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The Windsor-Essex Parkway Project

Geotechnical Investigation and Design Report (90%) Bridge B-3

E.C. Row Expressway

(Sta. 10+930.439E to 11+110.939E)

Revision History					
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1 Introduction

1.1 Preface

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

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

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

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

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

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

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

1.2 Report Introduction

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

This report presents the 90% geotechnical design of Bridge B-3 (Realigned E.C. Row EBL Expressway Underpass near Matchette Road) located between Stations 10+930.439E and 11+110.939E (E.C. Row Expressway stations) in the Windsor sector of the Windsor-Essex Parkway (WEP) project. The report includes the results of the additional geotechnical investigation carried out to support the design (available at the time of preparation of this report) and other relevant background information. This report is issued for review and discussion only. The final report will include design changes due to revision to the project layout and configuration, all relevant geotechnical investigation information and will address the review comments.

The proposed 180.5 m long, 4 span Bridge B-3 structure will carry the realigned east bound lanes (EBL) of E.C. Row Expressway over Highway 401. The proposed structural solution incorporates concrete box structures supported on true concrete abutments and piers on piles. Four retaining walls (MSEW-04R, MSEW-08L, MSEW-07L and HRW-03R) are indicated in the immediate vicinity of the Bridge B-3 abutments.

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-43)[1] which was recognized as 30% design. The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines as well as the Parkway Infrastructure Constructors (PIC).

[1] References are listed in Section 9.

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

The design of Bridge B-3 complies with the requirements of the execution version of the Project Agreement (PA) Schedule 15-2 Part 2, Article 5.

2 Background Information

2.1 Geological Setting

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

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

At the WEP project area, the glacial till like deposit is typically 20 to 35 m thick and consists primarily of silty clay and clayey silt gradation with a random distribution of coarser particles. Random and apparently discontinuous seams / lenses of silt, sand and or gravel are present at various depths within the mass of the silty clay deposit. A firm to hard surficial crust layer has formed due to desiccation. Up to 2 m thick surficial layers of lacustrine silty clay or silt and sand are also encountered in the western sector of the project. A 1 m to 6 m thick very dense or hard basal glacial till or dense silty sand may be found directly overlying the bedrock surface. The bedrock at the project area comprises the Devonian Dundee Formation of the Hamilton group of formation and the 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-26). The bedrock geology within the Essex Domain was formed as part of the midcontinent rift south-eastern extension. The midcontinent rift south-eastern extension is composed of Paleozoic cover rocks which form the bedrock foundation of the Essex Domain. The bedrock was deposited in the Paleozoic Era during the Middle Devonian period. Within the Essex Domain the following strata were deposited the Hamilton Group, Dundee Formation, and Detroit River Group Onondaga Formation all consisting of Limestone, Dolostone, and Shale.

2.2 Site Seismic Background

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

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

2.3 Existing Site Conditions and Proposed Bridge Layout

Bridge B-3 site is situated in the western part of the Windsor segment of the Parkway. The topography of the lands along the Bridge B-3 is generally varied with elevation ranging from approximately 179[2] to 180. Adjacent land use is typically both residential and commercial.

The bridge structure will be constructed under WEP Phase III development and will be used to carry realigned E.C. Row Expressway EBL traffic over Highway 401 and to connect with Matchette Road on the west side of the proposed Bridge B-3. Highway 401 at this location will be constructed on earth fill low embankment. A headwall-like concrete wing wall flared to the bridge abutment is indicated at each corner of the structure as shown on Drawing 285380-03-060-WIP3-0301.

2.4 Frost Depth

In accordance with MTO–SDO-90-01 Pavement Design and Rehabilitation Manual (ref. R-38) and OPSD 3090.101, the frost depth below the ground surface in Windsor area is estimated to 1.0 m[3]. 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 rip rap, or otherwise coarse rockfill cover, the insulation effects of such materials are considered to be one half of the insulation offered by soil deposits /cover, and the depth of frost penetration will have to be increased accordingly.

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

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

3 Geotechnical Investigations

3.1 Scope and Procedures of Geotechnical Investigations

Geotechnical investigations involving a number of boreholes, cone penetration tests (CPT) and Nilcon vane tests had been carried out in 2007-09 by Golder Associates (ref. R-16 to R-23) to develop the conceptual design and serve as background information for development of the WEP proposal designs. Additional geotechnical investigation was carried out in 2011 to supplement the available subsurface soil data, as required to support the detailed design development of the WEP embankment and structures. The additional investigation program at and around the proposed location of Bridge B-3 comprised a total of 9 boreholes, 2 Nilcon vane tests, 3 CPTs and 2 DMTs (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-3 Site

Reference	Boreholes	Nilcon Vane Tests	CPTs	DMTs
This Investigation (2011)	B3-1 + (CPT B3-1 & DMT B3-1)	NIL B3-1	CPT B3-1	DMT B3-1
	B3-2 + (CPT B3-2 & DMT B3-2)		CPT B3-2	DMT B3-2
	B3-3 + (NIL B3-3)	NIL B3-3		
	CPT 10-RW		CPT 10-RW	
Previous Studies (2007-09)	BH 341			
	CPT 159		CPT 159	
	CPT 339		CPT 339	
			CPT 339A	
	CPT 340		CPT 340	

Note: Test holes given in parentheses are shallow holes drilled to facilitate execution of Nilcon vane, CPTs and DMTs.

Drawing 285380-04-090-WIP3-0301 shows the locations of the test holes and an interpreted soil stratigraphic profile along the WEP centreline for the general area from Sta. 10+900W to Sta. 11+500W. The test hole locations and stratigraphic sections at the bridge location and immediate vicinity are illustrated on Drawings 285380-04-090-WIP3-0302 and 285380-04-091-WIP3-0303.

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

Soil sampling was generally carried out using a 50 mm diameter split spoon sampler. Thin-walled Shelby tube (70 mm diameter x 600 mm long) samples were also recovered in the cohesive soil deposits below the upper crust layer. Soil sampling was carried out generally at 0.75 m depth interval in the top 7 to 8 m and at 1.5 m depth intervals thereafter. All samples were identified by a field technologist and placed in airtight containers and transported to AMEC's Tecumseh (Windsor) laboratories for further examination and testing[4]. Rock coring of the bedrock was carried out using 1.5 m long NQ (OD=75.7 mm) or HQ (OD=96.0 mm) sized core barrels.

Standard Penetration Tests (SPT, ASTM D1586[5]) were carried out in conjunction with split spoon sampling. Field vane tests (using conventional vanes) were carried out in between sampling at selected depths. The Nilcon vane tests listed in Table 3-1 were carried typically adjacent the boreholes. Table 3-2 summarizes the depths of overburden penetration and rock coring as well as the list of instruments and the accompanying Nilcon vane tests.

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 testes were carried out on rock core samples selected from across the WEP length.

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

Nilcon vane blade was pushed into the ground from the bottom of shallow pre-augered holes through surficial soils using the hydraulic ram of the drill rig. The Nilcon vane tests were conducted in accordance with ASTM D2573-01. The CPT cone was pushed at a constant rate into the ground using hydraulic ram system of the drill rig (ASTM D5778). Pore pressure dissipation tests were carried out at CPT B3-2 and CPT 10-RW at 9.5 and 5.0 m, respectively, below ground surface. Similarly, the DMT probe was pushed in the ground in increments of 200 mm using the hydraulic ram of the drill rig. The tests were conducted following the provisions of ASTM D 6635. The Nilcon vane, CPT and DMT tests were carried out from the bottom of shallow auger holes drilled to remove hard surficial materials.

The locations of the test holes and inferred soil profile at and around Bridge B-3 are shown on Drawing 285380-04-090-WIP3-0302. 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.

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

[5] American Society for Testing and Materials

Table 3-2: Overburden Thickness and Instrumentation in Boreholes

Borehole	Location	Overburden Thickness, m	Test Name & Elevation					
			Rock Coring	Nilcon Vane	S-Piez.	VWP	MHSG	IN
B3-1 (2011)	N4682267.8, E329431.6	21.3	157.6 to 154.7	175.4 to 159.9		175.8, 167.9, 157.9	176.6, 170.4	
B3-2 (2011)	N4682224.9, E329491.0	22.0	156.0 to 153.6					
B3-3 (2011)	N4682180.9, E329559.0	25.3	153.7 to 152.2			176.0, 166.9, 156.6	176.3, 169.7	152.2
NIL B3-3	N4682184, E329556	3.5 (BTWO)		175.2 to 160.0				
CPT B3-1 (2011)	N4682270.6, E329419.6	2.0 (BTWO)						
CPT B3-2 (2011)	N4682176.2, E329573.0	2.0 (BTWO)						
CPT 10-RW (2011)	N4682295.7, E329387.8	2.0 (BTWO)						
DMT B3-1 (2011)	N4682286.4, E329420.5	2.0 (BTWO)						
DMT B3-2 (2011)	N4682177.6, E329571.6	2.0 (BTWO)						
BH 341 (Pre-bid)	N4682256, E329379	21.7	157.1 to 151.6			151.6		
CPT 159 (Pre-bid)	N4682293, E329332	1.8 (BTWO)						
CPT 339 (Pre-bid)	N4682147, E329759	3.7 (BTWO)						
CPT 340 (Pre-bid)	N4682203, E329539	2.9 (BTWO)						

Legend:
S-Piez. Standpipe Piezometer (Screen elevations)
VWP Vibrating Wire Piezometer (Sensor elevations)
MSG Spider Magnet Heave/Settlement Gauge
IN Inclinometer Casing
BTWO Borehole Terminated within the Overburden
Note: Location coordinates and elevations are in UTM-NAD 83 (Zone 17) and geodetic datum

3.3 Instrumentation

Geotechnical instruments (standpipe piezometers, vibrating wire piezometers – VWP, spider magnets heave/settlement gauges – MHSG and inclinometer casings – INC) were installed at selected locations on completion of boreholes to monitor pore water pressure and deformation behaviour of the soil strata during and after construction. A brief description follows.

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

Vibrating Wire Piezometers: The VWP transducers (RST Model VW2100, 0.35 MPa for shallow to mid-depth and 0.7 MPa for deep installations) were installed at selected depths and their electrical wires extended to the monitoring station at the ground surface. The installation of the piezometers was according with the manufacturer specifications. The instrumented 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: Spider magnets (RST, Model SSMM100 mechanical release spider target for 25 mm pipe) were installed in boreholes at select locations and depths to permit future measurement of heave and settlement. Each magnetic torus was placed around a 25 mm diameter pipe, which was extended to above the ground surface. The spider legs grip into the surrounding soil, which enables the magnetic torus to move up or down on the pipe as the soil settles or heaves. The locations of the magnetic torus are determined by lowering a magnetic probe inside the pipe.

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

Inclinometers: Inclinometer casing was installed in Borehole B3-3. The purpose of this device is to measure the lateral ground movement at the installed location. The bottom end of the casing was anchored approximately 1.5 m into bedrock, and the annular space around the casing was filled with bentonite-cement grout. The inclinometer comprised 70 mm diameter RST “Snap Seal Inclinometer Casing”, and probe is IC32005 MEMS digital inclinometer system (0.5 m long).

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

3.4 Geotechnical and Analytical Laboratory Testing

All recovered soil samples and rock cores were examined in the field and the laboratory. Natural moisture content tests were carried out on most of the recovered samples; grain size distribution and Atterberg limit tests were carried out on selected representative samples.

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

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

3.5 Data Interpretation

Field Vane Test Data Correction: The chart (Figure 3.1[6]) developed initially by Bjerrum (1972) and updated subsequently by Ladd et al (1977) based on circular arc failure analyses of embankment failures suggest correction by multiplying the field vane data by 1.05 to 1.10 for soils with plasticity index of

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

about 15 (ref. R-5 and R-31). However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), the Canadian Foundations Manual suggests that the vane test data for clays with PI<20 should not be corrected (ref. R-1 and R-8, and Figure 3.2). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI.

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

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

Where:

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

Q_t is the corrected total cone tip resistance;

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

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

The CPT based Su profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles. In this regard, the N_{kt} factor values used to calibrate the CPT strength profiles varied slightly for different segments of the WEP and the soil strata. Thus, an N_{kt} factor of 14 was used to estimate the undrained shear strength of the clay crust and transition layers. The N_{kt} factors used for the underlying grey silty clay to clayey silt stratum and the lower clayey silt stratum were 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 Su profiles follows the Stress History and Normalized Soil Engineering Properties (SHANSEP) method developed at MIT (Ladd and Foott, 1974, ref. R-31). The following relationship was used to compute the pre-consolidation pressures:

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

Where:

S_u is the undrained shear strength;

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

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

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

OCR is the overconsolidation ratio; and

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

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

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

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

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

Where:

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

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

The 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 Sta. 10+850W and 11+550W are presented in Figure 3.3. Also included on these figures are $0.18 \times \sigma_{vo}'$ curve (representing undrained strength profile for OCR=1 condition) and simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units. The constant 0.18 for S_u / σ'_{vo} for OCR=1 curve is based on average plasticity index of the silty clay to clayey silt stratum and Chandler 1988 relationship (ref. R-11).

4 Subsurface Conditions

The general soil stratigraphy at the borehole locations in the area of Bridge B-3 consists of the following successive strata: topsoil, surficial layers of occasional fills and upper granular deposit, an extensive cohesive silty clay to clayey silt deposit below about elevation 177.3 to 178.7, lower granular deposit below about elevation 156.1 to 159.1, overlying limestone bedrock below about elevations ranging from 153.7 to 157.6. The thickness of the silty clay to clayey silt stratum varied between 19.6 and 22.1 m.

The bedrock was encountered at depths ranging from about 21.3 m to 25.3 m below the ground surface.

4.1 Topsoil, Surficial Fills and Upper Granular Deposit

Brown to black topsoil was encountered at the ground surface in all boreholes except Boreholes B3-1, DMT B3-1 and CPT 159. In Borehole CPT 10-RW, topsoil was encountered below fill materials. The thickness of the topsoil varied from 0.2 to 0.9 m, but is expected to vary in quality and thickness through the project area.

Boreholes B3-1, DMT B3-1, CPT 159 and CPT 10-RW were advanced through the existing embankment of E.C. Row Expressway and the on-ramp from Matchette Road, and encountered surficial fills consisting of pavement materials, clayey topsoil and gravel to clayey silt. The total thickness of the fills varied from 0.3 to 1.5 m.

Upper granular deposit was encountered at all of the test locations except Boreholes B3-1, B3-2, DMT B3-1 and CPT 159. The upper granular deposit consisted of sandy silt to sand and gravel. The thickness of the deposit varied from 0.3 m to greater than 2.0 m. Sampling was terminated in the upper granular deposit at Borehole CPT B3-1.

4.2 Silty Clay to Clayey Silt Stratum

The cohesive silty clay material was encountered directly underlying the surficial topsoil or fill/granular deposit in all test holes at 0.2 m to greater than 2 m depth below existing ground surface. Based on the gradation, in-situ moisture content and strength characteristics, the stratum may be divided into four layers as follows: mottled brown-grey firm to stiff clay crust, transition zone, upper grey silty clay to clayey silt deposit (referred to hereafter as upper silty clay), and then a generally coarser lower grey clayey silt deposit (referred to hereafter as lower clayey silt). The natural water content, Atterberg limits and bulk unit weights determined on the samples of the clay sub-strata recovered during the pre-bid and additional geotechnical investigation are summarized in Table 4-1. The plasticity charts (Figures C.4 to C.6 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 Clay Stratum

Property	Clay Crust	Clay Transition	Upper Silty Clay	Lower Clayey Silt
Elevation Range (m)	179(1) – 177	177 – 175	175 – 161	161 – 156
Natural Water Content, wN, %	5.0 – 28.8	18.2 – 34.4	10.0 – 43.0	15.7 – 25.8
Liquid Limit, wL	35.0	35.0	24.0 – 49.0	26.0 – 34.0
Plastic Limit, wP	20.0	20.0	14.0 – 21.0	15.0 – 18.0
Plasticity Index, PI	15	15	10.0 – 29.0	11.0 – 18.0
Liquidity Index, LI	(-) 1.0	(-) 1.0	(-) 0.42 – 1.87	0.03 – 0.22
Unit Weight, γ, kN/m3	N/A	N/A	18.6 – 20.8	20.7 – 21.1

(1) - Elevation of clay crust surface varies
Index Properties are based on laboratory results on samples recovered from Boreholes B3-1, B3-2, B3-3, BH 341, NIL B3-3, CPT B3-1, CPT B3-2, DMT B3-1 and DMT B3-2.

The undrained shear strength (S_u) profiles of the stratum between Sta. 10+850W and Sta. 11+550W are illustrated on Figure 3.3. The S_u profiles at the Bridge B-3 site are illustrated on Figures 5.1 and 5.2 for west and east abutments, respectively.

As illustrated on Figures 3.3, 5.1 and 5.2, the undrained shear strength of the clay stratum varied with depth generally as follows:

- Crust layer: >60 kPa
- Transition layer: >60 kPa to 55±20 kPa
- Upper silty clay: 55±20 kPa to 35±10 kPa
- Lower clayey silt: >60±15 kPa.

The stress-strain properties and the effective shear strength properties of the silty clay deposit were based on test results from the pre-bid geotechnical investigations (ref. R-16, R-17, R-18 and R-19) and the one-dimensional consolidation tests, triaxial shear tests and direct shear tests performed during the additional geotechnical investigation described in Section 3.1. These interpreted trends are supported by published correlations in the literature (Kulhawy and Mayne, 1990, ref. R-30, Leroueil at al., 2001, ref. R-34 and Terzaghi et al., ref. R-42).

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

$$C_c = 0.0086w_N – 0.0086$$

$$C_r = 0.11C_c$$

$$C_s = 0.25C_c$$

$$C_\alpha = 0.028C_c$$

The interpreted average values used for the clay substrata for the Bridge B-3 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, wN, %	22	26	27 to 25	20
Virgin Compression Index, Cc	0.181	0.215	0.224 to 0.206	0.163
Recompression Index, Cr	0.0199	0.0237	0.0246 to 0.0227	0.0180
Swelling Index, Cs	0.0452	0.0538	0.0559 to 0.0516	0.0409
Secondary Compression Index, Cα	0.0051	0.0060	0.0063 to 0.0058	0.0046

The effective shear strength properties applicable to the silty clay to clayey silt stratum were determined form triaxial and direct shear tests performed during the pre-bid and additional geotechnical investigations and supported by published PI versus ǿ relationships (ref. R-34 and R-42). These strength parameters are summarized as follows (Figures 4.3 and 4.4):

Apparent cohesion, c' — 0 kPa

Angle of internal friction, ǿ — 30°

Friction angle at critical state, Φ_c — 25° to 26° [7]

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

Elastic Modulus (Undrained), $E_u = 300\ S_u$

Elastic Modulus (Drained), $E' = 0.9E_u$

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

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

Soils Stratigraphy	Elastic Modulus - Undrained, MPa	Poisson's Ratio – Undrained (*)	Elastic Modulus - Drained, MPa	Poisson's Ratio – Drained (*)
Clay Crust	28	0.49	25	0.35
Clay Transition	18		16	
Upper Silty Clay	12 to 9		11 to 8	
Lower Clayey Silt	17		15	

(*) Assumed values (ref. R-42)

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

Of the four boreholes advanced to bedrock, only Borehole BH 341 did not encounter a lower granular deposit. The gradation of the material varied from silt to sand and gravel with layers of clayey silt. Based

[7] Based on triaxial tests (ref. R-18).

on the limited Standard Penetration Test (SPT) "N" values ranging generally from 15 to 52, this material is considered to be in a compact to very dense state of compactness, or stiff to hard state of consistency. This layer, where present, was approximately 2.4 to 3.1 m thick but will vary significantly throughout the project area.

4.4 Bedrock

Where rock coring was undertaken, a grey to brown, limestone bedrock was encountered. The bedrock was coarse to very fine grained, occasionally pitted, faintly to strongly porous and fractured. Bedrock was encountered at elevations ranging from 153.7 to 157.6 in the vicinity of B-3. Photographs of rock cores recovered from the additional investigation are provided in Appendix E.

Rock core sample from Borehole B3-1 was tested and had an unconfined compressive strength of 35.5 MPa. The result of the compressive strength testing indicates that the limestone rock may be described as "medium strong" rock.

Over the entire project area, the Rock Quality Designation (RQD) of the recovered rock varied from 0 to 100 per cent, indicating a very poor to excellent quality. Based on this core logging the rock mass classification was estimated to range from 2.8 to 5 for the Q-System (Barton *et. al.*, 1974, ref. R-3) and 53 to 58 for the Rock Mass Rating (RMR) based on Bieniawski (1976, ref. R-5) and indicates that the rock mass can be considered as a Fair quality rock mass based on the later system. Rock quality generally increases with depth.

It was found during the preliminary investigations (ref. R-19) that little variation in the strength of the rock mass conditions was identified from site to site. For this reason in order to obtain a reasonable statistical sample, the density, unit weight and uniaxial compressive strength of the samples from all of the key sites have been grouped and are summarised in Table 4-4. A total of 12 samples were included for density and unit weight, while 16 were included for unconfined compressive strength. The average strength of the limestone is determined to be 85.5 MPa and is 'strong rock' based on the ISRM (1978). Additionally, based on the coefficient of variation, enough tests have been performed to characterise the compressive strength.

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

	Density (kg/m3)	Unit Weight (kN/m3)	UCS (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 mi = 12 for a crystalline limestone, a disturbance factor of 0.7, and a factor of safety of 3.0, an allowable bearing capacity of the rock has been calculated to range from 5.3 MPa to 13.5 MPa. The mean allowable bearing capacity is determined to be 9.2 MPa using the Hoek and Brown strength criterion for determining the bearing capacity of a fractured rock mass (Wyllie, 1999, ref. R-44).

4.5 Groundwater Conditions

Shallow and deep vibrating wire piezometers (VWPs) were installed in selected boreholes to measure the water levels within overburden and bedrock (Table 3-2). The reported water levels are presented in Table 4-5, below.

The most stabilized readings in the table indicate the following trend: Water level elevations 177.1 and 178.4 in the upper part of the upper silty clay stratum, 177.7 and 178.9 at mid height of the silty clay stratum, and about 180.3 to 180.6 in the lower granular deposit and limestone bedrock. These readings indicate artesian conditions with piezometric head roughly 1.7 m above the ground surface.

During drilling at Borehole B3-3, slightly artesian groundwater flow developed approximately one hour after contact with bedrock. An odour associated with hydrogen sulphide was also noted however no measured concentrations are available.

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 Elevation	Piezometer Type	Screen / Sensor Elevation	Strata Type at Screen / Sensor Depth	Measured Water level	
					Date	Elevation
B3-1	178.9	VWP	175.8	Silty Clay	June 25, 2011 July 11, 2011 July 22, 2011	176.2 176.0 177.1
		VWP	167.9	Silty Clay	June 25, 2011 July 11, 2011 July 22, 2011 Aug. 25, 2011	172.8 177.8 178.4 178.9
		VWP	157.9	Lower Granular	June 25, 2011 July 11, 2011 July 22, 2011 Aug. 25, 2011	180.6 180.6 180.6 180.6
B3-3	179.0	VWP	176.0	Silty Clay	Aug. 22, 2011	178.4
		VWP	166.9	Silty Clay	Aug. 22, 2011	177.7
		VWP	156.6	Lower Granular	Aug. 22, 2011	180.3
BH 341	178.8	VWP	151.6	Limestone	Jan. 6, 2010 Feb. 24, 2010	180.4 180.5

Legend: VWP Vibrating Wire Piezometer

4.6 Subsurface Gases

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

The H_2S gas can frequently be detected by odour at approximate concentrations of 0.5 mg/L and can be corrosive at concentrations of about 2 mg/L to 3 mg/L in the groundwater. The presence of the gas was noted during the current drilling at the Bridge B-3 site. During drilling at Borehole B3-3, slightly artesian groundwater flow developed approximately one hour after contact with bedrock on June 22, 2011 accompanied by an odour associated with hydrogen sulphide. No measured concentrations of gas are available for this occurrence.

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. More details about the pumping test results and interpretations are provided in the "Hydrogeological Assessment of H_2S Migration" report (Document No. 285380-83-119-0005).

Table 4-6: Pumping Tests Data

Test Number	Approximate Location	H_2S Gas Concentration (mg/L)
TOW-1	East of Tunnel T-10A	<0.2
TOW-2	North of Tunnel T-7	20.0
TOW-3	South of Tunnel T-4	7.0

The understanding of the engineering behaviour (related to the impact on design and construction) of the gassy soils is rather limited. In the case of low permeability cohesive soils, it is known that these soils may experience rapid drop in undrained shear strength during unloading. Due to the relatively high compressibility of the pore water fluid in gassy soils, the immediate pore water pressure response (ΔU) to total stress changes can be very low. This phenomena leads to reduction in effective stress and hence shear strength (ref. R-25 and R-41). It is, therefore, recommended that the design and construction methodologies should be developed in consideration of the potential presence of these gases (ref. R-14).

5 Development of Geotechnical Design

5.1 Bridge Configuration

Bridge B-3 (Realigned E.C. Row EBL Expressway Underpass near Matchette Road) will be constructed along the realigned E.C. Row Expressway EBL on east of Matchette Road over Highway 401 between Sta. 10+930.439E and Sta. 11+110.939E (E.C Row Expressway stations), and will accommodate the traffic of E.C. Row Expressway (Drawing 285380-03-060-WIP3-0301). The proposed Bridge B-3 is 180.5 m long and the width varies between 16.050 and 19.013 m.

As shown on Drawing 285380-03-060-WIP3-0301, Bridge B-3 is a four-span concrete box structure incorporating concrete true abutments and piers. Bridge deck elevations were estimated using the elevations of WP #1, 2, 3, 4 and 5, and calculated for the selected design section locations using the grades shown on Drawing 285380-03-060-WIP3-0301 (60% submission). The abutments consist of 20.0 m wide × 1.5 thick pile cap founded on deep end-bearing HP 310×110 steel batter piles with various batter (1H:10V to 1H:3V). The piers include 7.2 to 9.4 m wide x 2.0 m high pile caps supported on batter H-piles (1H:5V) as shown on Drawings 285380-03-061-WIP3-0304 and 285380-03-061-WIP3-0305. A concrete wing wall flared to the bridge abutment is indicated at each corner of the abutment structure.

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

Table 5-1: Summary of Control and Interpreted Elevations at Abutments and Piers

Location	Station	Approximate Existing Ground Surface*	Top of Deck	Top of Pile Cap	Approximate Highway 401 Pavement Subgrade*
Centerline of Bridge & West Abutment (WP#1)	10+930.439E	180.0	188.784	182.5	180.5
Centerline of Bridge & Pier #1 (WP#2)	10+964.439E	179.5	188.912	179.8	180.0
Centerline of Bridge & Pier #2 (WP#3)	11+013.439E	179.0	188.806	179.1	179.0
Centerline of Bridge & Pier #3 (WP#4)	11+071.939E	179.0	188.231	177.0	179.0
Centerline of Bridge & East Abutment (WP#5)	11+110.939E	179.5	187.576	179.5	179.0

(*) Indicate elevations as interpreted from highways drawing sections.
Notes: 1-Top of deck elevations were interpreted from Drawing 285380-03-060-WIP3-0301.
2-Details (dimensions and elevations) for the wing walls were not available at the time of this report preparation.

5.2 Geotechnical Design Criteria and Considerations

The geotechnical design has been completed in compliance with the requirements of the execution version of the Project Agreement (PA) Schedule 15-2 Part 2, Article 5 for the Windsor-Essex Parkway Project. The foundation design was as per the principles of Limit States Design (LS Method) based on Load and Resistance Factors (CFEM, ref. R-8 and CHBDC, ref. R-9).

Working Stress Design (WS Method) was employed for global stability of the earthworks and the soil mass containing earth retaining structures as well as for the external stability (bearing, sliding, and overturning) of the retained soil system (RSS) structures. The stability of the soil mass containing the abutments and wing-walls was checked for all potential surfaces of sliding.

The embankments for the E.C. Row Expressway on west and east sides of Bridge B-3 will be built in general with compacted silty clay fill. The design side slopes of the embankment sides are generally 3H:1V. The design height of these embankments above original ground surface was about 9 to 10 m. Where full height side slopes at 3H:1V cannot be accommodated to support the approachway embankment due to geometric restrictions, Light Weight Fill (LWF) was incorporated in conjunction with adequate retaining walls.

The design and construction of the high embankments at this area are complex due to the very weak foundation soils, insufficient time available to achieve consolidation and strength gain in the clay deposit without acceleration of the consolidation by means of wick drains, space restrictions preventing slope flattening and surcharging for preloading, and stringent settlement constraint. These conditions necessitated use of wick drains to expedite consolidation of the foundation stratum and strength improvement, multi-stage construction (three stages at this bridge site) and surcharge loading to minimize future long-term settlement.

Presently, Perforated Vertical Drains (PVD) or wick drains (100 mm wide with 2 mm core thickness) are being installed in triangular pattern at the site. The bottom of wick drains was established at about elevation 163 at this site for environmental reasons.

The wick drain design and construction should be as per the requirements of the OPSS 220, "Construction Specification for Wick Drain Installation". Details of the wick drains and multi-stage construction of the high embankments are provided in "Design Report - High Embankments" (Document No. 285380-04-119-0003).

Bridge B-3 construction (including wick drains and embankments constructions) is expected to involve the following sequence of earthwork, design elements and loading stages:

- Site clearing/grubbing and topsoil 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 surcharge fill to expedite ground consolidation and strength improvement;
- Removal of excess sloped backfill used as a surcharge over wick drain areas to the design elevation of underside of pile cap to facilitate the installation of piles and reconstruction of the approachway;
- Installation of piles (HP310×110) for the abutment and pier supports;
- Construction of the pile caps at abutments and piers;
- Construction of the concrete true abutments including associated permanent subdrainage works, and approved backfill behind and in front of the concrete abutments;
- Construction of pier stems and bridge deck;

- Completion of the road materials; and
- Completion of the Highway 401.

5.3 Design Soil Properties

The test holes located at the Bridge B-3 site and included in the current assessment included 13 boreholes, 7 CPTs, 2 Nilcon vane profile and 2 DMT probes (listed in Table 3-1).

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. The undrained shear strength (S_u) and preconsolidation pressure (σ'_p) profiles inferred from the CPT, DMT and Nilcon tests advanced at and around Bridge B-3 and the design values obtained from these profiles are shown in Figures 5.1 and 5.2, and summarized hereafter in Table 5-2. Effective cohesion for the upper clay crust and transition zone layers has been neglected due to long term weathering, moisture ingress and fissuring effects. As indicated in Figures 5.1 and 5.2, and Table 5.2, the undrained shear strength of the silty clay stratum in the west abutment area was lower than the east abutment area.

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

Clay Substratum	Elevation Range		Undrained Shear Strength (Su), kPa		Effective Strength Parameters	Preconsolidation Pressure (σ_p'), kPa	OCR
	West Abutment	East Abutment	West Abutment	East Abutment			
Clay Crust	179(*) to 177	179(*) to 177	60 (**)	70 (**)	c' = 0, ǿ = 30°	500	>4.00
Clay Transition	177 to 175	177 to 175	60 to 50	70 to 60		500 to 250	3.50
Upper Silty Clay - 1	175 to 166	175 to 163	50 to 25	60 to 36		375 to 150	1.80
Upper Silty Clay - 2	166 to 161	163 to 161	25 to 37	36 to 40		150 to 290	1.05 to 1.40
Lower Clayey Silt	161 to 156	161 to 156	37 to 70	40 to 75		205 to 500	2.00

(*) Elevation varies
c' = Cohesion intercept
ǿ° = Effective Angle of Internal Friction (ǿ)°
(**) Lower bound of shear strength used for global stability
OCR = Over Consolidation Ratio

The estimated undrained shear strength gain (ΔS_u) after completion of consolidation due to embankment fill loading with the selected PVD configurations was used in the approachway embankment area in addition to above in-situ undrained shear strength for global stability analyses for abutments and wing walls. The strength gain in the silty clay stratum should be verified by CPT and Nilcon vane testing in the general area of the new fill embankment as stated in “High Embankment” report before construction of the final approachway for the bridge. The design should be reviewed and refined based on actual strength gain in the silty clay stratum.

The design values of the coefficient of horizontal permeability (k_h), the hydraulic conductivity anisotropy ratio ($A = k_h/k_v$) and the in-situ void ratios required for the analysis of stress-deformation response of the soils are provided in Table 5-3. The permeability values are slightly (2 to 5 times) higher than the values interpreted from the field test results (Figure 4.5) and are considered to be within range of precision of the measurements.

Table 5-3: Design Hydraulic Conductivity Parameters and Initial Void Ratio

Clay Substratum	Horizontal Permeability, cm/sec	Anisotropy ratio, kh/kv	Initial Void Ratio, e0
Clay Crust	6.8 × 10-7	1	0.59
Clay Transition	3.9 × 10-7	2	0.70
Upper Silty Clay - 1	1.1 × 10-7		0.73
Upper Silty Clay - 2	1.1 × 10-7		0.68
Lower Clayey Silt	1.1 × 10-7		0.54
Lower Granular	1.2 × 10-5	1	0.54

For design purposes the long-term groundwater level in the overburden was considered at elevation 180 on west side and 179.5 on east side of the structure.

5.4 Pile Foundations

5.4.1 Resistance to Axial Loads

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

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

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

Based on the available borehole data at this structure, the bedrock surface elevation varies between 153.7 and 157.6, where the tips of piles are anticipated to be set. In cases where some of the piles cannot be driven to bedrock due to presence of dense till lying immediately above the bedrock, and/or a perceived risk of damaging the piles by overdriving is apparent, consideration should be given to supplementing the field testing to prove the actual mobilized resistance. If lower mobilized pile resistances are proven, options based on the most economical approaches may be considered (e.g., changes to the driving method and equipment, or addition of more piles).

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

The following general pile installation recommendations should be considered:

- The steel H-piles should be installed and monitored in accordance with OPSS 903 requirements. The piles should be reinforced with Type I shoe flanges as shown in OPSD 3000.100, or approved alternatives.
- Survey of all the pile head elevations should be completed at the end of driving and just prior to forming the pile cap. Re-tapping of the piles will be necessary where uplift exceeding 5 mm is noted, or as directed by engineer.
- Considering the general geologic conditions in the region and the experience during investigation, indications of natural gas venting, water and fines washout should be monitored during driving. Provision to mitigate such occurrences should be in place (heavy mud pours within the gaps between soil and pile shaft, temporary soil mounding around the pile, etc.). It is recommended that the pile splicing be avoided; if this is not possible, splicing by butt-welding (OPSD 3000.150, Section A-A) should be considered 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.

5.4.2 ULS and SLS Resistance to Lateral Loads

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

The SLS geotechnical resistance to lateral loads is dependent on the acceptable levels of the lateral pile deflections under the design loads and should be obtained on the basis of field load tests. In the absence of field tests, the preliminary design may be based on a conventional SLS resistance of 75 kN along the strong axis, and 50 kN along the weak axis of the HP310x110. This conventional SLS resistance represents the lateral shear force applied on a free-head pile that causes a lateral deflection of 10 mm measured at the ground surface.

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 225 kN and 110 kN along the strong axis and weak axis, respectively.

The above SLS and ULS resistances were estimated using the "p-y" model (LPile 5.0 model Ensoft 2010). The pile model assumed to be embedded within stiff to soft silty clay below elevation 181. 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 following soil parameters described in Table 5-4.

