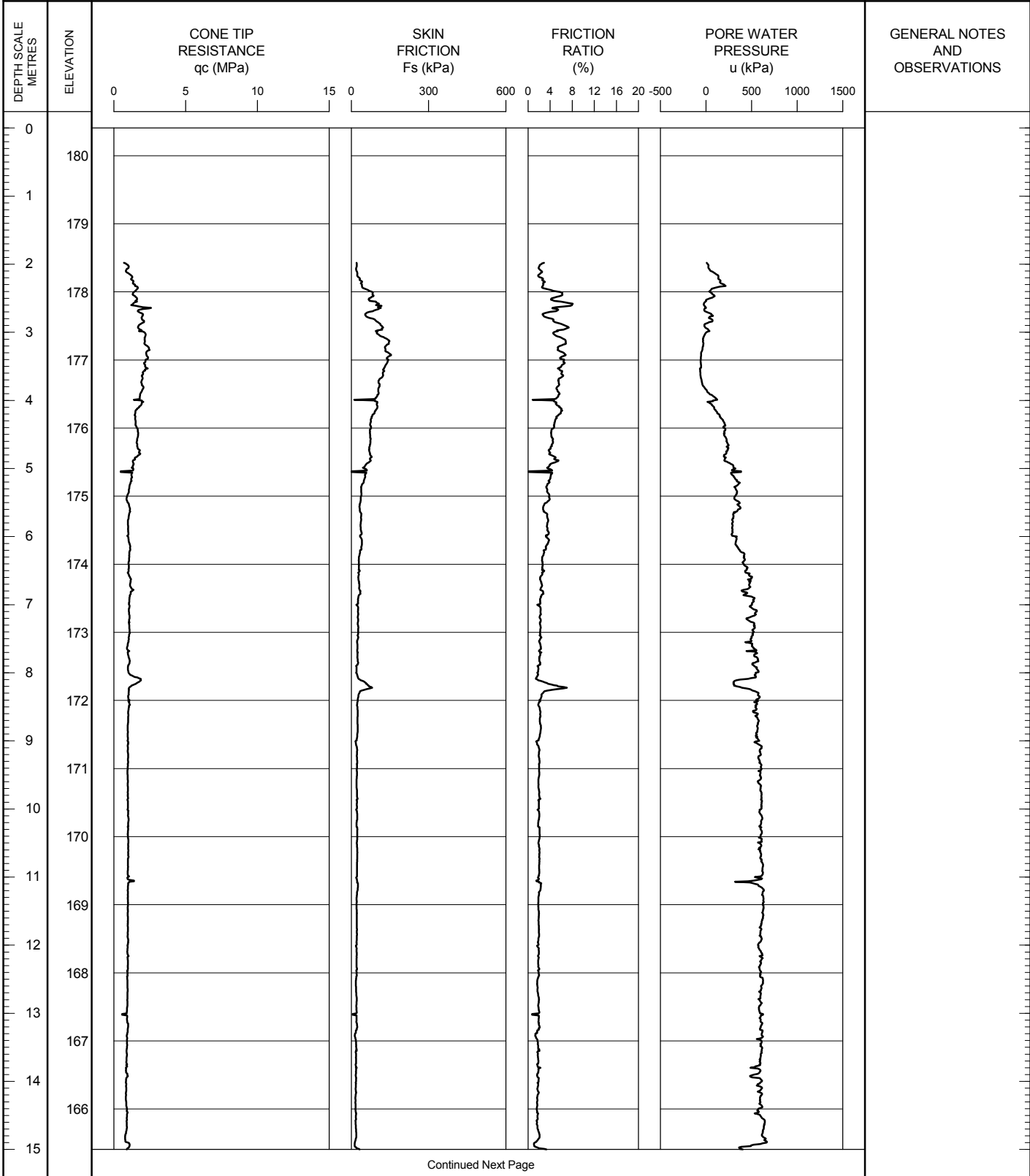
METRIC

PROJECT Windsor-Essex Parkway
LOCATION N4682001.9; E330097.3

TEST DATE 6/7/2011 - 6/7/2011

SHEET 1 OF 2
DATUM Geodetic

GROUND SURFACE ELEVATION: 180.4 PREDRILL DEPTH: 2 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA
CHECKED: DD

WEP CPT LOG CPT-RW.GPJ ONTARIO MOT.GDT 14/12/12

RECORD OF CONE PENETRATION TEST CPT 14-RW

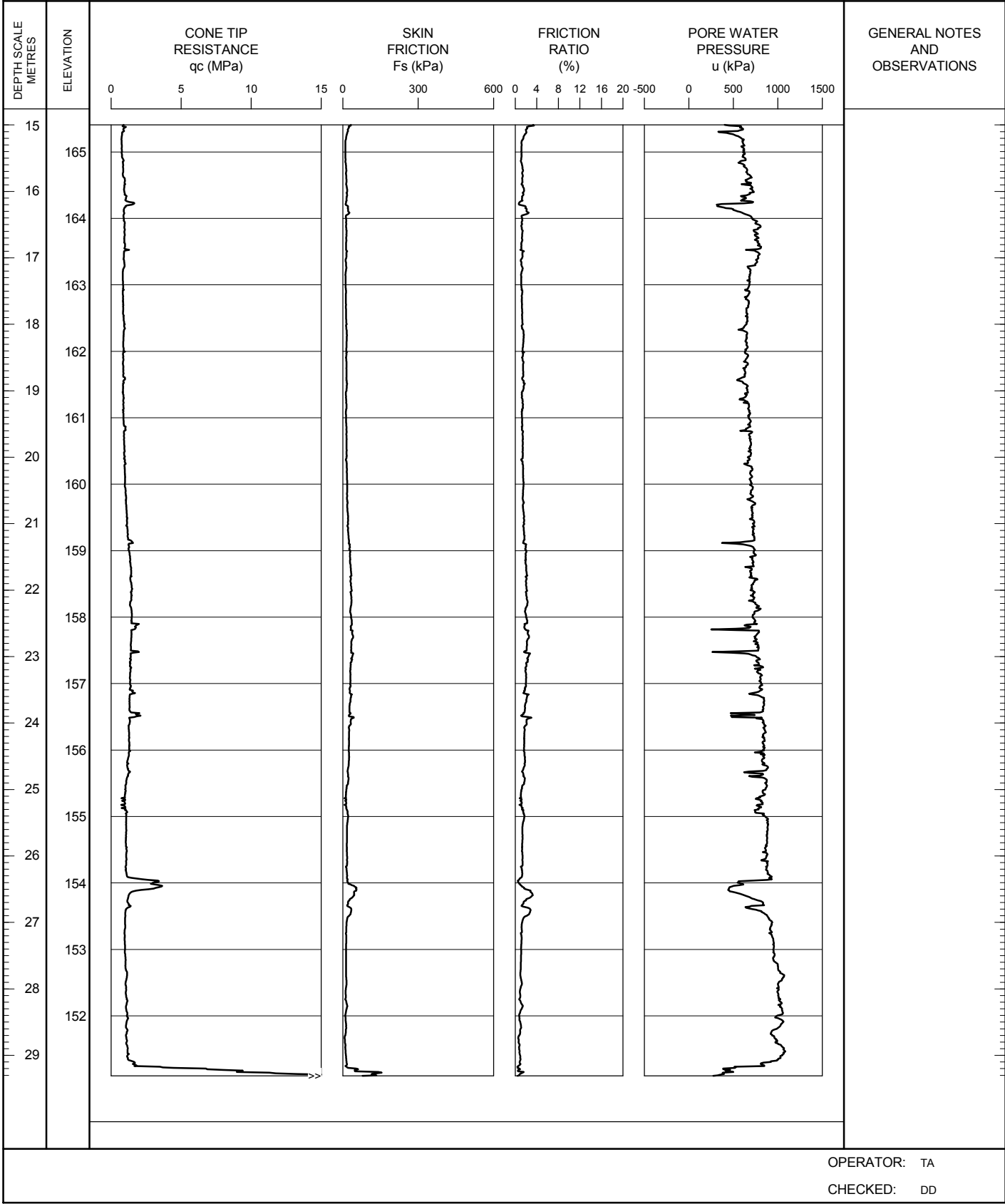
METRIC

PROJECT Windsor-Essex Parkway
LOCATION N4682001.9; E330097.3

TEST DATE 6/7/2011 - 6/7/2011

SHEET 2 OF 2
DATUM Geodetic

GROUND SURFACE ELEVATION: 180.4 PREDRILL DEPTH: 2 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



RECORD OF CONE PENETRATION TEST CPT 15-RW

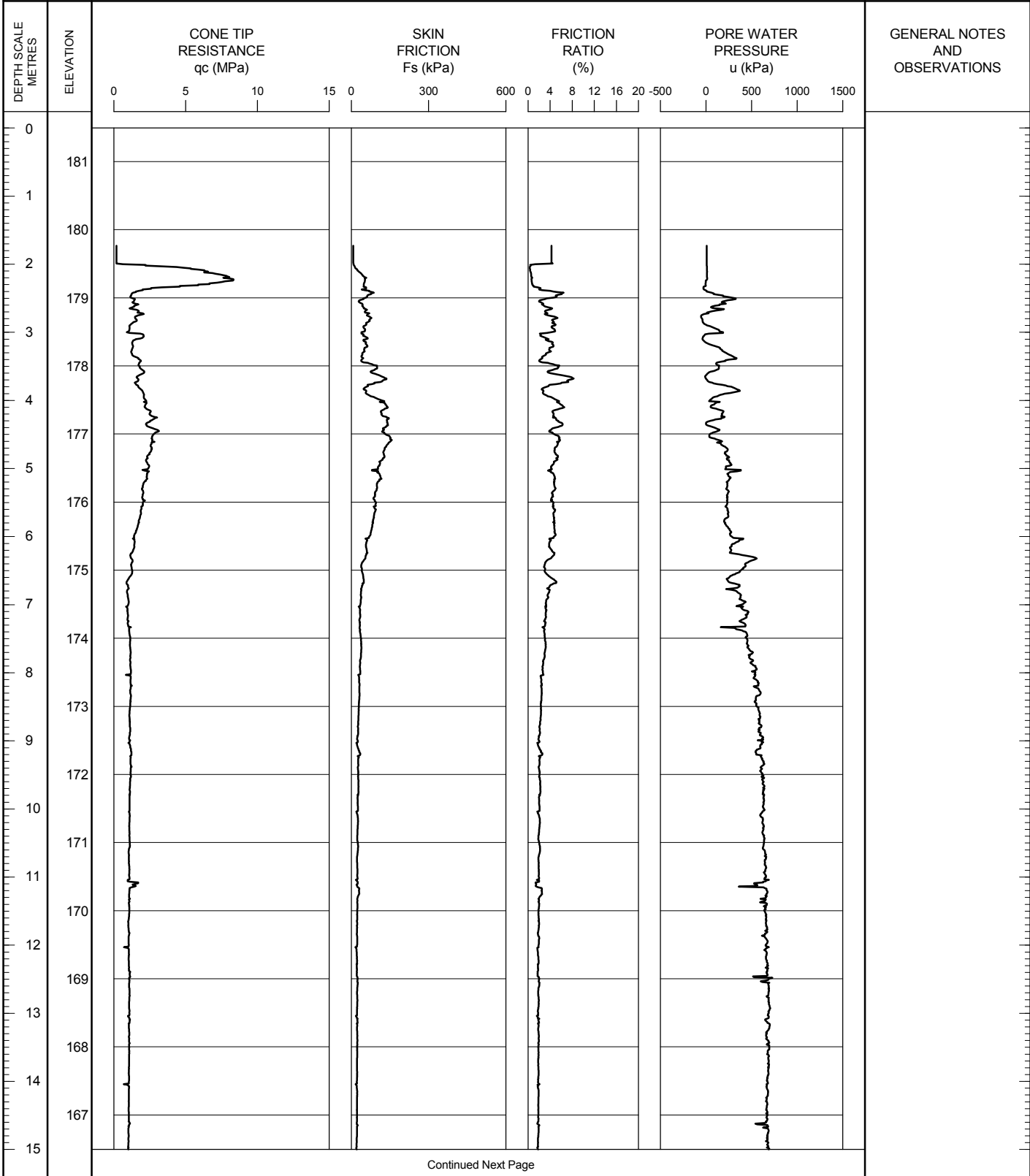
METRIC

PROJECT Windsor-Essex Parkway
LOCATION N4681954.3; E330300.3

TEST DATE 6/8/2011 - 6/8/2011

SHEET 1 OF 3
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.5 PREDRILL DEPTH: 1.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA
CHECKED: DD

WEP CPT LOG CPT-RW.GPJ ONTARIO MOT.GDT 14/12/12

RECORD OF CONE PENETRATION TEST CPT 15-RW

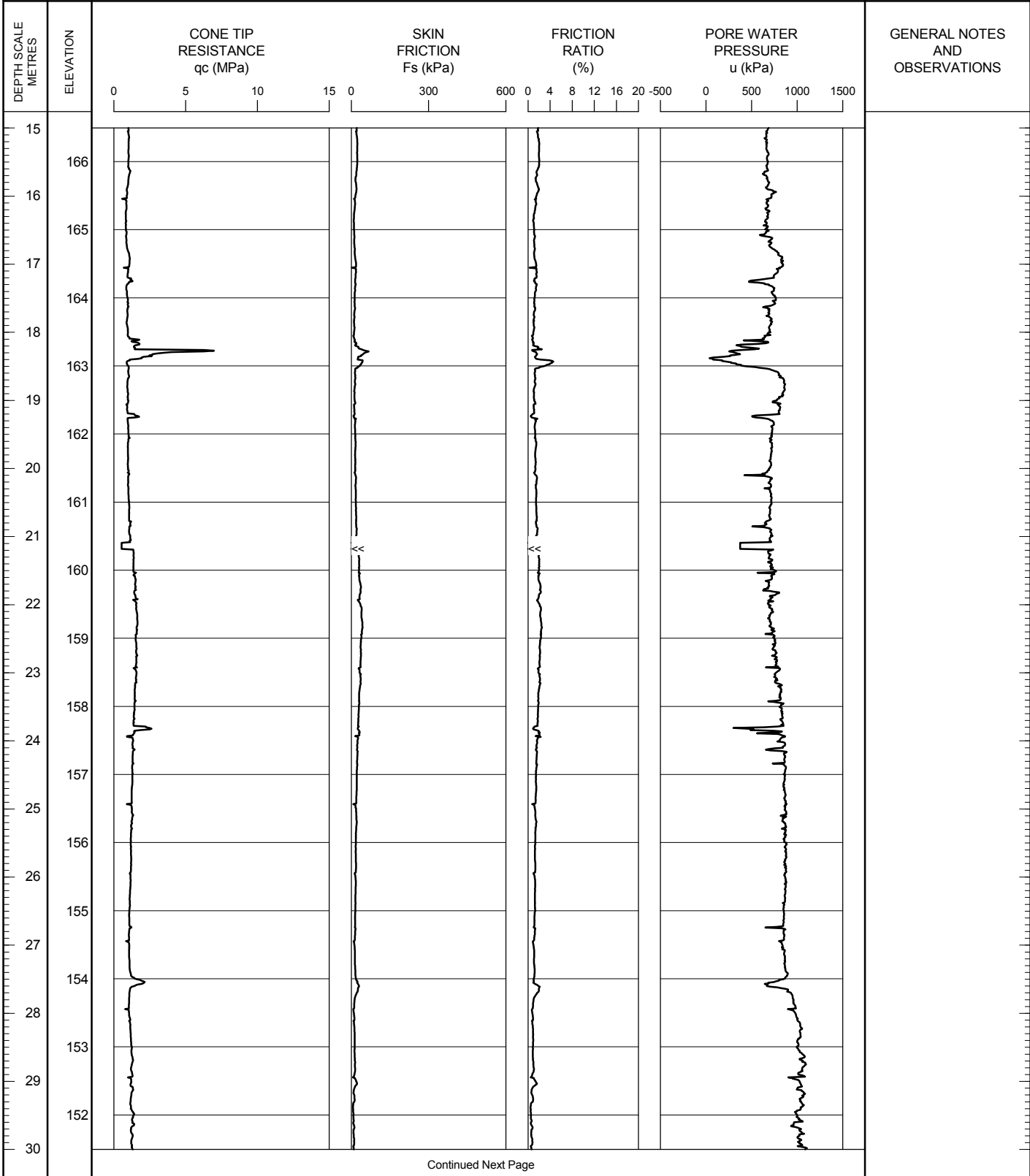
METRIC

PROJECT Windsor-Essex Parkway
LOCATION N4681954.3; E330300.3

TEST DATE 6/8/2011 - 6/8/2011

SHEET 2 OF 3
DATUM Geodetic

GROUND SURFACE ELEVATION: 181.5 PREDRILL DEPTH: 1.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA
CHECKED: DD

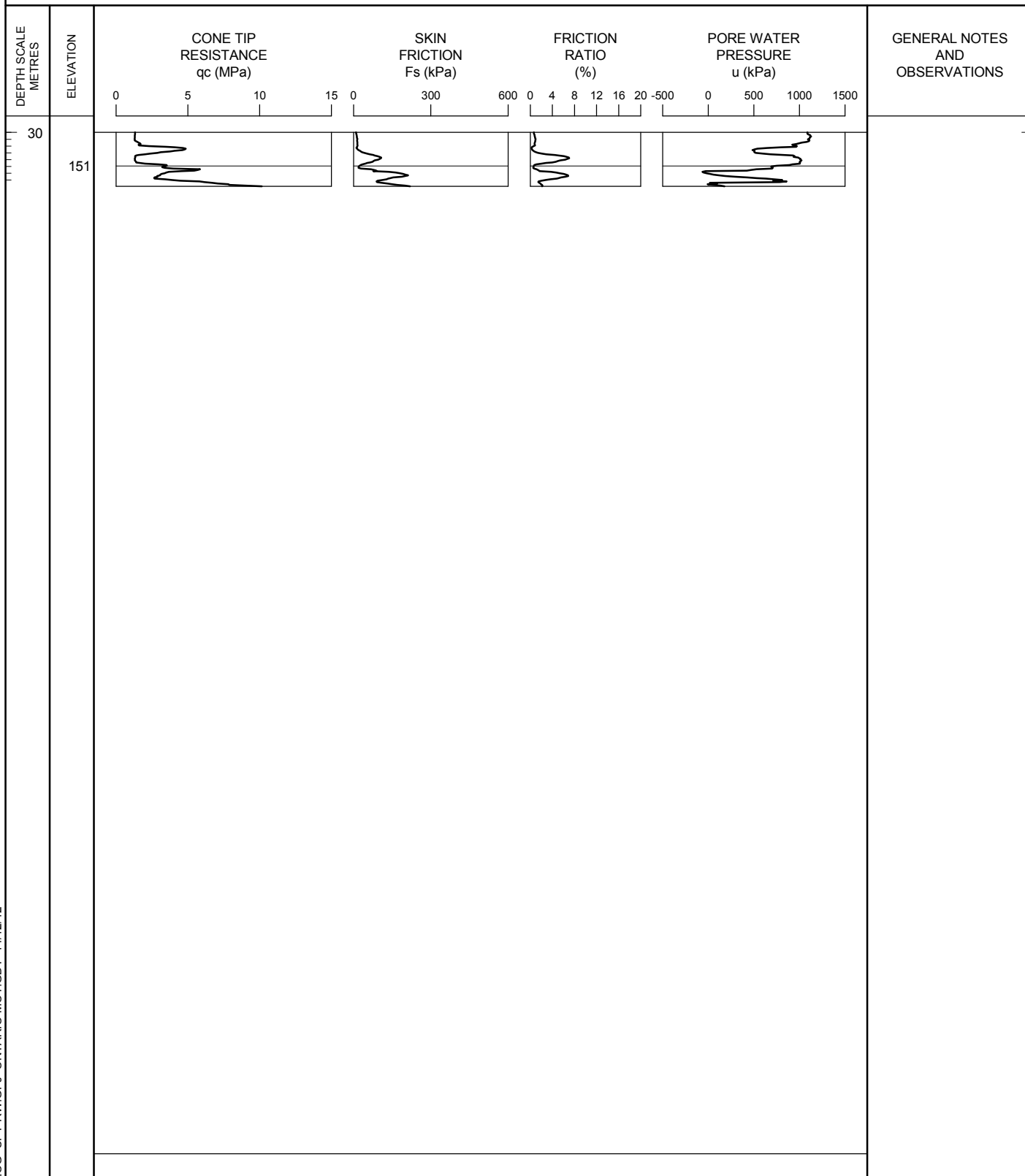
WEP CPT LOG CPT-RW.GPJ ONTARIO MOT.GDT 14/12/12