Table 5-4: Soil Parameters for Pile Interaction Assessment

Soils Around the Piles	Elevation	Design Bulk Unit Weight, kN/m^3	Undrained Shear Strength, S_u, kPa	ε_{50}
Compacted Clay Fill	181 to 179	21	50	0.007
Clay Crust	179 to 177	22	60	0.007
Clay Transition	177 to 175	22	60 to 50	0.007
Upper Silty Clay - 1	175 to 166	20.5	50 to 25	0.007 to 0.010
Upper Silty Clay - 2	166 to 161	20.5	25 to 37	0.010
Lower Clayey Silt	161 to 156	20.5	37 to 70	0.010 to 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}.

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

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

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

Significant lateral loads in excess of the 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 up to 1H:3V may be considered.

The stress-deformation analysis of the piles to lateral loads may be carried out using one of the following methods.

Horizontal Subgrade Reaction Method:

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

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

$= 67 \left(\frac{Su}{d}\right)$ for cohesive soils.

Where:

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

n_h (MPa/m) = Soil coefficient

S_u (MPa) = Undrained shear strength

z (m) = Depth below finished grade

d (m) = Pile diameter/width

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

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 5-5. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed here.

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

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

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

Alternative Nonlinear ‘p-y’ Curve Method:

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

The general procedure for computing p-y curves is summarized in the Canadian Foundation Engineering Manual (ref. R-8). A detailed description for the generation of the ‘p-y’ curves can be found in the Technical Manual for the commercial software LPile Plus by Ensoft Inc (ref. R-15). For a given foundation configuration, pile size, and soil stratification, the soil properties required for the generation of the ‘p-y’ curves are provided in Table 5-4. “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 require to be scaled by a factor (“modifier”) to account for batter and for group effects. The modifier factor applies to the ‘p’ values.

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

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

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

Where:

β_{ki} is the influence factor of pile ‘k’ in the group on pile ‘i’, with k ≠ i, and is calculated with one of the following expressions (depending on the relative position of pile ‘k’ in the group with respect to pile ‘i’ (Table 5-6).

Table 5-6: Lateral Load Capacity Reduction Factor for Pile Groups using Nonlinear for ‘p-y’ Curve Method

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

Reduction factors as listed in Table 5-6 would apply on the piles.

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

5.4.3 Soil Pile Interaction Assessment

Downdrag Loads (Negative Skin Friction – NSF):

Potential for downdrag loads on piles was examined in conjunction with the anticipated creep that is assumed to occur following completion of bridge constructions.

Soil stress-deformation analyses described later in Section 5.6 were conducted using the SIGMA/W software. The estimated ground vertical movement (settlement/heave) are presented in Figures G.1, G.2 and G.3 in Appendix G. The estimated vertical movements correspond to the following stages: completion of embankment construction with wick drains, completion of the bridge construction (End of Construction - EOC) and the long-term steady state condition (LT). Excess pore water pressure at wick drain, pore water pressure at wick drain, and vertical effective stresses along pile line are illustrated in Figures G.11, G.12 and G.13, respectively. 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 after completion of embankments and substantial consolidation of the foundation soils.
- A potential post construction settlement due to secondary consolidation (creep) of up to 70 mm is expected to occur over a period of time.

Considering the construction staging and the anticipated settlement of the soils described above, a residual (long-term) downdrag of about 300 kN is estimated to develop for the abutment piles.

Soil stress-deformation analyses indicate that ground settlement will occur at the pier location (Figure G.8). Based on this anticipated settlement of the soils, a downdrag of about 275 kN is estimated to develop for the pier piles.

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

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

Shaft Bending due to Lateral Soil Displacement:

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

- Lateral ground movement (Figure G.14) that causes pile shaft bending was estimated using the stress-deformation analysis described below in Section 5.6.
- The model was run with two options with the pile head assumed to be a free-head or fixed-head.

- The above soil deformation field was imposed as "loads" along the pile shaft. The calculation was conducted using the 'p-y' model (LPile 5.0 model Ensoft 2010). The 'p-y' curves were generated using the Reese method described in the Technical manual for LPILE, using the soil parameters indicated in Table 5-4.

Based on the above approach and anticipated lateral ground displacement, the estimated maximum unfactored bending moments in the shaft were 109 kN-m for the strong axis pile loadings for a free-head condition and 227 kN-m for a fixed-head condition. These results should be considered in the structural design of the piles. These bending moments, shear forces and deflections are in addition to those caused by bridge loads applied to the piles.

The maximum computed moment in the pile under assumed pile head load equal to the conventional SLS resistance (75 kN) was 84 kN-m for the strong axis pile loadings. Accordingly, a potential combination of the maximum bending stresses from pile head shear force and ground displacement field would lead to a maximum bending moment of 193 kN-m for the free-head condition and 311 kN-m for a fixed-head condition, which is less than the yield moment of the pile.

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

5.5 Global Stability

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

The properties of the proprietary products and backfill materials assumed in the geotechnical analyses are summarized in Tables 5-7 and 5-8.

Table 5-7: Assumed Proprietary Product Properties

Material	Unit weight, kN/m³	Limit Equilibrium Analyses (Slope/W Models)		Stress Deformation Analyses (Sigma/W Models)	
		Friction Angle, °	Apparent Cohesion, kPa	Modulus of Elasticity, E, MPa	Poisson's ratio, μ
RSS (with Approved Granular Fill)	21.0	35	50	60	0.35
RSS (with LWF)	12.0	35	50	40	0.35
LWF	12	35	0	30	0.35

Table 5-8: Assumed Backfill Material Properties

Backfill Material	Unit weight, kN/m³	Undrained Shear Strength, kPa	Drained Angle of Internal Friction, degree	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	35	40.0	0.35

Figures F.1 to F.18 illustrate the stability models for the abutments and wing walls at cantilever portion at west and east sides. The global stability analyses have been carried out for short-term during construction (using undrained soil properties), end of construction (using undrained soil properties) and long-term steady state (using drained soil properties with stabilized water levels) loading conditions. The short-term analysis simulated temporary condition during construction in which the toe berm of the structure was not present. The end of construction (undrained) and the long-term steady (drained) analyses assumed that all the components of the structure were present.

The undrained shear strength at completion of the approachway embankment was based on the in-situ shear strength plus the strength gain during the preloading. The increase in the undrained strength (ΔS_u) of the clay deposit following excess pore pressure dissipation and consolidation of the clay strata under successive surcharge loads was calculated based on the net increase in the pre-consolidation pressure ($\Delta P'_c$) generated by the preloading using the relationship ΔS_u = Ux0.18 $\Delta P'_c$, where U (%) is the degree of consolidation.

The global stability analyses have been carried out for west and east abutments at representative sections at Stations 10+930.439E and 11+110.939E where LWF is not present. These sections have been chosen in order to assess the highest impact of the abutment on global stability. The presence of the piles was not considered in the stability models (somewhat conservative approach). Live Loads of 12 kPa for short-term and long-term model were applied at the top of ground surface for the roadway, while tension crack was assumed for short-term only. The global stability analyses have been carried out on the soil mass containing concrete wing walls at cantilever portion as well as RSS walls adjacent to the wing walls.

The calculated factors of safety (FS) against global instability of the abutments and cantilever wing walls are shown in Figures F.1 to F.18 and summarized in Table 5-9.

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

Abutment/Wing Wall	Factor of Safety for Loading Condition			Reference Figure
	Short-term Undrained Loading Condition(1)	End of Construction Undrained Loading Condition(2)	Long-term Drained Loading Condition(3)	
West Abutment	1.36 (1.28)(4)	1.37 (1.30)	1.72 (1.59)	F.1 to F.3
East Abutment	1.42 (1.33)	1.43 (1.35)	1.64 (1.49)	F.4 to F.6
West Wing Wall (South)	1.41 (1.34)	1.62 (1.49)	1.73 (1.58)	F.7 to F.9
West Wing Wall (North)	1.41 (1.30)	1.41 (1.30)	1.66 (1.64)	F.10 to F.12
East Wing Wall (South)	1.87 (1.77)	1.87 (1.77)	1.56 (1.50)	F.13 to F.15
East Wing Wall (North)	1.83 (1.70)	2.03 (1.85)	2.23 (2.12)	F.16 to F.18

Note: Values outside parentheses refer to circular failure surfaces and the values in parentheses refer to non-circular failure surface.

(1) Short-term (temporary) undrained response without toe berm

(2) Undrained response with all design component present

(3) Drained response with all design components present

(4) Toe berm should be built before any backfill is placed above the bridge seat level.

Based on the global stability and geotechnical bearing analyses, abutment and wing wall configurations, and dimension of RSS walls and LWF were determined and listed in Table 5-10. The general configurations of abutments and wing walls with RSS walls and LWF are shown on Figure H.1.

Table 5-10: Dimension of LWF and RSS Walls at Wing Wall

Abutment/Wing Wall	Average Thickness of LWF, m	Average Thickness of Granular Fill, m	RSS Wall Size at North (Width x Height)[1], m	RSS Wall Size at South (Width x Height)[1], m
West Cantilever Wing Wall	3.5	1.5	7.0 x 5.0	7.0 x 5.0
East Cantilever Wing Wall	5.5[(2)] and 7.0[(3)]	0.5	15.0 x 7.5	9.0 x 6.0

(1) The RSS supplier may require wider walls to meet the internal design requirement. The effects of a wider wall on bearing capacity will need to be assessed
(2) On south side
(3) On north side.

5.6 Stress Deformation Analyses

Stress-deformation analyses (SDA) were carried out by finite element modeling using SIGMA/W software Version 2007. The main purpose 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 to G.6. 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 (based on published data [ref. R-42] and confirmed by DMT at the site) for the soil deposit (0 days);

(b) Installation of wick drains with construction of preloading embankment (240 days duration – day 1 to 240);

(c) Removal of the preloading and replacement by LWF and granular fill, and subsequent construction of the concrete true abutment and the associated backfill (assumed 30 days duration – day 240 to 270);

(d) Completion of the backfill at toe of abutment – end of construction (1 days duration – day 270 to 271); and

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

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 concrete abutment was simulated by 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 subgrade surfaces. Elastic-plastic Mohr-Coulomb models were used for all soil layers except the unweathered firm to stiff silty clay, which was described by the Modified Cam-Clay model. Hydraulic conductivity properties described in Table 5-3 were assigned to the different soil layers.

The scenario of stress-deformation model suggests dissipation of major proportion of the excess pore water pressures generated by the soil loading of the listed construction stages (loading steps 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.

The SIGMA model was developed for the west abutment where the height of the retained soils measured from the top of finished grade to the existing ground surface is 9 m high and LWF is not present. The west abutment model will provide the upper limits for the deformation estimates.

Figures G.1, G.2 and G.3 show the cumulative settlement/heave for the end of construction of approachway embankment with wick drains (240 days), end of construction ("271 days") of the bridge and the long-term ("11,271 days") drained loading conditions. Figures G.4 and G.5 show the cumulative lateral deformation at the end of construction and the long-term drained loading condition. Figure G.6 illustrates the stabilized pore water pressure contours at the end of dissipation (long-term) period.

5.6.1 Serviceability Limit States (SLS) Assessment

The SLS performance was assessed on the basis of the SDA described above in Section 5.6. The cumulative deformations are summarized in Table 5-11.

Table 5-11: Summary of Calculated Deformations

Parameter	End of Construction of Bridge(1), mm	Long-term (Drained)(2), mm	Net Deformation	Remarks
Settlements on Top of Ground at Distances (m) from the Edge of Bridge Deck of (†)				Figure G.7
0 m†	-405	-405	Nominal	
5 m	-480	-480		
10 m	-530	-530		
20 m	-565	-565		
30 m	-560	-560		
50 m	-525	-525		
75 m	-500	-500		
Maximum Settlement/Heave at Pier #1	35	20	-15 mm	Figure G.8

(-)ve denotes settlements
(†) Distances measured perpendicular to the bridge abutment.
(1) Cumulative deformation at top of abutment backfill to be compensated during construction.
(2) Cumulative deformation without potential creep
The cumulative deformations are rounded up to closest 5 mm.

Figure G.9 shows the soil settlement at the existing ground surface. Figures G.10 and G.14 show soil settlement and lateral soil displacement along the pile line. These deformations were estimated from SDA, which were used in pile calculation in Section 5.4.

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

The settlement/heave magnitudes presented 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.

5.7 Bearing Capacity and Sliding Resistance

The external stability factors of safety against base sliding, overturning about the toe and bearing capacity failures were checked for RSS walls abutting the wing walls and supporting the approachway embankment. The use of LWF was required in order to meet the external stability for bearing and sliding. 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 was employed.

Bearing Capacity:

Bearing capacity analyses were carried out to estimate the mobilized undrained shear strength of the improved soils under the abutments/wing walls. Based on the estimated mobilized shear strength of the soils obtained, the following net ultimate geotechnical bearing resistance values (q_u) were determined for the native subgrade soils at the RSS walls for short-term (undrained) and long-term (drained) loading conditions.

Table 5-12: Subgrade Ultimate Bearing Capacity

Wing Wall	Assumed Lowest Subgrade Elevation	Loading Condition	q_u (kPa)
West Side	182.0	Short-Term (Undrained)	240(1)
		Long-Term (Drained)	300(2)
East Side	179.5	Short-Term (Undrained)	270(3)
		Long-Term (Drained)	300(2)

(1) Based on estimated mobilized average cohesion of 47 kPa within the zone of influence
(2) Based an assumed soil friction angle phi = 30^0
(3) Based on an estimated mobilized average cohesion of 53 kPa within the zone of influence.

Sliding Resistance:

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

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

Where:

A' (m^2)	= effective contact area of the base;
c' (kPa)	= cohesion/adhesion at sliding interface;
δ (º)	= friction angle at sliding interface;

V (kN) = vertical force (kN); and

H_f (kN) = design horizontal load.

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

Table 5-13: Soil Properties for use in ULS at Sliding

Interface	Undrained (Short-Term)		Drained (Long-Term)	
	δ, deg	c, kPa	δ', deg	c', kPa
LWF to Silty Clay	0	60	30	0

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 CHBDC (ref. R-9). Construction notes for backfill are provided in Drawing 285380-04-094-WIP3-0372. Construction notes for lightweight fill material (LWF) are provided in Drawing 285380-04-094-WIP3-0373.

The conventional soil 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 CHBDC.

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

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

Table 5-14: Soil Parameters for Earth Pressure Calculations

Soil Parameter	Group I Soils	Group II Soils	Group III Soils
Fill Unit Weight, kN/m3	22	21	20.5
Friction Angle, φ, degrees	33 to 35	29 to 32	22 to 30
Coefficients of Static Lateral Earth Pressure:			
'Active' or Unrestrained, Ka(*)	0.27 to 0.30	0.31 to 0.35	0.33 to 0.45
'At Rest' or Restrained, Ko(*)	0.43 to 0.46	0.47 to 0.52	0.50 to 0.62
'Passive', Kp(*)	3.3 to 3.7	2.9 to 3.2	2.2 to 3.0

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

- Group I Soils: Coarse grained soils (e.g. Granular A and B Type 2).
- Group II Soils: Finer grained than Group I non-cohesive soils (e.g. Granular B Type 1, pit run, etc).
- Group III Soils: Finer grained soils (e.g. approved site generated silty clay).

5.9 Permanent Subdrainage System

A permanent subdrainage system as per OPSD 3101.150 and OPSD 3102.100 should be provided behind the abutments and connected to the roadway drainage system.

6 Other Geotechnical Recommendations

6.1 Construction Dewatering

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

Considering the excavations at this site which will be shallow along with the relatively low permeability of the silty clay deposit, groundwater seepage is anticipated to be minor, which should be controllable by conventional temporary dewatering methods. However, significant seepage into the excavations from perched groundwater from the fill, old farm tiles and/or utility trenches, and upper granular layers are likely to occur. In adverse conditions, the runoff and seepage from perched groundwater and sand/silt pockets can be significant and accompanied by piping and wash-outs of the fines causing sloughing of the slopes.

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

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

6.2 General Construction Requirements

The anticipated construction conditions in this report are discussed only to the extent of their potential influence on the design of the permanent elements of the tunnel. References to construction methods are not intended to be the suggestions or directions on the construction methodologies. Contractors should be aware that the data presented in this report and their interpretations may not be sufficient to assess all factors that may affect the construction. Construction requirements related to wick drains, embankment and bridge approachways are described in "Design Report - High Embankments".

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

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

- To protect the integrity of subgrade for foundations and pavements, the final excavation lift above the design elevation should not be less than 500 mm and should be carried out only when the Contractor is ready to prepare and cover the subgrade with the materials specified in the design same day the final excavation is exposed and approved. No construction traffic should be permitted over subgrade without approved protective covers.
- The final excavation layer 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.
- In recognition of potential for soil gases as described in Section 4.6, air quality and subgrade pore pressure monitoring should be carried out during construction. The equipment operating in confined spaces should be selected to safely operate in a potentially gaseous environment. Excavation lifts should be decided in consideration of the pore pressure monitoring data and the potential ground softening.

6.3 Instrumentation and Monitoring during Construction

As mentioned earlier in Section 5.4, a program of site instrumentation and monitoring of the temporary works during construction should be implemented by the Contractor in addition to the limited instrumentation already installed during the geotechnical investigation (Table 3-2).

Recommendations for additional instrumentations and monitoring programme as well as guidelines for interpretation, alert levels and contingencies are provided in a separate report (Document No. 285380-04-118-0001).

The Contractor is responsible for planning, installation and maintenance of instrumentation as well as the completion of monitoring of the response of the excavations (ground movement) during construction. Detailed plans and procedures should be submitted to HMQ for approval at least three months 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 to clayey silt stratum obtained in Boreholes B3-1 (Sample 18), B3-2 (Sample 16) and B3-3 (Sample 1). Table 6-3 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 B3-1 (Sample 18)	154.7	7.87	195	2170	<0.2	246
Borehole B3-2 (Sample 16)	160.6	7.65	158	2580	<0.2	449
Borehole B3-3 (Sample 1)	178.2	7.70	147	7410	<0.2	<20

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

Based on the measured electrical resistivity, pH, redox potential, sulphide contents etc., the tested soil would be considered noncorrosive to buried metallic elements (ref. R-2).

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

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

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

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

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

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

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

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

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

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

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

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

8 Closure

The geotechnical report for Bridge B-3 was prepared by Mr. Nazmur Rahman, P.Eng and checked by Dr. Dan Dimitriu, P.Eng. The project was executed under the technical direction of Dr. Narendra S. Verma, P.Eng. who also provided the senior review of the report. Mr. Matt Oldewening, P.Eng., managed the geotechnical investigation and Mr. Brian Lapos, P.Eng., was 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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Senior Geotechnical Engineer

Dan Dimitriu, Ph.D., P.Eng.
Associate Geotechnical Engineer
(Project Lead Designer)

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

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Drawings

METRIC

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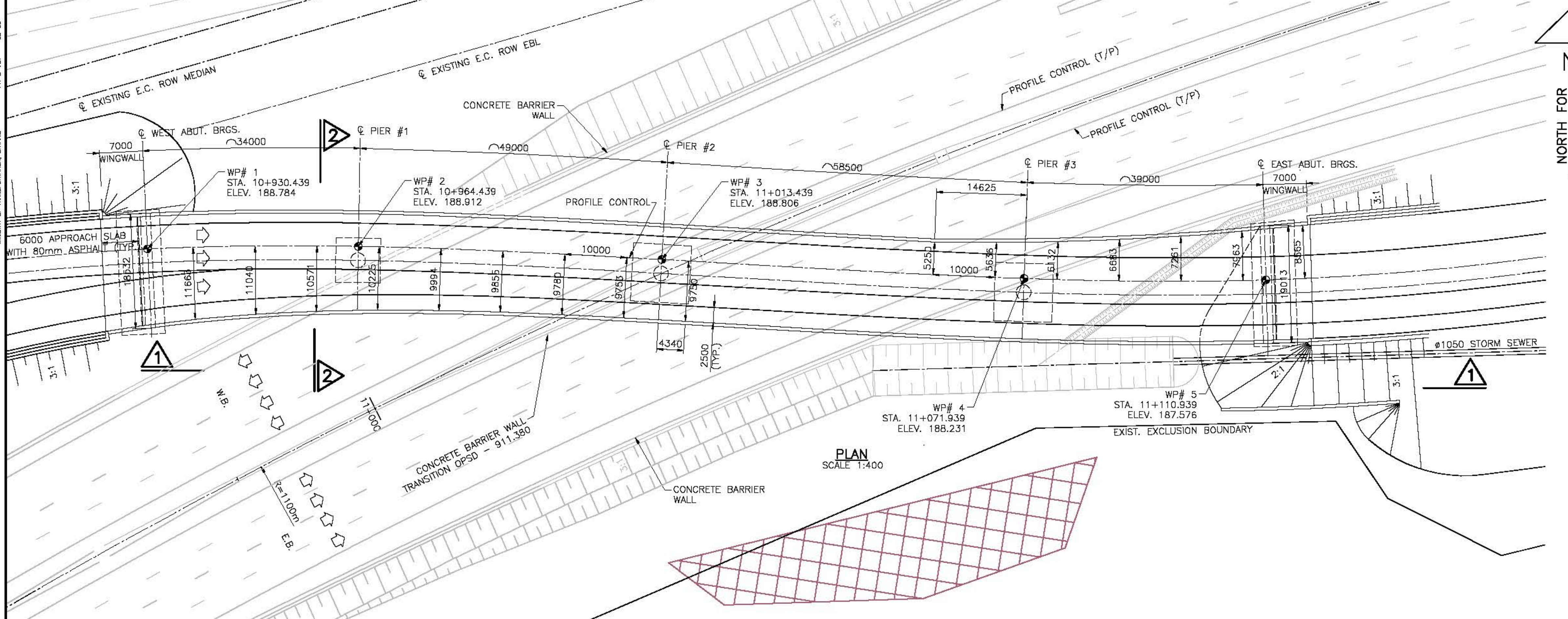
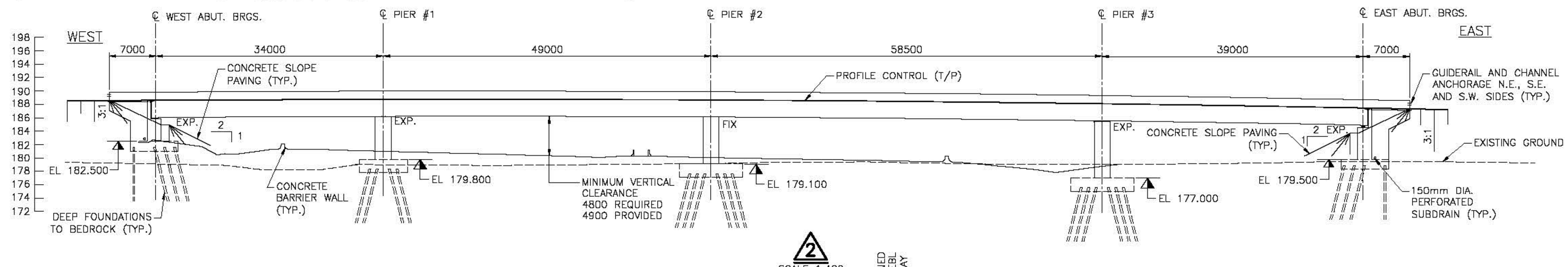
Windsor–Essex Parkway Project
RFP No. 09–54–1007

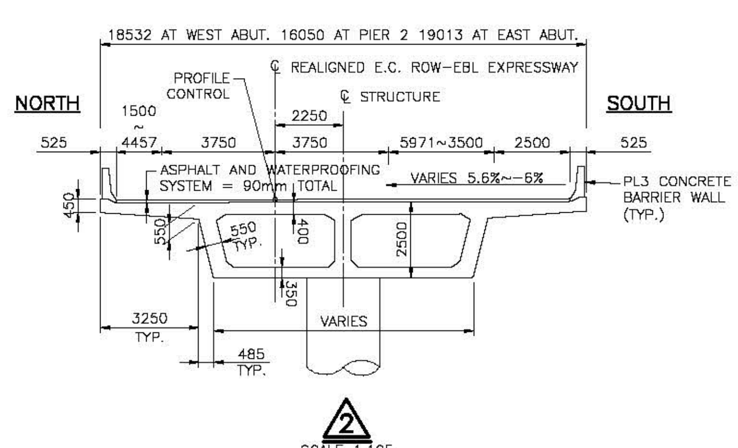
NEW CONSTRUCTION
BRIDGE B–3
REALIGNED E.C. ROW–EBL EXPRESSWAY UNDERPASS NEAR MATCHETTE ROAD
GENERAL ARRANGEMENT

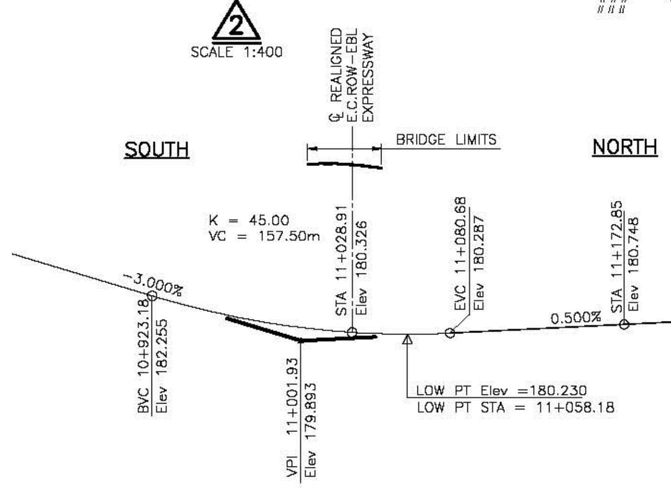
SHEET S––––

Phase 3
60% Sub

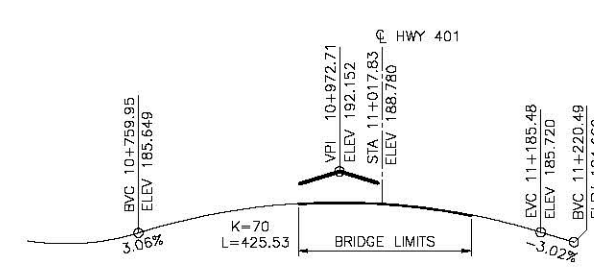








PROFILE OF HWY 401
N.T.S.



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

GENERAL NOTES:

1. CLASS OF CONCRETE:
   - DECK....................50 MPa
   - FOOTING....................30 MPa
   - REMAINDER....................30 MPa
2. CLEAR COVER TO REINFORCING STEEL:
   - FOOTINGS....................100 ± 25
   - ABUTMENT AND WINGWALLS....................70 ± 20
   - PIER....................80 ± 20
   - DECK :
     - TOP SLAB, TOP....................70 ± 20
     - TOP SLAB, BOTTOM....................40 ± 10
     - BOTTOM SLAB, TOP....................40 ± 10
     - BOTTOM SLAB, BOTTOM....................60 ± 10
   - WEBS....................60 ± 10
   - REMAINDER....................70 ± 20

   UNLESS OTHERWISE NOTED.
3. REINFORCING STEEL:
   - REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
   - BAR MARKS WITH PREFIX 'C' DENOTE COATED 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.
   - BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
   - 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 INDICATED OTHERWISE.

CONSTRUCTION NOTES:

1. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESS FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESS ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
2. LOCATION OF EXISTING UTILITIES TO BE DETERMINED DURING DETAIL DESIGN. ALL EXISTING UTILITIES ARE TO REMAIN UNLESS NOTED OTHERWISE.
3. ALL EXISTING UTILITIES SHALL BE PROTECTED FROM DAMAGE DURING CONSTRUCTION OF EMBANKMENTS,DRIVING OF DEEP FOUNDATION AND CONSTRUCTION OF ABUTMENTS.
4. EMBANKMENTS SHALL BE CONSTRUCTED IN STAGES AND MAY REQUIRE WICK DRAINS AND PRELOADING. ALTERNATIVELY LIGHTWEIGHT FILL MAY BE USED.
5. SETTLEMENT OF THE EMBANKMENTS SHALL BE MONITORED DURING AND AFTER CONSTRUCTION.

LIST OF ABBREVIATIONS

ABUT.	ABUTMENT	STA.	STATION
BRGS.	BEARINGS	SHLD	SHOULDER
DIA.	DIAMETER	T/P	TOP OF PAVEMENT
E.B.	EASTBOUND	THK.	THICK
EL.	ELEVATION	TYP.	TYPICAL
HORIZ.	HORIZONTAL	VERT.	VERTICAL
N.T.S.	NOT TO SCALE	W.B.	WESTBOUND
RW	RETAINING WALL	WP	WORKING POINT
SCL	SPEED CHANGE LANE		

APPLICABLE STANDARD DRAWINGS

- OPSD 3101.150 ABUTMENTS, BACKFILL,MINIMUM GRANULAR REQUIREMENT
- OPSD 3121.150 WALLS, RETAINING, BACKFILL, MINIMUM GRANULAR REQUIREMENT
- 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 2mm WIDE AND CONSTRUCTION JOINTS
- OPSD 3419.100 BARRIERS AND RAILINGS, STEEL GUIDERAIL AND CHANNEL ANCHORAGE
- OPSD 3941.200 FIGURES IN CONCRETE, SITE NUMBER AND DATE, LAYOUT
- 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

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METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

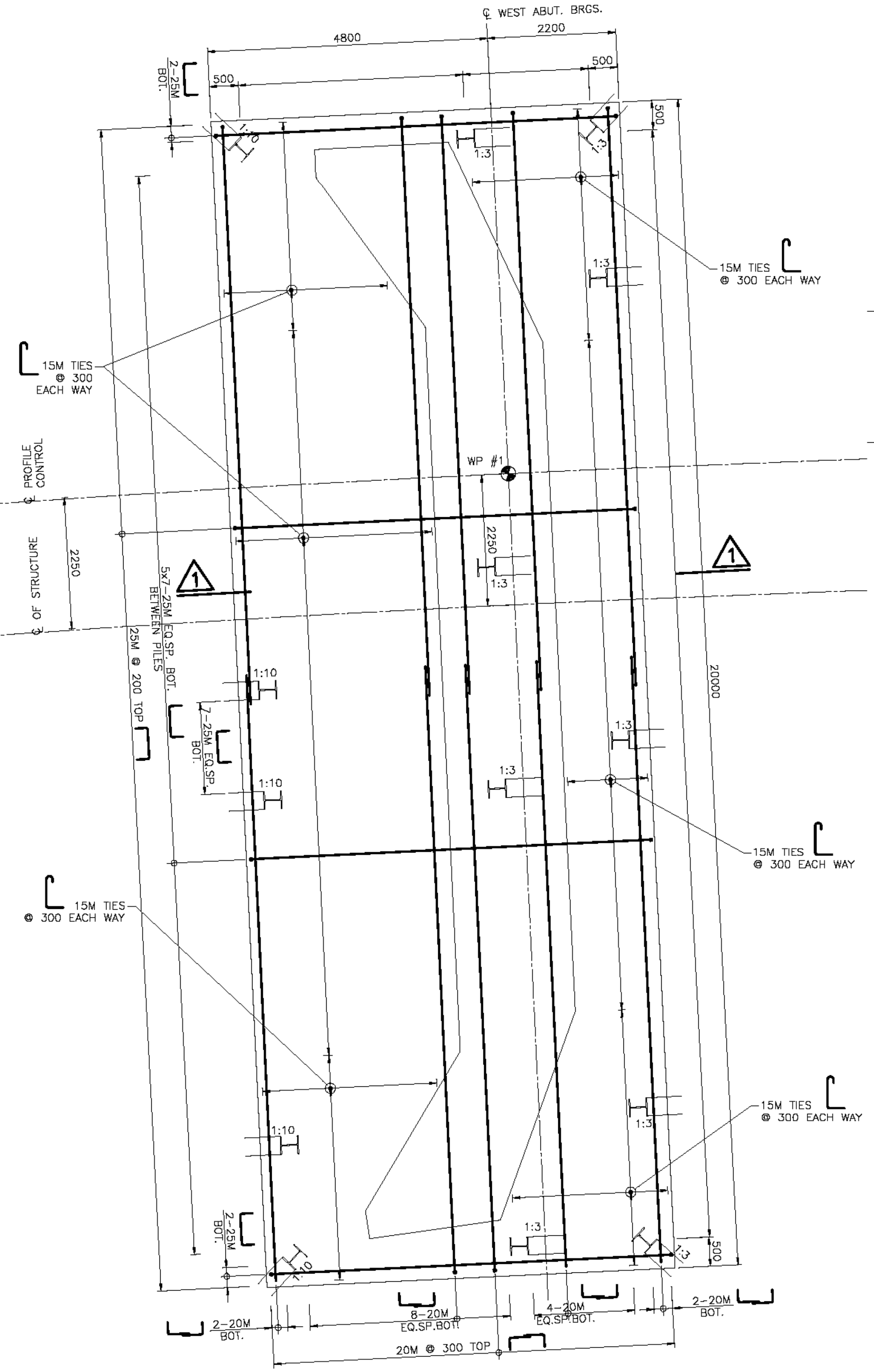
Parkway Infrastructure Engineers | Hatch Mott MacDonald

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

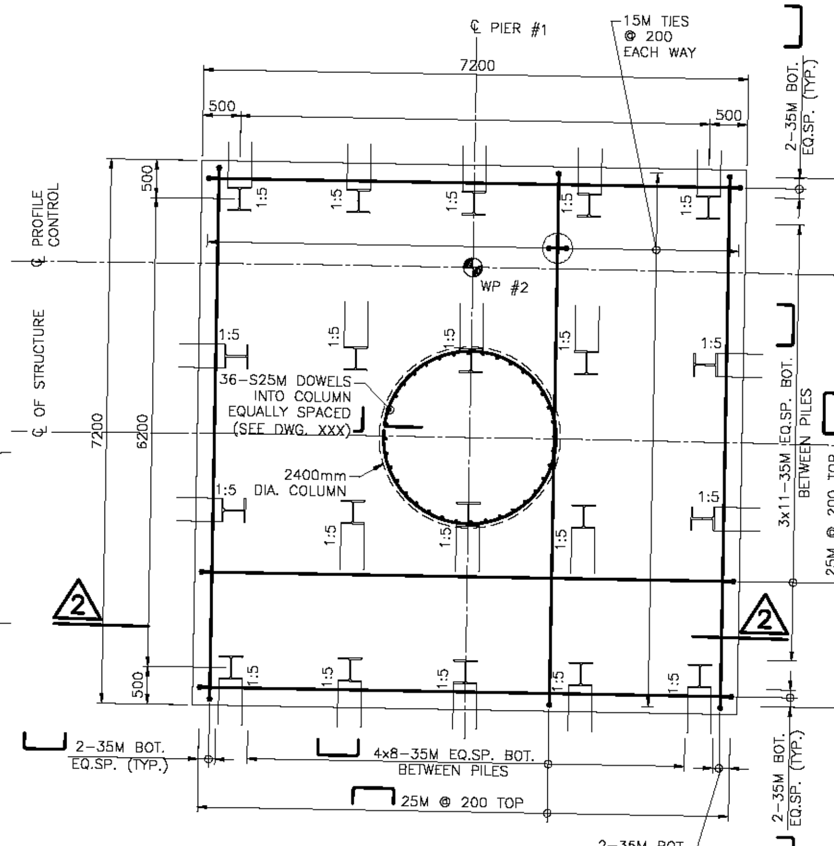
NEW CONSTRUCTION
BRIDGE B–3
REALIGNED E.C. ROW–EBL EXPRESSWAY UNDERPASS NEAR MATCHETTE ROAD
FOUNDATION LAYOUT & REINFORCEMENT I

SHEET
S––––

Phase 3
60% Sub



WEST ABUTMENT PLAN
SCALE 1:50



PIER #1 PLAN
SCALE 1:50

WORKING POINT DATA		
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WP #1	4 682 273.495	329 452.616
WP #2	4 682 253.766	329 480.299

PILE DATA			
LOCATION	No. REQUIRED	LENGTH (m)	BATTER
WEST ABUTMENT	–	–	SEE PLAN
PIER #1	–	–	SEE PLAN

APPLICABLE STANDARD DRAWINGS:

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

OPSD–3000.150 FOUNDATION PILES – STEEL H–PILE SPLICE

LIST OF ABBREVIATIONS

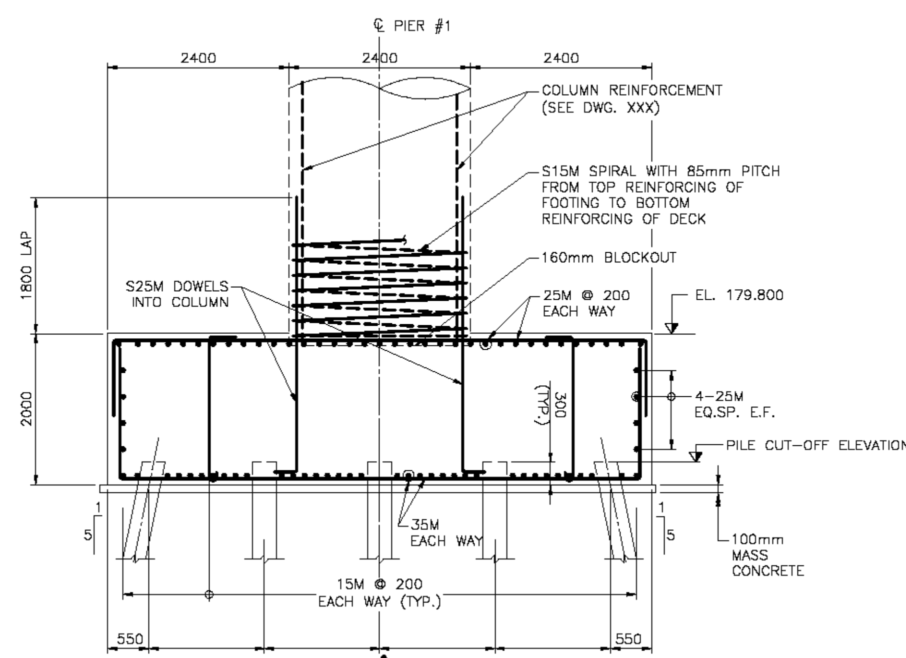
BOT.	BOTTOM
O.F.	OUTSIDE FACE
I.F.	INSIDE FACE
E.F.	EACH FACE
EQ.SP.	EQUALLY SPACED
T/F	TOP OF FOOTING
TYP.	TYPICAL
EL.	ELEVATION
WP	WORKING POINT

NOTES: (STANDARD)

1. FOR GENERAL NOTES SEE DWG. S0301.
2. THIS DRAWING TO BE READ IN CONJUNCTION WITH DWG. S0305 AND S0307.

PILE NOTES:

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



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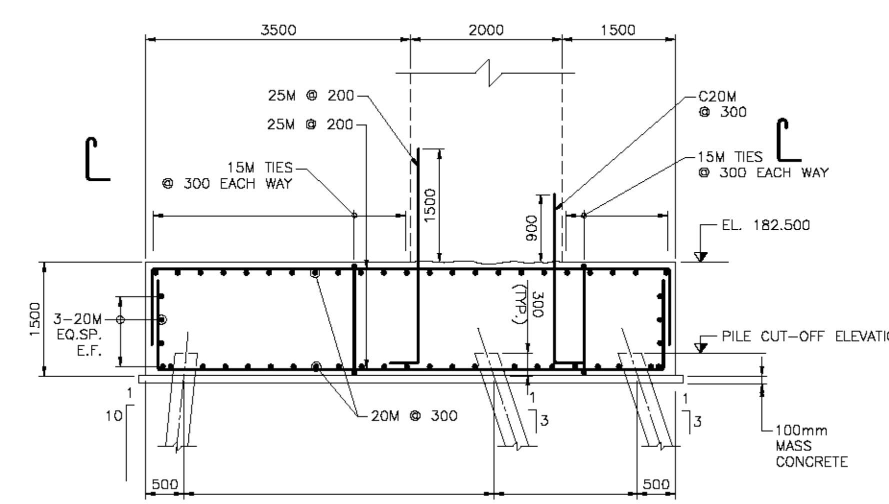

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METRIC

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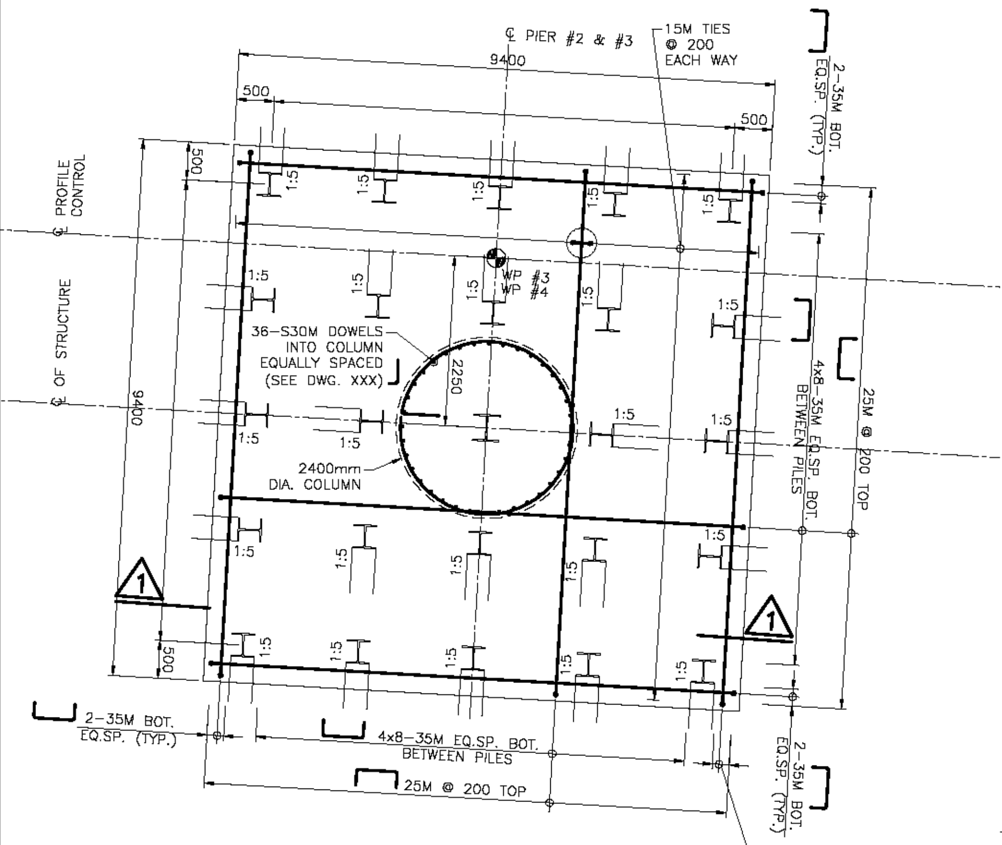
Windsor-Essex Parkway Project
RFP No. 09-54-1007

NEW CONSTRUCTION
BRIDGE B-3
REALIGNED E.C. ROW-EBL EXPRESSWAY UNDERPASS NEAR MATCHETTE ROAD
FOUNDATION LAYOUT & REINFORCEMENT II

SHEET S----

Phase 3
60% Sub



PIER #2 & #3 PLAN
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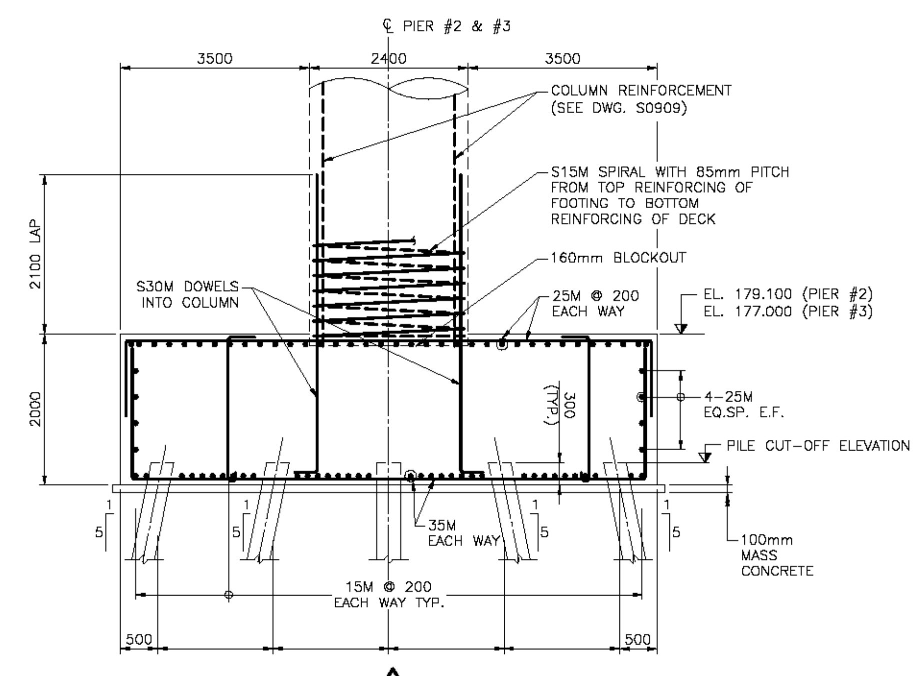
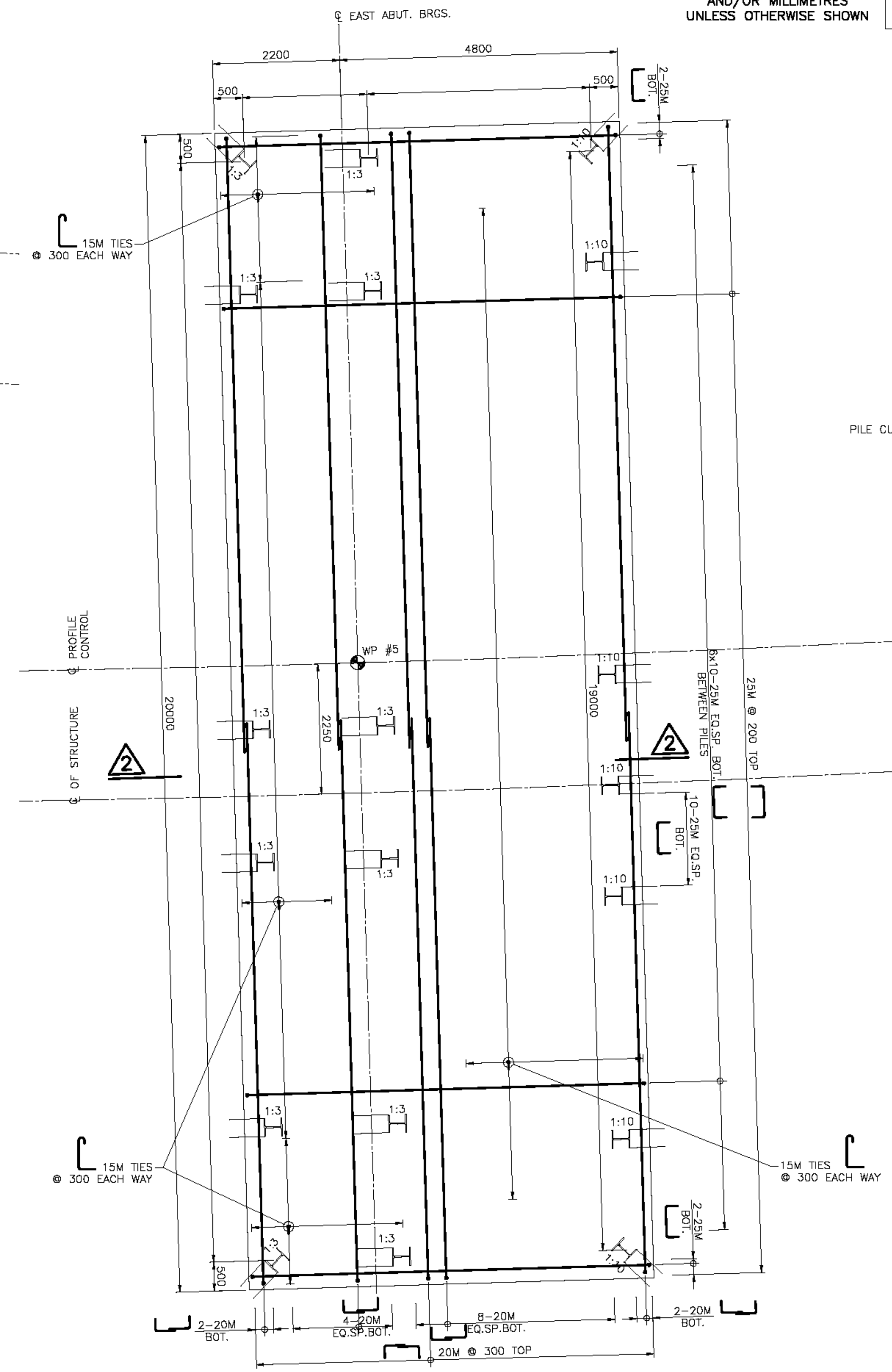


1 — 1:50



EAST ABUTMENT PLAN
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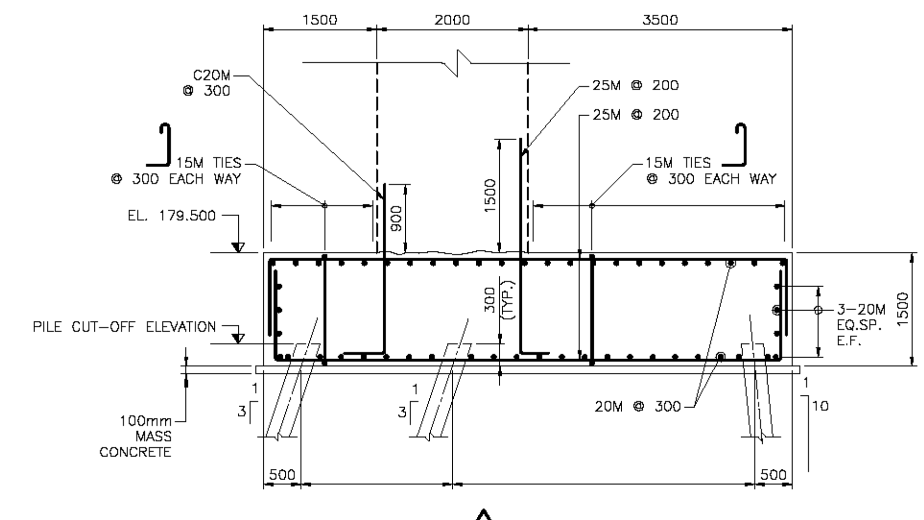

2 — 1:50

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WP #4	4 682 186.217	329 563.918
WP #5	4682163.017	329 595.258

PILE DATA			
LOCATION	No. REQUIRED	LENGTH (m)	BATTER
PIER #2	–	–	SEE PLAN
PIER #3	–	–	SEE PLAN
EAST ABUTMENT	–	–	SEE PLAN

LIST OF ABBREVIATIONS

BOT.	BOTTOM
O.F.	OUTSIDE FACE
I.F.	INSIDE FACE
E.F.	EACH FACE
EQ.SP.	EQUALLY SPACED
T/F	TOP OF FOOTING
TYP.	TYPICAL
EL.	ELEVATION
WP	WORKING POINT

APPLICABLE STANDARD DRAWING:

OPSD 3000.150 FOUNDATION PILES STEEL H-PILES SPLICE

NOTES:

THIS DRAWING TO BE READ IN CONJUNCTION WITH DWG. S0301, S0304 AND S0308

DRAWING NOT TO BE SCALED
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REVISIONS	DATE	REV.	BY	DESCRIPTION

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METRIC

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

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

INVESTIGATION PLAN & INTERPRETED STRATIGRAPHIC PROFILE
STA 10+900W TO STA 11+500W

REVISIONS			
25–MAY–12	A	NR	90% MTO SUBMISSION
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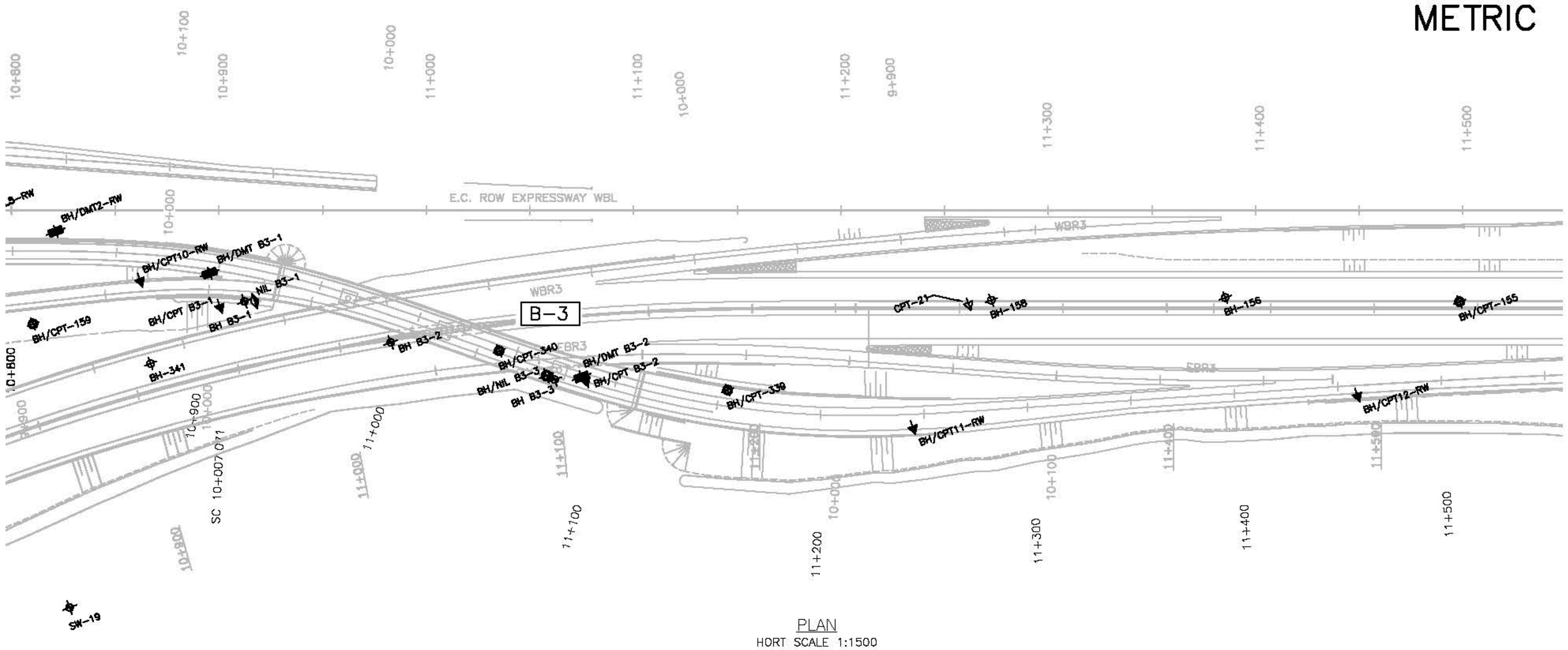
DESIGN SF APR NSV DATE 30–MAY–11

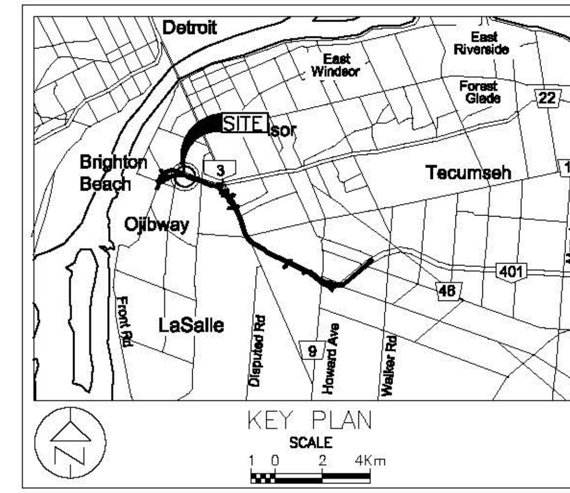
SHEET G0301

Phase 3

90% Sub



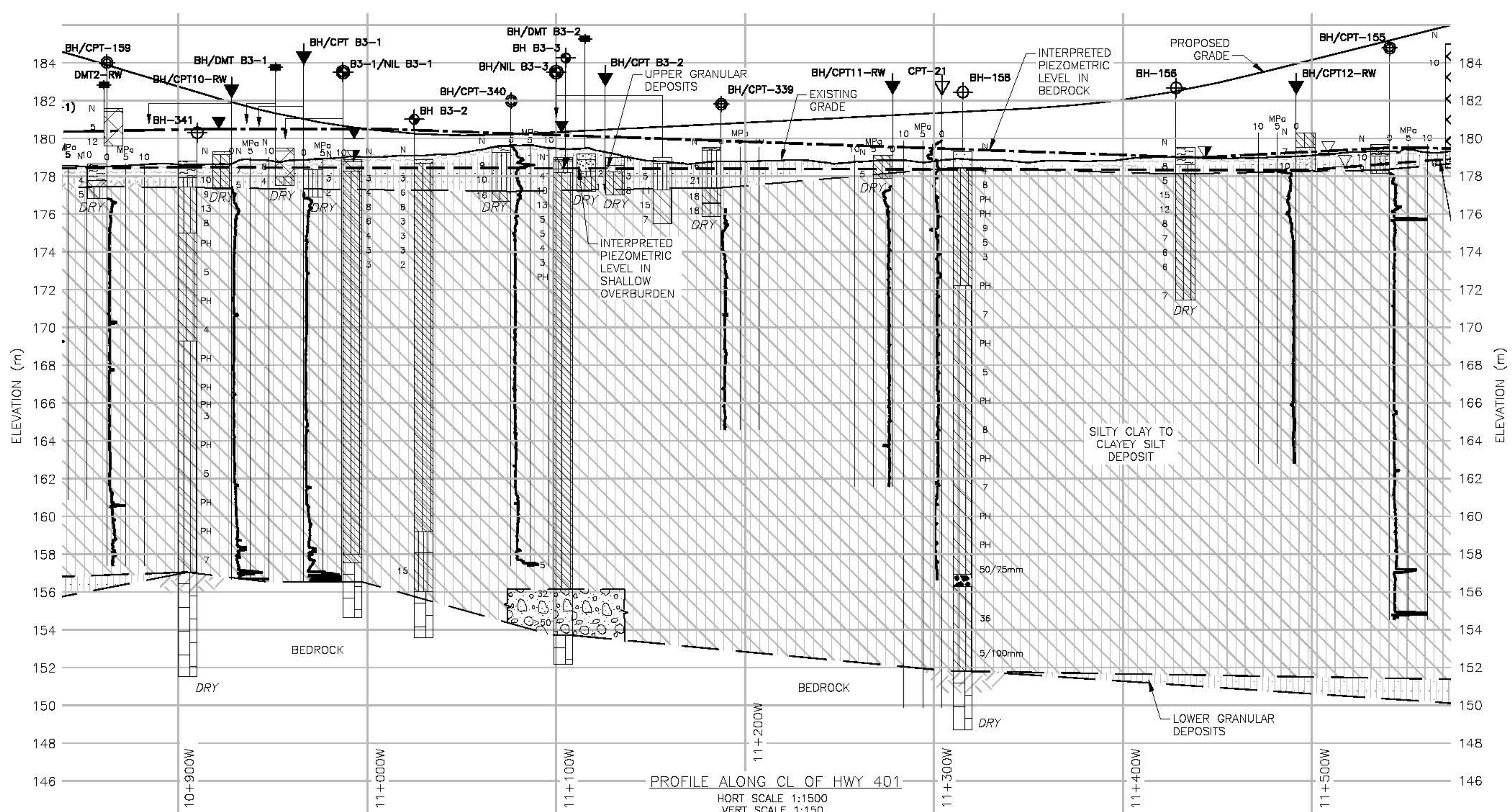
PLAN
HORT SCALE 1:1500



KEY PLAN
SCALE
1 0 2 4Km

LEGEND

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



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

NOTES

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

SCALES
15 0 30 Horizontal
1.5 0 3 Vertical

DATE PLOTTED: 5/25/2012 4:47:32 PM

DOC: 285380–04–090–WIP3–0301

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

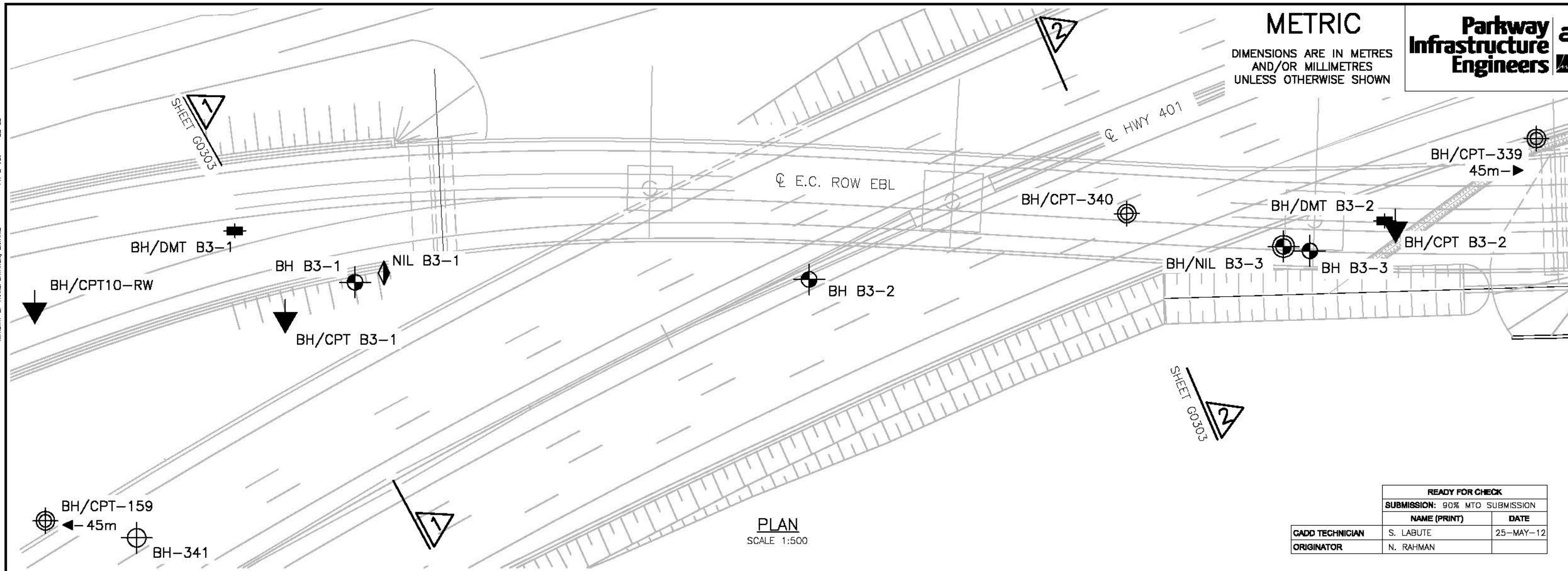
Windsor–Essex Parkway Project RFP No. 09–54–1007

NEW CONSTRUCTION
BRIDGE B–3
REALIGNED E.C. ROW–EBL EXPRESSWAY UNDERPASS NEAR MATCHETTE ROAD
BOREHOLE LOCATIONS & SOIL STRATA

SHEET G0302

Phase 3

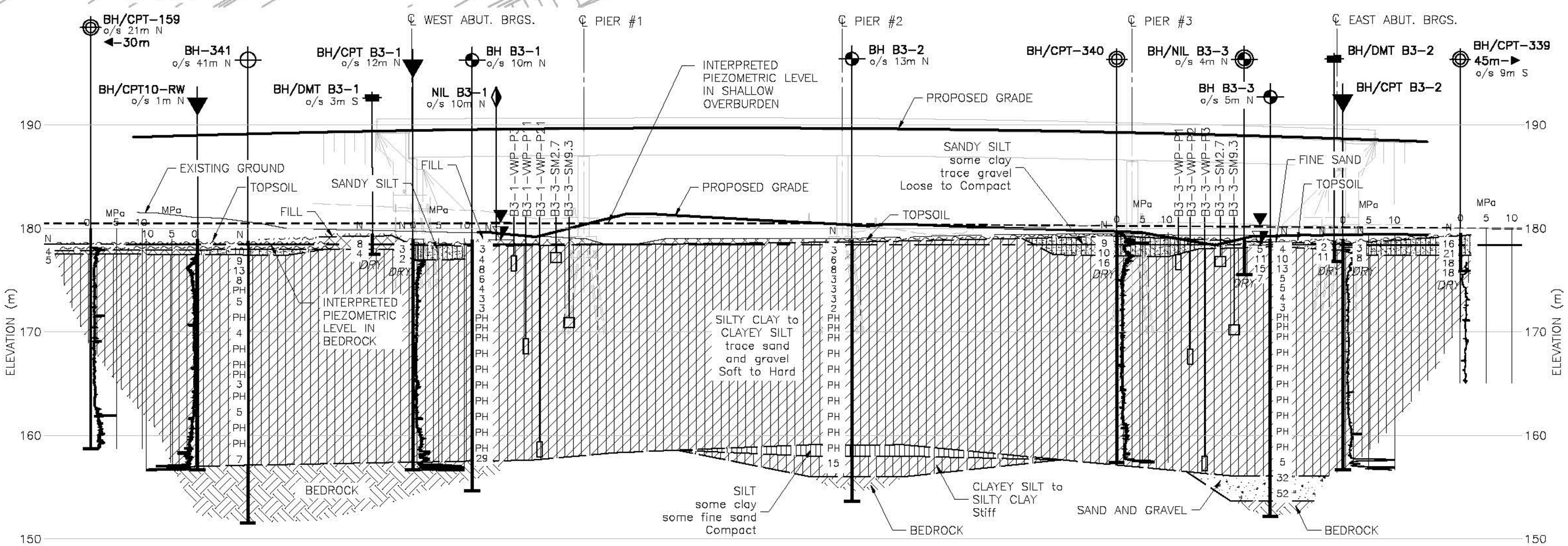
90% Sub



PLAN

SCALE 1:500

READY FOR CHECK		
SUBMISSION:	90% MTO SUBMISSION	
	NAME (PRINT)	DATE
CADD TECHNICIAN	S. LABUTE	25–MAY–12
ORIGINATOR	N. RAHMAN	



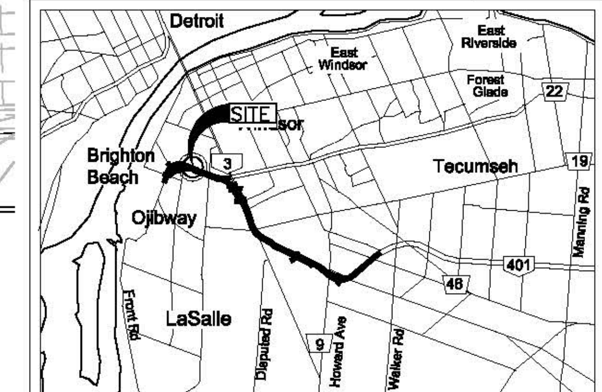
PROFILE ALONG E.C. ROW EXPRESSWAY EBL THROUGH BRIDGE B–3

HORT SCALE 1:500
VERT SCALE 1:250

No.	ELEVATION	CO–ORDINATES (UTM, NAD 83 ZONE 17)	
		NORTHING	EASTING
AMEC BOREHOLES			
BH B3–1	178.9	4682267.8	329431.6
BH B3–2	178.9	4682224.9	329491.0
BH B3–3	179.0	4682180.9	329559.0
BH/DMT B3–1	179.5	4682286.4	329420.5
BH/DMT B3–2	179.2	4682177.6	329571.6
BH/CPT B3–1	179.0	4682270.6	329419.6
BH/CPT B3–2	179.1	4682176.2	329573.0
BH/NIL B3–3	179.0	4682184.0	329556.0
BH/CPT10–RW	179.3	4682295.7	329387.8
NIL B3–1	178.9	4682266.3	329436.2
PREVIOUS BOREHOLES			
BH–341	178.8	4682255.5	329378.7
BH/CPT–159	178.8	4682292.8	329332.1
BH/CPT–339	179.5	4682147.4	329635.6
BH/CPT–340	179.6	4682203.2	329538.7

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING



KEY PLAN

SCALE
1 0 2 4Km

LEGEND

- BOREHOLE CURRENT INVESTIGATION
- BOREHOLE AND NILCON VANE CURRENT INVESTIGATION
- SW/SP HOLE (HYDROGEOLOGY) CURRENT INVESTIGATION
- NILCON VANE CURRENT INVESTIGATION
- CPT – CURRENT INVESTIGATION
- DMT – CURRENT INVESTIGATION
- BOREHOLE PREVIOUS INVESTIGATION
- BOREHOLE, CPT AND NILCON VANE PREVIOUS INVESTIGATION
- CPT –PREVIOUS INVESTIGATION
- N SPT N–VALUE
- 16 BLOWS/0.3m UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
- MHSG – MAGNETIC HEAVE/SETTLEMENT GAUGE
- P – VIBRATING WIRE PIEZOMETER
- DRY BOREHOLE DRY DURING DRILLING
- WATER LEVEL DURING DRILLING
- WATER LEVEL (SHALLOW PIEZO)
- WATER LEVEL (DEEP PIEZO)
- MPa 0 5 10 CPT, qc

NOTES

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LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

REVISIONS			
25–MAY–12	A	NR	90% MTO SUBMISSION
DATE	REV.	BY	DESCRIPTION

DESIGN SF	CHK SF	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN SC	CHK NSV	SITE 6–603	DATE 01–DEC–11

DOC: 285380–04–090–WIP3–0302

DATE PLOTTED: 5/25/2012 4:50:19 PM
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MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

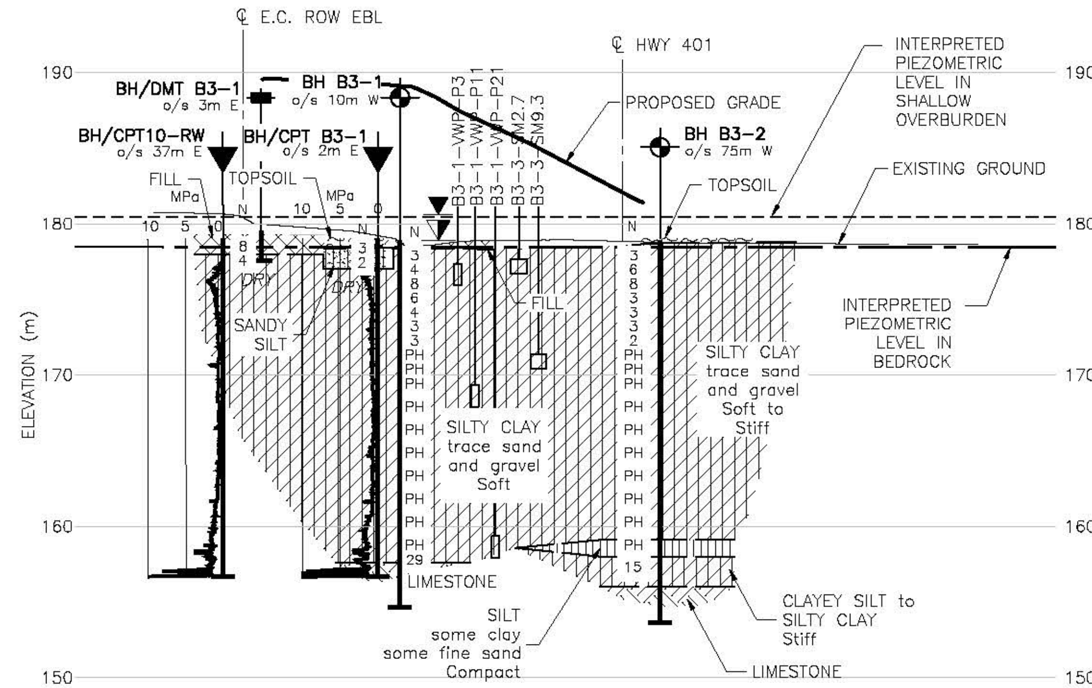
Windsor-Essex Parkway Project RFP No. 09-54-1007