METRIC

SHEET 3 OF 3

DATUM Geodetic

GROUND SURFACE ELEVATION: 181.5 PREDRILL DEPTH: 1.7 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

Appendix B Borehole and CPT Logs from Previous Investigations

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report - Bridge B-5 (Malden Rd. Overpass – Realigned E.C. Row EBL, Sta. 11+704 to 11+736, Windsor)
Doc No.: 285380-04-119-0115 (Geocres No. 40J6-48)

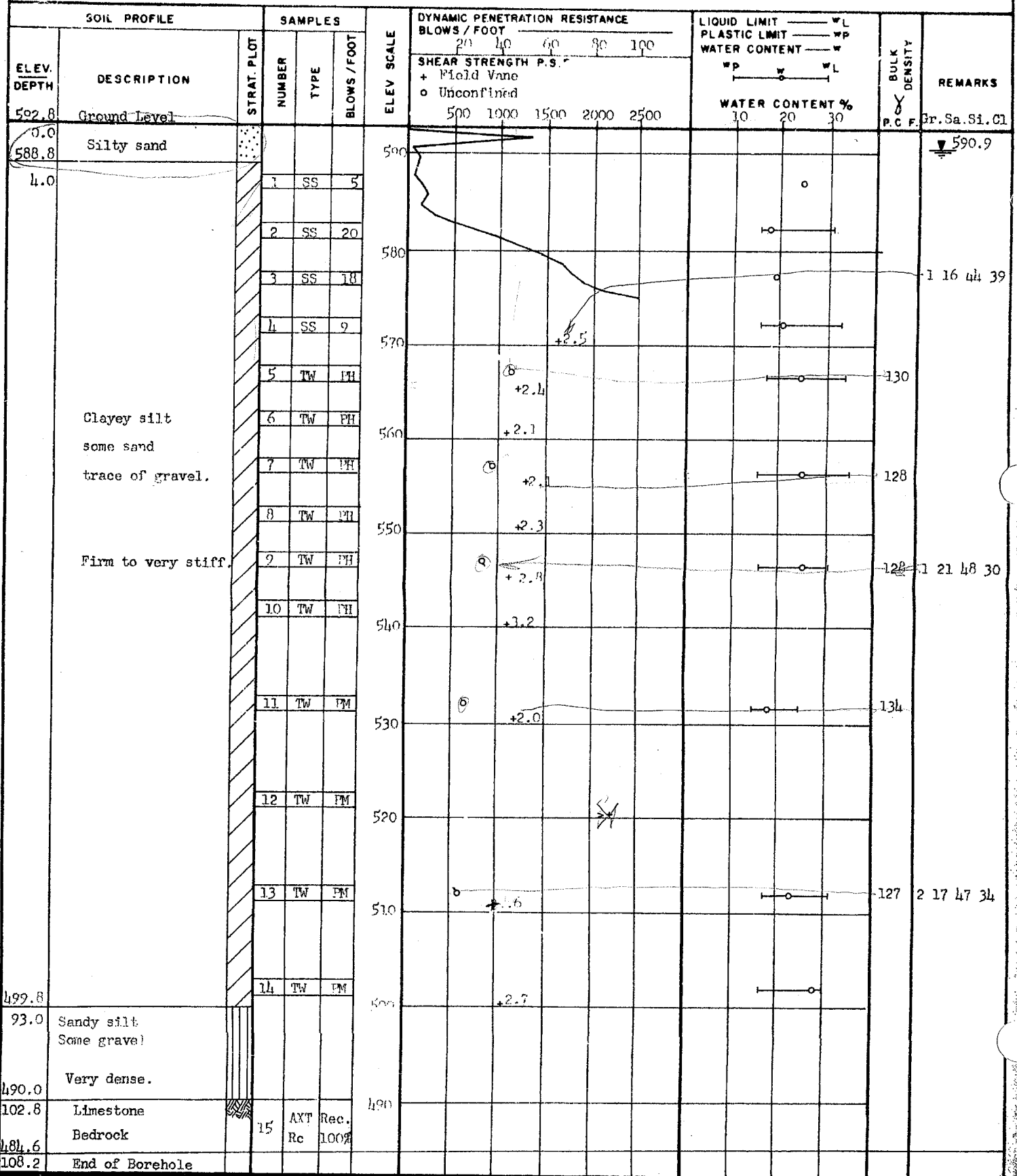
Date: December / 2012
Rev: 0
Page No.: Appendix B

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 68-F-15-1 LOCATION Co-ords. 100,901 N; 55,440 E. ORIGINATED BY AMS
W.P. 260-66-030 BORING DATE Feb. 21 & 22, 1968 COMPILED BY AMS
DATUM Geodetic BOREHOLE TYPE Cont. Flight auger (Bombardier) AXT Rock Core CHECKED BY [Signature]



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

JOB 68-E-15 LOCATION Co-ords. 101,015 N; 55,440 E. ORIGINATED BY AMS
W.P. 260-66-6 BORING DATE Feb. 23, 1968 COMPILED BY AMS
DATUM Geodetic BOREHOLE TYPE Cont. flight auger (bombardier) CHECKED BY AMS

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	W.P.	W.L.	W		
593.0	Ground Level															
0.0	Silty sand.					590										590.5
4.0	Clayey silt some sand trace of gravel. Firm.		1	SS	6											
			2	TW	PH											
			3	TW	PH	580									124	1 11 38 50
			4	TW	PM											
			5	TW	PM	570									127	
			6	TW	PM											
			7	TW	PM	560										
			8	TW	PM											
551.5	End of Borehole															3 17 45 35
41.5																

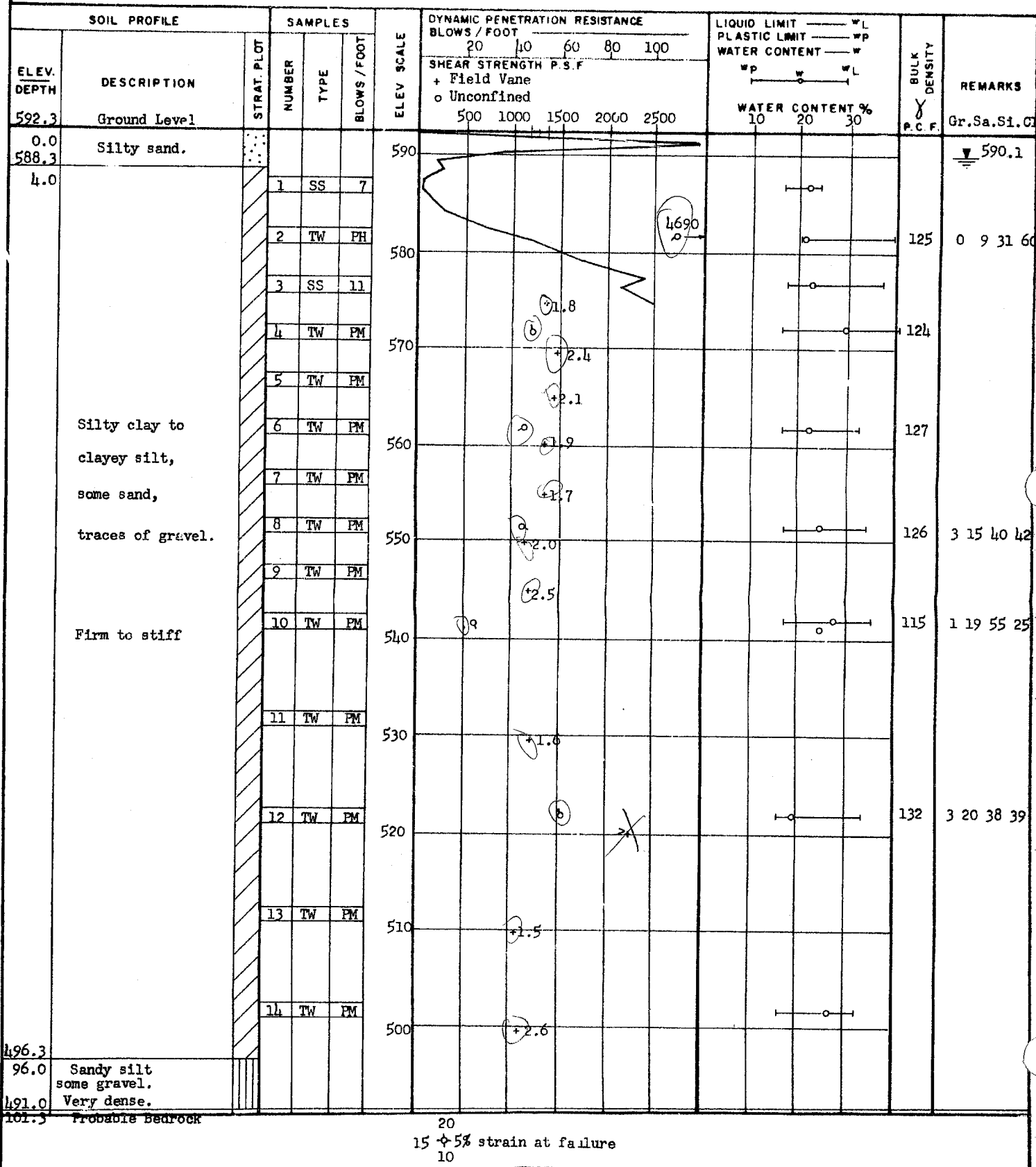
20
15-5 % strain at failure
10

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

JOB 68-F-15-1 LOCATION Co-ords. 101,090 N; 55,320 E. ORIGINATED BY AMS
W.P. 260-66-030 BORING DATE Feb. 26 & 27, 1968 COMPILED BY AMS
DATUM Geodetic BOREHOLE TYPE Cont. flight auger (bombardier) CHECKED BY SL



DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 8

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 68-F-15-1 LOCATION Co-ords. 100,964 N; 55,361 E. ORIGINATED BY AMS
W.P. 260-66-030 BORING DATE Feb. 27, 1968 COMPILED BY AMS
DATUM Geodetic BOREHOLE TYPE Cont. flight auger (bombardier) CHECKED BY 12

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. FLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	WP	WL	W		
593.6	Ground Level															
0.0																
589.6	Silty sand					590										
4.0			1	SS	4											
			2	TW	PH											
			3	TW	PH											
	Silty clay		4	TW	PH											
	Some sand		5	TW	PH											
	trace of gravel.		6	TW	PH											
	Firm		7	TW	PH											
			8	TW	PH											
550.6																
43.0	End of Borehole															

20
15 \pm 5 % strain at failure
10

0 13 41 46

127.5 1 16 44 39

PROJECT 07-1130-207-0		RECORD OF BOREHOLE No 154		1 OF 4	METRIC
W.P. _____		LOCATION N 4681959 9 , E 330200 6		ORIGINATED BY SM	
DIST WEST HWY 401/3		BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY BRS	
DATUM GEODETIC		DATE July 22, 2008 - July 24, 2008		CHECKED BY SSS	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
180.87	GROUND SURFACE										
0.00	TOPSOIL, silty Black										
0.30	SAND, fine, some silt Loose Brown		1	SS	7						
179.50	SILT, some clay, trace sand Loose Grey		2	SS	4						
178.74	SILTY CLAY to CLAYEY SILT, some sand, trace gravel, with silt seams Firm to stiff Grey		3	TO	PH						
			4	TO	PH						
			5	SS	14						
			6	TO	PH						
			6A	SS	6						
			7	SS	7						
			8	TO	PH						
			9	SS	7						
			10	TO	PH						
			11	SS	11						
			12	TO	PH						

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 8/29/09

Continued Next Page

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

RECORD OF BOREHOLE No 154

2 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4681959.9 :E 330200.6

ORIGINATED BY SM

DIST WEST HWY 401/3

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

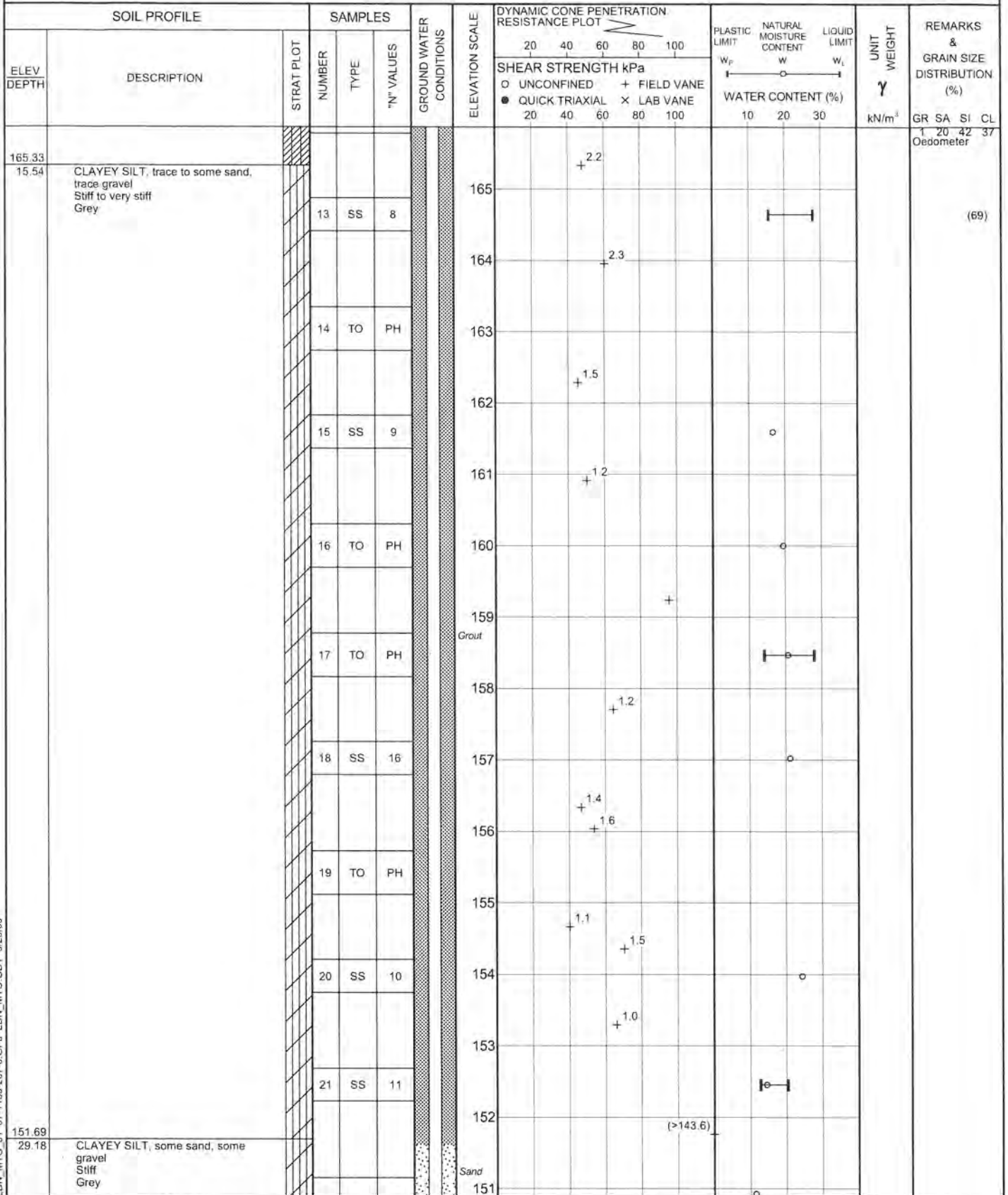
COMPILED BY BRS

DATUM GEODETIC

DATE

July 22, 2008 - July 24, 2008

CHECKED BY *SB*



Continued Next Page

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

LDN MTO_01 07-1130-207-0.GPJ LDN MTO GDT 6/29/09

RECORD OF BOREHOLE No 154

3 OF 4

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4681959.9 ; E 330200.6

ORIGINATED BY SM

DIST WEST HWY 401/3

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

COMPILED BY BRS

DATUM GEODETIC

DATE

July 22, 2008 - July 24, 2008

CHECKED BY *SS*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	*N* VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
	CLAYEY SILT, some sand, some gravel Stiff Grey		22	SS	11							
149.63			23	SS	60/ 125mm		150					
31.24	LIMESTONE, fresh, medium strong, weakly laminated to bedded, fine to coarse grained, faintly porous Light grey to tan to brown (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		24	NQ RC			149 Screen					
			25	NQ RC			148	100 98 97				
			26	NQ RC			147	T.C.R. (%) 94 S.C.R. (%) 87 R.Q.D. (%) 84				
			27	NQ RC			146	86 75 36				
144.22							145	97 92 83				
36.65	END OF BOREHOLE Borehole dry during drilling between July 22 and July 24, 2008. Water level measured in piezometer at elev. 178.97m on July 28, 2008. Water level measured in piezometer at elev. 180.42m on September 19, 2008. Water level measured in piezometer at elev. 177.23m on November 11, 2008. Water level measured in piezometer at elev. 178.27m on January 28, 2009.											UC

+³, X³

Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 154

SHEET 4 OF 4

LOCATION: N 4681959.9 E 330200.6

DRILLING DATE: July 22, 2008 - July 24, 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)	FLUSH % RETURN	ELEVATION	RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL PORT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)	CORRECTION					TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION			
		ROCK SURFACE		149.63														
32	MUD ROTARY NO ROCK CORE	LIMESTONE, fresh, medium strong, very fine to coarse grained, faintly porous with occasional pits, stylolitic, fossiliferous (up to 5 cm diameter), mottled light grey to grey		31.24					149									
		LIMESTONE, fresh, medium strong, weakly laminated, fine to medium grained, faintly porous, tan		146.70	1				148									
				147.59														
		LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, tan to grey		33.28	2				147									
				147.27														
		LIMESTONE, fresh, medium strong, coarse grained with up to 3 cm diameter inclusions in fine grained matrix, faintly porous, brown to grey with dark grey zone at 33.62m		33.60														
				33.74														
		LIMESTONE, fresh, medium strong, very fine to fine grained, laminated, faintly porous, light brown to grey		146.68	3				146									
				34.19														
				34.38														
35		LIMESTONE, fresh, medium strong, very fine to fine grained, laminated, faintly porous, light brown to grey		146.10					145									
		LIMESTONE, fresh, medium strong, laminated very fine to fine grained, faintly porous stylolitic, mottled grey		34.77	4													
				145.74														
		LIMESTONE, fresh, medium strong, laminated very fine to fine grained, faintly porous stylolitic, mottled grey		35.13														
				145.31														
		LIMESTONE, fresh, medium strong, very fine grained, very weakly laminated, faintly porous, light grey		35.56														
				144.56														
		LIMESTONE, fresh, medium strong, fine grained, laminated, faintly porous, light brown and grey		36.40														
				36.65														
36		LIMESTONE, fresh, medium strong, weakly laminated, very fine to fine grained, porous with occasional pits, weakly stylolitic, light grey to grey																
		LIMESTONE, fresh, medium strong, bedded, fine grained, faintly porous, tan to brown																
		LIMESTONE, fresh, medium strong, up to 2 cm inclusions (angular) within fine grained matrix, faintly porous, brown, Breccia like texture																
		LIMESTONE, fresh, medium strong, laminated, fine grained, faintly porous, tan to brown																
37		END OF DRILLHOLE																
38																		
39																		
40																		
41																		
42																		
43																		
44																		
45																		
46																		

DEPTH SCALE

1 : 75



LOGGED: SG

CHECKED: SJB

PROJECT <u>09-1132-0080</u>		RECORD OF BOREHOLE No CPT-338		1 OF 1		METRIC	
W.P. _____		LOCATION <u>N 4681980.3 ; E 330141.6</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 15, 2009</u>		CHECKED BY _____			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W		W _L	GR	SA	SI	CL
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						WATER CONTENT (%)						
181.22	GROUND SURFACE																			
0.00	TOPSOIL, sandy, trace clay Black						181													
0.23	FILL, clayey silt, some sand, trace gravel																			
180.61	Brown																			
0.61	SILTY FINE SAND, trace organics Loose to compact Dark brown		1	SS	8		180							○						
179.39			2	SS	10															
1.83	SANDY SILT, some clay Compact Grey																			
2.13	CLAYEY SILT, some sand, trace gravel, with occasional silt partings Stiff Grey						179													
178.32			3	SS	10									○						
2.90	END OF BOREHOLE																			
	Groundwater encountered at about elev. 179.6m during drilling on December 15, 2009.																			

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

Depth (m)	Elevation (m)	Undrained Shear Strength (kPa)			Sensitivity
		Natural	Post-Peak	Remoulded	
7.6	174.7	99	82	78	1.3
8.6	173.7	70	59	61	1.1
9.6	172.7	62	53	57	1.1
10.6	171.7	59	45	49	1.2
11.6	170.7	60	49	57	1.1
12.6	169.7	53	53		
13.6	168.7	55	45	55	1.0
14.6	167.7	51	40	43	1.2
15.6	166.7	64	19	42	1.5
16.6	165.7	43	38		
17.6	164.7	30	13		
18.6	163.7	34	30		
19.6	162.7	30	30		

Field Vane Location 154 (Borehole BH-154)

5.6	175.3	91	57	57	1.6
6.6	174.3	66	40	47	1.4
7.6	173.3	76	68	64	1.2
8.6	172.3	57	26		
9.6	171.3	74	40	21	3.5
10.6	170.3	70	55	30	2.3
11.6	169.3	44	25	28	1.6
12.6	168.3	53	30		
13.6	167.3	57	40	40	1.4
14.6	166.3	51	45	28	1.8

Field Vane Location 158 (Borehole BH-158)

3.7	175.6	110	64	55	2.0
5.7	173.6	57	21	28	2.0
6.7	172.6	57	13	38	1.5
7.7	171.6	62	34	42	1.5
8.7	170.6	51	17	38	1.4
9.7	169.6	62	17	38	1.7
10.7	168.6	53	43	40	1.3
11.7	167.6	40	19		
12.7	166.6	32	11	26	1.2
13.7	165.6	26	15	23	1.2
14.7	164.6	45	13	25	1.8
15.7	163.6	55	28	34	1.6
16.7	162.6	26	8		
17.7	161.6	25	13	13	1.9

Field Vane Location 160 (Borehole BH-160)

3.7	175.0	54	23	15	3.5
5.7	173.0	32	13	11	2.8
6.7	172.0	19	2	11	1.7
7.7	171.0	34	13	13	2.6
8.7	170.0	23	6	17	1.3
9.7	169.0	38	15	20	1.9

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-154

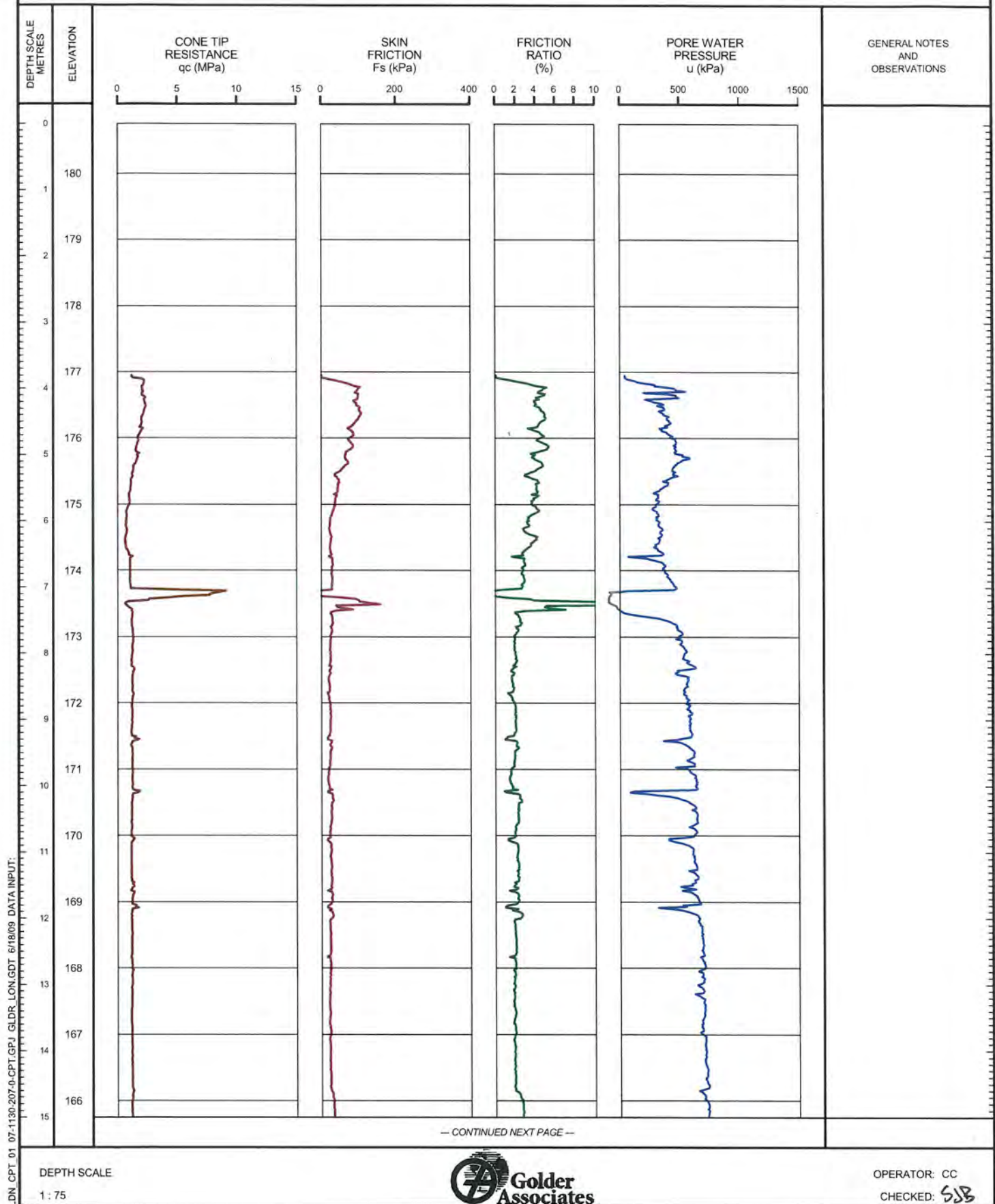
SHEET 1 OF 2

LOCATION: N 4681963.3 ; E 330191.0

TEST DATE:

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 4.58m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



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

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-154

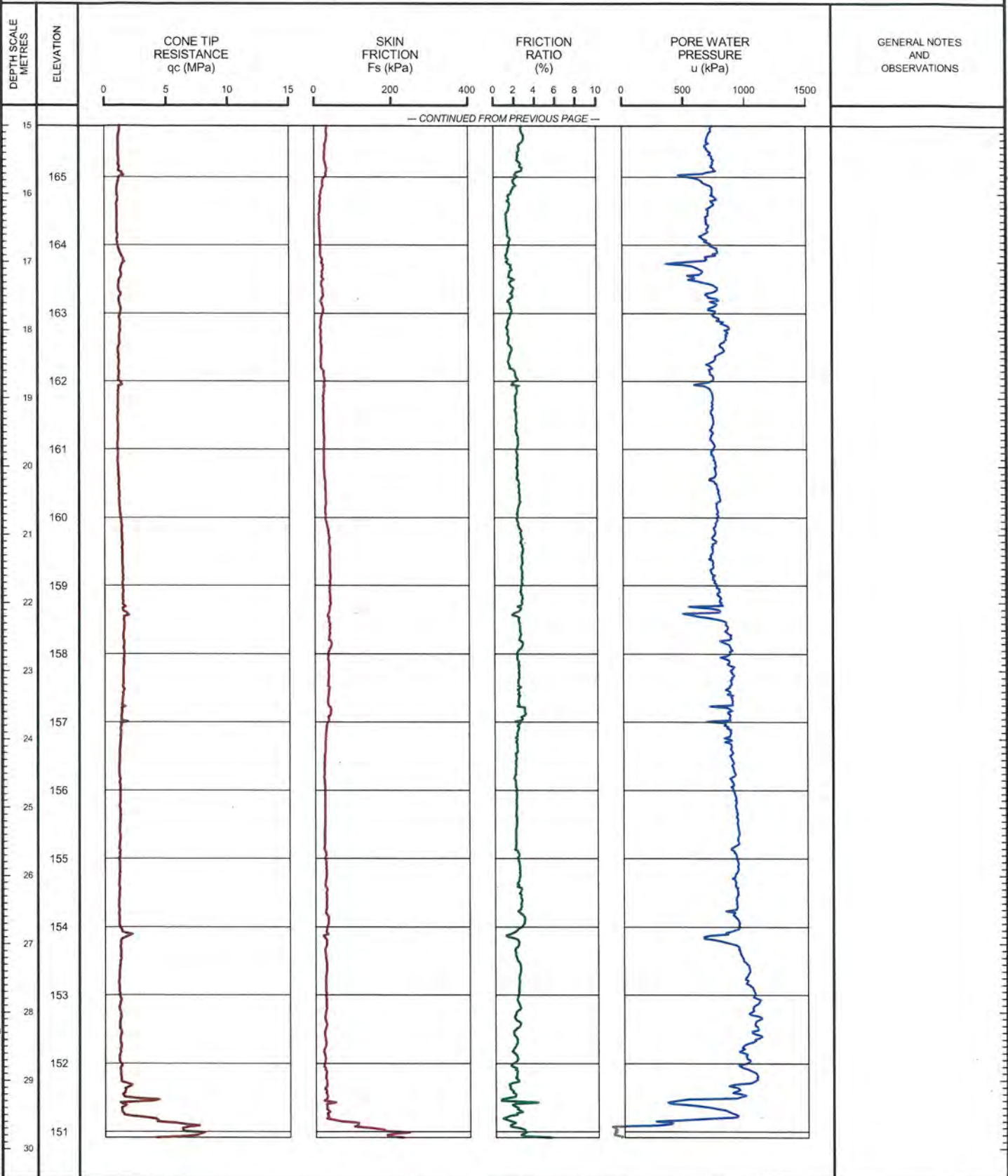
SHEET 2 OF 2

LOCATION: N 4681963.3 ; E 330191.0

TEST DATE:

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 4.58m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



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

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: SJB

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-338

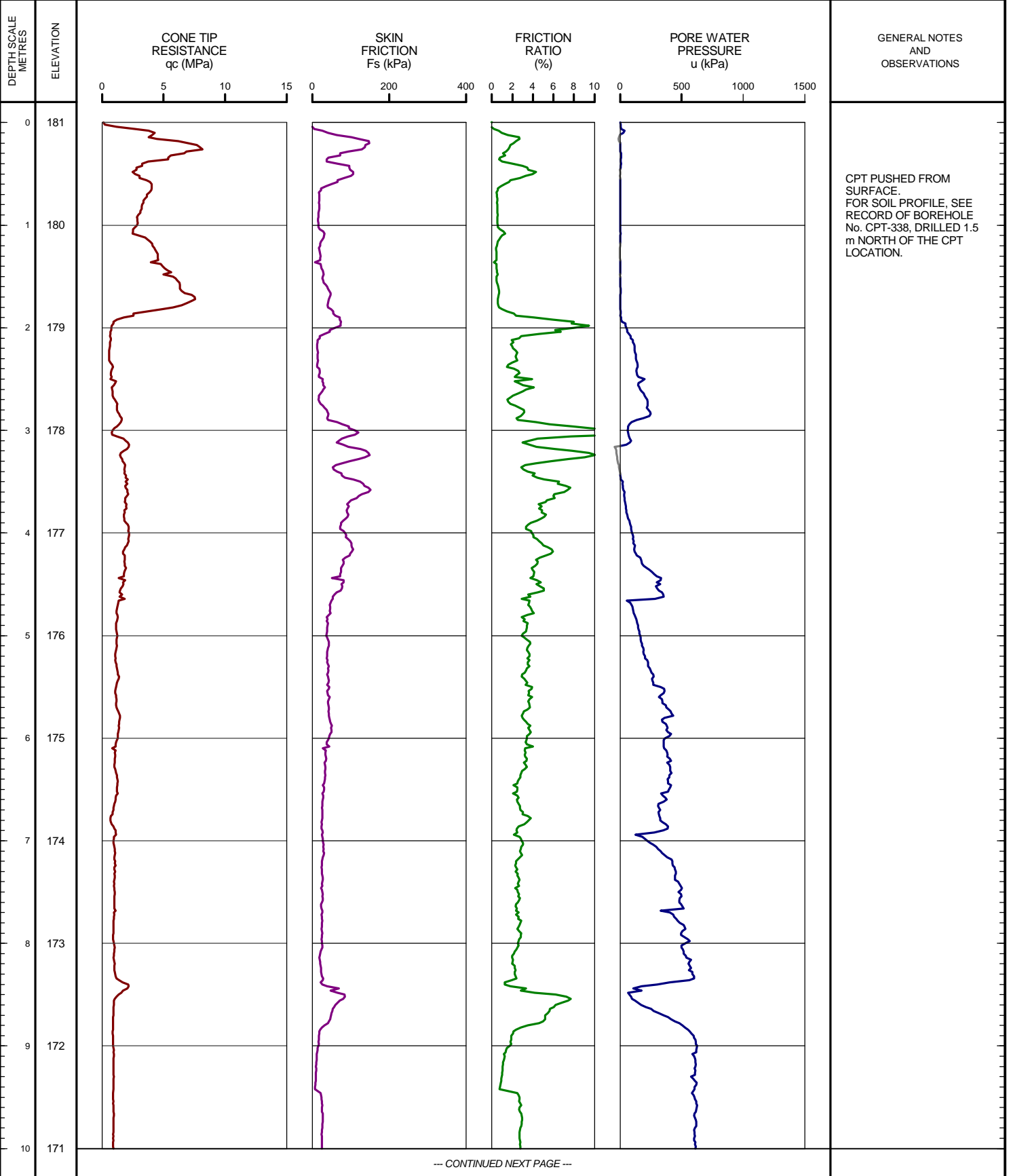
SHEET 1 OF 3

LOCATION: N 4681980.3 ;E 330141.6

TEST DATE: December 15, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.22m 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-338

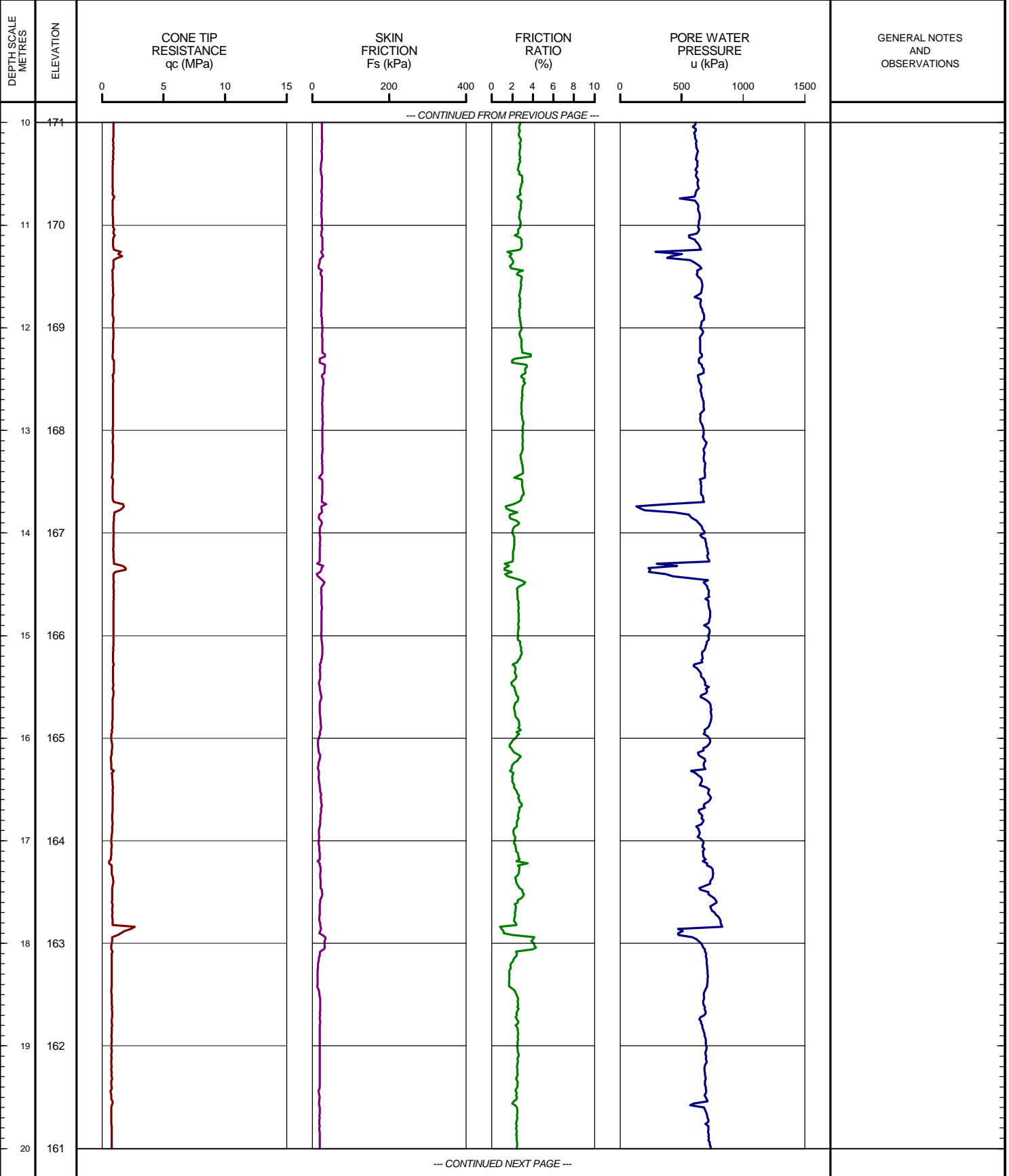
SHEET 2 OF 3

LOCATION: N 4681980.3 ;E 330141.6

TEST DATE: December 15, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.22m 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-338