NEW CONSTRUCTION
BRIDGE B-3
REALIGNED E.C. ROW-EBL EXPRESSWAY UNDERPASS NEAR MATCHETTE ROAD
SOIL STRATIGRAPHY

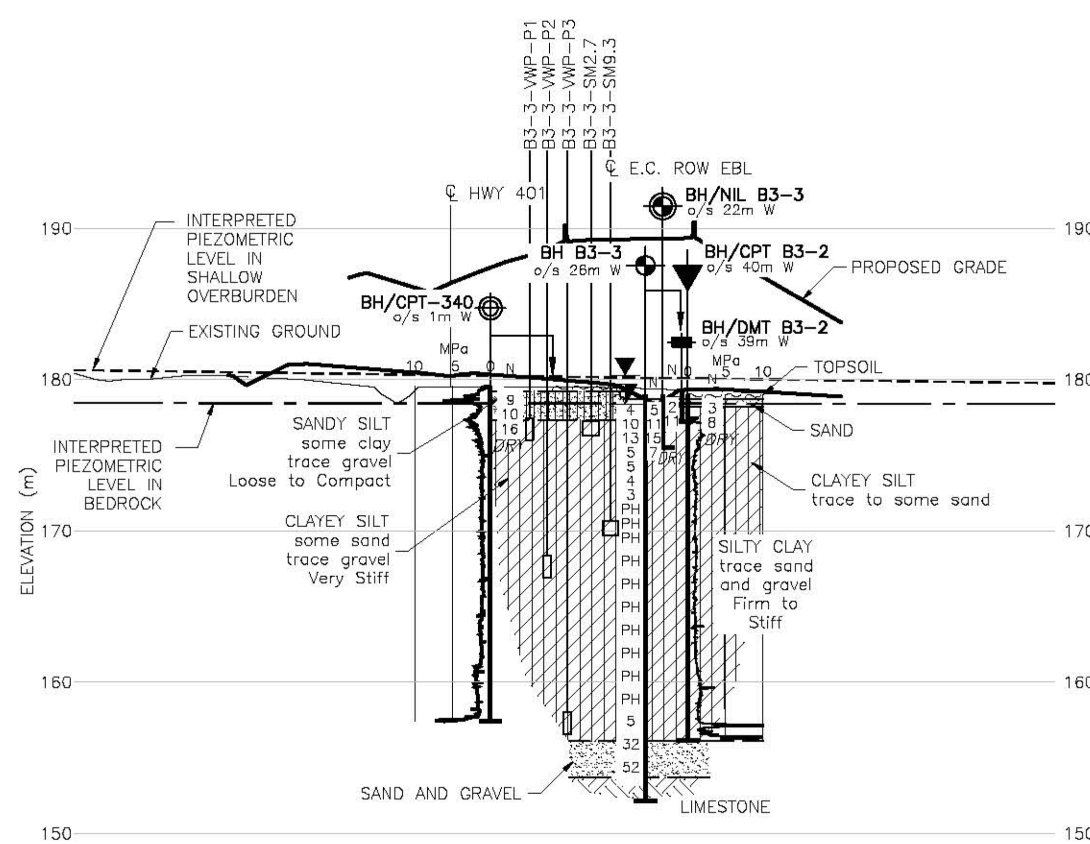
SHEET G0303

Phase 3

90% Sub

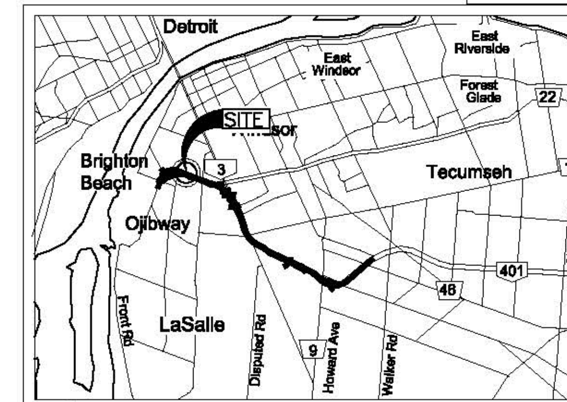


1
HORT SCALE 1:500
VERT SCALE 1:250



2
HORT SCALE 1:500
VERT SCALE 1:250

READY FOR CHECK		
SUBMISSION:	90% MTO SUBMISSION	
	NAME (PRINT)	DATE
CADD TECHNICIAN	S. LABUTE	25-MAY-12
ORIGINATOR	N. RAHMAN	



KEY PLAN
SCALE 1 0 2 4Km

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- SAND
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3. ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

DRAWING NOT TO BE SCALED 100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

REVISIONS				
25-MAY-12	A	NR	90% MTO SUBMISSION	
DATE	REV.	BY	DESCRIPTION	

DESIGN SF	CHK SF	CODE CAN/CSA S6-06	LOAD CL-625-ONT
DRAWN SC	CHK NSV	SITE 6-603	DATE 01-DEC-11

DOC: 285380-04-091-WIP3-0303

MINISTRY OF TRANSPORTATION, ONTARIO PR-D-707 89-06

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METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

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

NEW CONSTRUCTION
BRIDGE B–3
REALIGNED E.C. ROW–EBL EXPRESSWAY UNDERPASS NEAR MATCHETTE ROAD
CONSTRUCTION NOTES – BACKFILL AT STRUCTURES

SHEET G0372

Phase 3

90% Sub

CONSTRUCTION NOTES – BACKFILL AT STRUCTURES

1.0 GENERAL REQUIREMENTS

1.1. THESE CONSTRUCTION NOTES RELATE TO THE SUPPLY AND PLACEMENT OF BACKFILL MATERIALS AT THE STRUCTURES AT THE WINDSOR–ESSEX PARKWAY (WEP) PROJECT AS ILLUSTRATED ON THE ACCOMPANYING DRAWINGS. THE REQUIREMENTS GIVEN HEREAFTER ARE THE GENERAL REQUIREMENTS. FOR DETAILED REQUIREMENTS, THE CONTRACTOR SHOULD REFER TO APPROPRIATE ONTARIO PROVINCIAL STANDARD SPECIFICATIONS (OPSS) LISTED IN SECTION 1.6.

1.2. THESE CONSTRUCTION NOTES ARE TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN DRAWINGS AND REPORT.

1.3. FOR LIGHTWEIGHT FILL (LWF), REFER TO CONSTRUCTION NOTES FOR LIGHTWEIGHT FILL MATERIAL.

1.4. FOR EXPANDED POLYSTYRENE (GEOFOAM, EPS) FILL, REFER TO CONSTRUCTION NOTES FOR EXPANDED POLYSTYRENE FILL.

1.5. THESE REQUIREMENTS DO NOT APPLY TO THE HIGHWAY PAVEMENT CONSTRUCTION.

1.6. THE CONSTRUCTION WORKS SHALL BE EXECUTED IN ACCORDANCE WITH THE GEOTECHNICAL DESIGN ILLUSTRATED ON THE ACCOMPANYING DRAWINGS, THE SUPPLIER SPECIFICATIONS AND THE REQUIREMENTS SPECIFIED IN THE FOLLOWING STANDARDS, SPECIFICATIONS AND PUBLICATIONS:

- ASTM D422 — PARTICLE–SIZE ANALYSIS OF SOILS
- ASTM D2216 — MOISTURE CONTENT OF SOILS
- ASTM D2850 — UNCONSOLIDATED–UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
- ASTM D2922 — DENSITY OF SOIL AND SOIL–AGGREGATE IN PLACE BY NUCLEAR METHODS
- ASTM D3017 — WATER CONTENT OF SOIL AND ROCK IN PLACE BY NUCLEAR METHODS
- ASTM D5856 — HYDRAULIC CONDUCTIVITY OF POROUS MATERIALS USING A RIGID WALL PERMEAMETER

- OPSS 201 — CLEARING, CLOSE CUT CLEARING, GRUBBING, REMOVAL OF SURFACE AND PILES BOULDERS
- OPSS 206 — GRADING
- OPSS 212 — BORROW
- OPSS 401 — TRENCHING, BACKFILLING AND COMPACTING
- OPSS 501 — COMPACTING
- OPSS 517 — DEWATERING AT PIPELINE, UTILITY AND ASSOCIATED STRUCTURE EXCAVATION
- OPSS 518 — CONTROL OF WATER FROM DEWATERING OPERATIONS
- OPSS 805 — TEMPORARY EROSION AND SEDIMENT CONTROL MEASURES
- OPSS 902 — CONSTRUCTION SPECIFICATIONS FOR EXCAVATING AND BACKFILLING – STRUCTURES
- OPSS 1001 — AGGREGATES – GENERAL
- OPSS 1004 — AGGREGATES – MISCELLANEOUS
- OPSS 1010 — AGGREGATES – BASE, SUBBASE, SELECT SUBGRADE AND BACKFILL MATERIAL
- OPSS 1860 — GEOTEXTILE
- OPSD 208.010 — BENCHING OF EARTH SLOPES

1.7 IF THERE IS ANY CONFLICT BETWEEN THE REQUIREMENTS GIVEN ON THIS DRAWING AND THE STANDARDS AND SPECIFICATIONS DOCUMENTS LISTED IN SECTION 1.6, THE DESIGNER SHOULD BE CONSULTED FOR CLARIFICATION AND RECOMMENDATIONS.

1.8 IN THE FOLLOWING CONSTRUCTION NOTES, THE CONTRACTOR MEANS PIC AND ITS SUB–CONTRACTORS, THE SUPPLIER MEANS THE MANUFACTURER AND PROPRIETARY SUPPLIER, THE ENGINEER MEANS THE GEOTECHNICAL SITE ENGINEER, AND THE DESIGNER MEANS THE GEOTECHNICAL DESIGNER OF THE PROJECT.

2.0 SITE PREPARATION AND EXCAVATION

2.1 CLEARING AND GRUBBING AREA SHALL EXTEND MINIMUM 3 m BEYOND THE FOOTPRINT AREA OF THE STRUCTURE, OR AS REQUIRED BY THE ENGINEER. THE TREES AND SHRUBS REMOVED FROM THE GROUND SHALL BE TRANSPORTED TO DESIGNATED AREAS.

2.2 THE STRIPPING AREA SHALL EXTEND MINIMUM 1 m BEYOND THE FOOTPRINT AREA OF THE STRUCTURE, OR AS REQUIRED BY THE ENGINEER. ALL PEAT/MUSKEG, WETLAND VEGETATION AND OTHER UNSUITABLE MATERIAL SHOULD BE STRIPPED AND TRANSPORTED TO DESIGNATED AREAS.

2.3 CONTRACTOR IS FULLY RESPONSIBLE FOR THE DESIGN, CONSTRUCTION METHODS AND PERFORMANCE OF THE TEMPORARY SLOPES AND WORKS.

2.4 ALL EXCAVATION WORKS SHOULD BE CARRIED OUT IN ACCORDANCE WITH THE GUIDELINES OUTLINED IN OCCUPATIONAL HEALTH AND SAFETY ACT (OHSA) AND ONTARIO PROVINCIAL STANDARD SPECIFICATION (OPSS) 902. NATIVE DEWATERED SOILS AT THE SITE AND COMPACTED FILLS MAY BE CLASSIFIED IN GENERAL AS TYPE 3 SOILS. UNDEWATERED FILLS, NATIVE SAND AND SILTS, AND WATER BEARING BACKFILL WITHIN TRENCHES OF ACTIVE AND/OR ABANDONED UTILITIES MAY DEVELOP TYPE 4 SOIL CONDITIONS AND SHALL BE ADDRESSED ACCORDINGLY.

2.5 THE SOILS AT THE PROJECT SITE ARE HIGHLY SUSCEPTIBLE TO RAPID DETERIORATION WHEN EXPOSED TO ELEMENTS, WEATHERING, WATER INFLOW AND PONDING, DISTURBANCE FROM CONSTRUCTION TRAFFIC, AND THE LIKE. SUBGRADE SOILS AND BACKFILL IN PROGRESS SHALL BE APPROPRIATELY PROTECTED AT ALL TIMES AGAINST SURFACE EROSION, DESICCATION, AND FREEZE–THAW EFFECTS, REGULARLY INSPECTED AND MONITORED, AND TREATED AS REQUIRED.

2.6 TO PROTECT THE SUBGRADE INTEGRITY, THE FINAL EXCAVATION LAYER ABOVE THE DESIGN ELEVATION IN GENERAL SHOULD NOT BE LESS THAN 0.5 m AND SHOULD BE CARRIED OUT ONLY WHEN THE CONTRACTOR IS READY TO PREPARE AND COVER/PROTECT THE SUBGRADE SAME DAY THE FINAL EXCAVATION IS EXPOSED AND APPROVED.

2.7 NO CONSTRUCTION TRAFFIC SHOULD BE PERMITTED OVER THE SUBGRADE WITHOUT APPROVED PROTECTIVE COVERS.

2.8 THE SUBGRADE EXCAVATION SHALL BE CUT TO NEAT LINES AND GRADES USING BUCKETS EQUIPPED WITH SMOOTH LIPS. ONCE EXPOSED, THE SUBGRADE MUST BE IMMEDIATELY INSPECTED. UPON APPROVAL, THE SUBGRADE SURFACE SHOULD BE COVERED WITH SKIM COAT OF LEAN CONCRETE MUD MAT, GRANULAR OVER GEO–FABRIC, GRANULAR OVER SUBGRADE, ETC., AS APPROVED BY THE ENGINEER, FOR PROTECTION AGAINST DISTURBANCE AND TO PROVIDE A WORKING SURFACE.

2.9 THE TEMPORARY EXCAVATION SURFACES SHALL BE BENCHED ACCORDING TO OPSD 208.010. UNLESS THE GRANULAR BACKFILL IS FILTER GRADED WITH RESPECT TO THE NATIVE SUBGRADE MATERIAL, A GEOTEXTILE LAYER (TERRAFIX 360R OR EQUIVALENT) SHALL BE PLACED AT THE BENCHED INTERFACE BETWEEN THE EXCAVATED SURFACE AND THE GRANULAR BACKFILL TO FUNCTION AS A SEPARATOR AND PREVENT MIGRATION OF FINES.

2.10 IF PRESENCE OF GASSY SOILS IS EVIDENCED (FOR EXAMPLE, DISSOLVED GAS BUBBLES COMING OUT OF SOLUTION AND/OR SOFTENING OF THE EXCAVATION FACE), THE EXCAVATION PROGRESS SHALL BE REVIEWED WITH THE ENGINEER IN TERMS OF TIMING, STAGING AND OTHER MITIGATION MEASURES.

2.11 THE CONTRACTOR SHOULD EMPLOY APPROPRIATE GROUND IMPROVEMENT APPROACH (E.G., SUITABLE FILL LAYER, GEOGRID SHEET, ETC.) TO FACILITATE CONSTRUCTABILITY, WHERE REQUIRED, AS APPROVED BY THE ENGINEER.

2.12 THE SUBGRADE SHOULD BE SLOPED APPROPRIATELY TO ACHIEVE POSITIVE DRAINAGE OF SEEPAGE AND SURFACE WATER TO SUBDRAINS, DITCHES OR SUMPS TO AVOID PONDING BENEATH ANY FILL PLACED. NO PONDING OR FLOODING SHALL BE ALLOWED TO OCCUR IN AREAS OF FINAL EARTHWORKS (SEE SECTION 6 ON DRAINAGE – REQUIREMENTS).

3.0 REINFORCED GRANULAR MAT (RGM)

3.1 THE RGM ARE REINFORCED SOIL MATS COMPRISING SELECT COMPACTED GRANULAR FILL AND REINFORCEMENT (GEOSYNTHETICS OR METALLIC)

3.2 GRANULAR FILL FOR RGM: THE FILL MATERIAL SHALL BE GRANULAR 'A' OR GRANULAR 'B' TYPE II (OPSS 1010) PLACED AS PER NOTE 5.3 AND COMPACTED TO NOT LESS THAN 98%.

3.3 REINFORCEMENT FOR RGM: AS PER CONTRACT DOCUMENTS.

4.0 FILL MATERIALS

4.1 ALL FILL MATERIALS TO BE USED FOR TUNNEL AND BRIDGE CONSTRUCTION SHALL BE INERT MATERIAL, FREE OF ORGANIC MATERIAL AND DELETERIOUS SUBSTANCES. ALL FILL MATERIALS SHALL BE APPROVED BY THE ENGINEER AT THE BORROW SOURCE AND AT PLACEMENT LOCATION.

4.2 SILTY CLAY FILL: THE UPPER CLAY CRUST ZONE MATERIAL OBTAINED FROM REQUIRED EXCAVATIONS IN THE DEPRESSED SEGMENTS OF THE WEP OR OTHER SOURCES APPROVED BY THE ENGINEER SHALL BE USED AS PER DRAWINGS PROVIDED IT MEETS THE OPSS 902 REQUIREMENTS AND CAN BE COMPACTED TO AT LEAST 95% SPMDD. THE SUITABILITY OF THE CLAY FILL MATERIALS SHALL BE VERIFIED IN TERMS OF ITS GRADATION (E.G., SILTY CLAY TO CLAYEY SILT), PLASTICITY CHARACTERISTICS (LOW TO MEDIUM PLASTICITY INDEX) AND THE IN–SITU MOISTURE CONTENT. ALL SUITABLE METHODS TO ACHIEVE THE SPECIFIED PLACEMENT MOISTURE CONTENT SHALL BE EMPLOYED.

4.3 GRANULAR FILL FOR GENERAL BACKFILL: THE GRANULAR FILL MATERIAL SHALL BE GRANULAR 'B' TYPE I OR II, OR ALTERNATIVE GRANULAR MATERIALS APPROVED BY THE ENGINEER. THE SUITABILITY OF GRANULAR FILL MATERIALS SHALL BE DETERMINED AS PER THE OPSS 1010 STANDARD AND THE REQUIREMENTS OF THE RSS/RGM SUPPLIER.

4.4 RIPRAP: THE RIPRAP MATERIAL FOR EROSION PROTECTION OF PERMANENT SLOPES AND CHANNEL SURFACES SHALL BE R–10 (MINUS 180 mm) FOR LIGHT TO MEDIUM EROSION RISK CONDITIONS AND R–50 (MINUS 305 mm) FOR HIGH RISK CONDITIONS, AS SHOWN ON THE DESIGN DRAWINGS OR AS REQUIRED BY THE ENGINEER (OPSS 1004). GEOTEXTILE SHALL BE USED AT INTERFACE BETWEEN THE SOIL SLOPES AND RIPRAP LAYER TO PREVENT LOSS OF MATERIAL FROM THE SOIL SLOPE.

4.5 LWF AND EPS: SEE RESPECTIVE CONSTRUCTION NOTES.

5.0 FILL PLACEMENT AND COMPACTION

5.1 GENERAL:

- THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER THEIR QC/QA INSPECTION AND TEST PLAN FOR REVIEW/COMMENT PRIOR TO THE PLACEMENT/COMPACTION OF FILL.
- FILL SHALL NOT BE PLACED ON SURFACES HAVING STANDING WATER, OR SURFACES WHICH HAVE BEEN RUTTED AND HEAVED BY TRAFFICKING. FILL SHALL NOT BE PLACED ON FROZEN SURFACES. FROZEN FILL IS DEFINED AS MATERIALS WITH SOIL WATER IN FROZEN STATE.
- ALL EARTHWORKS TO BE ADEQUATELY PROTECTED AGAINST EROSION, FROST AND WATER INGRESS UNTIL THE LANDSCAPING REQUIREMENTS HAVE BEEN INSTALLED (SEE SECTIONS 2.6 TO 2.8).

5.2 IF NOT SPECIFIED IN THE CONTRACT DOCUMENTS, TARGET DENSITIES WILL BE ESTABLISHED UTILIZING CONTROL STRIPS AS PRESENTED IN OPSS 501. THE MINIMUM TARGET DENSITIES SHALL BE AS PER NOTES 5.3 AND 5.4.

5.3 THE SILTY CLAY FILL SHALL BE PLACED IN MAXIMUM 200 mm THICK LOOSE LIFTS AND COMPACTED AT WOPT±2% MOISTURE CONTENT TO A MINIMUM OF 95% SPMDD UNLESS OTHERWISE SPECIFIED IN THE CONTRACT DOCUMENTS. THE TERMS WOPT AND SPMDD REFER TO OPTIMUM WATER CONTENT AND MAXIMUM DRY DENSITY, RESPECTIVELY, DETERMINED BY STANDARD PROCTOR TESTS.

5.4 THE GRANULAR FILL MATERIALS SHALL BE PLACED IN MAXIMUM 300 mm THICK LOOSE LIFTS AND COMPACTED AT WOPT±2% MOISTURE CONTENT TO A MINIMUM OF 95% SPMDD UNLESS OTHERWISE SPECIFIED IN THE CONTRACT DOCUMENTS.

5.5 THE COMPACTION EQUIPMENT SHALL BE APPROPRIATE FOR THE MATERIAL TO BE COMPACTED AND THE SITE CONDITIONS, AND SHOULD BE PROPOSED TO THE ENGINEER FOR APPROVAL. ADEQUATE NUMBER OF PASSES SHALL BE EMPLOYED TO ACHIEVE THE SPECIFIED PLACEMENT DENSITIES. HEAVY COMPACTION EQUIPMENT SHOULD NOT BE EMPLOYED NEAR STRUCTURAL WALLS.

5.6 COMPACTION AND PLACEMENT OF GRANULAR MATERIALS FOR RSS WALLS SHALL CONFORM TO THE MANUFACTURER'S RECOMMENDATIONS.

5.7 FILL PLACEMENT SHALL CONFORM TO THE REQUIREMENTS PRESENTED IN OPSS 501. THE CONTRACTOR SHOULD USED APPROPRIATELY SIZED EQUIPMENT TO AVOID DAMAGING ANY STRUCTURES, DEGRADING THE AGGREGATE, OR EPS BLOCKS.

6.0 DRAINAGE – DEWATERING

6.1 REFER TO OPSS 518 FOR DEWATERING REQUIREMENTS.

6.2 THE CONSTRUCTION SITE WILL BE KEPT CLEAN AND DRY, FREE OF WATER PUDDLES, MUD AND DEBRIS.

6.3 MINOR TO SIGNIFICANT SEEPAGE FROM RUNOFF INFILTRATIONS OR PERCHED WATER WITHIN UPPER GRANULAR DEPOSITS AND/OR FILL IS ANTICIPATED. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE TEMPORARY DEWATERING SYSTEM.

7.0 USE

7.1 THIS DRAWING PROVIDES CONSTRUCTION REQUIREMENTS FOR GEOTECHNICAL ASPECTS OF BACKFILLING AT BRIDGES.

	READY FOR CHECK	
	SUBMISSION: 90% MTO SUBMISSION	
	NAME (PRINT)	DATE
CADD TECHNICIAN	S. LABUTE	25–MAY–12
ORIGINATOR	N. RAHMAN	

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

REVISIONS			
25–MAY–12	A	NR	90% MTO SUBMISSION
DATE	REV.	BY	DESCRIPTION

DESIGN SF	CHK NSV	CODE CAN/CSA S6–06	LOAD CL–625–ONT
DRAWN MM	CHK DD	SITE 6–603	DATE 20–DEC–11

DOC: 285380–04–094–WIP3–0372

MINISTRY OF TRANSPORTATION, ONTARIO PH–D–707 86–08

DATE PLOTTED: 5/25/2012 4:54:46 PM
FILE LOCATION: C:\pwworking\hmmg_285380\stephen.labute@amec.com\dms09845\285380–04–094–WIP3–0372.dwg

CONSTRUCTION NOTES – LIGHTWEIGHT FILL MATERIAL

1.0 GENERAL REQUIREMENTS

1.1. THE CONSTRUCTION NOTES ON THIS DRAWING COVER THE REQUIREMENTS FOR THE SUPPLY AND PLACEMENT OF WATER COOLED ULTRA LIGHTWEIGHT BLAST FURNACE SLAG TO BE USED FOR CONSTRUCTION OF THE STRUCTURES FOR THE WINDSOR-ESSEX PARKWAY (WEP) PROJECT. AT THE WEP PROJECT, THE ULTRA LIGHTWEIGHT BLAST FURNACE SLAG MATERIAL IS GENERALLY REFERRED TO AS THE LIGHT WEIGHT FILL (LWF).

1.2. THESE CONSTRUCTION NOTES ARE TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING DESIGN DRAWING(S), OTHER RELEVANT CONSTRUCTION NOTES AND GEOTECHNICAL REPORT.

1.3. THE CONSTRUCTION WORKS SHALL BE EXECUTED IN ACCORDANCE WITH THE DESIGN ILLUSTRATED ON THE ACCOMPANYING DRAWINGS, AND THE REQUIREMENTS SPECIFIED IN THE FOLLOWING STANDARDS, SPECIFICATIONS AND PUBLICATIONS:

- MTO — NSSP ULTRA LIGHTWEIGHT BLAST FURNACE SLAG (WATER COOLED)
- ASTM D422 — PARTICLE-SIZE ANALYSIS OF SOILS
- ASTM D2216 — MOISTURE CONTENT OF SOILS
- ASTM D2850 — UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS
- ASTM D2922 — DENSITY OF SOIL AND SOIL-AGGREGATE IN PLACE BY NUCLEAR METHODS
- ASTM D3017 — WATER CONTENT OF SOIL AND ROCK IN PLACE BY NUCLEAR METHODS
- OPSS 212 — BORROW
- OPSS 501 — COMPACTION
- OPSS 517 — DEWATERING
- OPSS 1010 — AGGREGATES-BASE, SUBBASE, SELECT SUBGRADE, AND BACKFILL MATERIAL
- OPSS 1860 — GEOTEXTILES

1.4 IF THERE IS ANY CONFLICT BETWEEN THE REQUIREMENTS GIVEN ON THIS DRAWING AND THE STANDARDS AND SPECIFICATIONS DOCUMENTS LISTED IN SECTION 1.3, THE DESIGNER SHOULD BE CONSULTED FOR CLARIFICATION AND RECOMMENDATIONS.

1.5 IN THE FOLLOWING SPECIFICATIONS, THE CONTRACTOR MEANS PIC AND ITS SUB-CONTRACTORS, AND THE ENGINEER MEANS THE GEOTECHNICAL SITE ENGINEER, AND THE DESIGNER MEANS THE GEOTECHNICAL DESIGNER OF THE PROJECT.

2.0 SITE PREPARATION AND EXCAVATION

2.1 THE SITE PREPARATION AND EXCAVATION REQUIREMENTS ON THE CONSTRUCTION NOTES FOR THE BACKFILL AT STRUCTURES ARE APPLICABLE.

3.0 SUBMISSION AND DESIGN REQUIREMENTS

3.1 THE CONTRACTOR SHALL SUBMIT TO PIC AND THE ENGINEER CERTIFICATES OF CONFORMANCE SEALED AND SIGNED BY THE QUALITY VERIFICATION ENGINEER AS FOLLOWS:

a. PRIOR TO THE PLACEMENT OF THE LIGHTWEIGHT FILL MATERIAL ON THE PROJECT, THE CONTRACTOR SHALL SUBMIT TO THE CONTRACT ADMINISTRATOR A CERTIFICATE OF CONFORMANCE STATING THAT THE MATERIAL SATISFIES THE MATERIAL PROPERTIES SPECIFIED IN SECTION 4.1.

b. FOLLOWING FILL PLACEMENT, THE CONTRACTOR SHALL SUBMIT TO THE CONTRACT ADMINISTRATOR A CERTIFICATE OF CONFORMANCE STATING THAT THE MATERIAL SATISFIES THE REQUIREMENTS OF THIS SPECIFICATION AND THAT THE WORK HAS BEEN CARRIED OUT IN GENERAL CONFORMANCE WITH THE CONTRACT DOCUMENTS AND SPECIFICATIONS. THE CONTRACTOR SHALL ALSO SUBMIT ALL QUALITY CONTROL TEST RESULTS FOR INFORMATION ONLY.

4.0 MATERIAL

4.1 THE LWF SHALL SATISFY THE FOLLOWING PHYSICAL, MECHANICAL AND CHEMICAL PROPERTY REQUIREMENTS:

- ANGLE OF INTERNAL FRICTION — >35° (ASTM 2850-85)
- HYDRAULIC CONDUCTIVITY — >8 E-03 CM/S (ASTM 5856-95, METHOD A)
- CHEMICAL COMPOSITION — THE MATERIAL SHALL MEET THE LEACHATE CRITERIA ESTABLISHED UNDER ONTARIO REGULATIONS 347
- IN SITU WET UNIT WEIGHT — <12.5 kN/m³ (ASTM D2922) (MAXIMUM WHEN PLACED AND COMPACTED IN ACCORDANCE WITH THE SPECIFICATIONS)

5.0 CONSTRUCTION

5.1 THE LWF (BLAST FURNACE SLAG) IS SUSCEPTIBLE TO CRUSHING IF OVERCOMPACTED AND CAREFUL CONSTRUCTION PROCEDURES AND SUPERVISION ARE REQUIRED. THE CONTRACTOR SHALL PLACE THE LWF MATERIAL AND SHALL ACHIEVE COMPACTION WITHOUT CRUSHING THE MATERIAL SINCE CRUSHING INCREASES ITS UNIT WEIGHT. THE CONTRACTOR SHALL PLACE THE LWF MATERIAL WITHOUT EXCEEDING THE SPECIFIED IN SITU UNIT WEIGHT AND MAINTAINING CRUSHING OF THE MATERIAL BELOW 5%.

5.2 TO PREVENT OVER-CRUSHING AND OVER-COMPACTION, THE LWF SHALL BE PLACED AS FOLLOWS:

a. FOR EMBANKMENTS THE LWF SHALL BE PLACED IN LIFTS OF 300 mm AND COMPACTED BY 3 PASSES OF SINGLE DRUM VIBRATORY EQUIPMENT APPROVED BY THE ENGINEER (E.G., BOMAG 142 OR EQUIVALENT, TABLE 1).

b. FOR BACKFILL TO STRUCTURES, THE LWF SHALL BE PLACED IN LIFTS OF 300 mm AND COMPACTED WITH 8 PASSES OF MANUALLY GUIDED TAMPER SUCH AS A BOMAG BPR 30/38 D OR EQUIVALENT (TABLE 1).

c. THE CONTRACTOR SHALL PLACE AND SPREAD THE LOOSE LIFTS USING A RUBBER TIRE FRONT-END LOADER SUCH AS A CATERPILLAR 980 F OR EQUIVALENT.

5.3 COMPACTION EQUIPMENT TECHNICAL DETAILS ARE PROVIDED IN TABLE 1.

5.4 THE LWF ZONES SHALL BE APPROPRIATELY WRAPPED IN GEOTEXTILE TO AVOID LOSS OF FINES FROM THE ADJACENT BACKFILL OR NATIVE MATERIALS IN CONTACT WITH THE LWF ZONES.

6.0 QUALITY CONTROL

6.1 QUALITY CONTROL (QC) TESTING SHALL BE CARRIED OUT BY THE CONTRACTOR TO ENSURE THAT THE LWF MATERIAL IS PLACED AND COMPACTED AS SPECIFIED. FIELD DENSITY AND FIELD MOISTURE DETERMINATION SHALL BE MADE IN ACCORDANCE WITH ASTM D2922 AND ASTM D3017, RESPECTIVELY.

6.2 THE CONTRACTOR SHALL BUILD A CONTROL STRIP TO VERIFY THAT THE PLACEMENT AND COMPACTION PROCEDURE WILL ACHIEVE THE REQUIREMENTS OF THESE SPECIFICATIONS WITHOUT EVIDENCE OF CRUSHING AND WITHOUT EXCEEDING THE SPECIFIED MAXIMUM IN SITU WET UNIT WEIGHT OF 12.5 kN/m³.

6.3 MATERIAL PLACED IN THE CONTROL STRIP SHALL HAVE THE MOISTURE CONTENT THAT WILL YIELD THE SPECIFIED IN-SITU UNIT WEIGHT. FOR THE CONTROL STRIP DETERMINATION, THE NUCLEAR GAUGE METHOD WILL NOT BE CONSIDERED AN ACCEPTABLE METHOD OF DETERMINING THE IN-SITU MOISTURE CONTENT OF THE LWF MATERIAL. MOISTURE CONTENT SHALL BE DETERMINED BY THE OVEN DRY METHOD ON SELECTED COMPACTED EMBANKMENT MATERIAL SAMPLES IN ACCORDANCE WITH ASTM D2216.

6.4 AFTER THE TRIAL AREA IS COMPLETE, SAMPLES FOR MOISTURE CONTROL AND IN SITU UNIT WEIGHT DETERMINATION TESTING SHALL BE AS PER ASTM D2922.

6.5 IN ADDITION, GRADATION AS PER ASTM D422-63 BEFORE AND AFTER COMPACTION EFFORT SHALL BE PERFORMED TO DETERMINE THAT CRUSHING IS KEPT WITHIN 5%.

6.6 THE REQUIREMENTS OF THE CONTROL STRIP MUST BE SATISFIED AS PART OF THE ACCEPTANCE CRITERIA OF ANY PROPOSED CHANGE TO THE SPECIFIED COMPACTION METHOD OF THIS SPECIAL PROVISION.