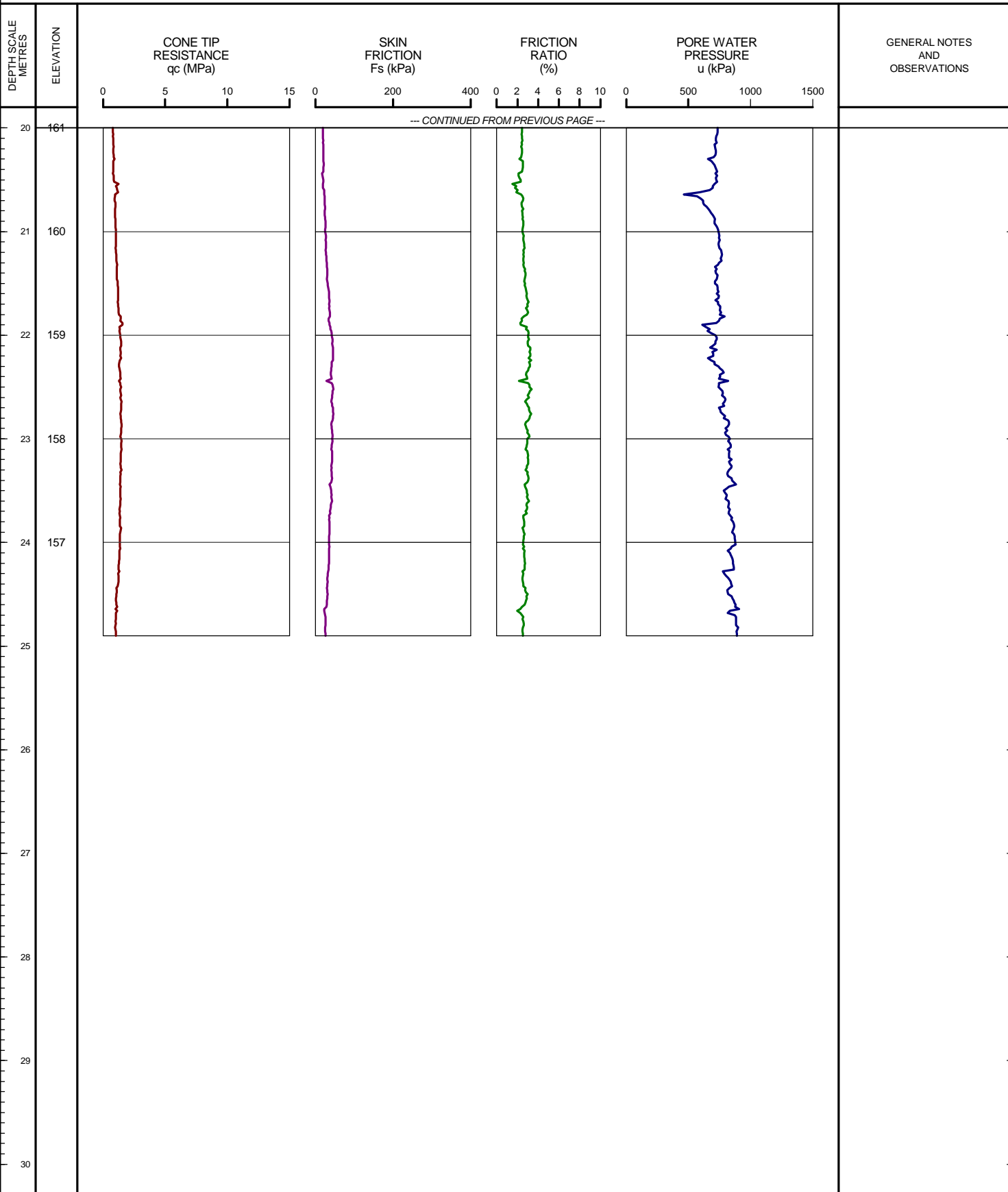
SHEET 3 OF 3

LOCATION: N 4681980.3 ;E 330141.6

TEST DATE: December 15, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 181.22m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 50



OPERATOR: TA

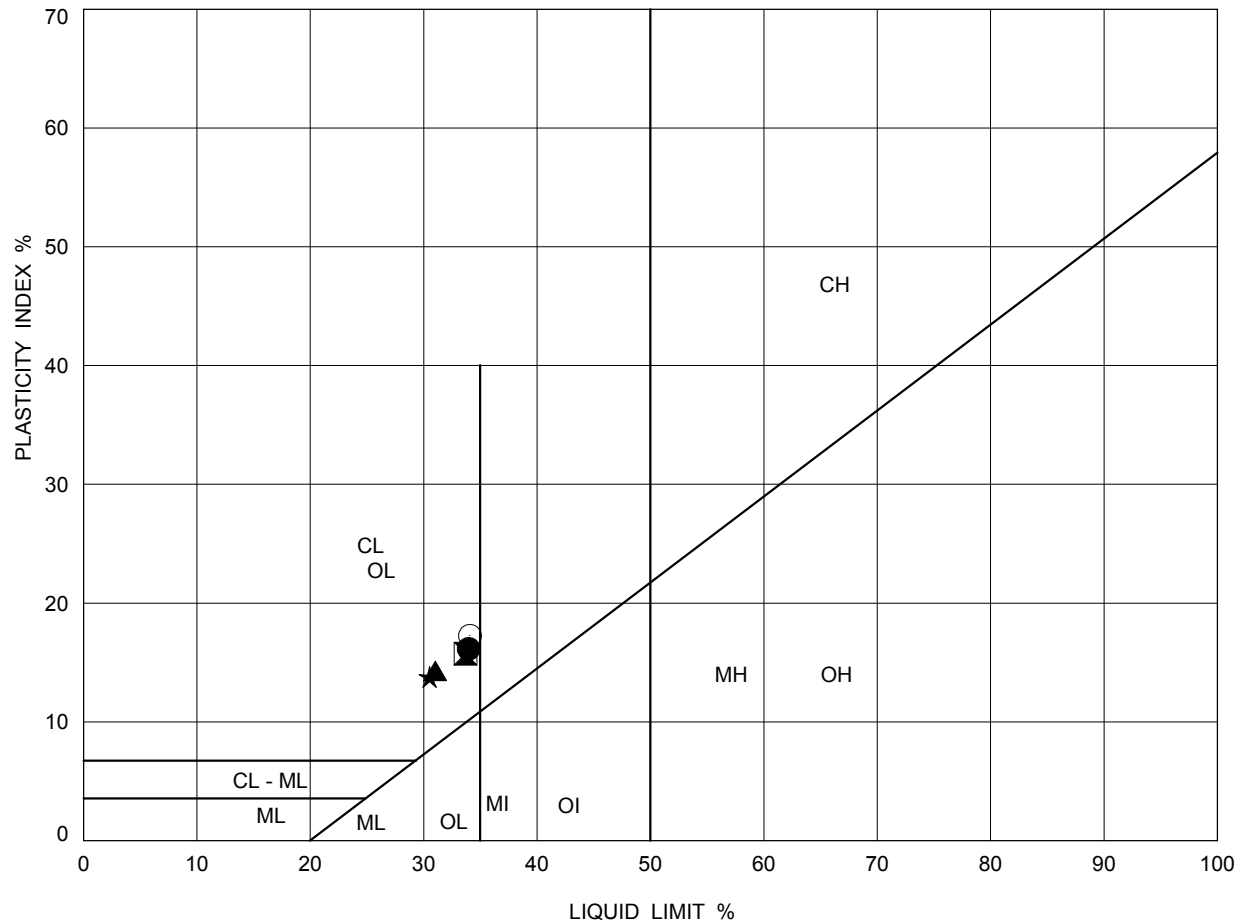
CHECKED:

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

Appendix C Geotechnical Laboratory Test Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report - Bridge B-5 (Malden Rd.
Overpass – Realigned E.C. Row EBL, Sta. 11+704 to 11+736, Windsor)
Doc No.: 285380-04-119-0115 (Geocres No. 40J6-48)

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



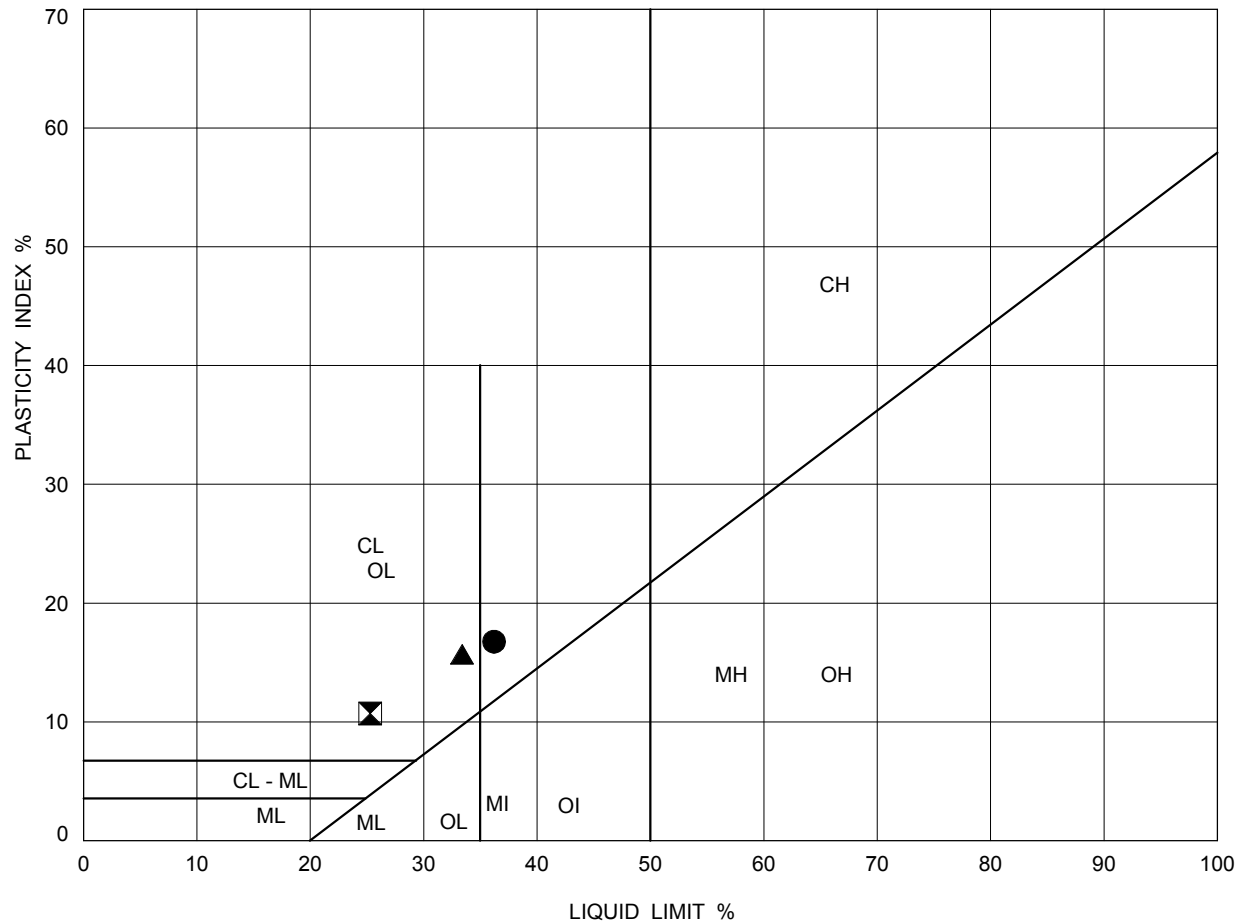
SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B4-1	8	6.1	34	18	16
⊠	B4-1	13	13.7	34	18	16
▲	B4-1	15	16.8	31	17	14
★	B4-1	19	22.9	31	17	14
○	BH06-RW	12	12.2	34	17	17

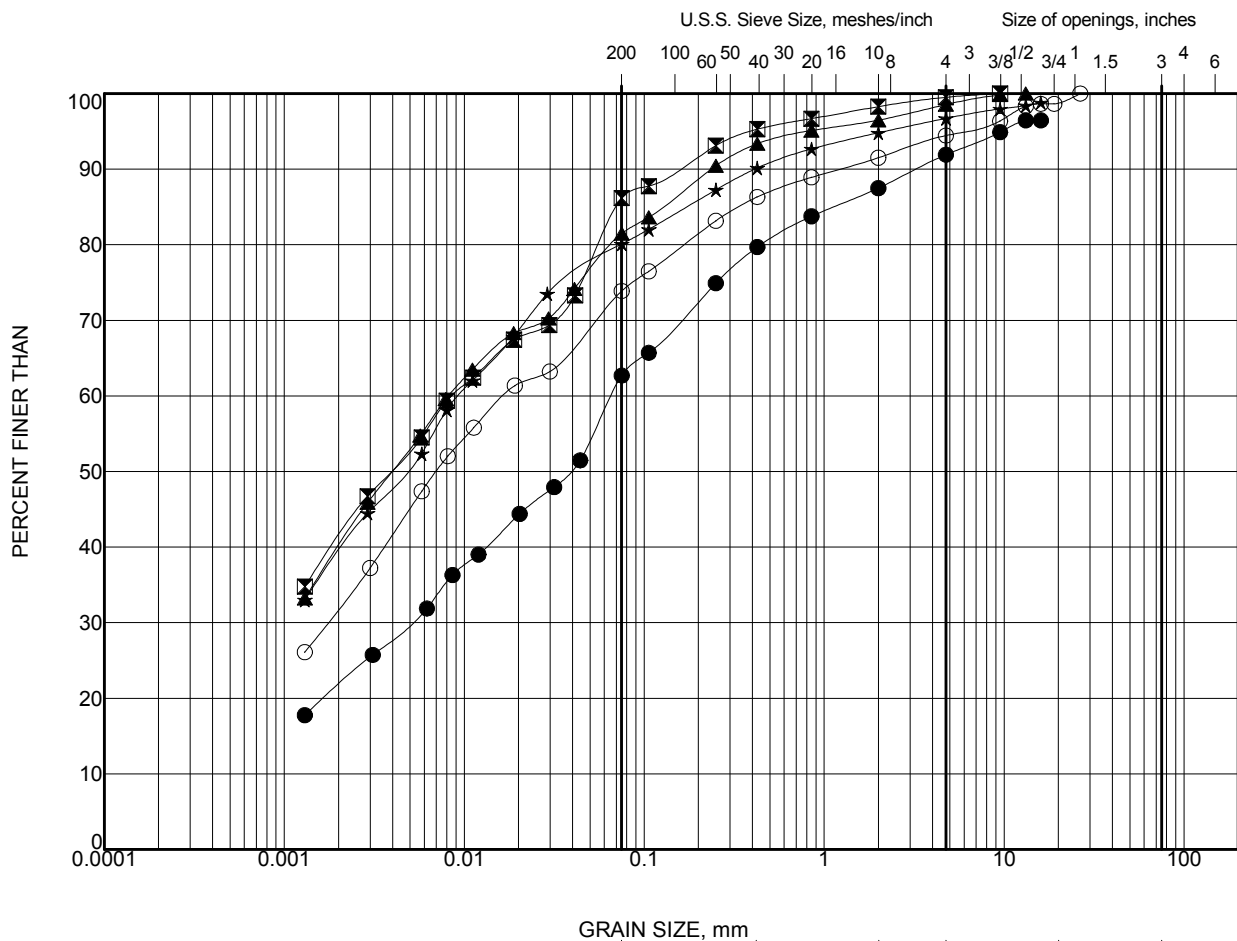
PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART Clayey Silt to Silty Clay	
	PROJECT No. SW8801.1004.101		FILE No.
	DRAWN	SS	SCALE
CHECK	SF		REV.
FIGURE C1			



LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	BH06-RW	14	15.2	36	19	17
⊠	BH06-RW	16	18.3	25	15	10
▲	BH06-RW	21	25.9	33	18	15

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		PLASTICITY CHART Clayey Silt to Silty Clay	
 		PROJECT No. SW8801.1004.101	FILE No.
DRAWN SS		SCALE	
CHECK SF		REV.	
		FIGURE C2	

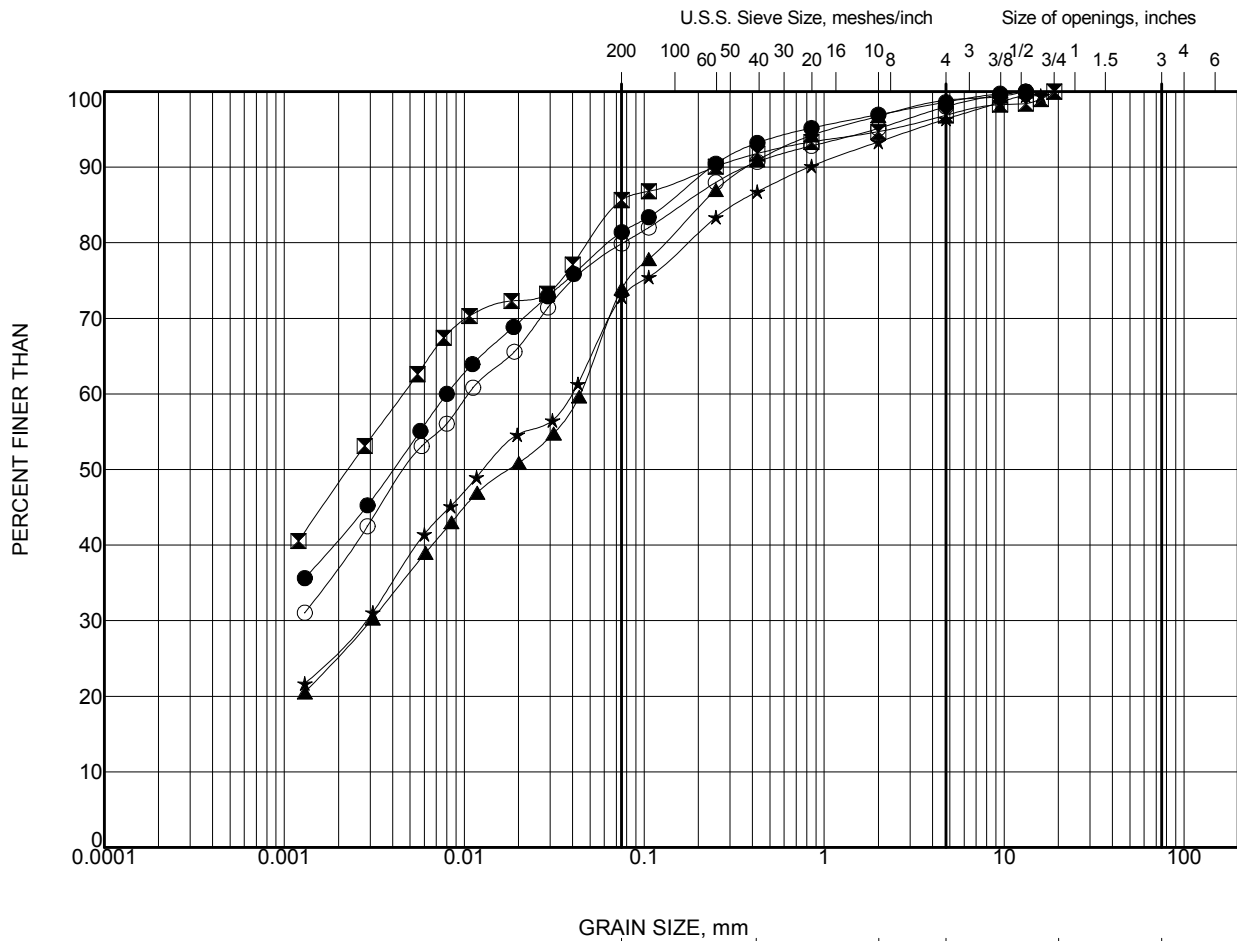


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B4-1	6	4.6
▣	B4-1	8	6.1
▲	B4-1	13	13.7
★	B4-1	15	16.8
○	B4-1	19	22.9

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		GRAIN SIZE DISTRIBUTION Clayey Silt to Silty Clay	
	PROJECT No. SW8801.1004.101	FILE No.	
	DRAWN SS	SCALE	REV.
	CHECK SF	FIGURE C3	



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

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	BH06-RW	12	12.2
▣	BH06-RW	14	15.2
▲	BH06-RW	15	16.8
★	BH06-RW	16	18.3
○	BH06-RW	21	25.9

PROJECT		Windsor Essex Parkway (WEP) Windsor, Ontario	
TITLE		GRAIN SIZE DISTRIBUTION Clayey Silt to Silty Clay	
	PROJECT No.	SW8801.1004.101	FILE No.
	DRAWN	SS	SCALE
	CHECK	SF	REV.
FIGURE C4			

Appendix D Analytical Laboratory Test Results

Project: Windsor-Essex Parkway
Document: Geotechnical Investigation and Design Report - Bridge B-5 (Malden Rd.
Overpass – Realigned E.C. Row EBL, Sta. 11+704 to 11+736, Windsor)
Doc No.: 285380-04-119-0115 (Geocres No. 40J6-48)

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



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

Date Received: 29-JUL-11
Report Date: 05-AUG-11 07:47 (MT)
Version: FINAL REV. 2

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1037972
Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 112832

Comments:

05-AUG-11: Redox Potential result changed due to lab error (incorrect extract used). GAB/LO

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

		Sample ID Description Sampled Date Sampled Time Client ID	L1037972-1 SOIL 28-JUL-11 B4- 1,SA2A@5',BROW N SAND, WET				
Grouping	Analyte						
SOIL							
Physical Tests	% Moisture (%)	18.1					
	pH (pH units)	7.59					
	Redox Potential (mV)	161					
	Resistivity (ohm cm)	2390					
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20					
Anions and Nutrients	Sulphate (mg/kg)	227					

Reference Information

Test Method References:

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

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

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

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

Chain of Custody Numbers:

112832

GLOSSARY OF REPORT TERMS

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

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

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

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

mg/L - milligrams per litre.