7.0 USE

7.1 THIS DRAWING PROVIDES CONSTRUCTION REQUIREMENTS FOR GEOTECHNICAL ASPECTS OF BACKFILLING AT BRIDGES.

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

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

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

NEW CONSTRUCTION
BRIDGE B-3
REALIGNED E.C. ROW-EBL EXPRESSWAY UNDERPASS NEAR MATCHETTE ROAD
CONSTRUCTION NOTES – LIGHTWEIGHT FILL MATERIAL

SHEET G0373
Phase 3
90% Sub

TABLE 1: COMPACTION EQUIPMENT TECHNICAL DETAILS

	BOMAG 142 D	BOMAG BPR 30/38 D
WEIGHTS		
• OPERATING WEIGHT (kg)	4690±	175±
• MASS PER SQUARE METRE OF BASE PLATE (kg/m²)	N/A	1439
DIMENSIONS		
• DRUM WIDTH (mm)	1426±	N/A
• DRUM DIAMETER (mm)	1058±	N/A
• WIDTH OF BASE PLATE (mm)	N/A	380
• LENGTH OF BASE PLATE (mm)	N/A	730
DRIVE		
• PERFORMANCE DIN 6271 IFN (kW)	37±	3.7
• PERFORMANCE SAE (kW)	39.5	N/A
• SPEED (RPM)	2300	3600
VIBRATORY SYSTEM		
• FREQUENCY (Hz)	32±	68±
• AMPLITUDE (mm)	1.24±	N/A
• CENTRIFUGAL FORCE (KN)	66±	30±

	READY FOR CHECK	
	SUBMISSION: 90% MTO SUBMISSION	
	NAME (PRINT)	DATE
CADD TECHNICIAN	S. LABUTE	25-MAY-12
ORIGINATOR	N. RAHMAN	

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

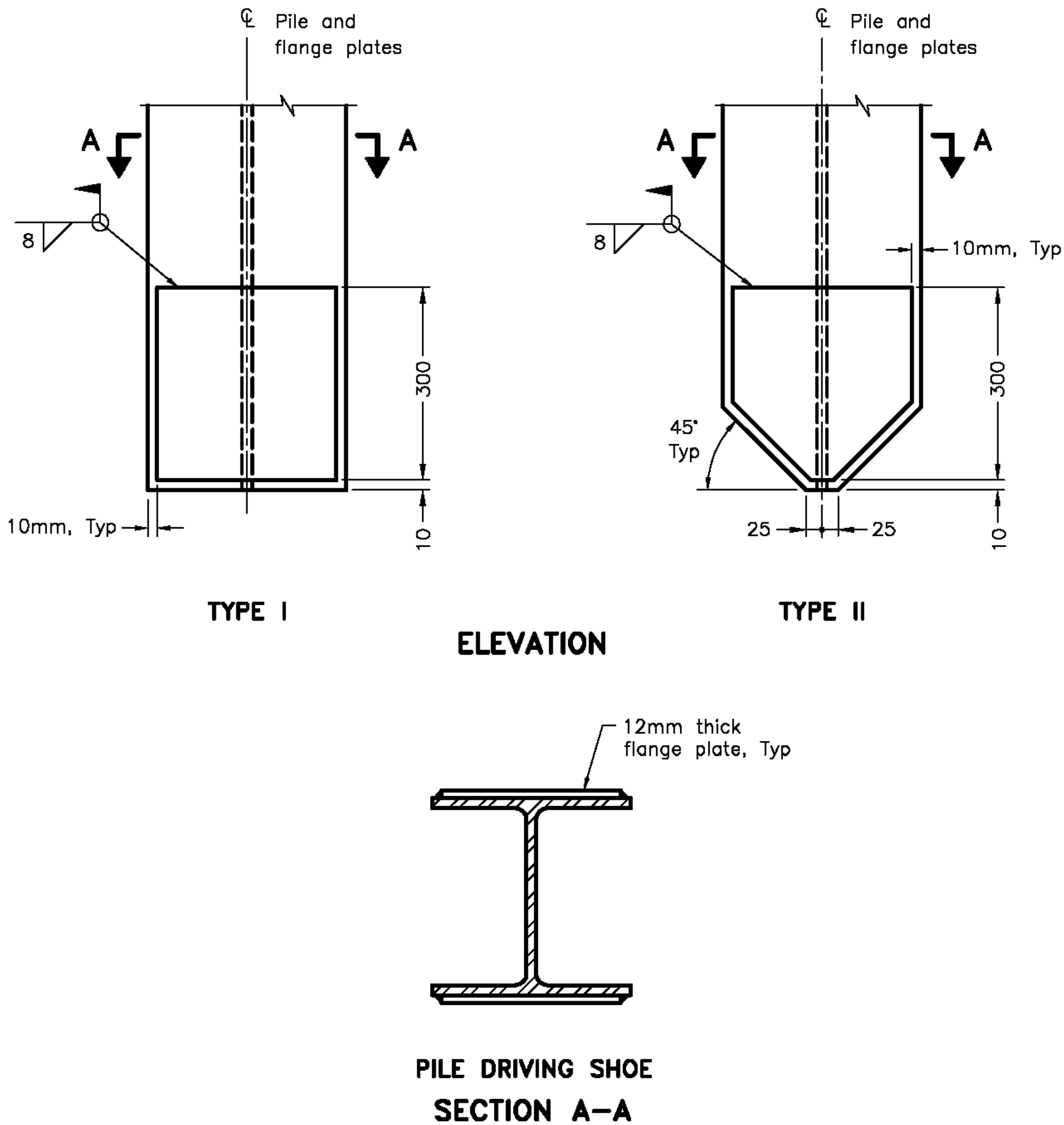
NOT FOR CONSTRUCTION

REVISIONS			
25-MAY-12	A	NR	90% MTO SUBMISSION
DATE	REV.	BY	DESCRIPTION

DESIGN SF	CHK NSV	CODE CAN/CSA S6-06	LOAD CL-625-ONT
DRAWN MM	CHK DD	SITE 6-603	DATE 20-DEC-11

DOC: 285380-04-094-WIP3-0373

Applicable OPSDs



NOTES:

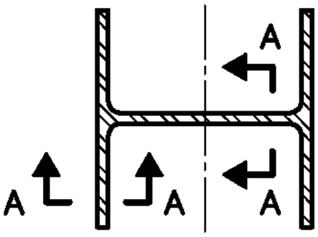
A Flange plates shall be according to CSA G40.20/G40.21, Grade 300W.

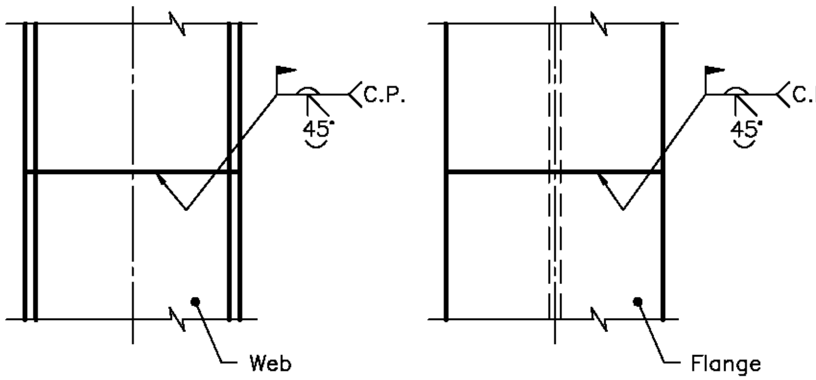
B Welding shall be according to CSA W59.

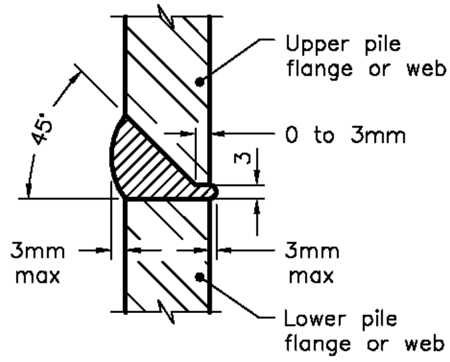
C Driving shoe Type I shall be used unless Type II is specified.

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	

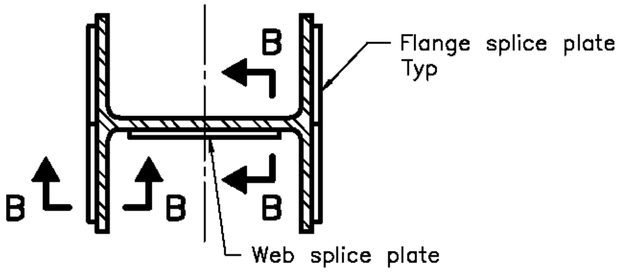


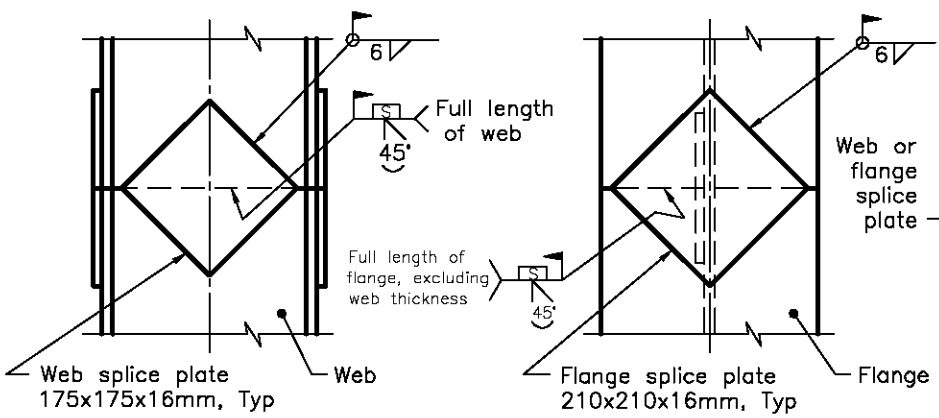


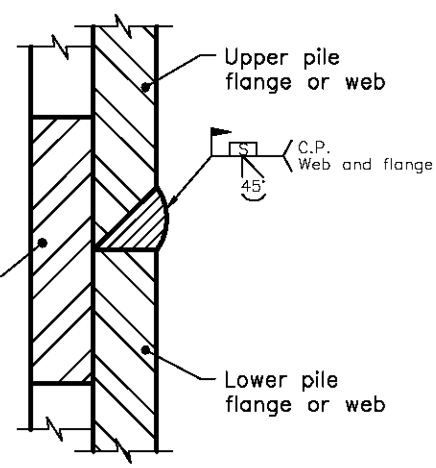


BUTT WELD

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.

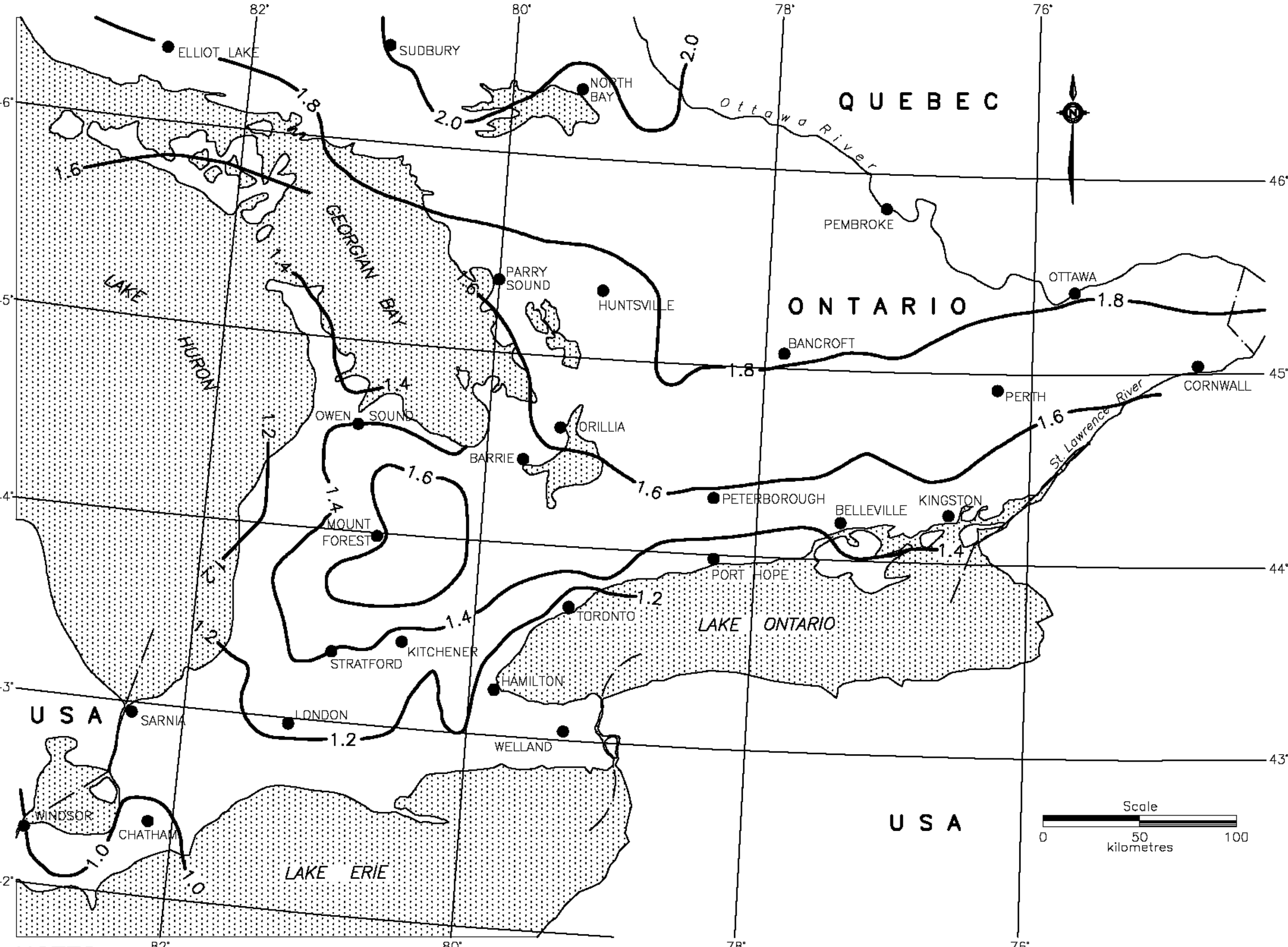
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
FOUNDATION PILES STEEL H–PILE SPLICE		OPSD 3000.150

ONTARIO PROVINCIAL STANDARD DRAWING

FOUNDATION FROST PENETRATION DEPTHS FOR SOUTHERN ONTARIO

Nov 2010 | Rev 1

OPSD 3090.101



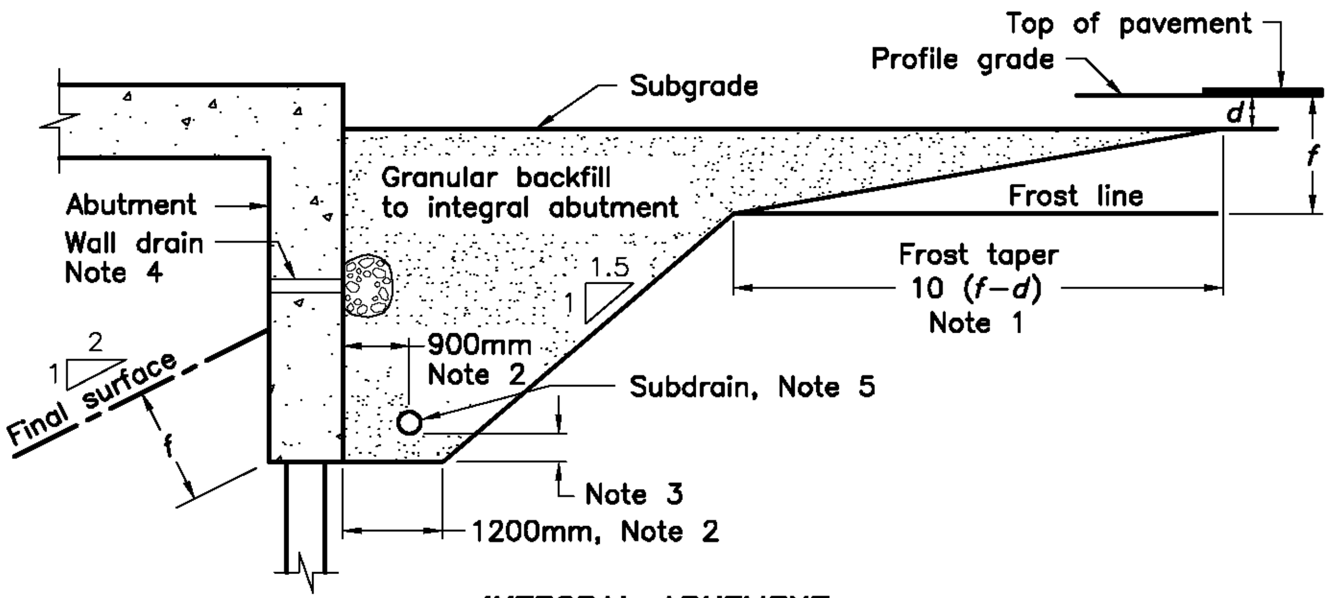
NOTES:

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

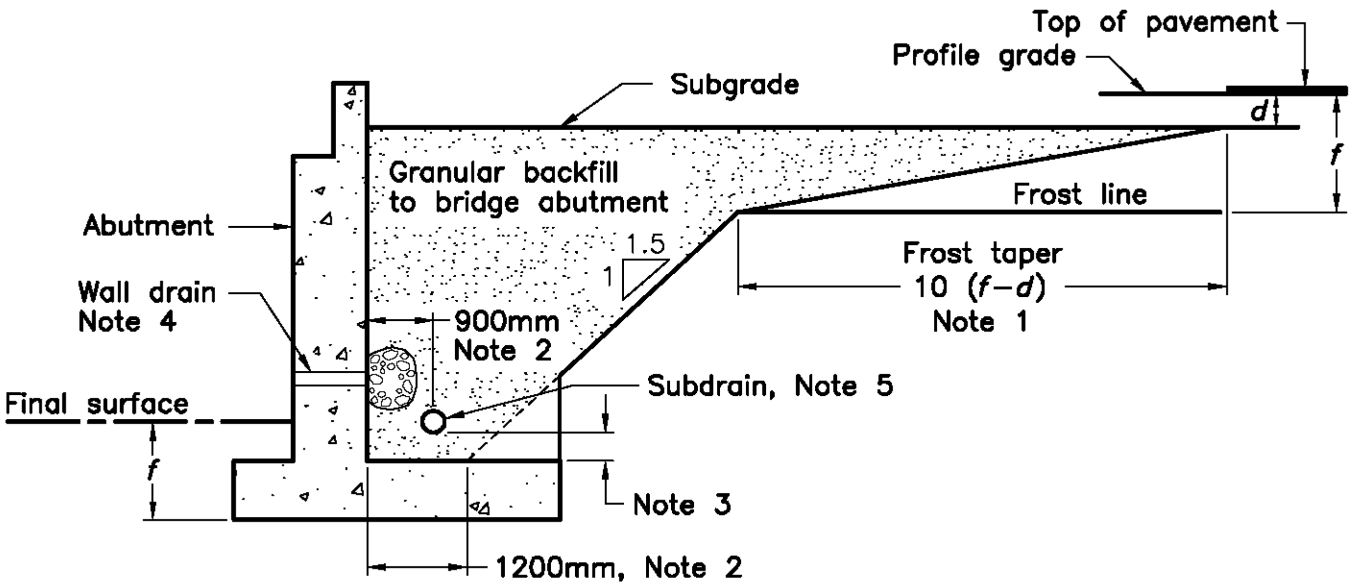
B This information is based on the Ministry of Transportation and Communications Research Publication RR225 "Aspects of Prolonged Exposure of Pavements to Sub-Zero Temperatures" dated December 1981.

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.



INTEGRAL ABUTMENT

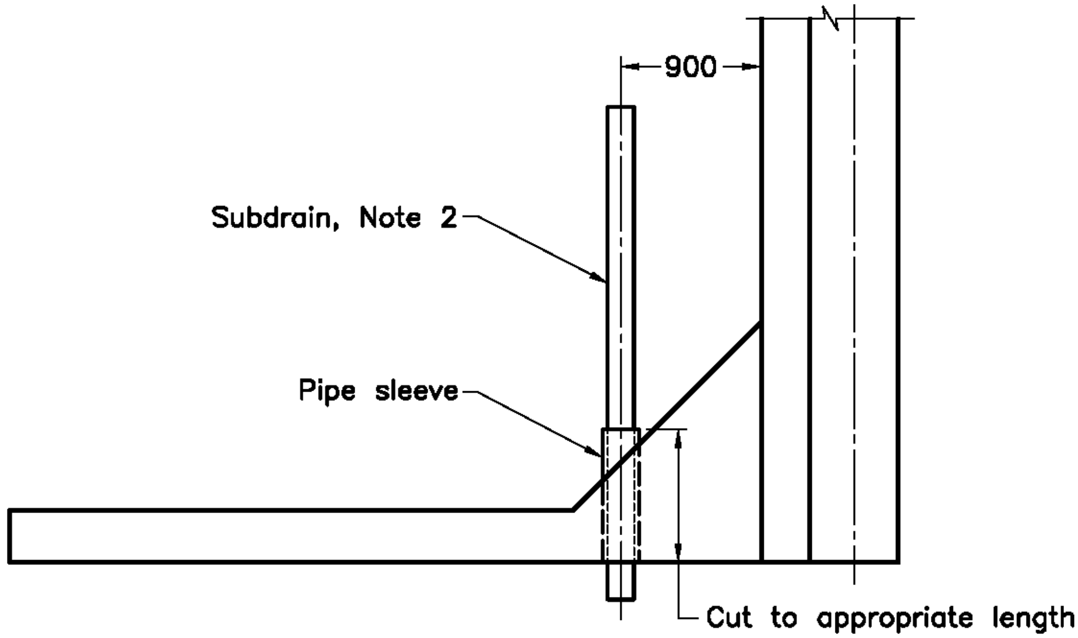


ABUTMENT

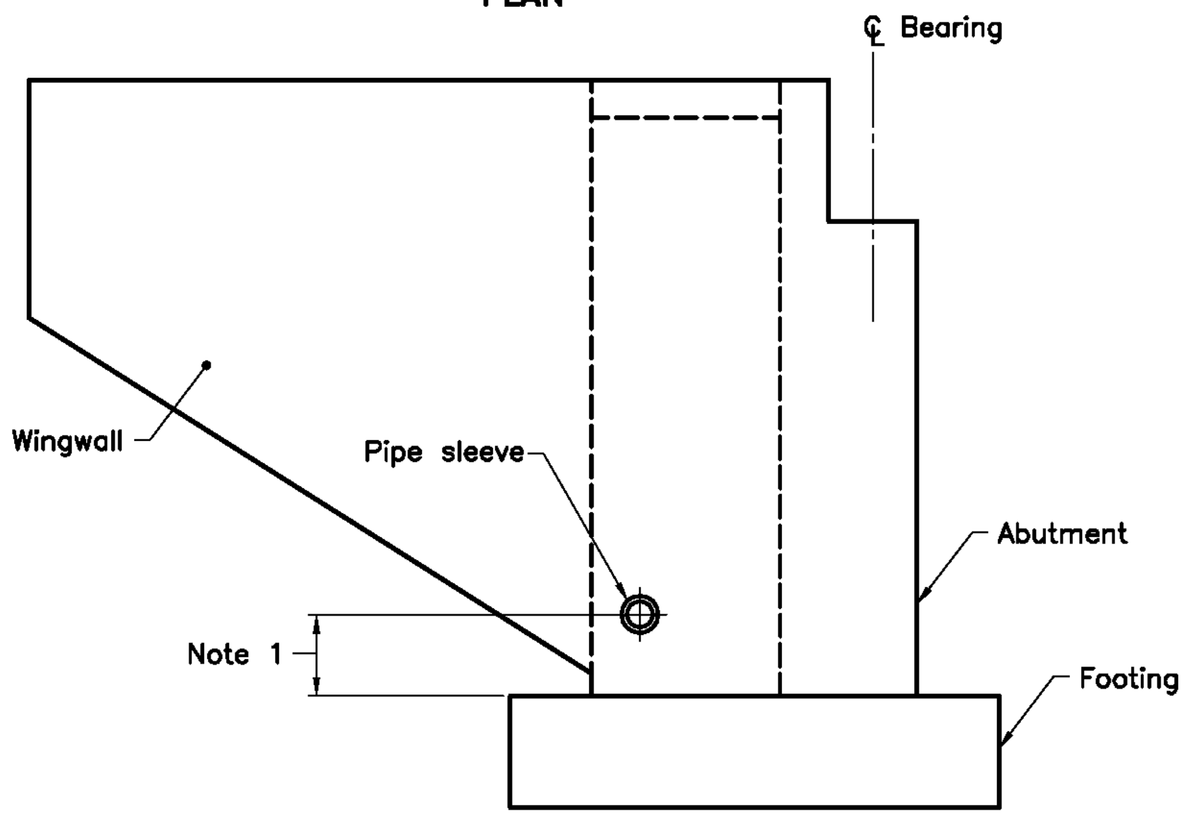
NOTES:

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



PLAN



ELEVATION

NOTE:

1 Height to be consistent with positive drainage of subdrain as specified.

2 150mm dia perforated pipe subdrain wrapped with geotextile.

A Subdrain shall be installed with a 2% gradient behind wall.

B All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2010	Rev 1
WALLS ABUTMENT BACKFILL DRAIN	OPSD 3102.100	

Figures

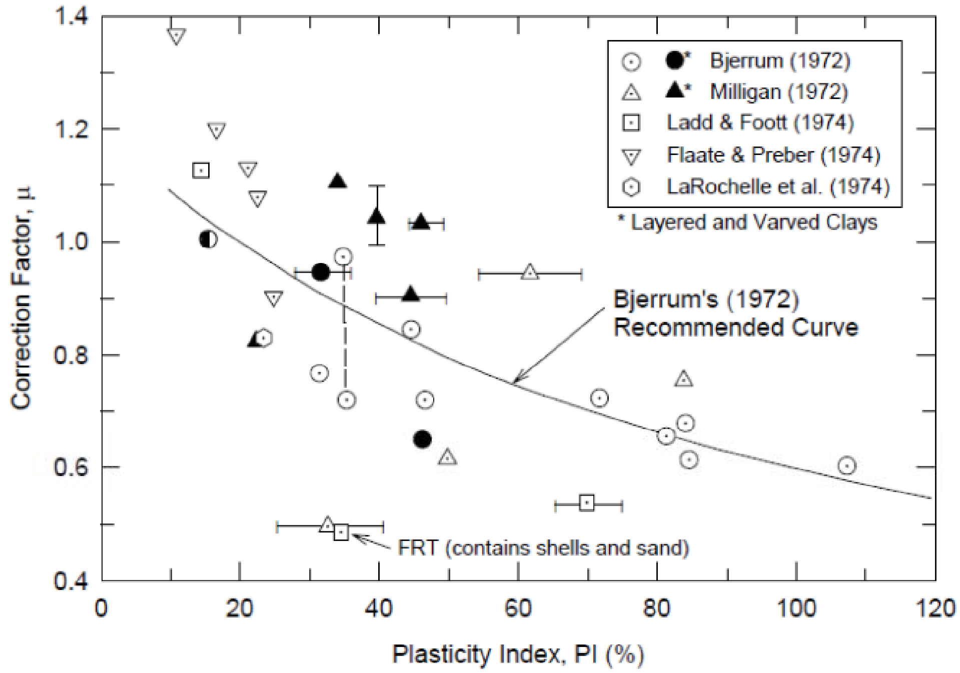


Figure 3.1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures (Ladd & DeGroot, 2004)

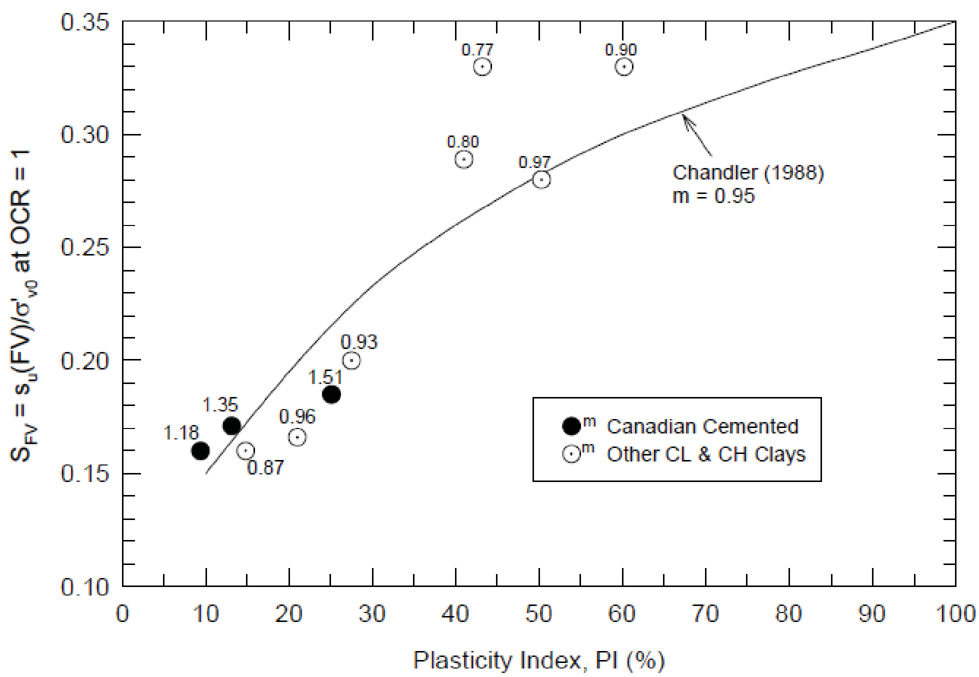
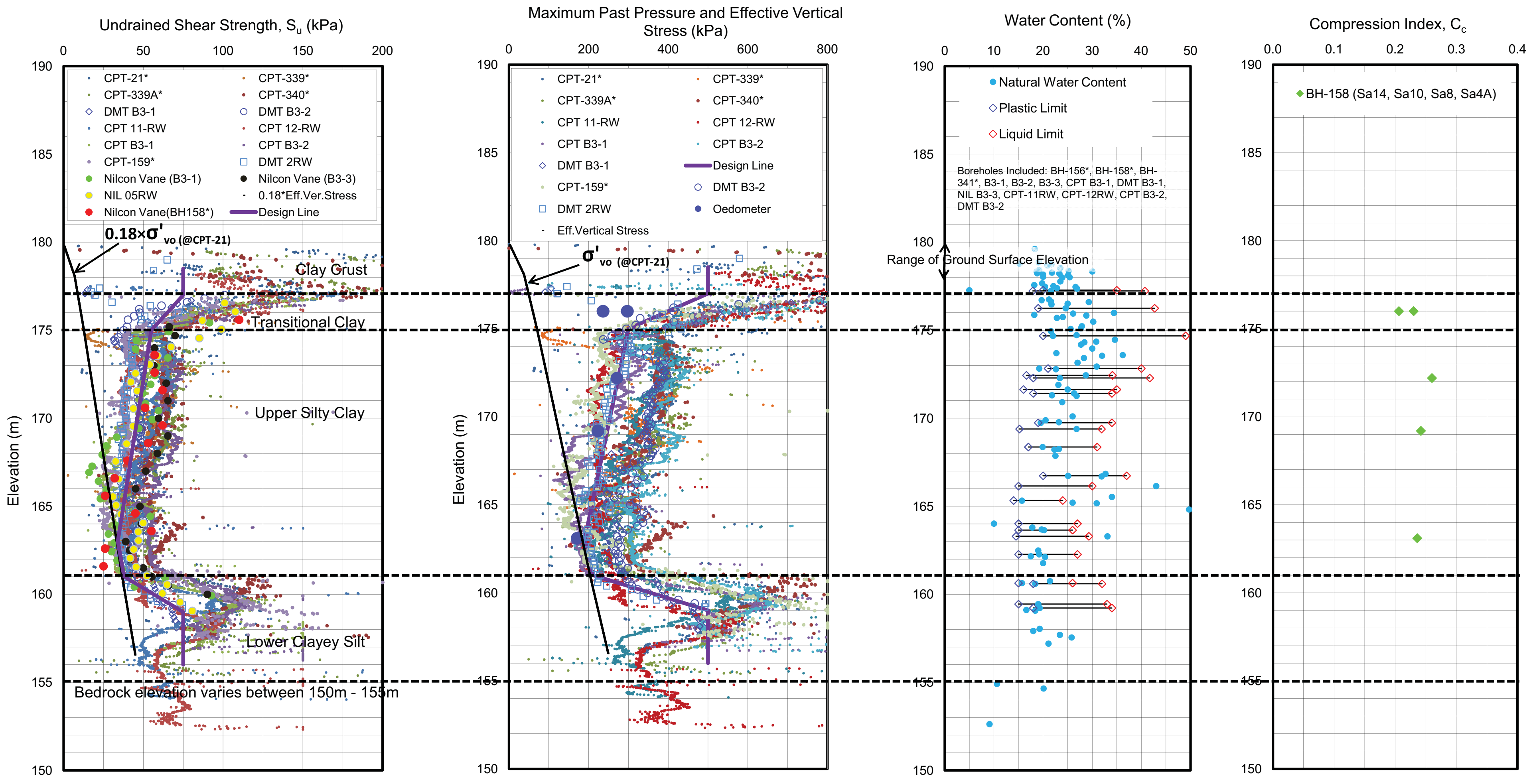


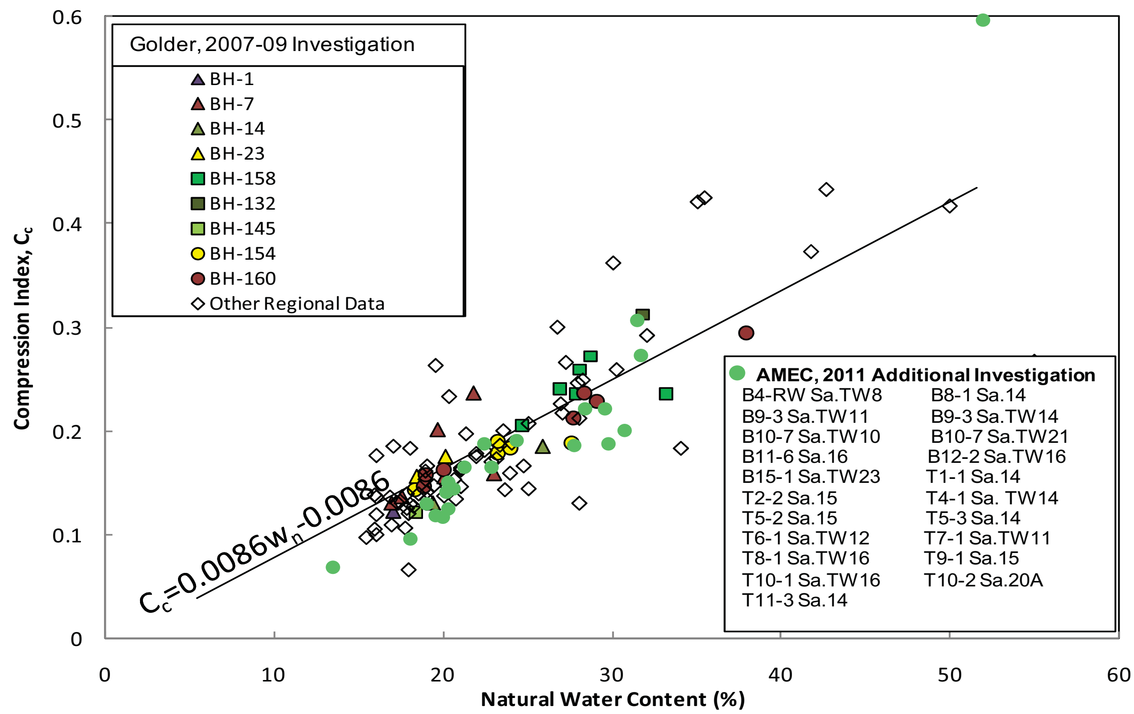
Figure 3.2: Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays (Ladd & DeGroot, 2004)



Notes:
1. Shear strength profiles were estimated from CPT data using the equation $S_u = (q_t - \sigma_{VO}) / N_{KT}$. The cone factor N_{KT} was estimated by comparing the CPT profiles with a nearby Nilcon Vane profile.
2. Maximum past pressure profiles estimated using SHANSEP method. $OCR = [(S_u/\sigma'_V)/S]^{1/m}$
*From previous investigations (ref. R-16 to R-23).

amec Environment & Infrastructure

CLIENT:	
PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	SOIL PROPERTIES PROFILES STA.10+850W TO 11+550W
DATE: Mar 2012	JOB NO.: SW8801.1002
CAD FILE:	FIGURE NO.: 3.3
REV.	



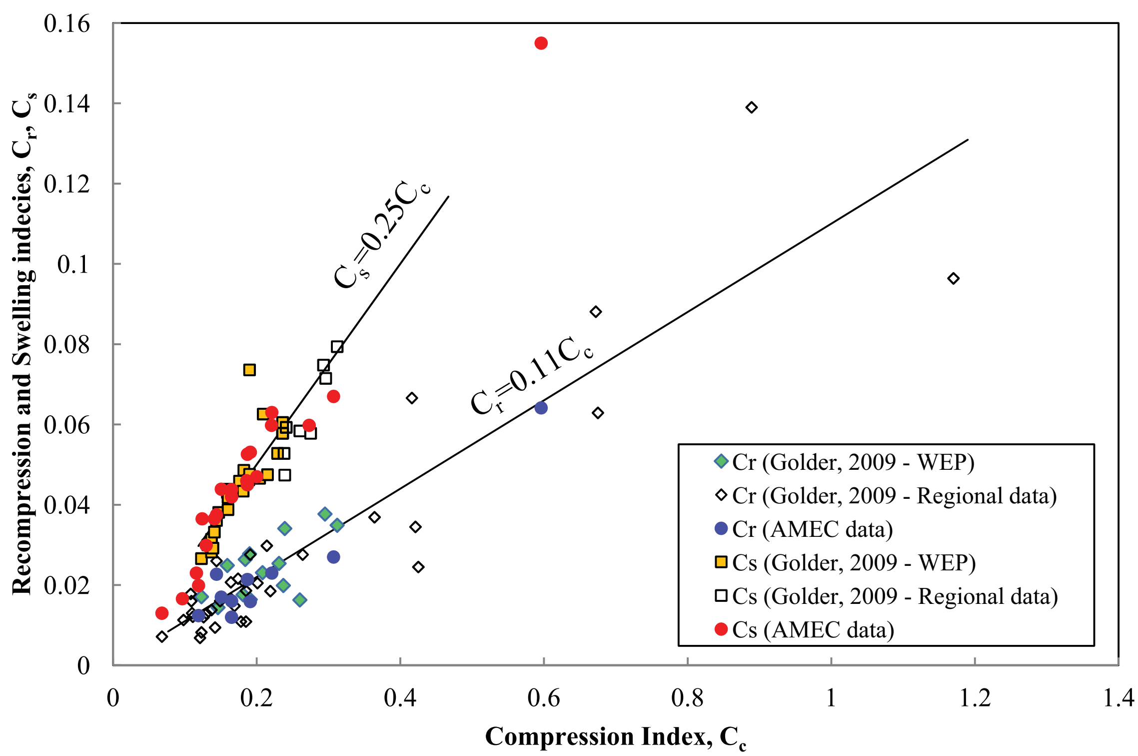


Figure 4.1: Compressibility Parameters at WEP

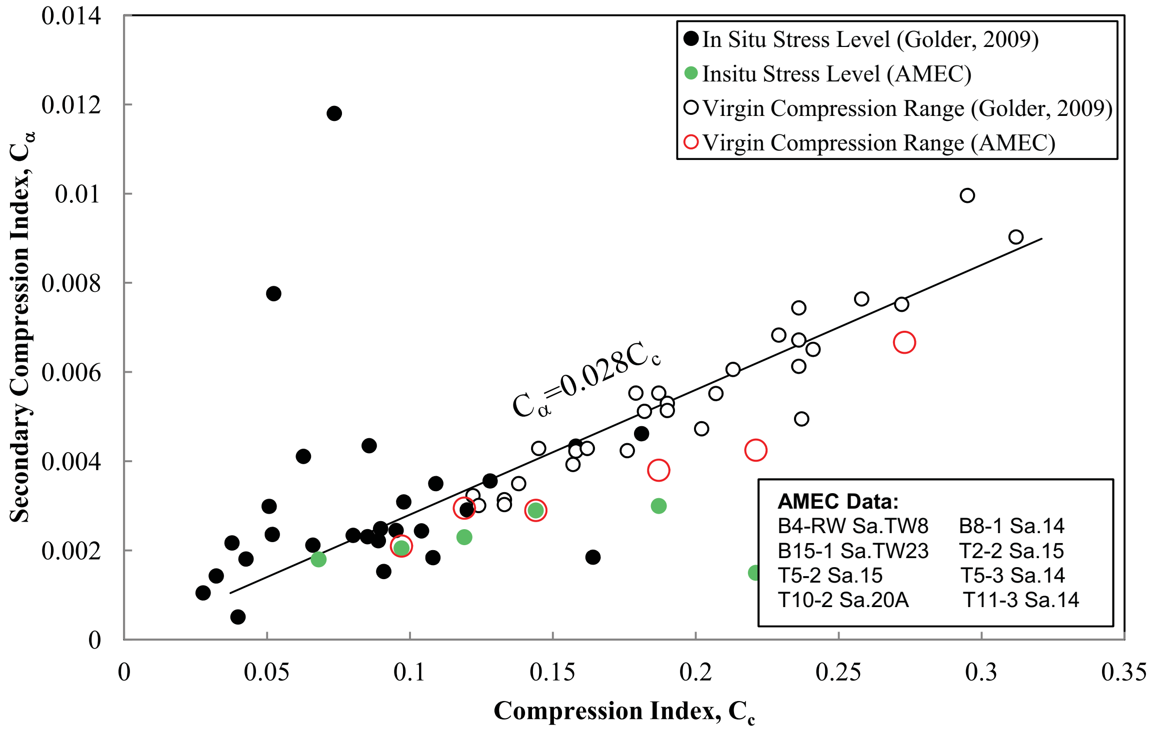


Figure 4.2: Cc versus Cα Relationship at WEP

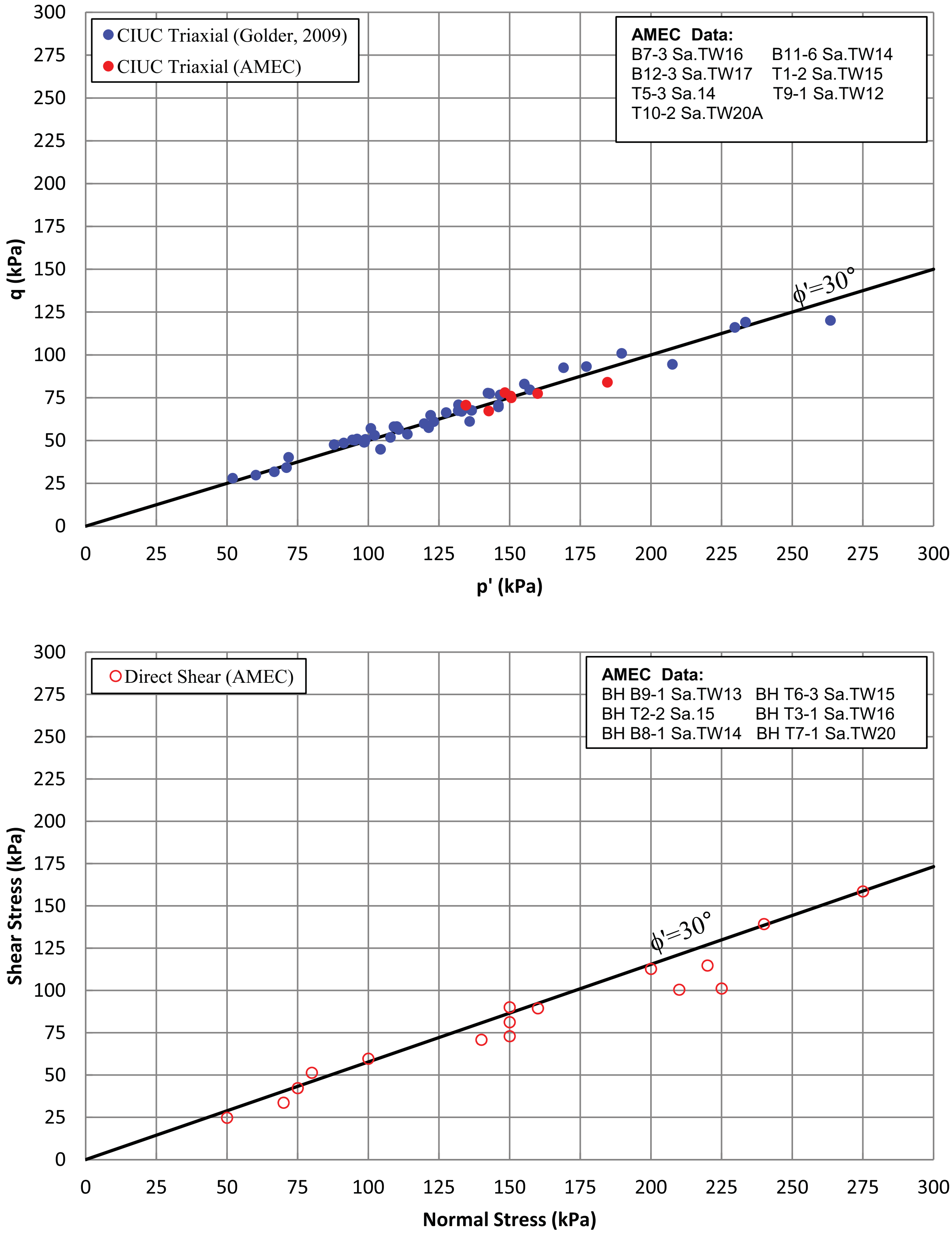


Figure 4.3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP

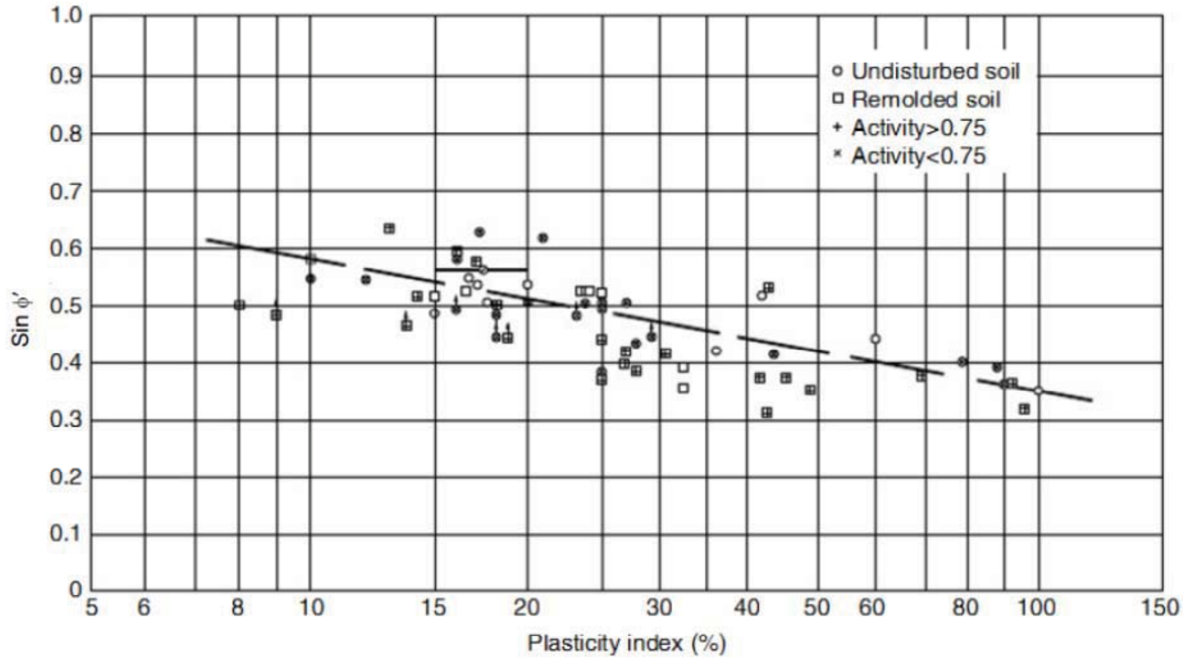


Figure 4.4: Relationship between sin ϕ' and Plasticity Index for Normally Consolidated Soils (Kenney, 1959)

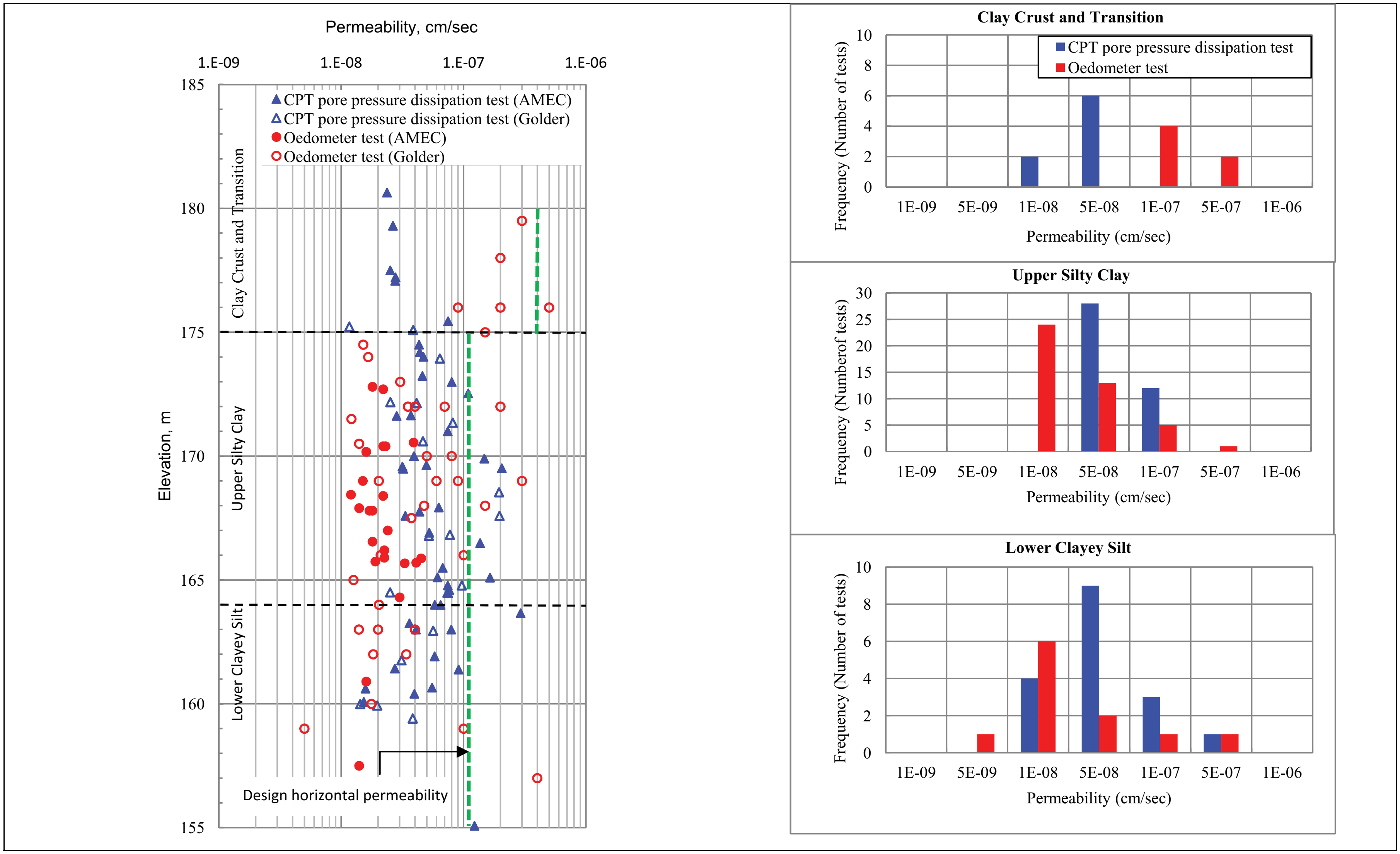
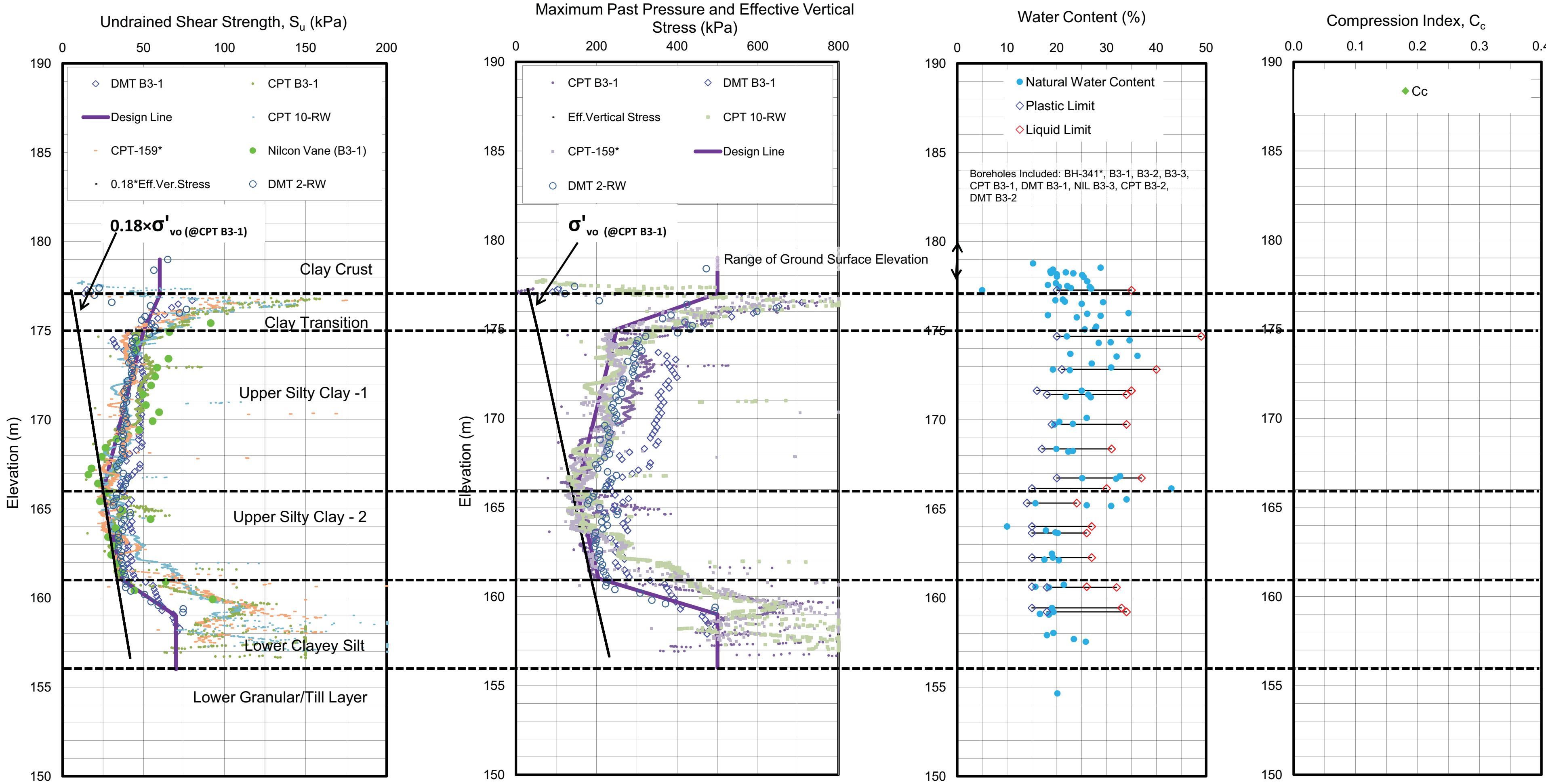
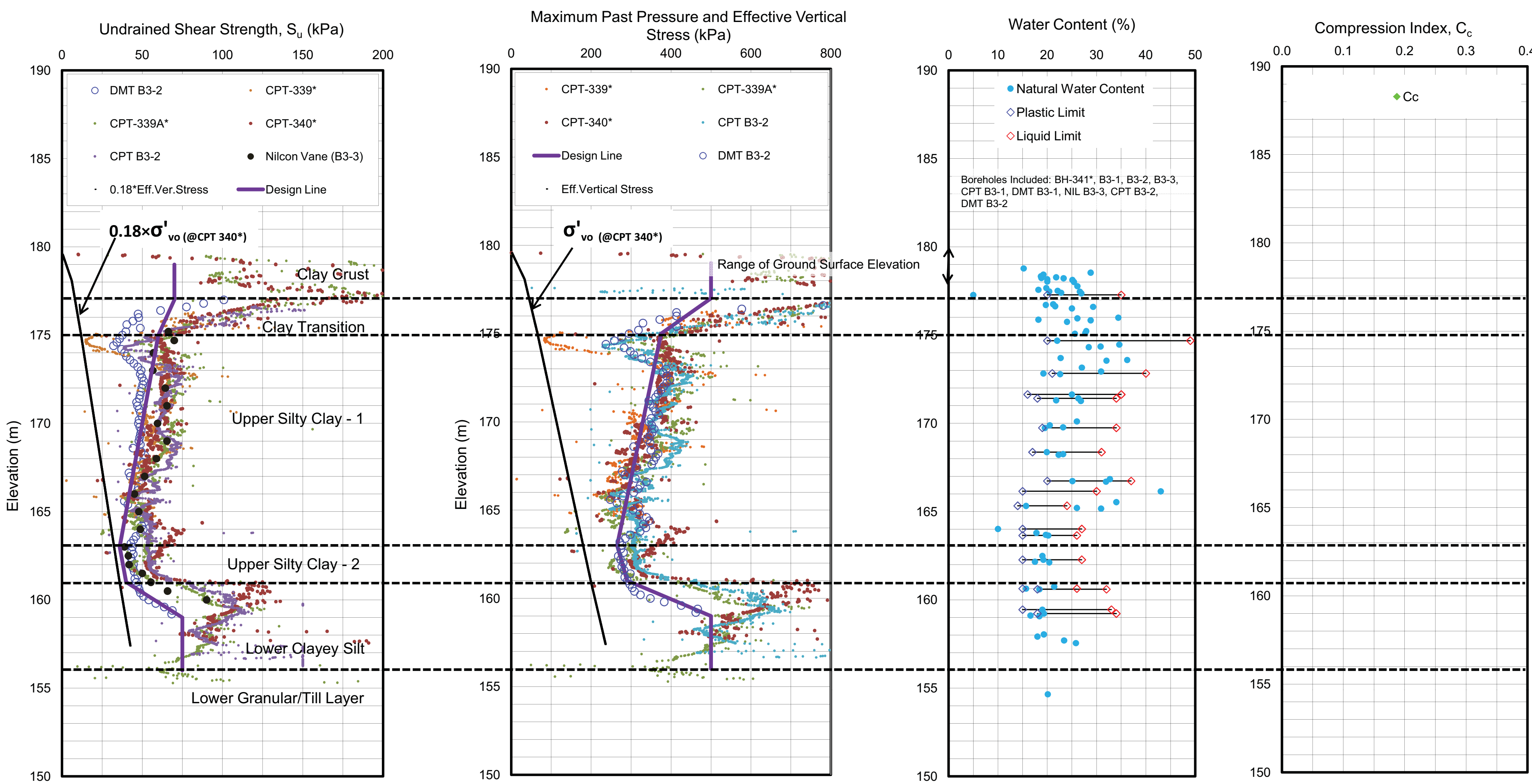


Figure 4.5: Inferred Clay Stratum Permeability from CPT Pore Pressure Dissipation and Oedometer Tests



Notes:
1. Shear strength profiles were estimated from CPT data using the equation $S_u = (q_t - \sigma_{VO}) / N_{KT}$. The cone factor N_{KT} was estimated by comparing the CPT profiles with a nearby Nilcon Vane profile.
2. Maximum past pressure profiles estimated using SHANSEP method. $OCR = [(S_u/\sigma'_V)/S]^{1/m}$
*From previous investigations (ref. R-16 to R-23).

amec Environment & Infrastructure	PROJECT:	WINDSOR ESSEX PARKWAY
CLIENT:	TITLE:	SOIL PROPERTIES PROFILES AT AND AROUND BRIDGE B-3 - WEST SIDE
	DATE: Jun 2012 / JOB NO.: SW8801.1002 / CAD FILE:	FIGURE NO.: 5.1 / REV.



Notes:
1. Shear strength profiles were estimated from CPT data using the equation $S_u = (q_t - \sigma_{VO}) / N_{KT}$. The cone factor N_{KT} was estimated by comparing the CPT profiles with a nearby Nilcon Vane profile.
2. Maximum past pressure profiles estimated using SHANSEP method. $OCR = [(S_u/\sigma'_V)/S]^{1/m}$
*From previous investigations (ref. R-16 to R-23).

amec Environment & Infrastructure

CLIENT:

PROJECT: WINDSOR ESSEX PARKWAY

TITLE: SOIL PROPERTIES PROFILES AT AND AROUND BRIDGE B-3 - EAST SIDE

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.:	REV.
Jun 2012	SW8801.1002		5.2	

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

Foundation Design

RECORD OF BOREHOLE No B3-1

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4682267.8, E329431.6	ORIGINATED BY	LC
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	Jun 14, 11 - Jun 14, 11	CHECKED BY	MSO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
178.9	Ground Surface												
0.0	Mixed **FILL** Sand and gravel, clayey silt with topsoil organics near surface												-vibrating wire piezometers (VWP) installed at bedrock surfce in sampled borehole; shallow and mid-depth VWPs installed in adjacent boring at (N4682267, E329434) -Spider Magnets (MG) installed in adjacent boring at (N4682267, E329435) -MG installed at 2.26m below ground surface -VWP #P3 installed at 3.05m below surface
178.3 0.6	**SILTY CLAY** Weathered Trace sand, trace gravel Soft to Stiff Mottled brown and grey -Fissures and wet sand seams -Soft wet seam	1	SS	3	178								
		2	SS	4	177								
	Grey -Pink clay inclusions	3	SS	8									
		4	SS	6	176								
		5	SS	4	175								
		6	SS	3	174								
	-Sand laminations (approx. 25mm thick)	7	SS	3	173								
		8	TW	PH				19.6	1	7	36	56	
			VT		172	2.1							
		9	SS	PH	171								-no recovery with shelby tube; sample retrieved by driving split spoon -MG installed at 8.48m below ground surface
			VT		170	2							
		10	TW	PH	169			19.9					
		11	TW	PH	168								-VWP #P11 installed at 10.97m below surface
			VT		167	2							
	-Silt and fine sand inclusions	12	TW	PH	166			18.7	1	10	36	53	
		13	TW	PH	165								
			VT		164	2							

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 19/01/12

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No B3-1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4682267.8, E329431.6 ORIGINATED BY LC

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

DATUM Geodetic DATE Jun 14, 11 - Jun 14, 11 CHECKED BY MSO

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY *(continued)* -Some coarse gravel	14	TW	PH		18.6	-shelby tube sank 0.6m under weight of rods	
					163		-switched to wash boring with NW casing at about 15.8m below ground surface	
	-Inferred sand layer	15	TW	PH	162			
			VT		161			
160.6 / 18.3	**CLAYEY SILT** Some sand, trace gravel Grey	16	TW	PH	160	21.1		9 23 37 31
		17	TW	PH	159			
			VT				-unable to push vane	
157.6	Some gravel	18	SS	29	158			
21.3	**LIMESTONE,** Fossiliferrous Laminated, fine grained and partially crystallized, stylolites present Grey to brown	19	RC		157		-VWP #P21 installed at 21.3m below surface RQD = 94% TCR = 100% SCR = 92% Rock Core Cu = 35.5 MPa	
		20	RC		156		RQD = 100% TCR = 100% SCR = 100%	
154.7 / 24.2	**END OF BOREHOLE**				155			

No groundwater observed during auger drilling

Water level measured in Piezometer VWP #P3 at elevation 176.2m on June 25, 2011
Water level measured in Piezometer VWP #P3 at elevation 176.0m on July 9, 2011
Water level measured in Piezometer VWP #P3 at elevation 177.1m on July 22, 2011

Water level measured in Piezometer VWP #P11 at elevation 172.8m on June 25, 2011
Water level measured in Piezometer VWP #P11 at elevation 177.8m on July 9, 2011
Water level measured in Piezometer VWP #P11 at elevation 178.4m on July 22, 2011
Water level measured in Piezometer VWP #P11 at elevation 178.9m on August 25, 2011

Water level measured in Piezometer VWP #P21 at elevation 180.6m on June 25, 2011
Water level measured in Piezometer VWP #P21 at elevation 180.6m on July 9, 2011

Elevation scale: 154, 153, 152, 151, 150, 149

Continued Next Page

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

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

RECORD OF BOREHOLE No B3-1

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4682267.8, E329431.6	ORIGINATED BY	LC
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	Jun 14, 11 - Jun 14, 11	CHECKED BY	MSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Water level measured in Piezometer VWP #P21 at elevation 180.6m on July 22, 2011 Water level measured in Piezometer VWP #P21 at elevation 180.6m on August 25, 2011										
							148				
							147				
							146				
							145				
							144				
							143				
							142				
							141				
							140				
							139				
							138				
							137				
							136				
							135				
							134				

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No B3-2

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4682224.9, E329491.10		ORIGINATED BY BS
DIST	HWY WEP	BOREHOLE TYPE CME 850 - 150mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE Jun 25, 11 - Jun 26, 11		CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
178.9	Ground Surface								
0.0	**TOPSOIL**								
0.2	**SILTY CLAY** Soft to stiff Mottled brown and grey -Some organics								
		1	SS	3					
	-Trace sand, trace gravel	2	SS	6					
		3	SS	8					
	Grey	4	SS	3					
		5	SS	3					
	-Some sand, trace gravel	6	SS	3		2	14	34	50
		7	SS	2					
		8	TW	PH	19.7				
			VT						
		9	TW	PH					
		10	TW	PH	19.5	1	13	39	47
			VT						
	Moist -Grey-pink with black clay inclusions	11	TW	PH					
		12	TW	PH	18.8				
			VT						
		13	TW	PH					

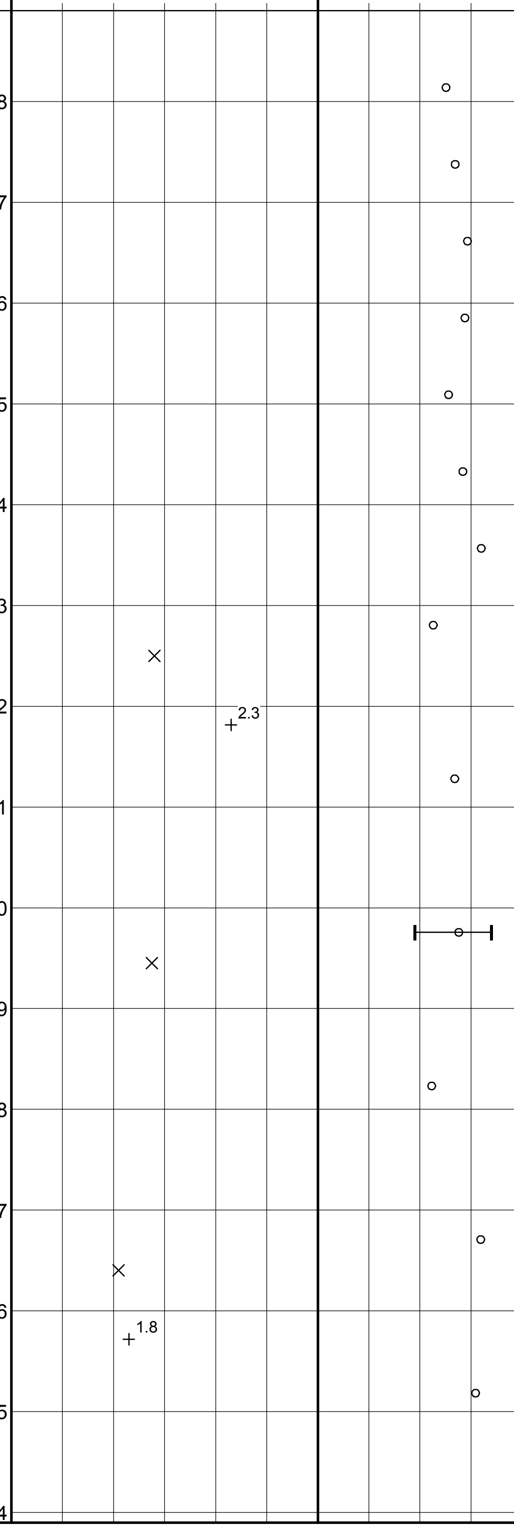
ELEVATION SCALE: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

GROUND WATER CONDITIONS; STRAT PLOT; REMARKS & GRAIN SIZE DISTRIBUTION (%)



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

Foundation Design

RECORD OF BOREHOLE No B3-2

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4682224.9, E329491.10
DIST		HWY WEP	
BOREHOLE TYPE	CME 850 - 150mm Dia. Continuous Flight Hollow Stem Augers		
DATUM	Geodetic	DATE	Jun 25, 11 - Jun 26, 11
ORIGINATED BY	BS	COMPILED BY	SS
CHECKED BY	MSO		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
163.7 / 15.2	**SILTY CLAY** *(continued)*
163.7 / 15.2	**CLAYEY SILT** Some sand, trace gravel, Stiff, Grey; -Trace sand, trace gravel, trace cobbles (inferred)
159.1 / 19.8	**SILT** Some clay, some fine sand, some silty clay seams, Compact, Grey
158.0 / 20.9	**CLAYEY SILT** Stiff, Grey
156.0 / 22.9	**LIMESTONE** Fine grained, Fossiliferrous, petroliferrous, laminated. Fractured at location between 22.9m and 23.0m. Brown
153.6 / 25.3	**END OF BOREHOLE** No groundwater observed during auger drilling

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ (kN/m³)	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	REMARKS
14	TW	PH	20.8	3 25 40 32	
	VT				-continued drilling by wash boring with casing at about 15.5 m below ground surface
15	SS	PH			-no retrieval with shelby tube; sample retrieved by pushing split spoon
16	TW	PH	20.7	5 18 40 37	
	VT				-attempt at vane shear test exceeded max torque of apparatus
17	TW	PH			
18	SS	15			
19	RC				-Split Spoon refusal at 22.9m RQD = 66.7% TCR = 67% SCR = 44%
20	RC				RQD = 100% TCR = 100% SCR = 88%

Elevation scale: 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153, 152, 151, 150, 149

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane value label: 1.1

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

WATER CONTENT (%): 10 20 30

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

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RECORD OF BOREHOLE No B3-3

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4682180.9, E329559	ORIGINATED BY	TA
DIST	HWY WEP	BOREHOLE TYPE	CME 850 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	Jun 20, 11 - Jun 22, 11	CHECKED BY	MSO

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
179.0	Ground Surface
0.0 / 178.7 / 0.3	**TOPSOIL**
178.2 / 0.8	**FINE SAND** Poorly graded, Some silt, some clay, Brown
177.6 / 1.4	**CLAYEY SILT** Trace to some sand, Mottled brown and grey
	CLAYEY SILT Trace to some sand, Stiff to firm, Brown, Grey, -Some sand, trace gravel, - Trace fissures
	-Trace sand
	-Trace pink clay nodules below approx. 4m
	-Some sand, trace gravel
	Moist

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	10	
3	SS	13	
4	SS	5	
5	SS	5	
6	SS	4	
7	SS	3	
8	SS	PH	
	VT		
9	TW	PH	1 13 44 42
10	TW	PH	
	VT		
11	TW	PH	1 19 41 39
12	TW	PH	
	VT		
13	TW	PH	4 28 42 26