< - Less than.

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

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

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

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

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

Quality Control Report

Workorder: L1037972

Report Date: 05-AUG-11

Page 1 of 3

Client: AMEC EARTH & ENVIRONMENTAL

11865 County Road 42

TECUMSEH ON N8N 2M1

Contact: SHANE MACLEOD

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT								
	Soil							
Batch	R2227442							
WG1321774-2	LCS							
% Moisture			104		%		70-130	29-JUL-11
WG1321774-1	MB							
% Moisture			<0.10		%		0.1	29-JUL-11
PH-WT								
	Soil							
Batch	R2228344							
WG1323322-1	CVS							
pH			100		%		80-120	03-AUG-11
RESISTIVITY-WT								
	Soil							
Batch	R2229143							
WG1323310-1	CVS							
Resistivity			101		%		70-130	04-AUG-11
SO4-WT								
	Soil							
Batch	R2229091							
WG1323641-4	DUP	L1037972-1						
Sulphate		227	225		mg/kg	1.1	30	03-AUG-11
WG1323641-3	LCS							
Sulphate			103		%		60-140	03-AUG-11
WG1323641-1	MB							
Sulphate			<20		mg/kg		20	03-AUG-11
SULPHIDE-WT								
	Soil							
Batch	R2228470							
WG1323787-1	CVS							
Sulphide			96		%		50-120	03-AUG-11
WG1323781-1	MB							
Sulphide			<0.20		mg/kg		0.2	03-AUG-11

Quality Control Report

Workorder: L1037972

Report Date: 05-AUG-11

Page 2 of 3

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Workorder: L1037972

Report Date: 05-AUG-11

Page 3 of 3

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential	1	28-JUL-11	04-AUG-11 07:34	24	164	hours	EHTL

Legend & Qualifier Definitions:

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

Notes*:

Where actual sampling date is not provided to ALS, the date (& time) of receipt is used for calculation purposes.
Where actual sampling time is not provided to ALS, the earlier of 12 noon on the sampling date or the time (& date) of receipt is used for calculation purposes. Samples for L1037972 were received on 29-JUL-11 10:11.

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

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

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

Appendix E Selected Photographs

Photograph E-1: Borehole B4-1 – Rock Core Elevation 150.5 to 149.0 m



Photograph E-2: Existing Site – Malden Road looking North (towards E. C. Row)



Photograph E-3: Existing Site – Malden Road looking South



Appendix F Slope Stability Analyses Results

Figure F-1: Slope Stability Result – Typical Abutment – Short-term Loading (Undrained properties)

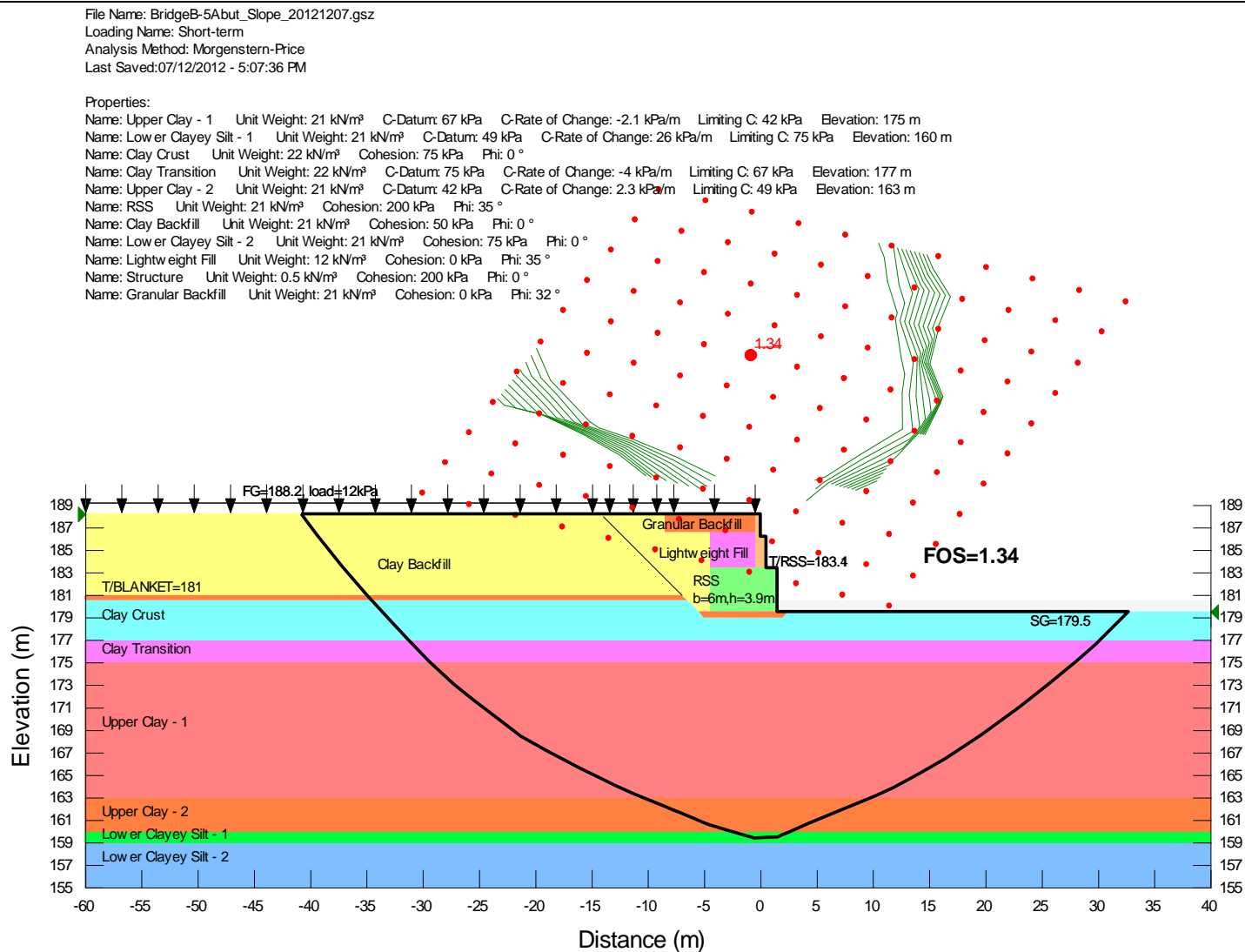


Figure F-2: Slope Stability Result – Typical Abutment – End of Construction Loading (Undrained properties)

File Name: BridgeB-5Abut_Slope_20121207.gsz
Loading Name: End of Construction
Analysis Method: Morgenstern-Price
Last Saved: 07/12/2012 - 5:07:36 PM

Properties:

Name: Upper Clay - 1 Unit Weight: 21 kN/m³ C-Datum: 67 kPa C-Rate of Change: -2.1 kPa/m Limiting C: 42 kPa Elevation: 175 m
Name: Lower Clayey Silt - 1 Unit Weight: 21 kN/m³ C-Datum: 49 kPa C-Rate of Change: 26 kPa/m Limiting C: 75 kPa Elevation: 160 m
Name: Clay Crust Unit Weight: 22 kN/m³ Cohesion: 75 kPa Phi: 0°
Name: Clay Transition Unit Weight: 22 kN/m³ C-Datum: 75 kPa C-Rate of Change: -4 kPa/m Limiting C: 67 kPa Elevation: 177 m
Name: Upper Clay - 2 Unit Weight: 21 kN/m³ C-Datum: 42 kPa C-Rate of Change: 2.3 kPa/m Limiting C: 49 kPa Elevation: 163 m
Name: RSS Unit Weight: 21 kN/m³ Cohesion: 200 kPa Phi: 35°
Name: Clay Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 0°
Name: Lower Clayey Silt - 2 Unit Weight: 21 kN/m³ Cohesion: 75 kPa Phi: 0°
Name: Lightweight Fill Unit Weight: 12 kN/m³ Cohesion: 0 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°
Name: Granular Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32°

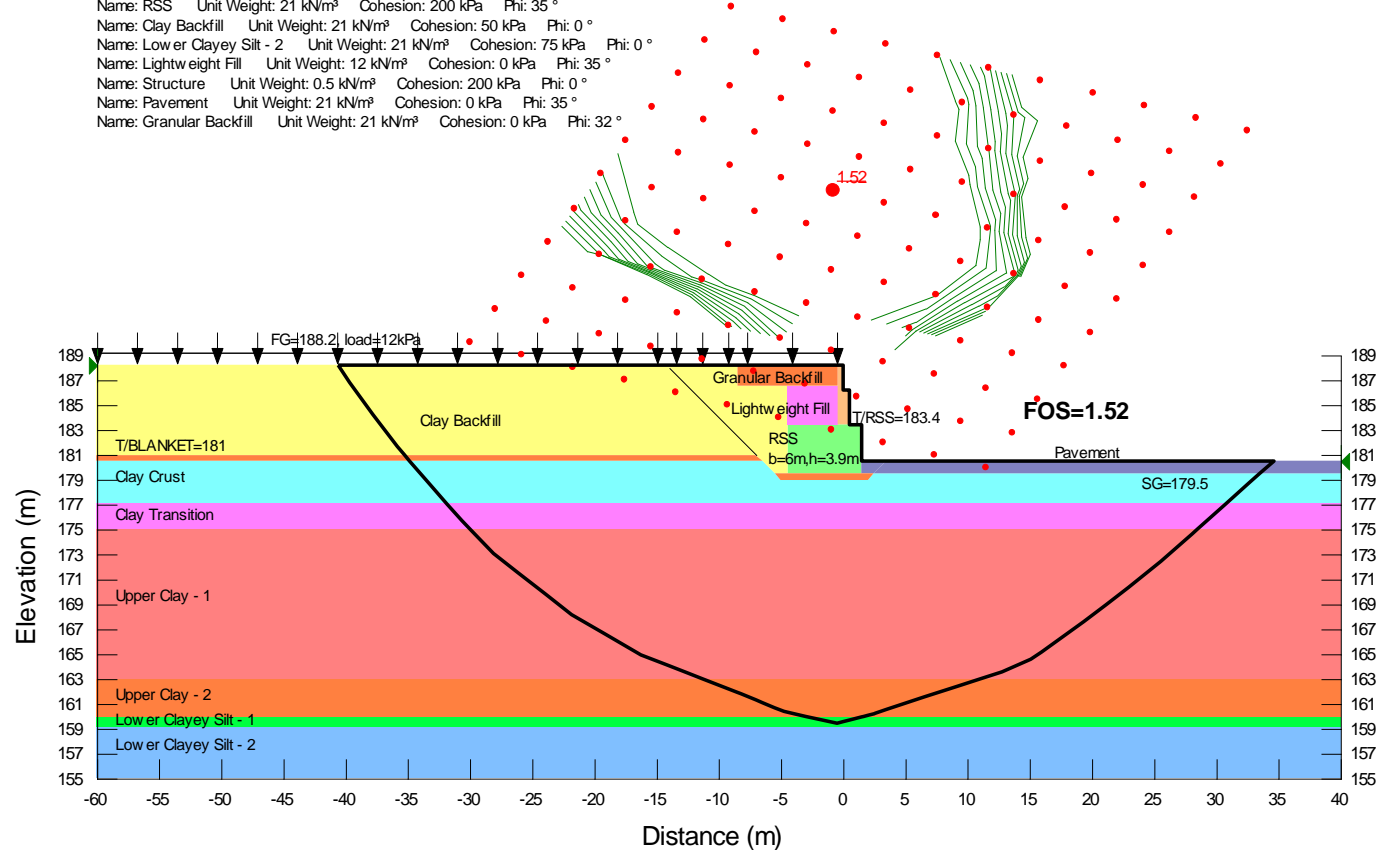
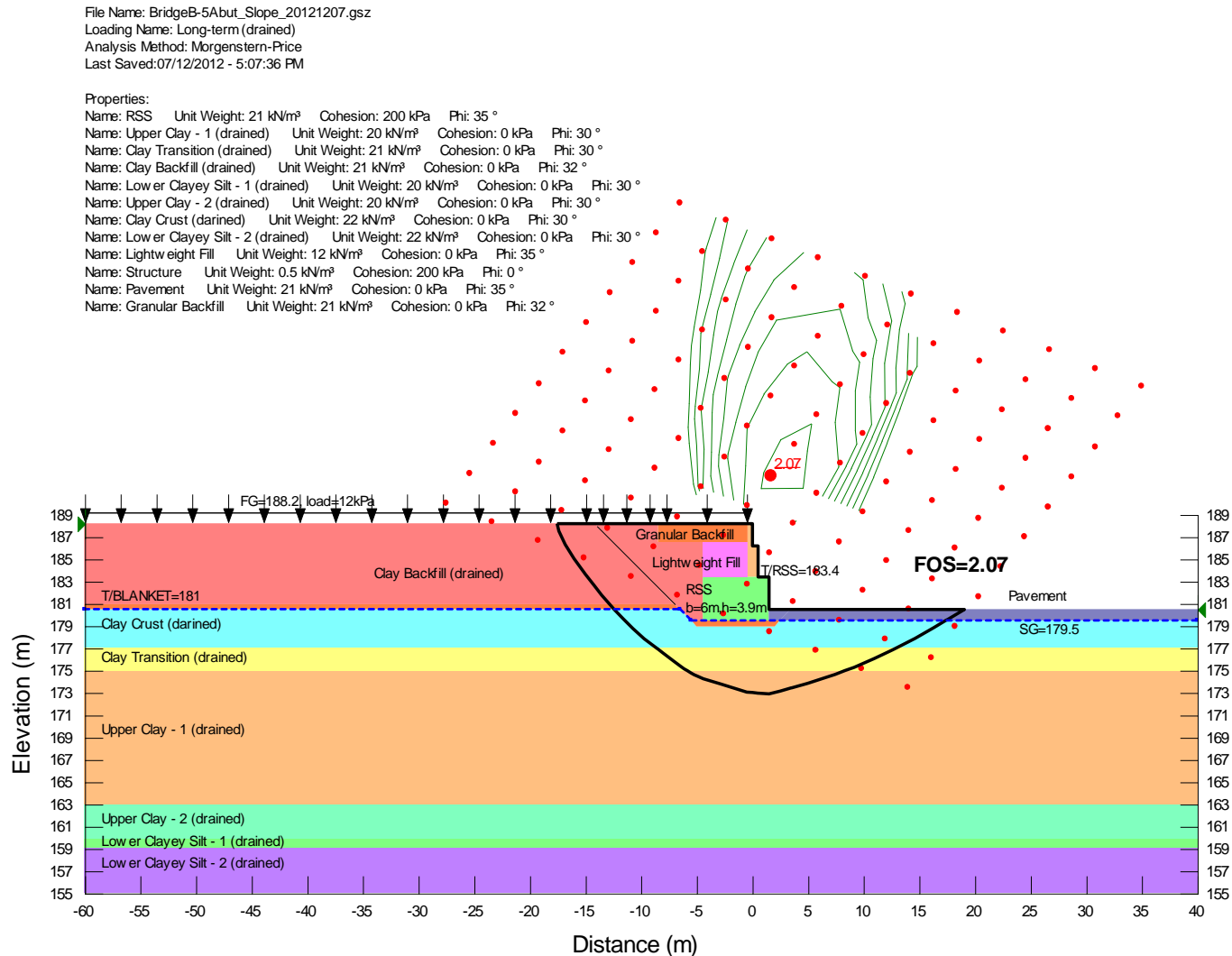


Figure F-3: Slope Stability Result – Typical Abutment – Long-term Loading (Drained properties)



Appendix G Stress-Deformation Analysis Results

Figure G-1: Cumulative Settlement Contours (m) – End of Preloading

Analysis Name: PVD-3rd Stage
File Name: B5-Abut_sigma_20121205.gsz
Date: 06/12/2012

Name: Clay Crust Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.0897 Kappa: 0.0099 Initial Void Ratio: 0.63 Unit Weight: 21 kN/m³ Phi': 25 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clayey Silt - 1 O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clayey Silt - 2 Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 21 kN/m³
Name: General Backfill Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Wick Drain Effective Young's Modulus (E'): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³
Name: Granular Fill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 21 kN/m³

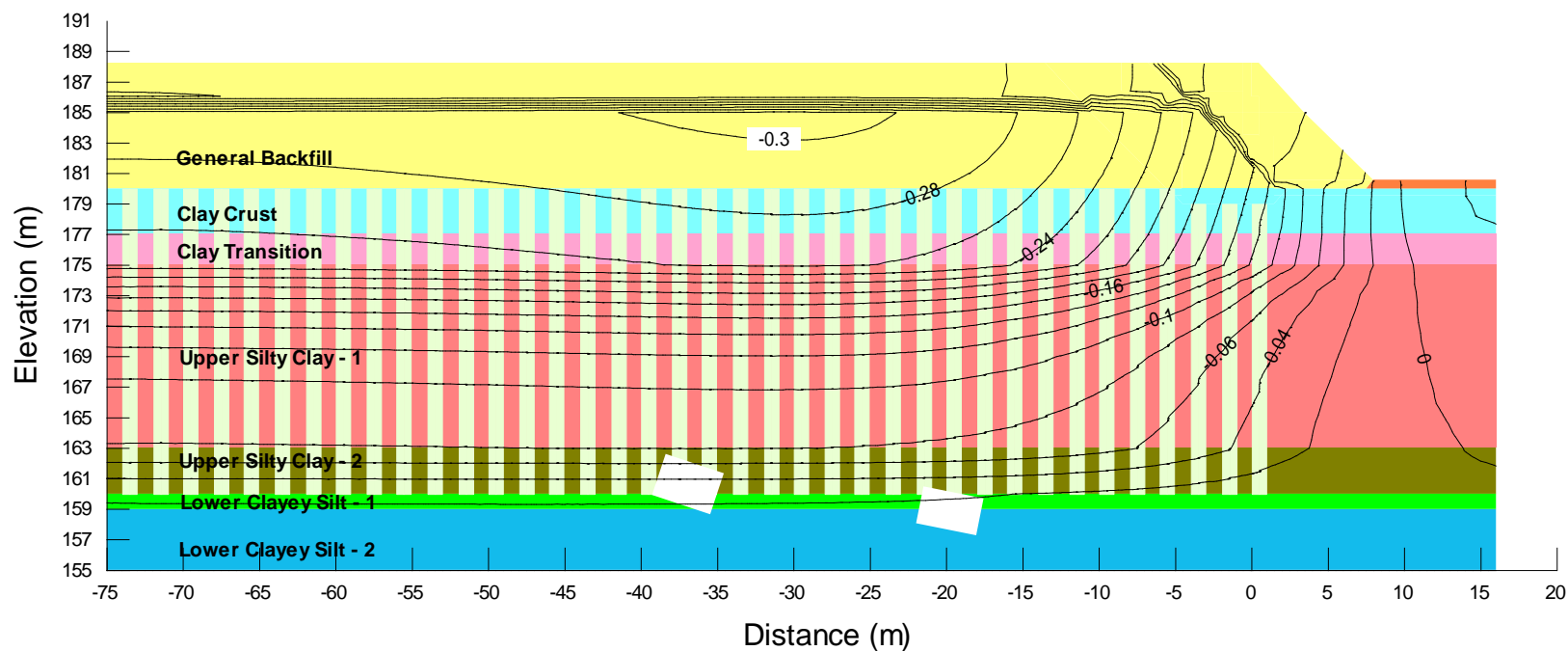


Figure G-2: Cumulative Settlement Contours (m) – Excavation Stage

Analysis Name: Temp Excavation
File Name: B5-Abut_sigma_20121205.gsz
Date: 06/12/2012