GROUND WATER CONDITIONS

ELEVATION SCALE: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165

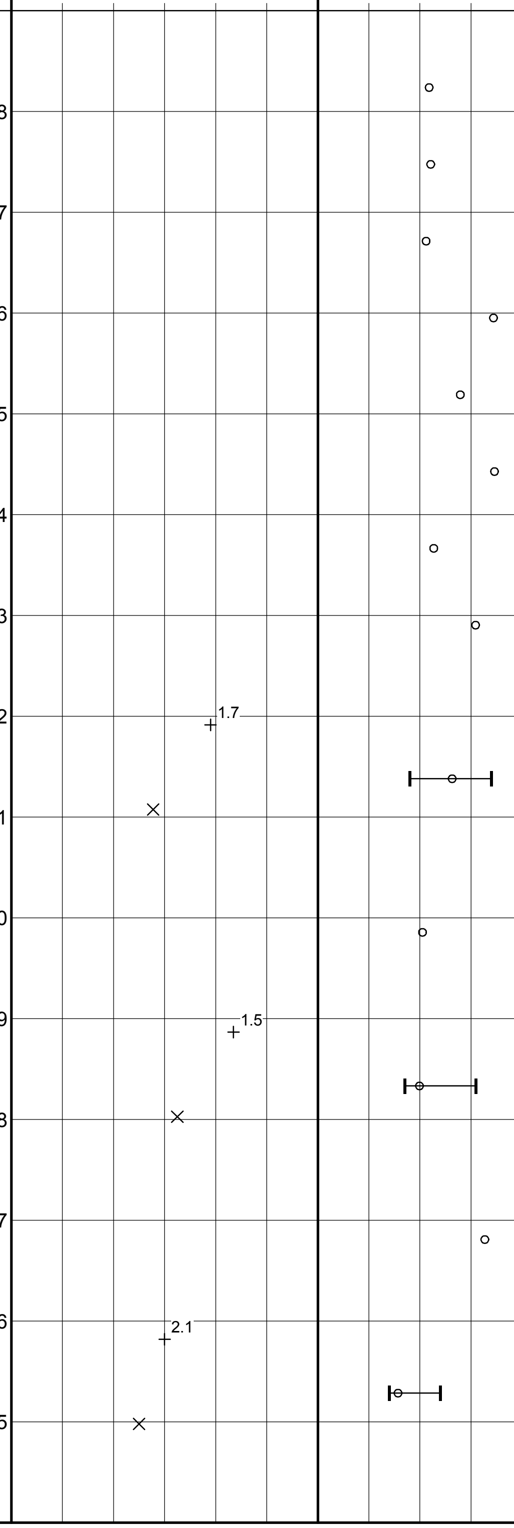
DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³



REMARKS

-Inclinometer casing installed in sampled borehole; Vibrating Wire Piezometers (VWP) installed in adjacent boring at (4682181N, 329557E) and (4682182N, 329556); Spider Magnets installed in adjacent boring at (4682181N, 329557E)
-SM installed at 2.67m below ground surface
-VWP #P1 installed at 3.05m below ground surface

-SM installed at 9.27m below ground surface

-VWP #P2 installed at 12.19m below ground surface

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

RECORD OF BOREHOLE No B3-3

METRIC

W.P. RFP No. 09-54-1007	LOCATION N4682180.9, E329559		ORIGINATED BY TA
DIST	HWY WEP	BOREHOLE TYPE CME 850 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY SS
DATUM Geodetic		DATE Jun 20, 11 - Jun 22, 11	CHECKED BY MSO

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	CLAYEY SILT *(continued)*	14	TW	PH	163	+ 1.4	-continued drilling by wash boring with casing at about 15.5 m below ground surface
			VT				
		15	TW	PH	162	×	4 23 45 28
					161		
		16	TW	PH	160	>>+ 1.4	
			VT				
	-Trace cobbles	17	TW	PH	159	×	2 23 46 29
					158		-damaged shelby tube
		18	SS	5	157		
156.1 / 22.9	**SAND** and **GRAVEL** Fine to Coarse Dense to very dense Grey and black	19	SS	32	156		-VWP #P3 installed at 24.2m below ground surface -no recovery with shelby tube; sample retrieved by driving split spoon
	-Trace limestone fragments	20	SS	52	155 / 154		
153.7 / 25.3	**LIMESTONE** Fine grained Grey	21	RC		153		RQD = not recorded TCR = 100% SCR = 97% -H2S detected approx. 1hr after contact with bedrock
25.5	**LIMESTONE** Fossiliferrous, fine grained, with oil stains Grey						
152.7 / 26.3	**LIMESTONE** Fine Grained, fossiliferrous Grey						
26.5 / 152.2 / 26.8	**LIMESTONE** Fine Grained, laminated Grey				152		

END OF BOREHOLE

No groundwater observed during auger drilling

Water level measured in Piezometer VWP #P1 at elevation 178.4m on August 22, 2011

Water level measured in Piezometer VWP #P2 at elevation 177.7m on August 22, 2011

Water level measured in Piezometer VWP #P3 at elevation 180.3m on August 22, 2011

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

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF BOREHOLE No NIL B3-3

METRIC

W.P. RFP No. 09-54-1007 | LOCATION N4682184.0, E329556.0 | ORIGINATED BY SD

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

DATUM Geodetic | DATE Jun 4, 11 - Jun 5, 11 | CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.0	Ground Surface					
0.0 / 178.7	**TOPSOIL** Black					
0.3	**SILT** Trace clay, Loose, Mottled brown and grey, Moist	1	SS	5		
177.3 / 1.7	**SILTY CLAY** Some sand, trace gravel, Stiff, Brown-grey, Damp to moist	2	SS	11		
		3	SS	15		
	Grey -Trace sand Moist to wet	4	SS	7		
175.5 / 3.5	**END OF SAMPLED BOREHOLE** (Continued with Nilcon Vane to refusal) Borehole dry on completion					

ELEVATION SCALE: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No CPT B3-1

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4682270.6, E329419.6 ORIGINATED BY TA

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

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

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PEN. × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.0	Ground Surface										
0.0	**TOPSOIL**										
178.4 0.6	**SANDY SILT** Fine Loose Brown		1	SS	3		178				
	-Some clay		2	SS	2						
177.0 2.0	**END OF SAMPLED BOREHOLE** (Continued with CPT to refusal) Borehole dry on completion						177				
							176				
							175				
							174				
							173				
							172				
							171				
							170				
							169				
							168				
							167				
							166				
							165				

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

RECORD OF BOREHOLE No CPT B3-2

METRIC

W.P. RFP No. 09-54-1007 | LOCATION N4682176.2, E329573 | ORIGINATED BY TA

DIST | HWY WEP | BOREHOLE TYPE CME 850 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY SS

DATUM Geodetic | DATE Jun 20, 11 - Jun 20, 11 | CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● POCKET PEN., × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.1	Ground Surface										
0.0	**TOPSOIL**						179				
178.7 / 0.4	**SAND** Poorly graded Trace silt Brown										
178.3 / 0.8	**SILTY CLAY** Some sand, trace gravel Mottled brown and grey Brown -Trace fissures		1	SS	3		178		○ (~19)		
			2	SS	8				○ (~19)		
177.1 / 2.0	**END OF SAMPLED BOREHOLE** (continued with CPT to refusal) Borehole dry on comletion						177				
							176				
							175				
							174				
							173				
							172				
							171				
							170				
							169				
							168				
							167				
							166				
							165				

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

RECORD OF BOREHOLE No DMT B3-1

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4682286.4, E329420.5	ORIGINATED BY	LC
DIST		HWY WEP	BOREHOLE TYPE: CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	Jun 19, 11 - Jun 19, 11	CHECKED BY	MSO

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.5	Ground Surface									
0.0	**FILL** Silty sand and gravel									
0.2	**FILL** Silty clay, some topsoil						179			
			1	SS	8			~15		
178.0 / 1.5	**SILTY CLAY** Weathered and thin, wet sand/silt seams Brown-grey						178			
177.5 / 2.0			2	SS	4			~25		
	END OF SAMPLED BOREHOLE (Continued with DMT)						177			
	Borehole dry on completion						176 … 165			

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

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

RECORD OF BOREHOLE No DMT B3-2

METRIC

W.P. RFP No. 09-54-1007 LOCATION N4682177.6, E329571.6 ORIGINATED BY LC

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

DATUM Geodetic DATE Jun 22, 11 - Jun 22, 11 CHECKED BY MSO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
179.2	Ground Surface				
0.0	**TOPSOIL** Sandy with roots				
178.9 / 0.3	**FINE SAND** Trace silt and roots Brown				
178.3 / 0.9	**CLAYEY SILT/SILTY CLAY** With numerous hairline sand/silt seams Mottled brown and grey	1A, B	SS	2	
178.0 / 1.2	**SILTY CLAY** Fissured, varved with 4" silt lens Grey	2	SS	11	
177.2 / 2.0	**END OF BOREHOLE** (Continued with DMT to refusal)				
	Borehole dry on competion				

ELEVATION SCALE: 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

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

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

RECORD OF BOREHOLE No CPT10-RW

METRIC

W.P.	RFP No. 09-54-1007	LOCATION	N4682295.7, E329387.8	ORIGINATED BY	TA
DIST	HWY WEP	BOREHOLE TYPE	CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	Jun 10, 11 - Jun 10, 11	CHECKED BY	MSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)
179.3	Ground Surface					
0.0	**FILL** Topsoil					
0.2	**FILL** Silty clay and topsoil	1	SS	7		~29
178.2 / 1.1	**TOPSOIL** (inferred from auger cuttings)					
177.9 / 1.4	**SAND** Poorly graded, Trace to some silt, Loose, Brown-grey	2	SS	5		~26
177.6 / 1.7	**SILTY CLAY** Firm, Mottled brown and grey					
177.3 / 2.0	**END OF SAMPLED BOREHOLE** (Continued with CPT to refusal)					
	Borehole dry on completion					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; ELEVATION SCALE (179 to 165); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● POCKET PEN., × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10, 20, 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity

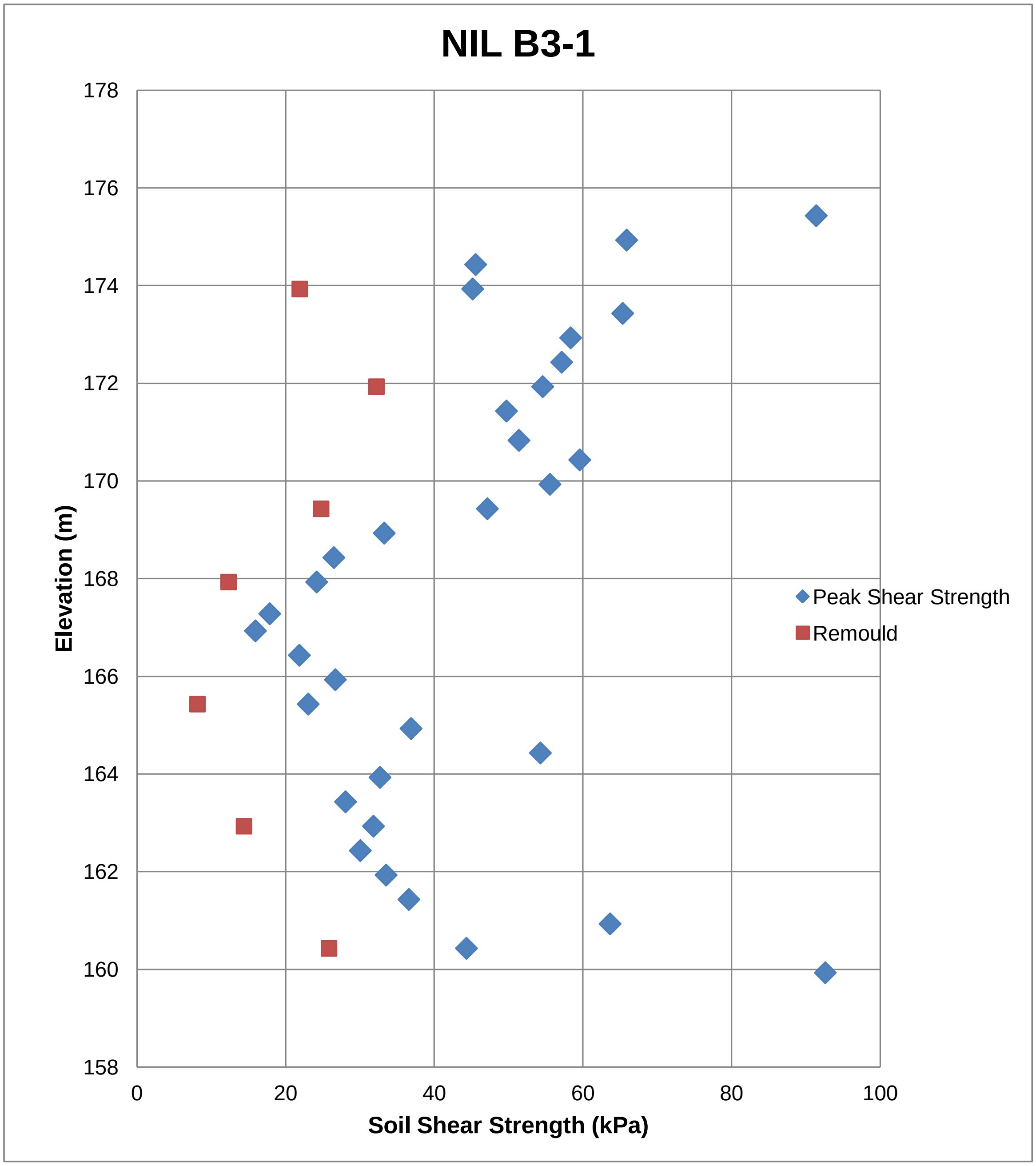
$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF NILCON VANE TEST NIL B3-1

Project : Windsor-Essex Parkway
Location: N4682266.3; E329436.2
Ground Surface Elevation: 178.9 m

Test Date: 6/17/2011
Predrill Depth : 3.5 m

Sheet 1 of 1
Datum Geodetic



Operator: NB

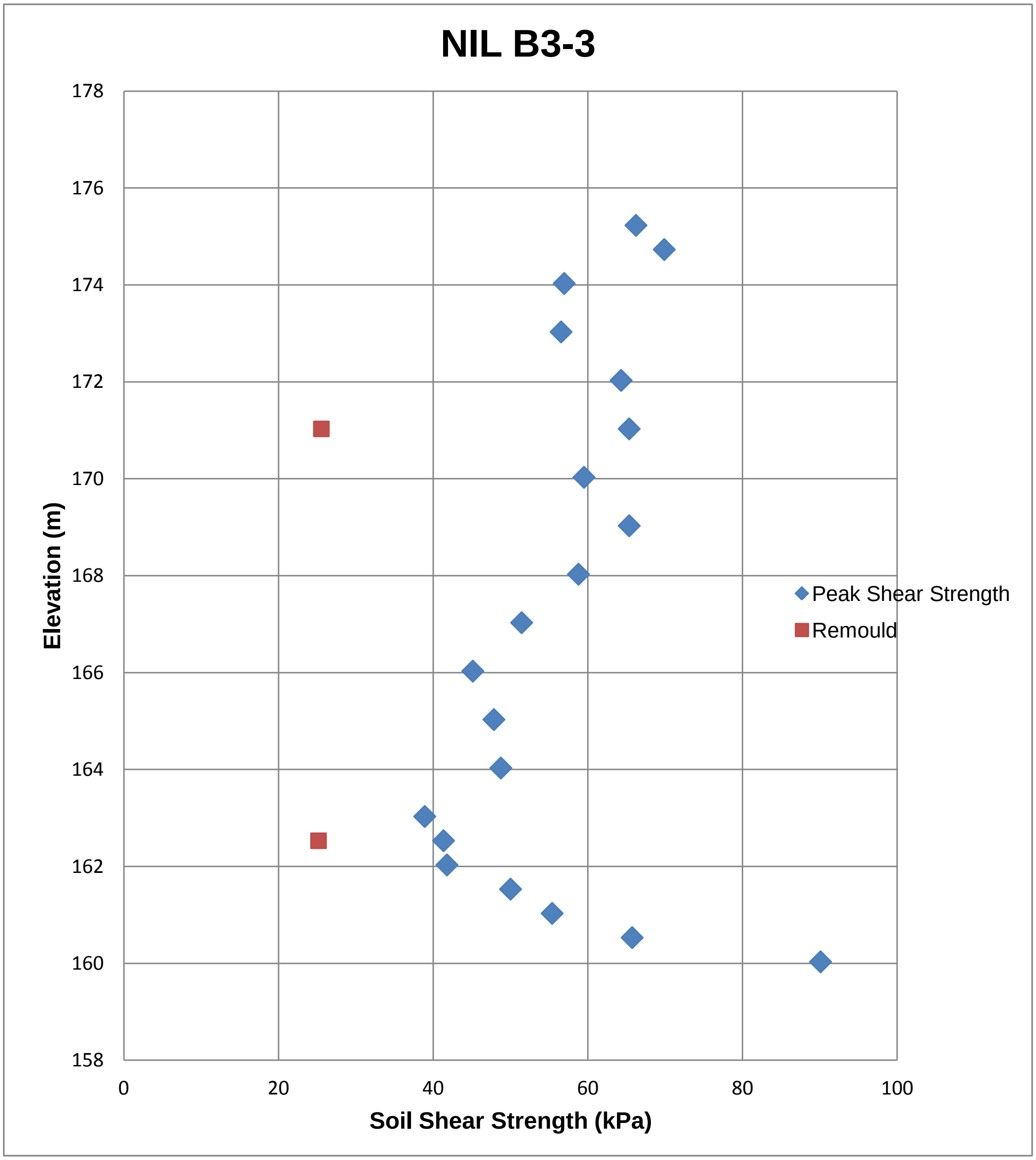
Checked: DD

RECORD OF NILCON VANE TEST NIL B3-3

Project : Windsor-Essex Parkway
Location: N4682184.0; E329556.0
Ground Surface Elevation: 179.0 m

Test Date: 6/4/2011 - 6/5/2011
Predrill Depth : 3.5 m

Sheet 1 of 1
Datum Geodetic



Operator: SD

Checked: DD

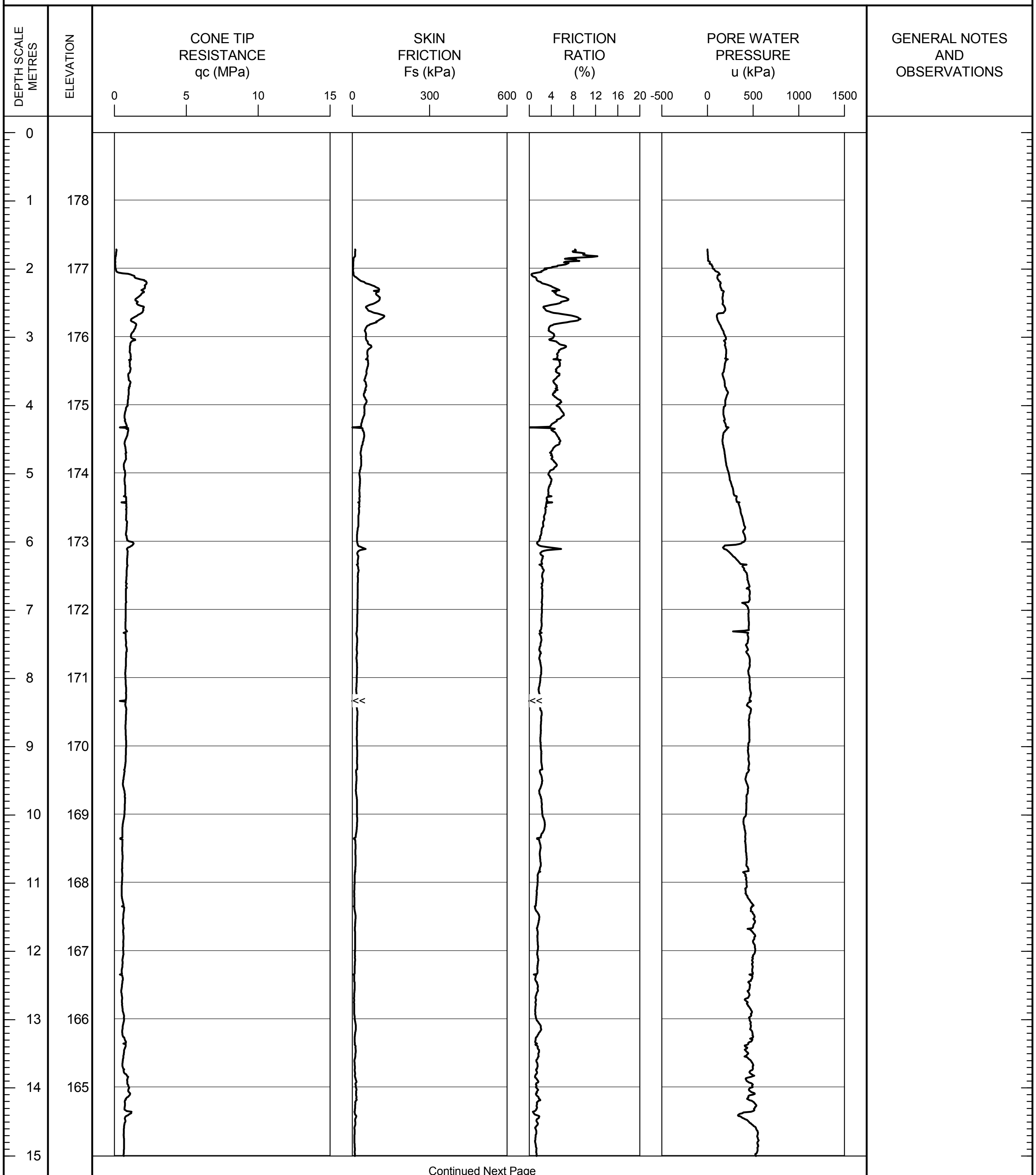
RECORD OF CONE PENETRATION TEST CPT B3-1

METRIC

PROJECT Windsor-Essex Parkway TEST DATE 6/10/2011 - 6/10/2011 SHEET 1 OF 2

LOCATION N4682270.6; E329419.6 DATUM Geodetic

GROUND SURFACE ELEVATION: 179.0 PREDRILL DEPTH: 1.55 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

CHECKED: DD

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

Foundation Design

RECORD OF CONE PENETRATION TEST CPT B3-1

METRIC

PROJECT Windsor-Essex Parkway

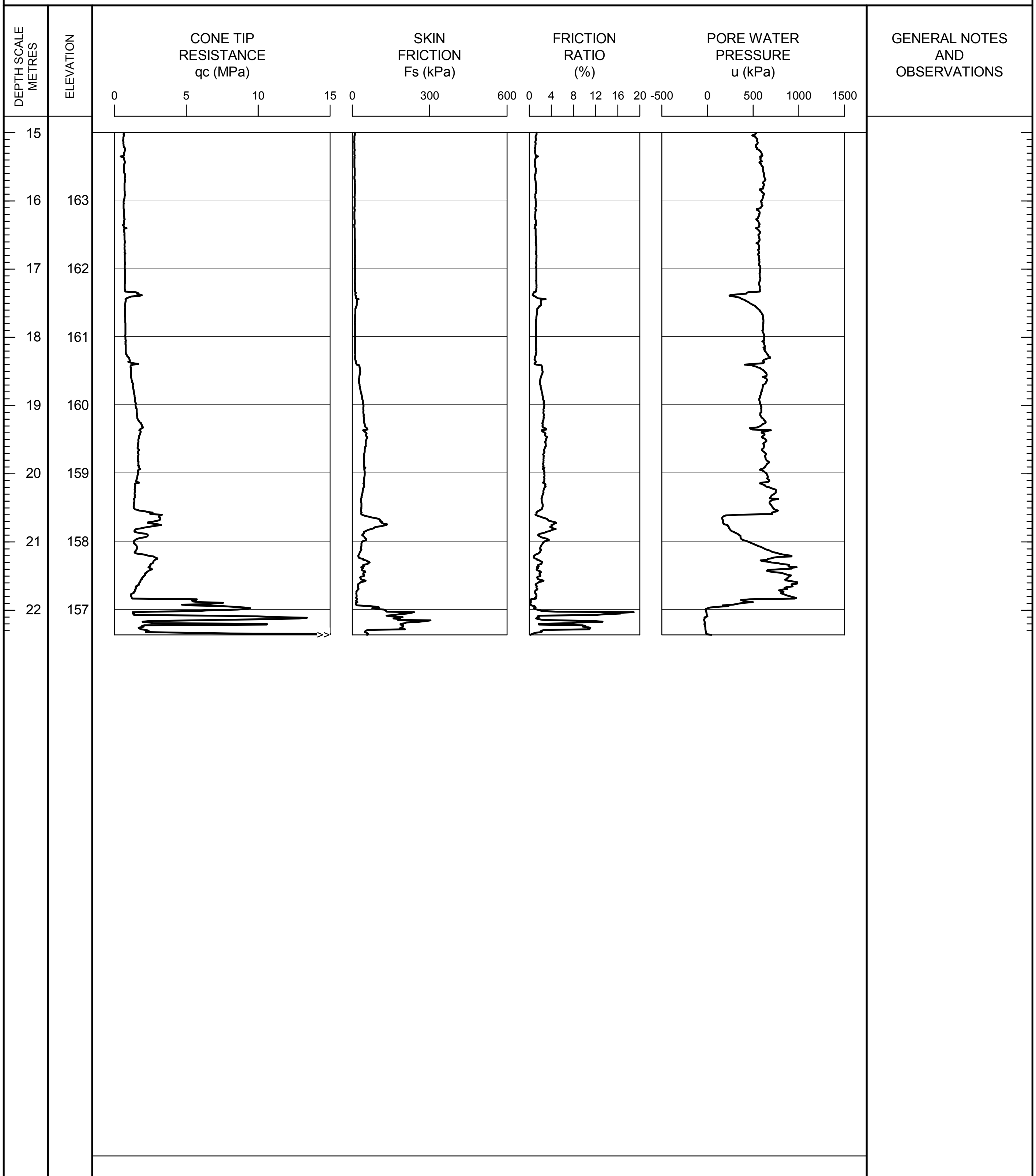
TEST DATE 6/10/2011 - 6/10/2011

SHEET 2 OF 2

LOCATION N4682270.6; E329419.6

DATUM Geodetic

GROUND SURFACE ELEVATION: 179.0 PREDRILL DEPTH: 1.55 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



OPERATOR: TA

CHECKED: DD

WEP CPT LOG CPT B-3.GPJ ONTARIO MOT.GDT 22/12/11

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF CONE PENETRATION TEST CPT B3-2

METRIC

PROJECT Windsor-Essex Parkway

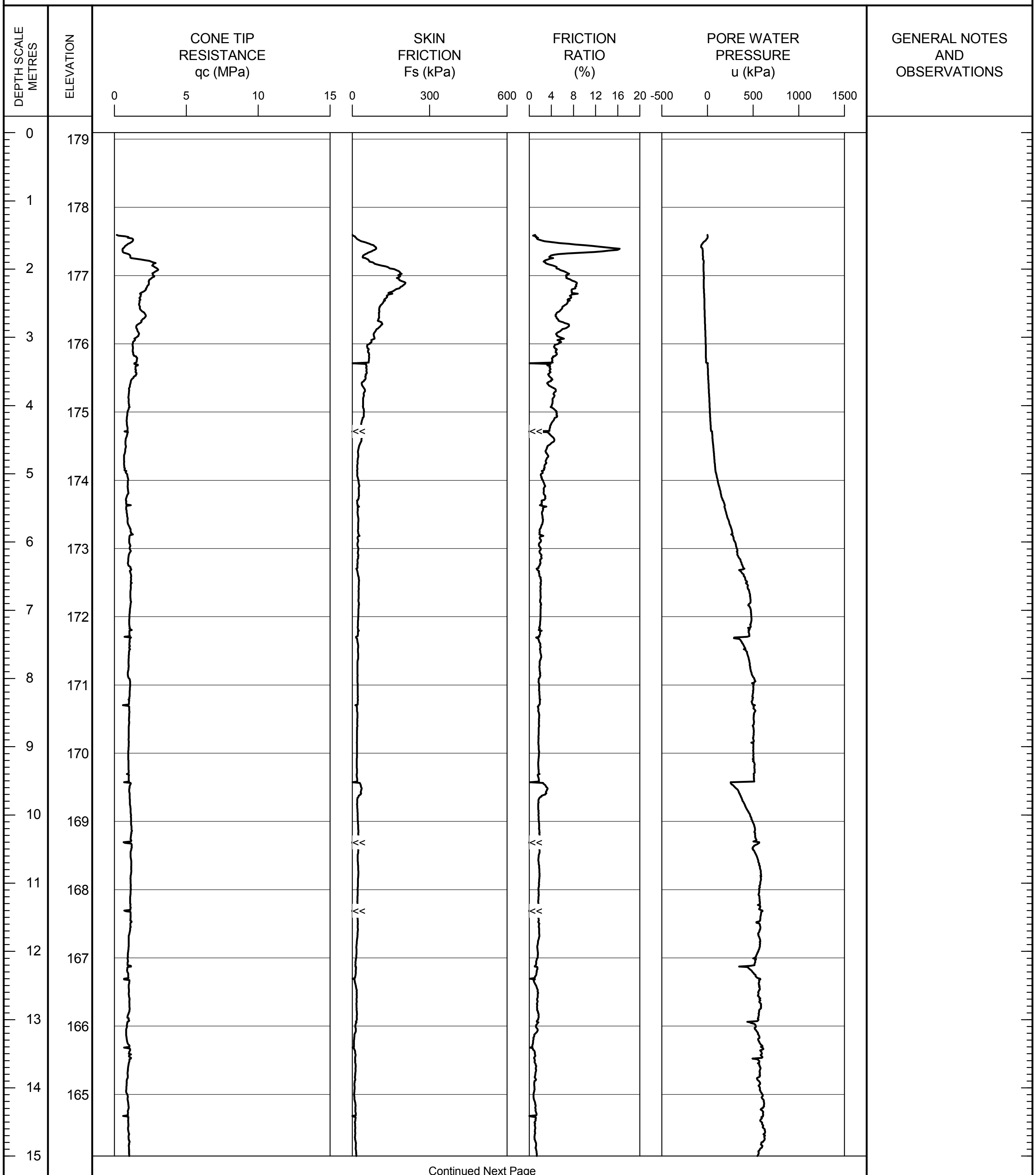
TEST DATE 6/20/2011 - 6/20/2011

SHEET 1 OF 2

LOCATION N4682176.2; E329573.0

DATUM Geodetic

GROUND SURFACE ELEVATION: 179.1 PREDRILL DEPTH: 1.37 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

CHECKED: DD

RECORD OF CONE PENETRATION TEST CPT B3-2

METRIC

PROJECT Windsor-Essex Parkway

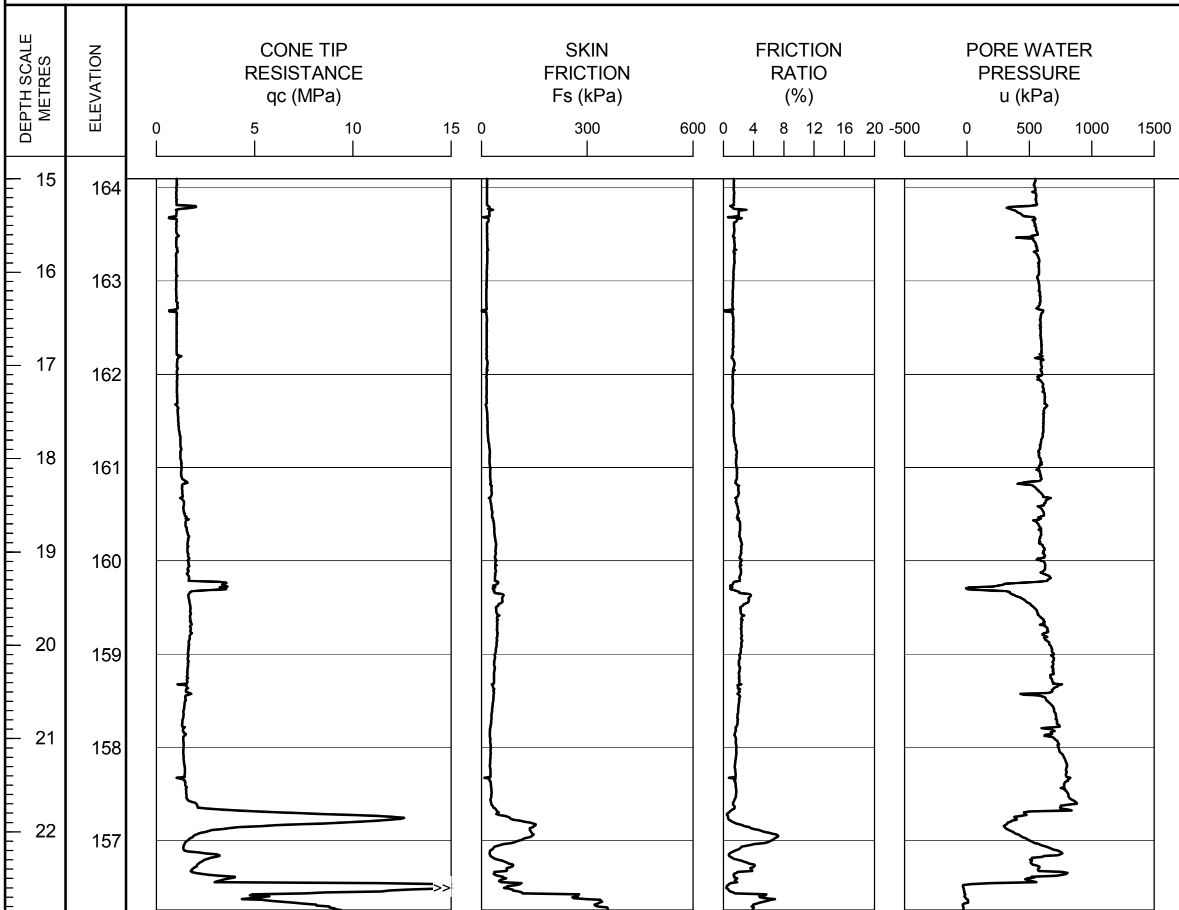
TEST DATE 6/20/2011 - 6/20/2011

SHEET 2 OF 2

LOCATION N4682176.2; E329573.0

DATUM Geodetic

GROUND SURFACE ELEVATION: 179.1 PREDRILL DEPTH: 1.37 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

OPERATOR: TA

CHECKED: DD

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF CONE PENETRATION TEST CPT 10-RW