Name: Clay Crust Effective Young's Modulus (E): 27000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E): 21000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.0897 Kappa: 0.0099 Initial Void Ratio: 0.63 Unit Weight: 21 kN/m³ Φ' : 25 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Φ' : 26 °
Name: Lower Clayey Silt - 1 O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Φ' : 26 °
Name: Lower Clayey Silt - 2 Effective Young's Modulus (E): 21000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 21 kN/m³
Name: General Backfill Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Wick Drain Effective Young's Modulus (E): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³

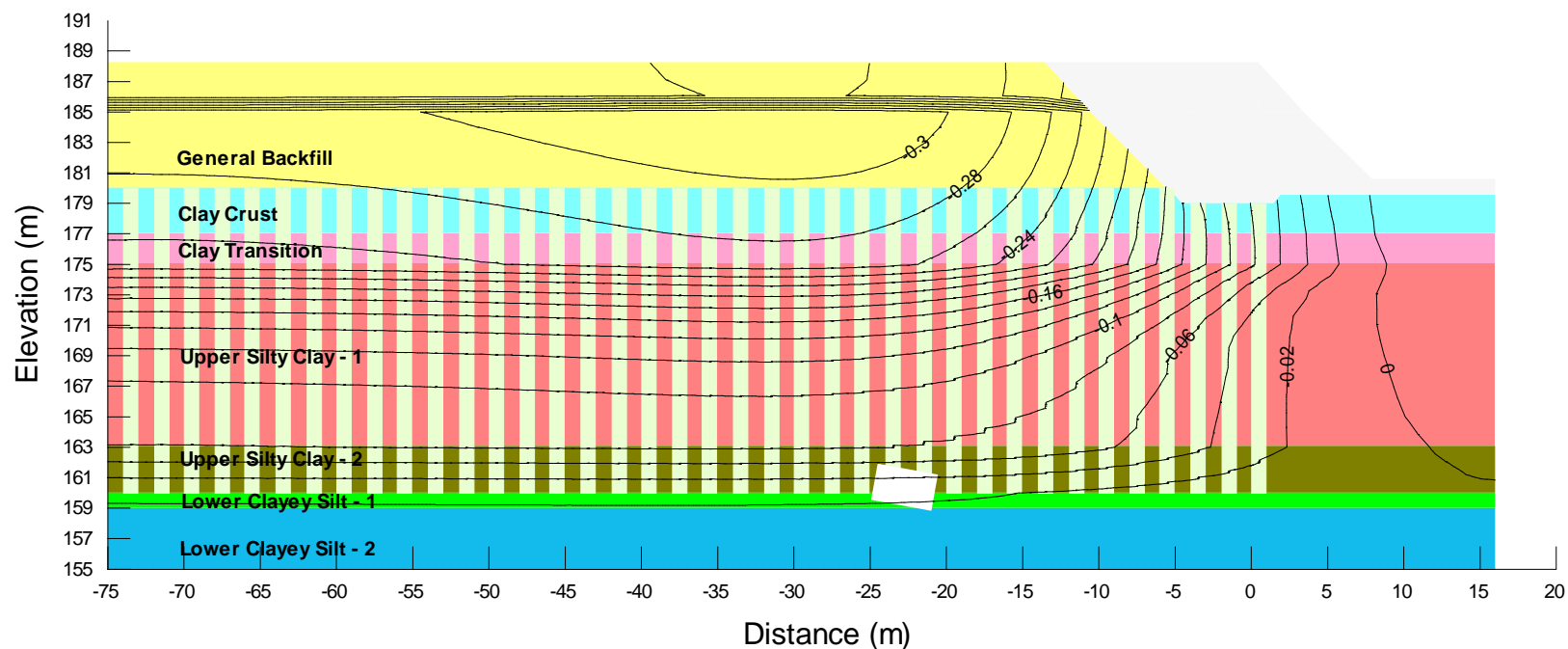


Figure G-3: Cumulative Settlement Contours (m) – End of Abutment Construction

Analysis Name: Abutment

File Name: B5-Abut_sigma_20121205.gsz

Date: 06/12/2012

Name: Clay Crust Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Clay Transition Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Upper Silty Clay - 1 O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.0897 Kappa: 0.0099 Initial Void Ratio: 0.63 Unit Weight: 21 kN/m³ Phi': 25 °
 Name: Upper Silty Clay - 2 O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
 Name: Lower Clayey Silt - 1 O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
 Name: Lower Clayey Silt - 2 Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 21 kN/m³
 Name: General Backfill Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
 Name: Concrete Abutment Young's Modulus (E): 25000000 kPa Unit Weight: 24 kN/m³ Poisson's Ratio: 0.2
 Name: Wick Drain Effective Young's Modulus (E'): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³
 Name: Granular Fill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 21 kN/m³
 Name: RSS Young's Modulus (E): 50000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 21 kN/m³
 Name: LWF Young's Modulus (E): 40000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 12 kN/m³

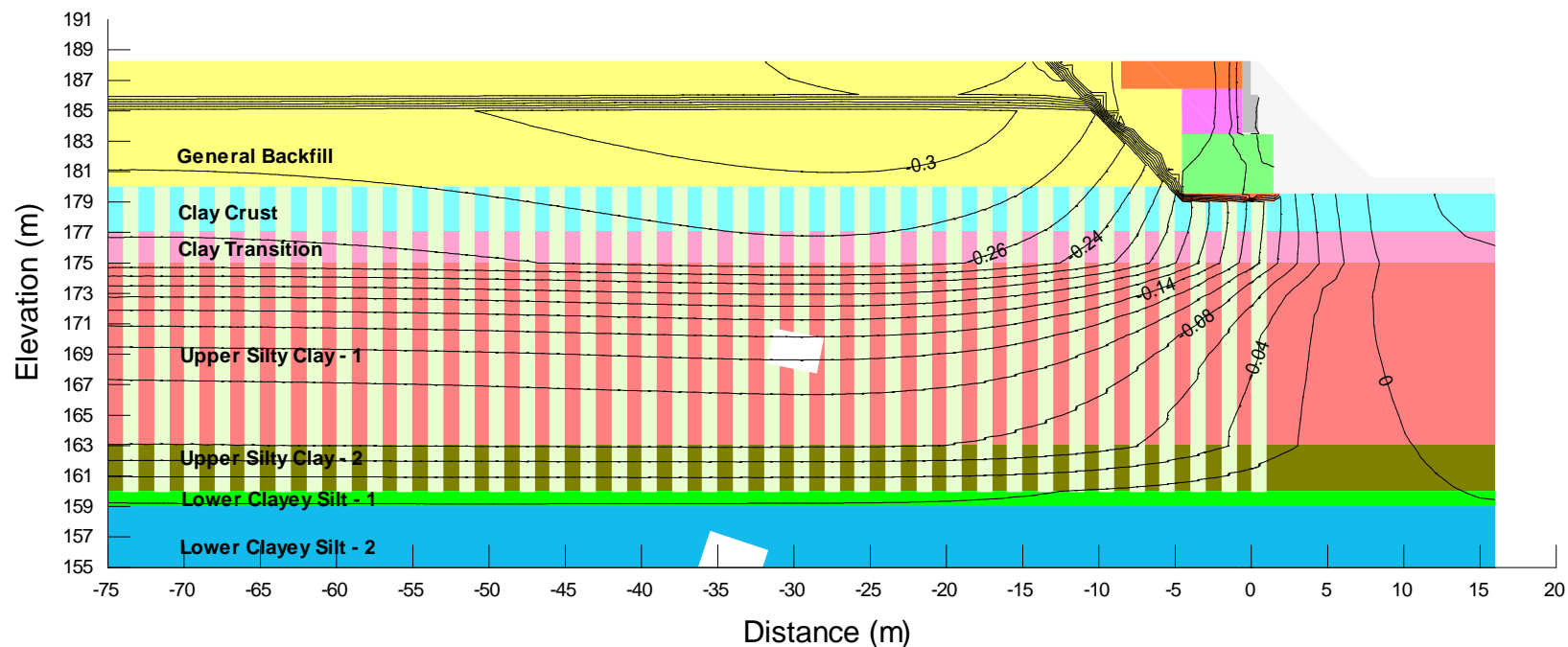


Figure G-4: Cumulative Settlement Contours (m) – Long-term

Analysis Name: Dissipation
File Name: B5-Abut_sigma_20121205.gsz
Date: 06/12/2012

Name: Clay Crust Effective Young's Modulus (E): 27000 kPa Poisson's Ratio: 0.35 Phi: 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E): 21000 kPa Poisson's Ratio: 0.35 Phi: 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.0897 Kappa: 0.0099 Initial Void Ratio: 0.63 Unit Weight: 21 kN/m³ Phi: 25 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi: 26 °
Name: Lower Clayey Silt - 1 O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi: 26 °
Name: Lower Clayey Silt - 2 Effective Young's Modulus (E): 21000 kPa Poisson's Ratio: 0.35 Phi: 30 ° Unit Weight: 21 kN/m³
Name: General Backfill Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Concrete Abutment Young's Modulus (E): 25000000 kPa Unit Weight: 24 kN/m³ Poisson's Ratio: 0.2
Name: Wick Drain Effective Young's Modulus (E): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³
Name: Granular Fill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 21 kN/m³
Name: RSS Young's Modulus (E): 50000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 21 kN/m³
Name: LWF Young's Modulus (E): 40000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 12 kN/m³

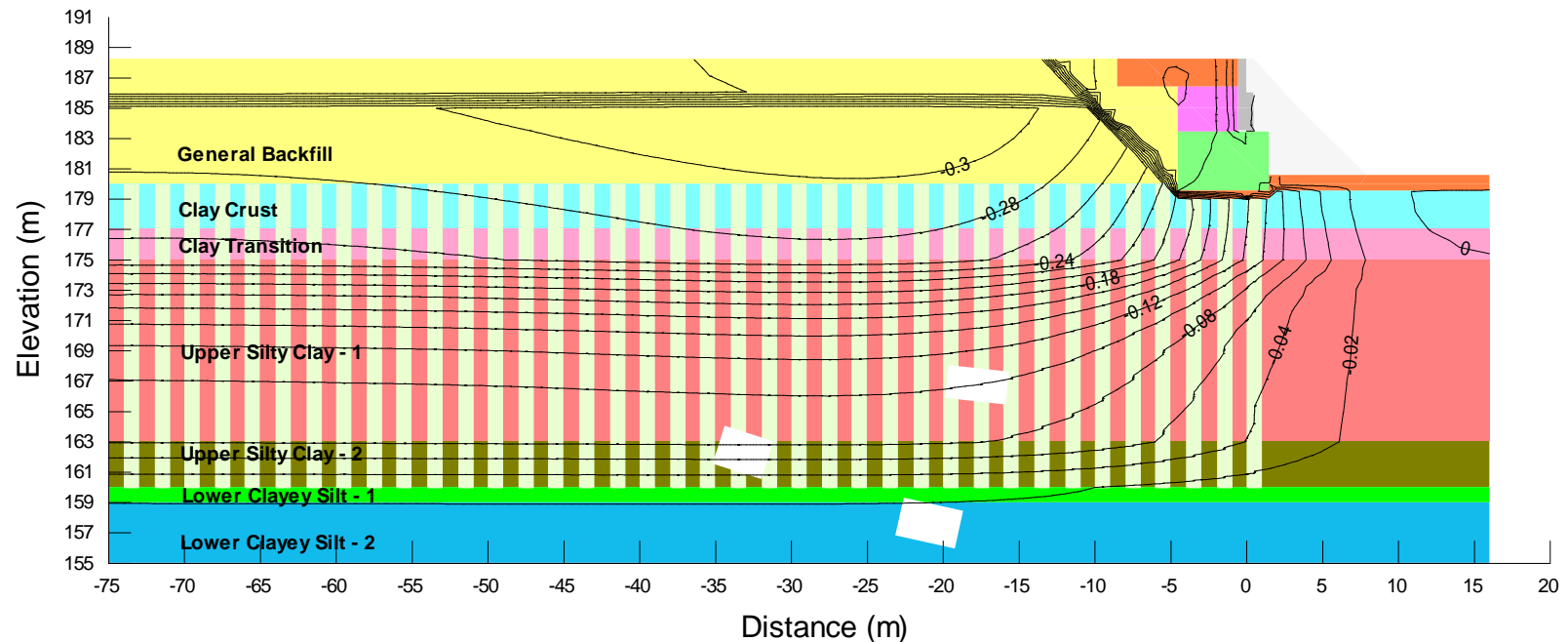


Figure G-5: Cumulative Lateral Deformation Contours (m) – End of Preloading

Analysis Name: PVD-3rd Stage
File Name: B5-Abut_sigma_20121205.gsz
Date: 06/12/2012

Name: Clay Crust Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.0897 Kappa: 0.0099 Initial Void Ratio: 0.63 Unit Weight: 21 kN/m³ Phi': 25 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clayey Silt - 1 O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clayey Silt - 2 Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 21 kN/m³
Name: General Backfill Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Wick Drain Effective Young's Modulus (E'): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³
Name: Granular Fill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 21 kN/m³

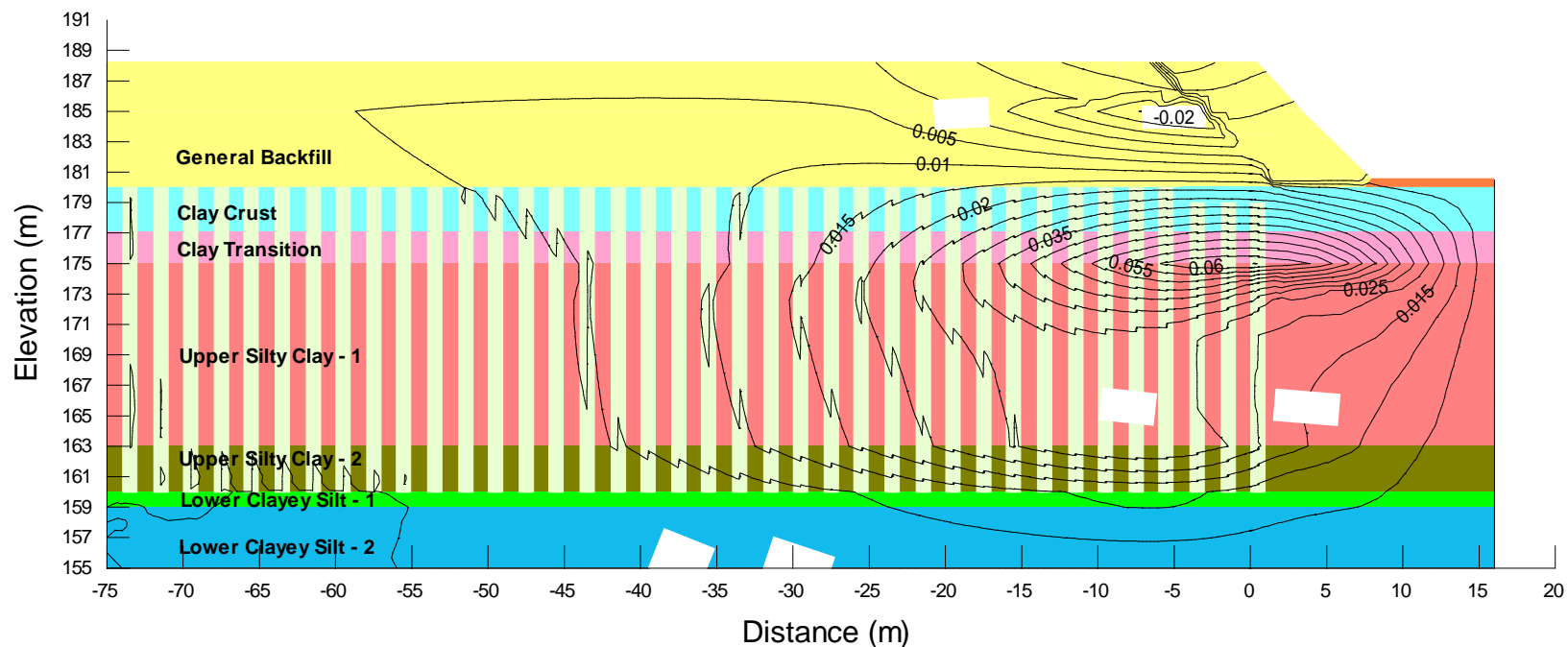


Figure G-6: Cumulative Lateral Deformation Contours (m) - Excavation Stage

Analysis Name: Temp Excavation
File Name: B5-Abut_sigma_20121205.gsz
Date: 06/12/2012

Name: Clay Crust Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.0897 Kappa: 0.0099 Initial Void Ratio: 0.63 Unit Weight: 21 kN/m³ Φ' : 25 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Φ' : 26 °
Name: Lower Clayey Silt - 1 O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Φ' : 26 °
Name: Lower Clayey Silt - 2 Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Φ' : 30 ° Unit Weight: 21 kN/m³
Name: General Backfill Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Wick Drain Effective Young's Modulus (E'): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³

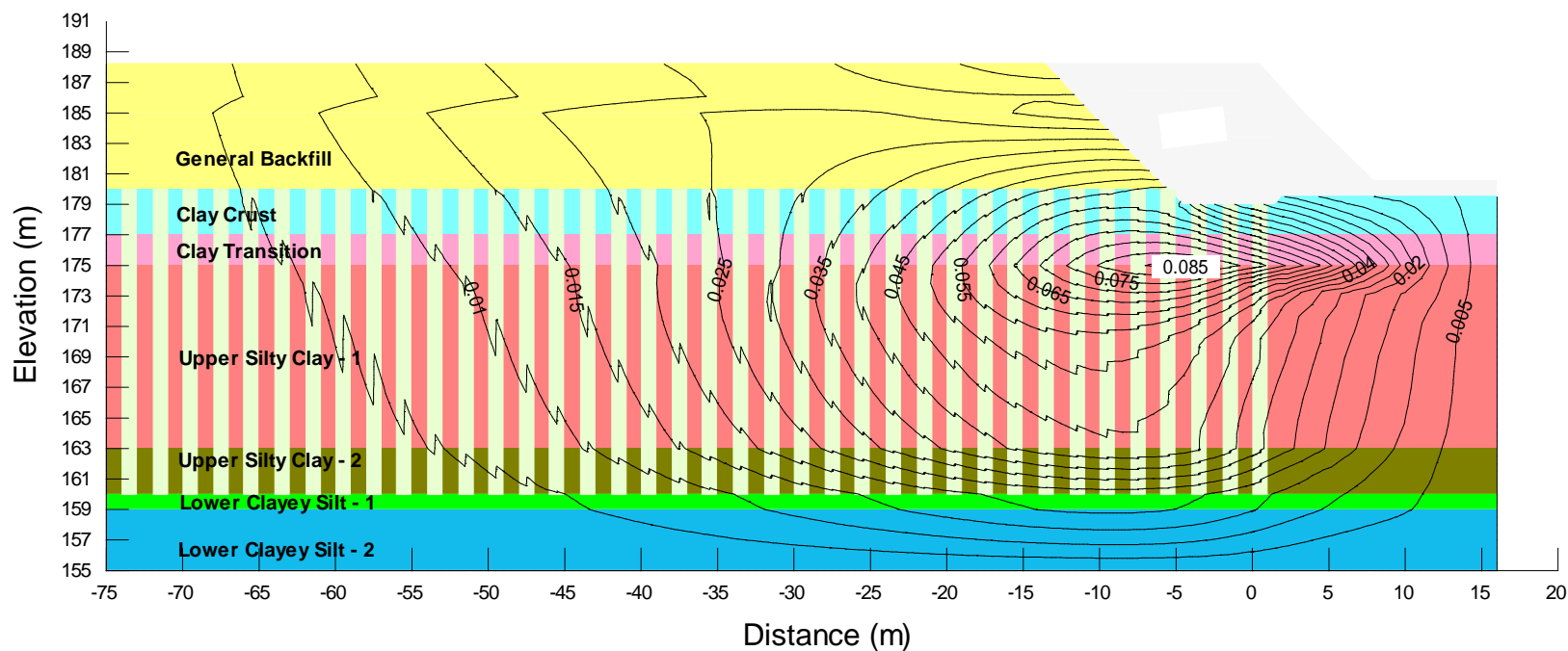


Figure G-7: Cumulative Lateral Deformation Contours (m) – End of Abutment Construction

Analysis Name: Abutment

File Name: B5-Abut_sigma_20121205.gsz

Date: 06/12/2012

Name: Clay Crust Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Clay Transition Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
 Name: Upper Silty Clay - 1 O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.0897 Kappa: 0.0099 Initial Void Ratio: 0.63 Unit Weight: 21 kN/m³ Phi': 25 °
 Name: Upper Silty Clay - 2 O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
 Name: Lower Clayey Silt - 1 O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
 Name: Lower Clayey Silt - 2 Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 21 kN/m³
 Name: General Backfill Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
 Name: Concrete Abutment Young's Modulus (E): 25000000 kPa Unit Weight: 24 kN/m³ Poisson's Ratio: 0.2
 Name: Wick Drain Effective Young's Modulus (E'): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³
 Name: Granular Fill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 21 kN/m³
 Name: RSS Young's Modulus (E): 50000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 21 kN/m³
 Name: LWF Young's Modulus (E): 40000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 12 kN/m³

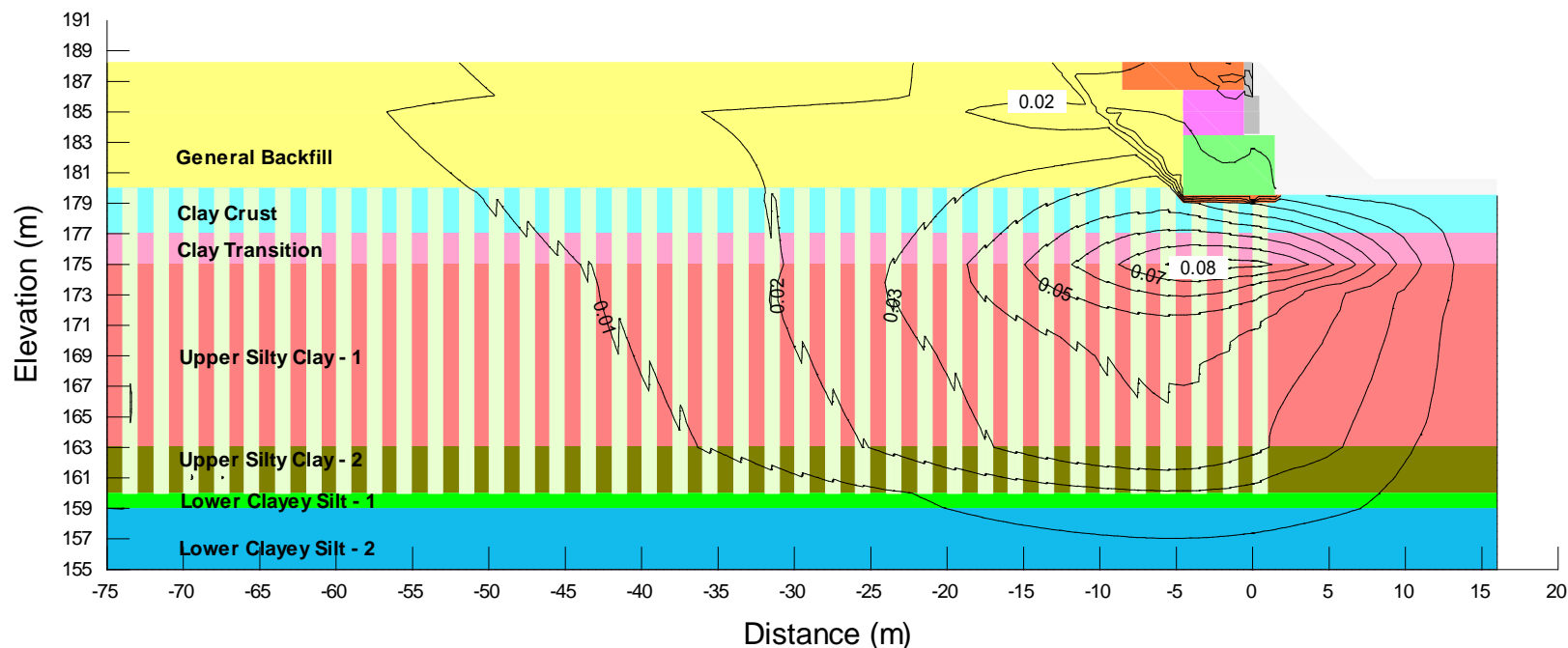


Figure G-8: Cumulative Lateral Deformation Contours (m) - Long-term

Analysis Name: Dissipation
File Name: B5-Abut_sigma_20121205.gsz
Date: 06/12/2012

Name: Clay Crust Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.0897 Kappa: 0.0099 Initial Void Ratio: 0.63 Unit Weight: 21 kN/m³ Phi': 25 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clayey Silt - 1 O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clayey Silt - 2 Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 21 kN/m³
Name: General Backfill Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Concrete Abutment Young's Modulus (E): 25000000 kPa Unit Weight: 24 kN/m³ Poisson's Ratio: 0.2
Name: Wick Drain Effective Young's Modulus (E'): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³
Name: Granular Fill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 21 kN/m³
Name: RSS Young's Modulus (E): 50000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 21 kN/m³
Name: LWF Young's Modulus (E): 40000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 12 kN/m³

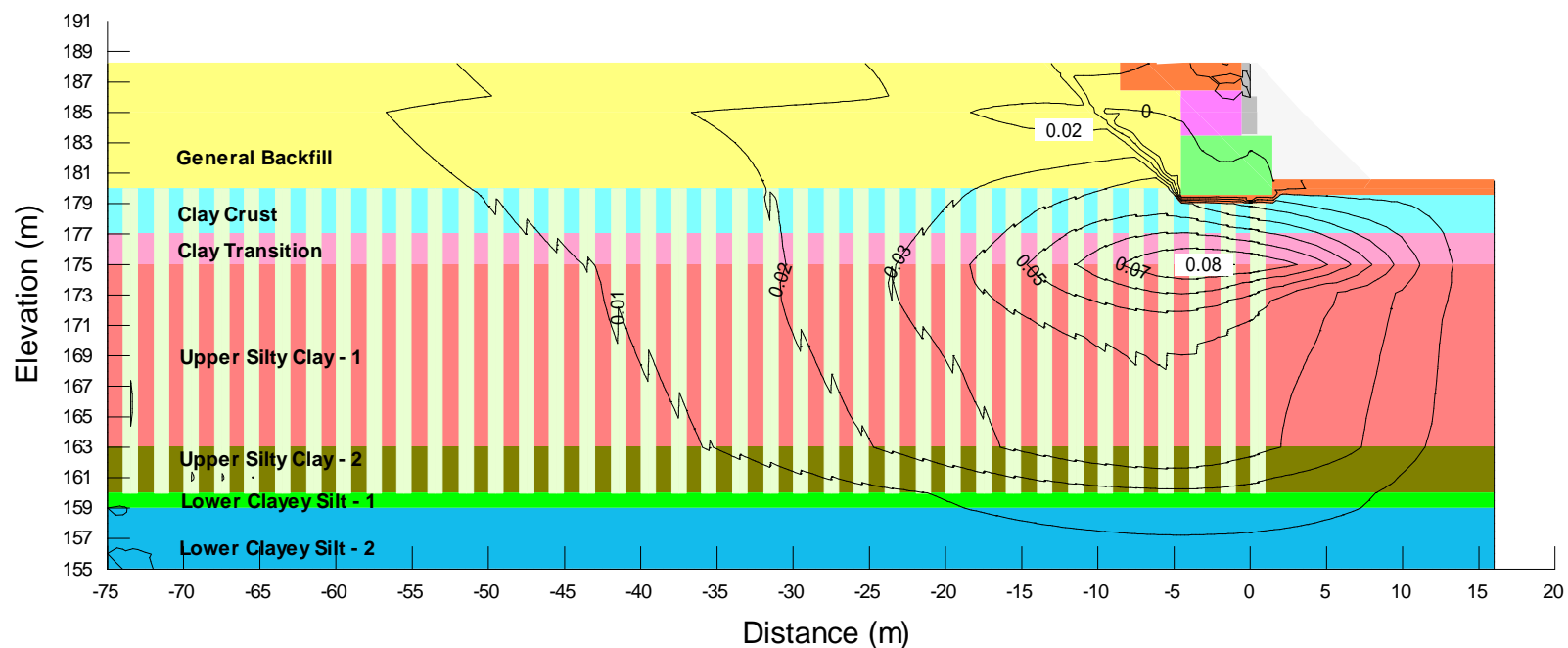


Figure G-9: Pore Water Pressure Contours (kPa) – Long-term

Analysis Name: Dissipation
File Name: B5-Abut_sigma_20121205.gsz
Date: 06/12/2012

Name: Clay Crust Effective Young's Modulus (E'): 27000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.0897 Kappa: 0.0099 Initial Void Ratio: 0.63 Unit Weight: 21 kN/m³ Phi': 25 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.1 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clayey Silt - 1 O.C. Ratio: 1.5 Poisson's Ratio: 0.35 Lambda: 0.0633 Kappa: 0.007 Initial Void Ratio: 0.5 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Lower Clayey Silt - 2 Effective Young's Modulus (E'): 21000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 21 kN/m³
Name: General Backfill Young's Modulus (E): 22500 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Concrete Abutment Young's Modulus (E): 25000000 kPa Unit Weight: 24 kN/m³ Poisson's Ratio: 0.2
Name: Wick Drain Effective Young's Modulus (E'): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³
Name: Granular Fill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 32 ° Unit Weight: 21 kN/m³
Name: RSS Young's Modulus (E): 50000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 21 kN/m³
Name: LWF Young's Modulus (E): 40000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 35 ° Unit Weight: 12 kN/m³

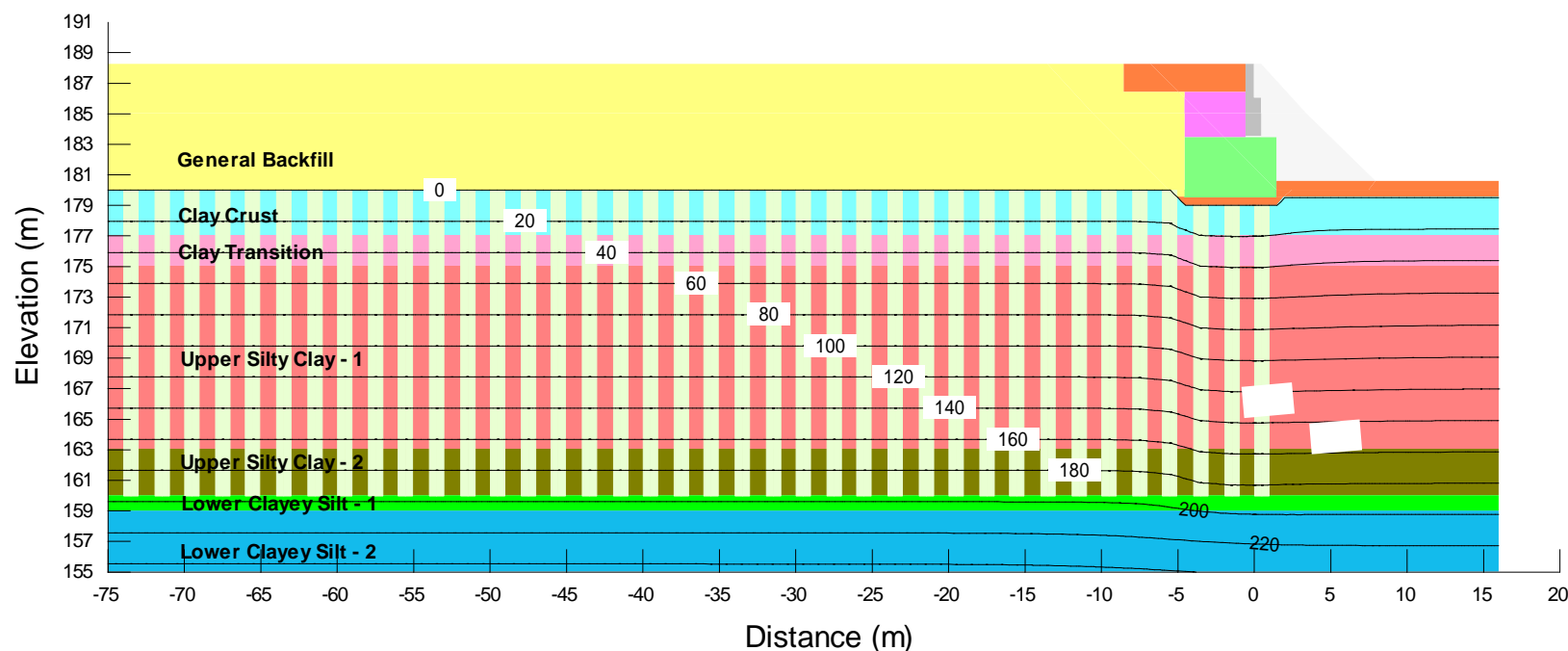
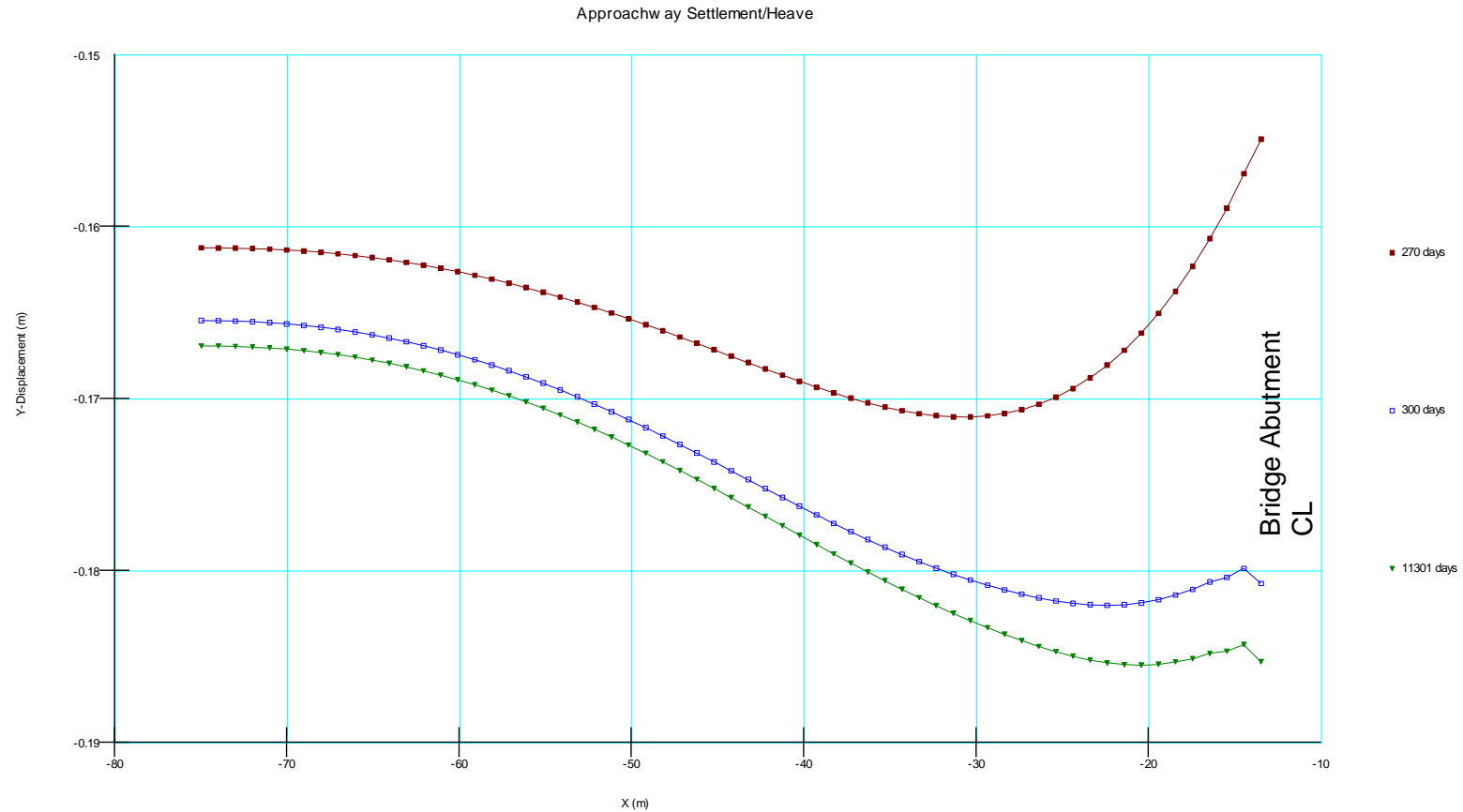
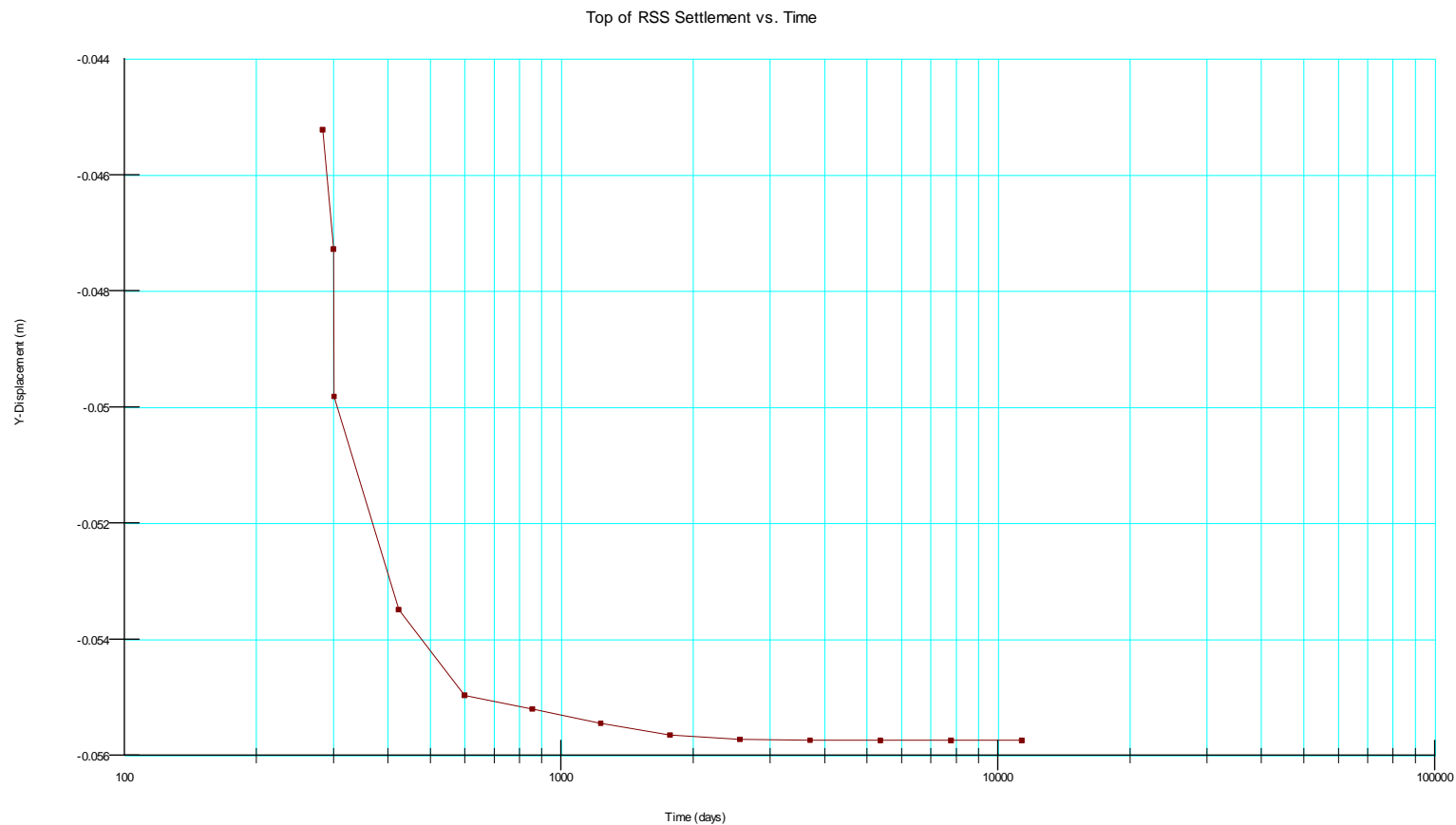


Figure G-10: Cumulative Ground Settlement at Bridge Approach way



Legend:
 270 day – End of Embankment Construction
 300 day – End of Abutment Construction
 11,301 day – Long-term Condition
 (-) Displacement = Settlement

Figure G-11: Settlement at Top of RSS Facing



Legend:

270 day – End of Embankment Construction

300 day – End of Abutment Construction

11,301 day – Long-term Condition

(-) Displacement = Settlement

Project: Windsor-Essex Parkway

Document: Geotechnical Investigation and Design Report - Bridge B-5
(Malden Rd. Overpass – Realigned E.C. Row EBL, Sta. 11+704 to 11+736, Windsor)

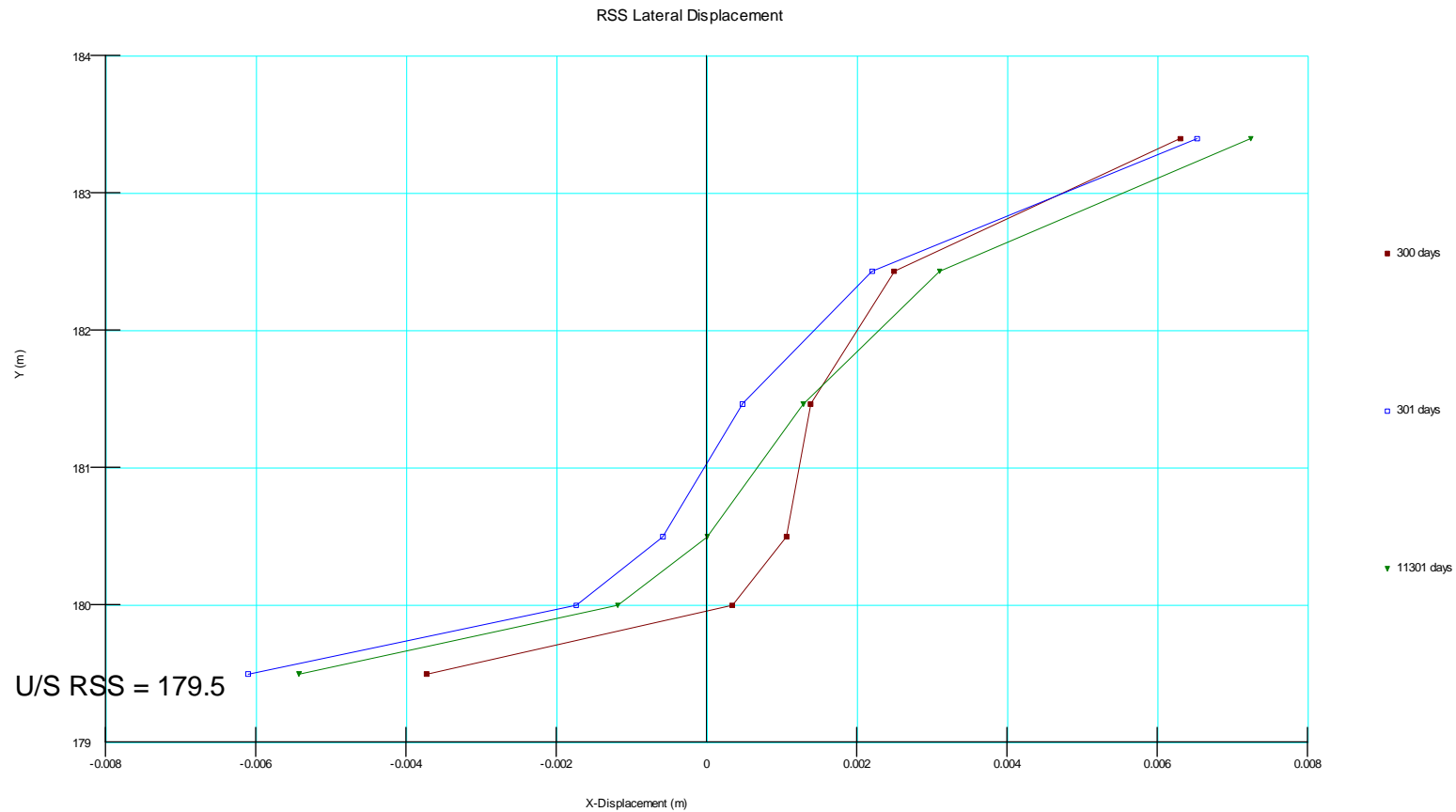
Doc No.: 285380-04-119-0115 (Geocres No. 40J6-48)

Date: December / 2012

Rev: 0

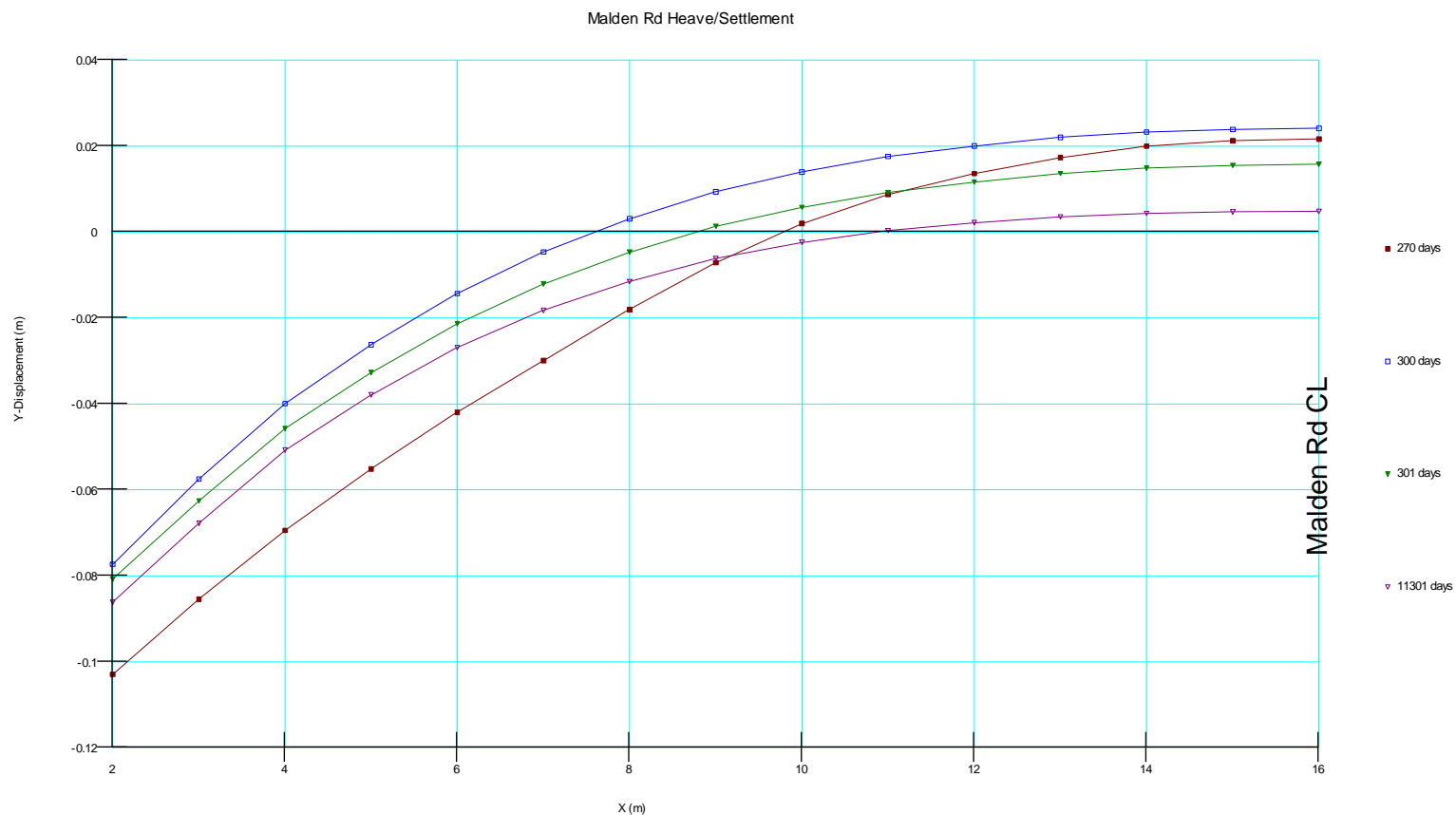
Page No.: Appendix G- Page 11 of 16

Figure G-12: Lateral Displacement of RSS Facing (Cumulative)



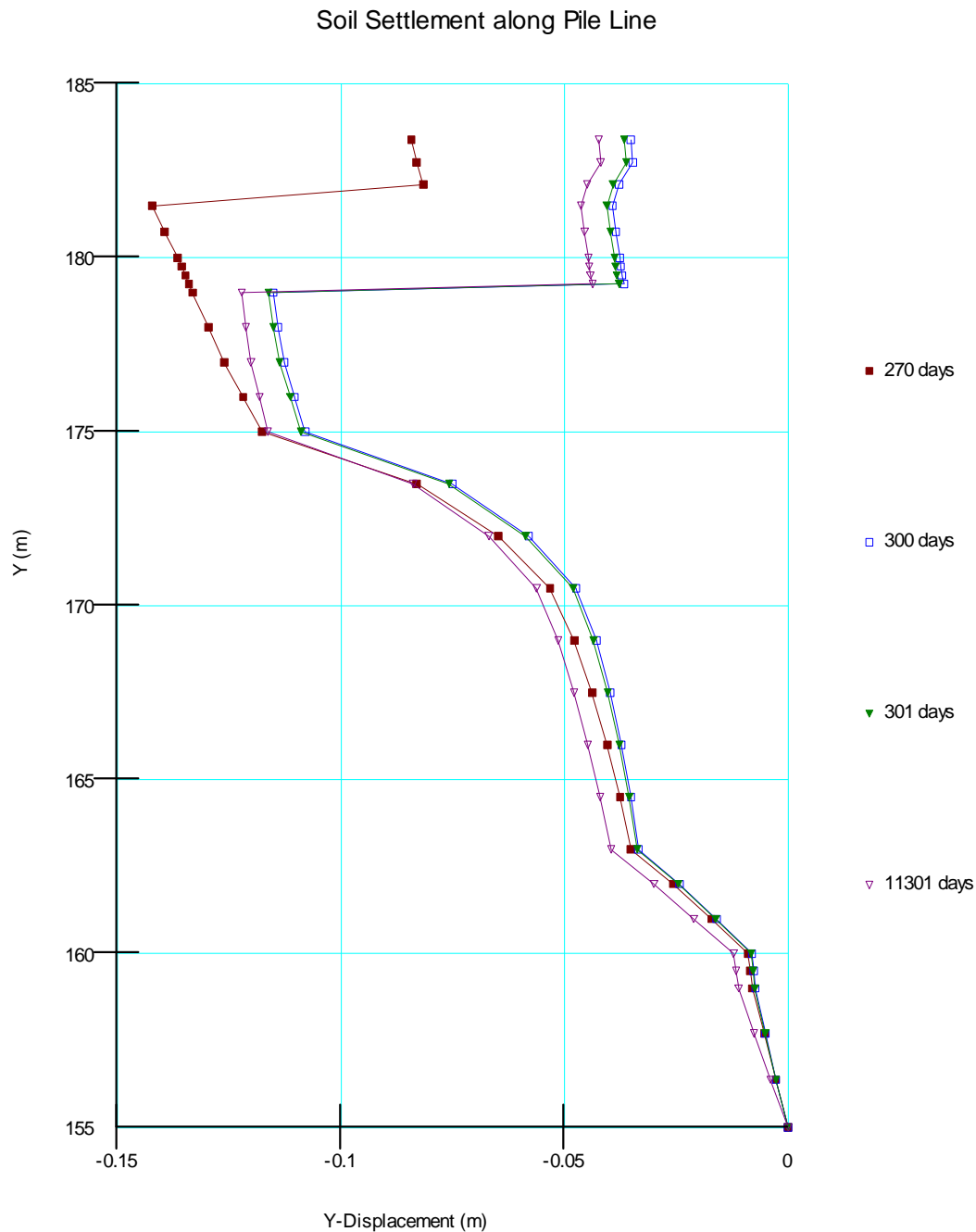
Legend:
 300 day – End of Abutment Construction
 301 day – End of Construction
 11,301 day – Long-term Condition

Figure G-13: Malden Road Heave / Settlement Profile (Cumulative)



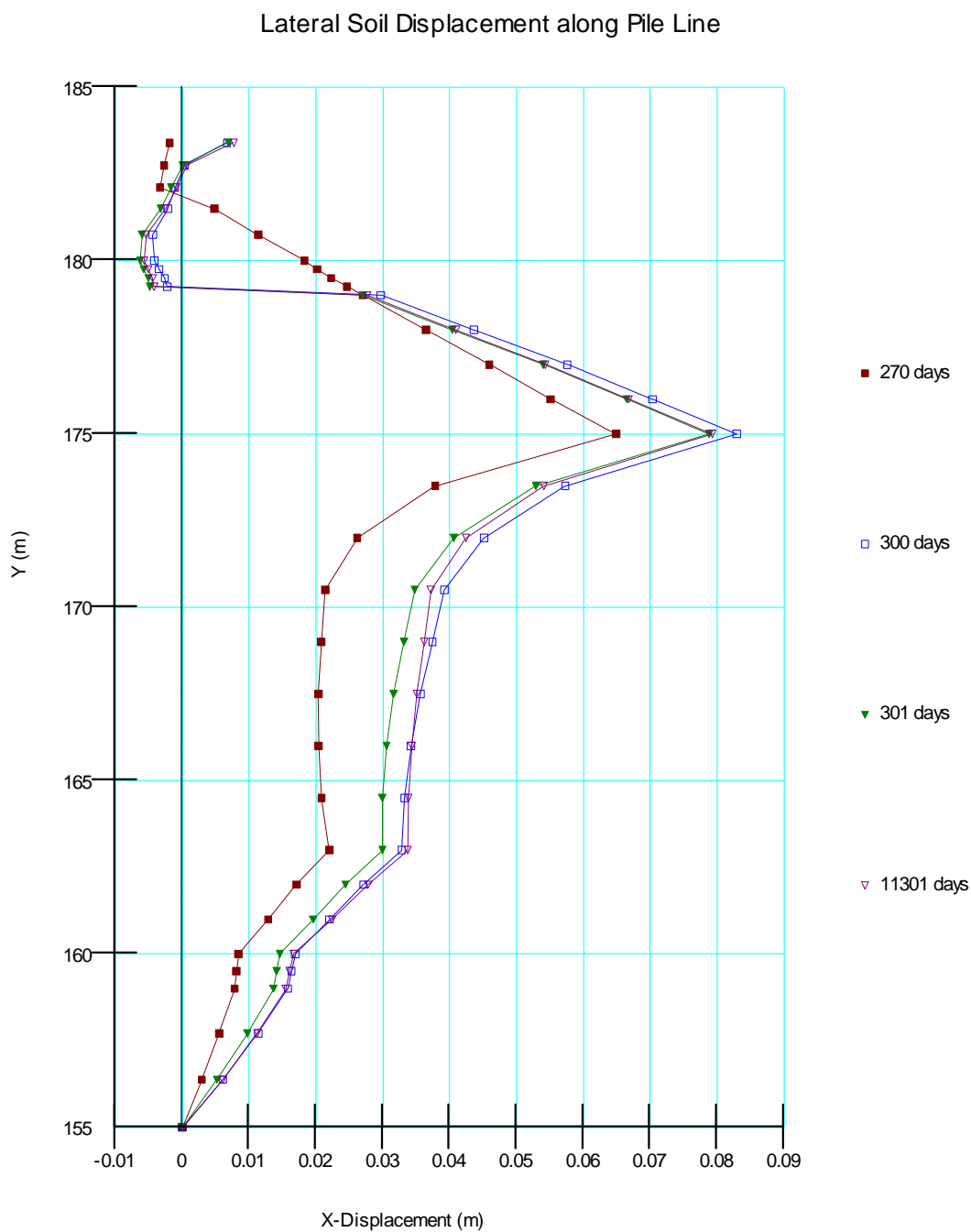
Legend:
 270 day – End of Embankment Construction
 300 day – End of Abutment Construction
 301 day – End of Construction
 11,301 day – Long-term Condition
 (-) Displacement = Settlement

Figure G-14: Cumulative Soil Settlement Profile along Pile Line



Legend:
 270 day – End of Embankment Construction
 300 day – End of Abutment Construction
 301 day – End of Construction
 11,301 day – Long-term Condition

Figure G-15: Cumulative Soil Lateral Displacement Profile along Pile Line



Legend:
270 day – End of Embankment Construction
300 day – End of Abutment Construction
301 day – End of Construction
11,301 day – Long-term Condition

Figure G-16: Soil Vertical Effective Stress Profile along Pile Line