METRIC

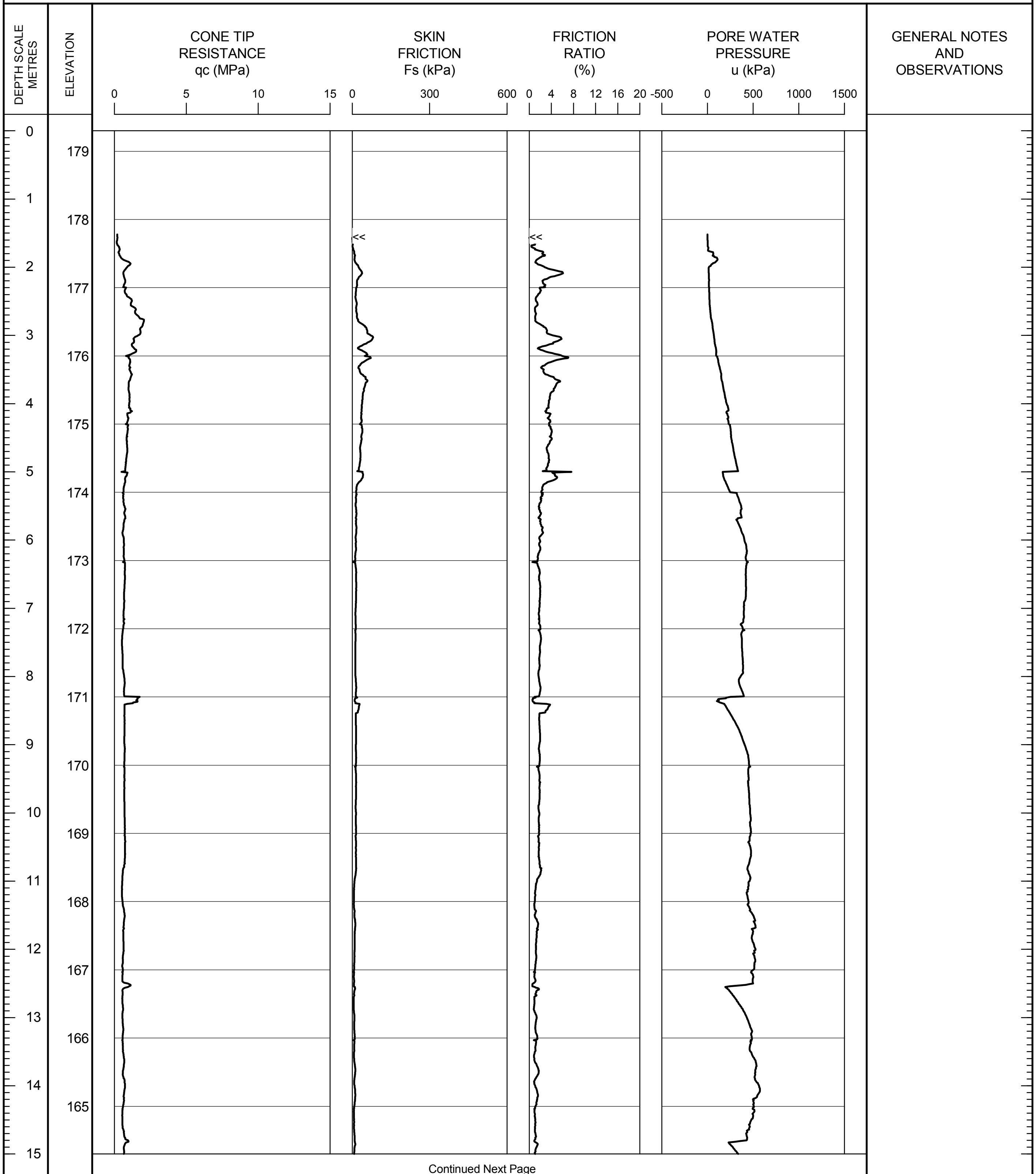
PROJECT Windsor-Essex Parkway

TEST DATE 6/10/2011 - 6/10/2011

LOCATION N4682295.7; E329387.8

DATUM Geodetic

GROUND SURFACE ELEVATION: 179.3 PREDRILL DEPTH: 1.5 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

CHECKED: DD

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

RECORD OF CONE PENETRATION TEST CPT 10-RW

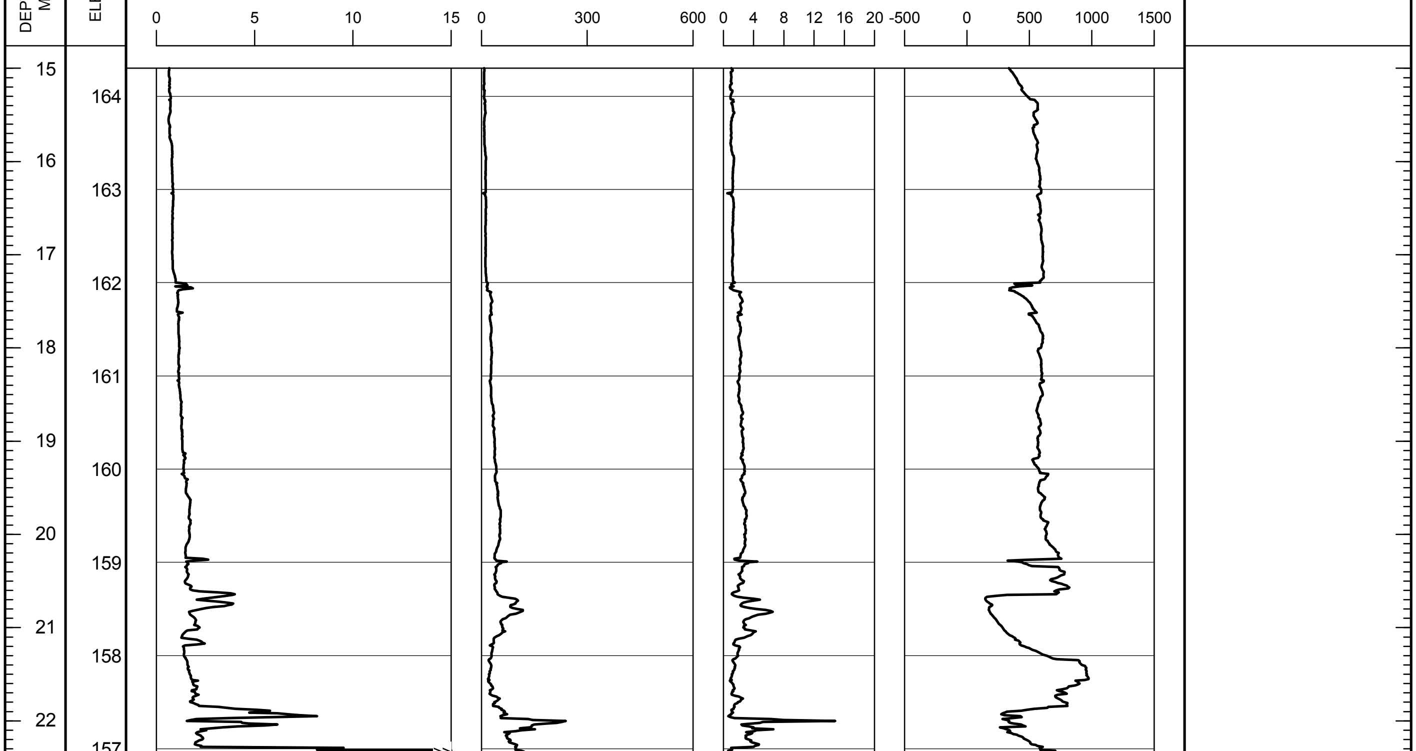
METRIC

PROJECT Windsor-Essex Parkway TEST DATE 6/10/2011 - 6/10/2011 SHEET 2 OF 2

LOCATION N4682295.7; E329387.8 DATUM Geodetic

GROUND SURFACE ELEVATION: 179.3 PREDRILL DEPTH: 1.5 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0

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



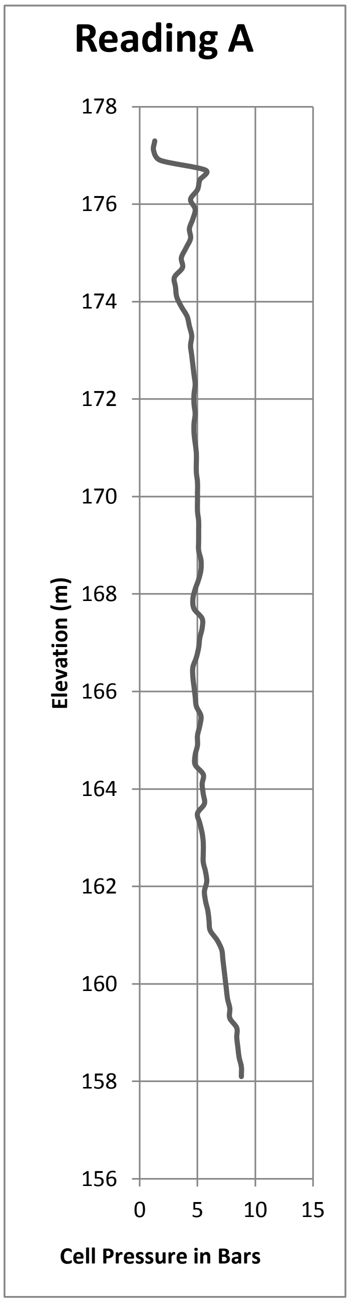
OPERATOR: TA

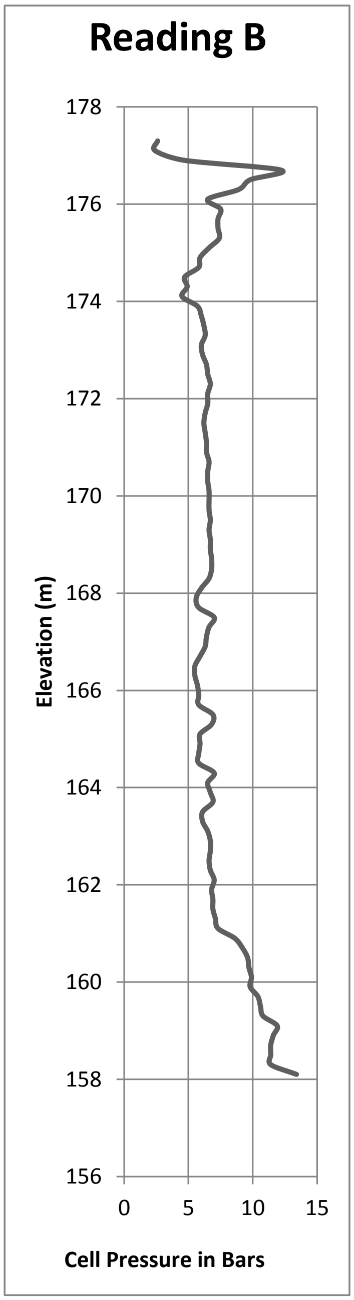
CHECKED: DD

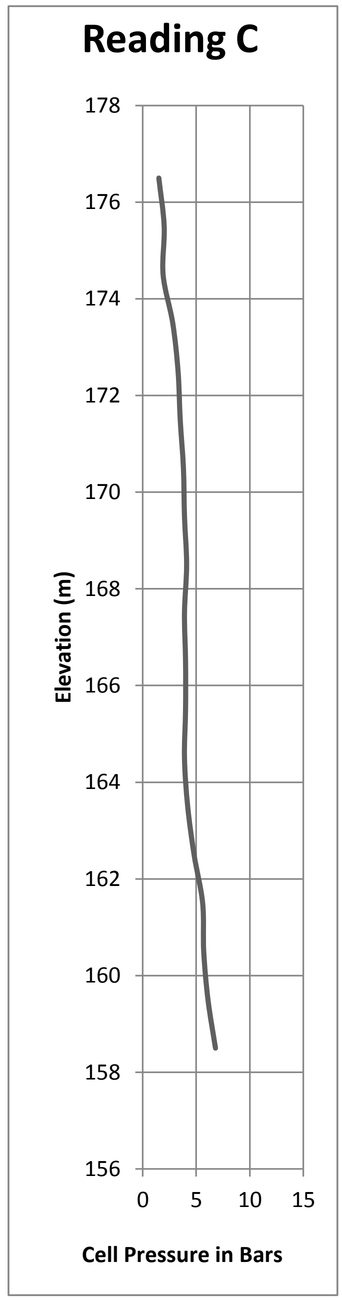
Foundation Design

RECORD OF DILATOMETER TEST DMT B3-1

Project : Windsor-Essex Parkway	Test Date: 6/19/2011	Sheet 1 of 1
Location: N 4682286.4; E 329420.5	Predrill Depth : 2.0 m	Datum Geodetic
Ground Surface Elevation : 179.5	Delta A: 0.20 Bar	Delta B: 0.25 Bar







Operator: LC

Checked: DD

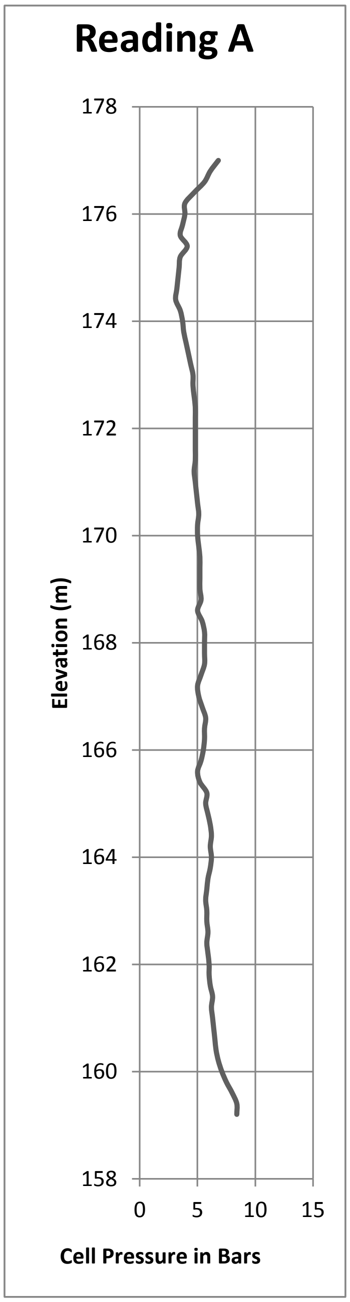
Foundation Design

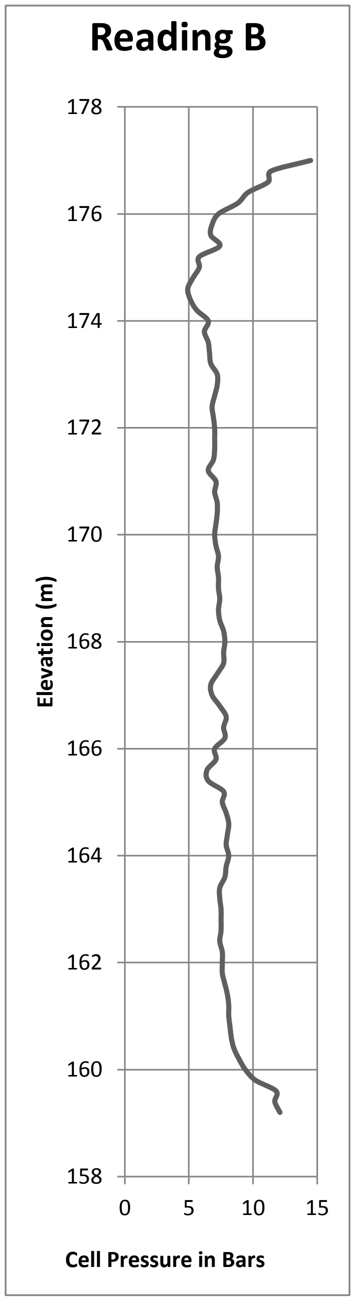
RECORD OF DILATOMETER TEST DMT B3-2

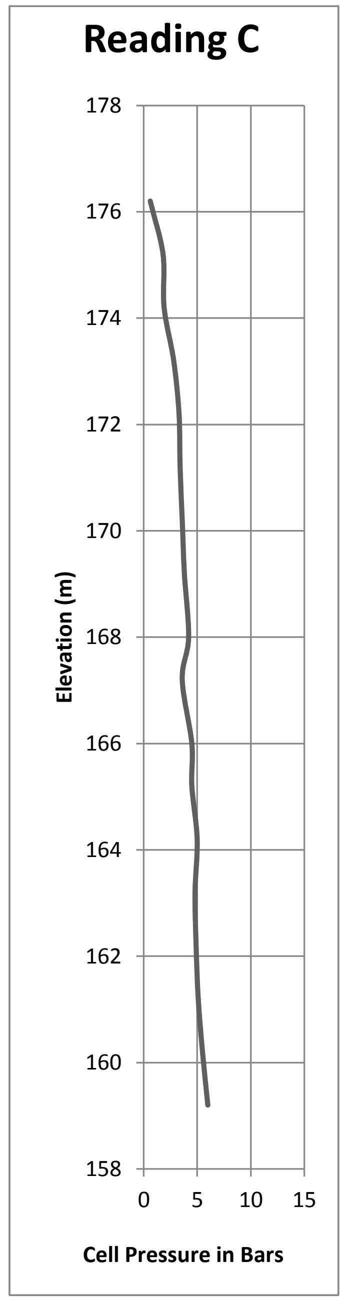
Project : Windsor-Essex Parkway
Location: N 4682177.6; E 329571.6
Ground Surface Elevation : 179.2

Test Date: 6/23/2011
Predrill Depth : 2.0 m
Delta A: 0.20 Bar

Sheet 1 of 1
Datum Geodetic
Delta B: 0.48 Bar







Operator: LC

Checked: DD

Appendix B Borehole and CPT Logs from Previous Investigations

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 341

METRIC

PROJECT 09-1132-0080

W.P.

LOCATION N 4682255.5 ;E 329378.7

ORIGINATED BY DB/MR

DIST WEST HWY 401 / 3

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

COMPILED BY LMK/DMB

DATUM GEODETIC

DATE November 24, 2009 - December 1, 2009

CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	FIELD VANE (kPa, sensitivity)	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
178.80	GROUND SURFACE						
0.00	TOPSOIL, sandy Black						
177.95 / 0.85	SANDY SILT, some clay Loose to compact Brown	1	SS	10			
177.43 / 1.37	CLAYEY SILT, some sand Firm to stiff Brown becoming grey below about elev. 175.9m	2	SS	9			
		3	SS	13			
		4	SS	8			
174.99 / 3.81	SILTY CLAY, some sand, trace gravel Firm to stiff Grey				2.2		
		5	TO	PH		49	0 14 27 59 Oedometer
					1.6		
		6	SS	5			
					1.8		
		7	TO	PH			3 17 40 40 Oedometer
					1.3		
		8	SS	4			
					1.5		
169.27 / 9.53	CLAYEY SILT, some sand, trace gravel Soft to very stiff Grey	9	TO	PH			
					1.4		
		10	TO	PH			
		11	TO	PH		43	0 15 53 32 Oedometer
		12	SS	3			

Ground water conditions: Grout

Elevation scale: 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m^3

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 341

METRIC

PROJECT 09-1132-0080
W.P.
LOCATION N 4682255.5 ;E 329378.7
ORIGINATED BY DB/MR
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY LMK/DMB
DATUM GEODETIC
DATE November 24, 2009 - December 1, 2009
CHECKED BY

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa / CORE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, some sand, trace gravel Soft to very stiff Grey					+ 1.4		
		13	TO	PH	164		o ├──┤	8 22 42 28 Oedometer
					163	+ 1.8		
		14	SS	5	162		o	
						+ 1.8		
		15	TO	PH	161			
					160	(>95.8)+		
		16	TO	PH	159		├o──┤	5 24 39 32 Oedometer
						(>143.6)+		
		17	SS	7	158		o	
157.06 21.74	LIMESTONE, fresh, medium strong, weakly laminated, very fine to fine grained, faintly porous Light brown to grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)				157	T.C.R. (%) / S.C.R. (%) / R.Q.D. (%)		
		18	NQ RC	-	156	91 / 74 / 73		
		19	NQ RC	-	155 / 154	89 / 74 / 83		
		20	NQ RC	-	153	98 / 92 / 92		
151.55 27.25		21	NQ RC	-	152	96 / 94 / 84		
	END OF BOREHOLE Borehole dry during drilling between Nov. 24 and Dec. 1, 2009. Borehole sealed with cement-bentonite grout.							

Ground water conditions: Grout; Vibrating Wire Piezometer

Water level measured at elev. 180.42 on February 24, 2010.
Water level measured at elev. 180.51 on January 6, 2010.

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100
Shear Strength kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE — 20 40 60 80 100
Plastic Limit W_P, Natural Moisture Content W, Liquid Limit W_L — Water Content (%) 10 20 30
Unit Weight γ kN/m³

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

PROJECT: 09-1132-0080

RECORD OF DRILLHOLE: 341

LOCATION: N 4682255.5 ;E 329378.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 24, 2009 - December 1, 2009

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: LANTECH

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	ELEVATION	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE	157.06				
22	MUD ROTARY / NQ ROCK CORE	LIMESTONE, fresh, medium strong, weakly laminated, very fine to fine grained, faintly porous with occasional pits and vugs, light brown to grey, zones of hydrocarbon staining, fossiliferous	21.74		157		
23				1	156		
24				2	155	JN, PL, SM	
25					154		
26				3	153		
27			151.55 / 27.25	4	152		
		END OF DRILLHOLE					

Other columns: SYMBOLIC LOG; PENETRATION RATE (m/min); FLUSH COLOUR % RETURN; RECOVERY TOTAL CORE % (80 60 40 20); SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 (5 10 15 20); DIP w.r.t. CORE AXIS (0 30 60 90); HYDRAULIC CONDUCTIVITY k, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6).

Depth scale continues: 28, 29, 30, 31, 32, 33, 34, 35, 36

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: SG

CHECKED:

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-159

METRIC

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

LOCATION N 4682292.8 ;E 329332.1
BOREHOLE TYPE POWER AUGER, SOLID STEM
DATE August 12, 2008

ORIGINATED BY CC
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
178.77	GROUND SURFACE										
0.00	TOPSOIL, sandy silt, trace clay Loose Black										
177.86 0.91	FILL, silt, some sand, some clay, trace topsoil, trace organics Loose Mottled brown and grey		1	SS	4		178		o		
177.55 1.22									o		
176.94 1.83	CLAYEY SILT, trace sand Firm Mottled brown and grey		2	SS	5		177		o		
	END OF BOREHOLE Borehole dry during drilling on August 12, 2008.										

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-339

METRIC

PROJECT 09-1132-0080
W.P.
LOCATION N 4682147.4 ;E 329635.6
ORIGINATED BY TA
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, SOLID STEM
COMPILED BY DMB
DATUM GEODETIC
DATE December 8, 2009
CHECKED BY

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.53	GROUND SURFACE						
0.00	TOPSOIL, sandy Black						
0.15	SANDY SILT, some clay, trace gravel, with sand pockets, silt partings and seams Compact Brown	1	SS	16			
		2	SS	21		~23	
177.40 2.13	CLAYEY SILT, some sand, trace gravel Very stiff Grey	3	SS	18			
176.56 2.97	SILTY CLAY, some sand, trace gravel Very stiff Grey	4	SS	18		~26	
175.87 3.66	END OF BOREHOLE Borehole dry during drilling on December 8, 2009.						

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; ELEVATION SCALE (179, 178, 177, 176); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No CPT-340

METRIC

PROJECT 09-1132-0080

W.P.

LOCATION N 4682203.2 ;E 329538.7

ORIGINATED BY TA

DIST WEST HWY 401 / 3

BOREHOLE TYPE POWER AUGER, SOLID STEM

COMPILED BY DMB

DATUM GEODETIC

DATE December 10, 2009

CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS
179.58	GROUND SURFACE						
0.00	TOPSOIL, sandy						
179.25	Black						
0.33	SANDY SILT, some clay, trace gravel, with silt partings Loose to compact Brown					179	
		1	SS	9			
		2	SS	10		178	
177.19							
2.39	CLAYEY SILT, some sand, trace gravel, with occasional silt partings Very stiff	3	SS	16		177	
176.68	Very stiff						
2.90	Grey						
	END OF BOREHOLE						
	Borehole dry during drilling on December 10, 2009.						

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

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

PROJECT: 07-1130-207-0

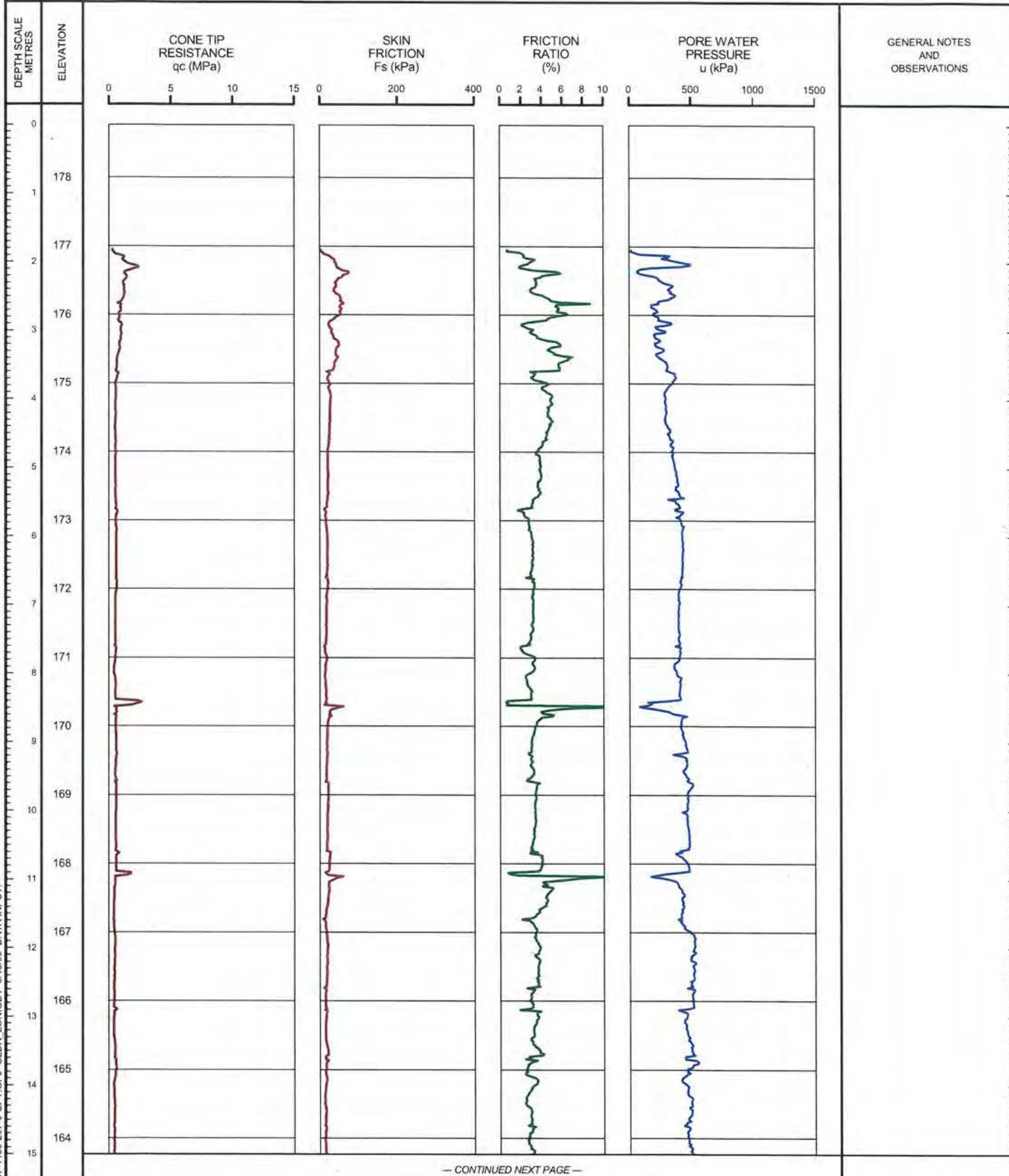
RECORD OF CONE PENETRATION TEST CPT-159

LOCATION: N 4682292.8 ;E 329332.1

TEST DATE: August 12, 2008

DATUM: GEODETIC

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



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DEPTH SCALE
1 : 75

Golder Associates

OPERATOR: CC
CHECKED: SSB

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

PROJECT: 07-1130-207-0

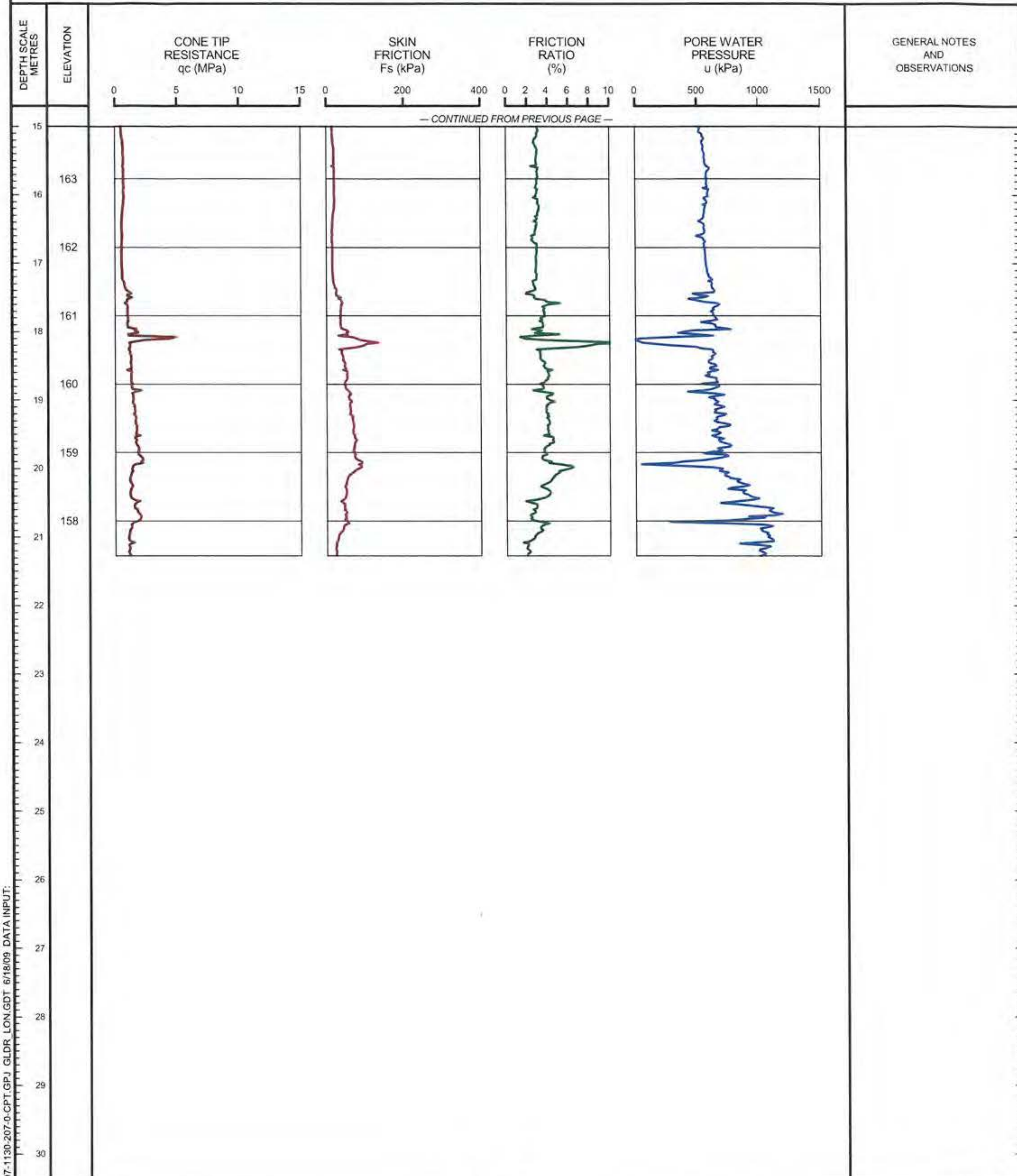
RECORD OF CONE PENETRATION TEST CPT-159

LOCATION: N 4682292.8 ;E 329332.1

TEST DATE: August 12, 2008

DATUM: GEODETIC

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



DEPTH SCALE

1 : 75

Golder Associates

OPERATOR: CC

CHECKED: SJB

RECORD OF CONE PENETRATION TEST CPT-339

PROJECT: 09-1132-0080

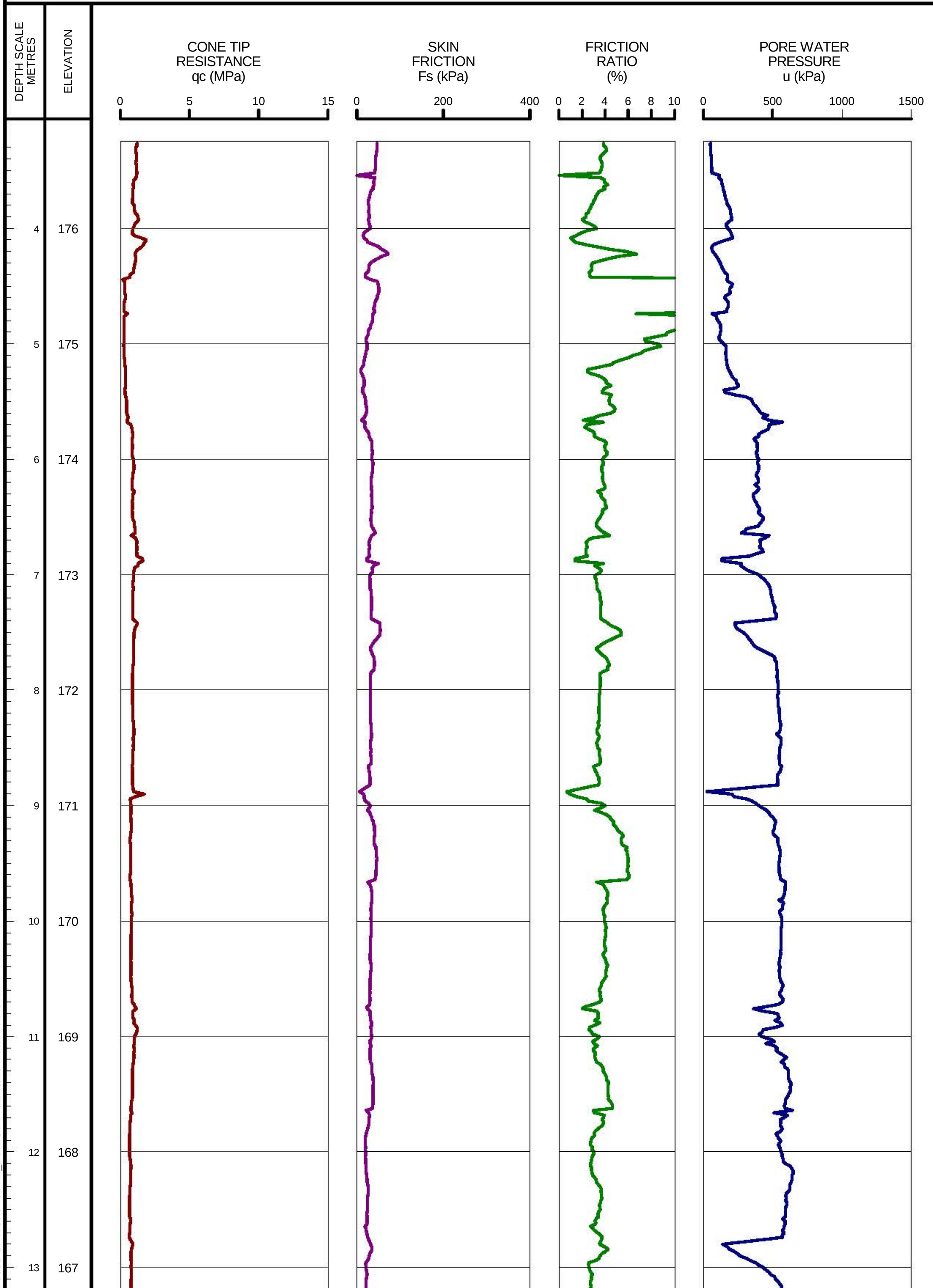
SHEET 1 OF 2

LOCATION: N 4682147.4 ;E 329635.6

TEST DATE: December 8, 2009 - December 9, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.53m PREDRILL DEPTH: 3.25m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



GENERAL NOTES AND OBSERVATIONS

CPT PUSHED AFTER PREDRILLING WITHOUT SAMPLING.
FOR SOIL PROFILE, SEE RECORD OF BOREHOLE No. CPT-339, DRILLED 3.0 m SOUTH OF THE CPT LOCATION.
DISSIPATION TESTS WERE CONDUCTED AT THE FOLLOWING DEPTHS: 4.44 m, 7.39 m and 12.74 m.

--- *CONTINUED NEXT PAGE* ---

DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

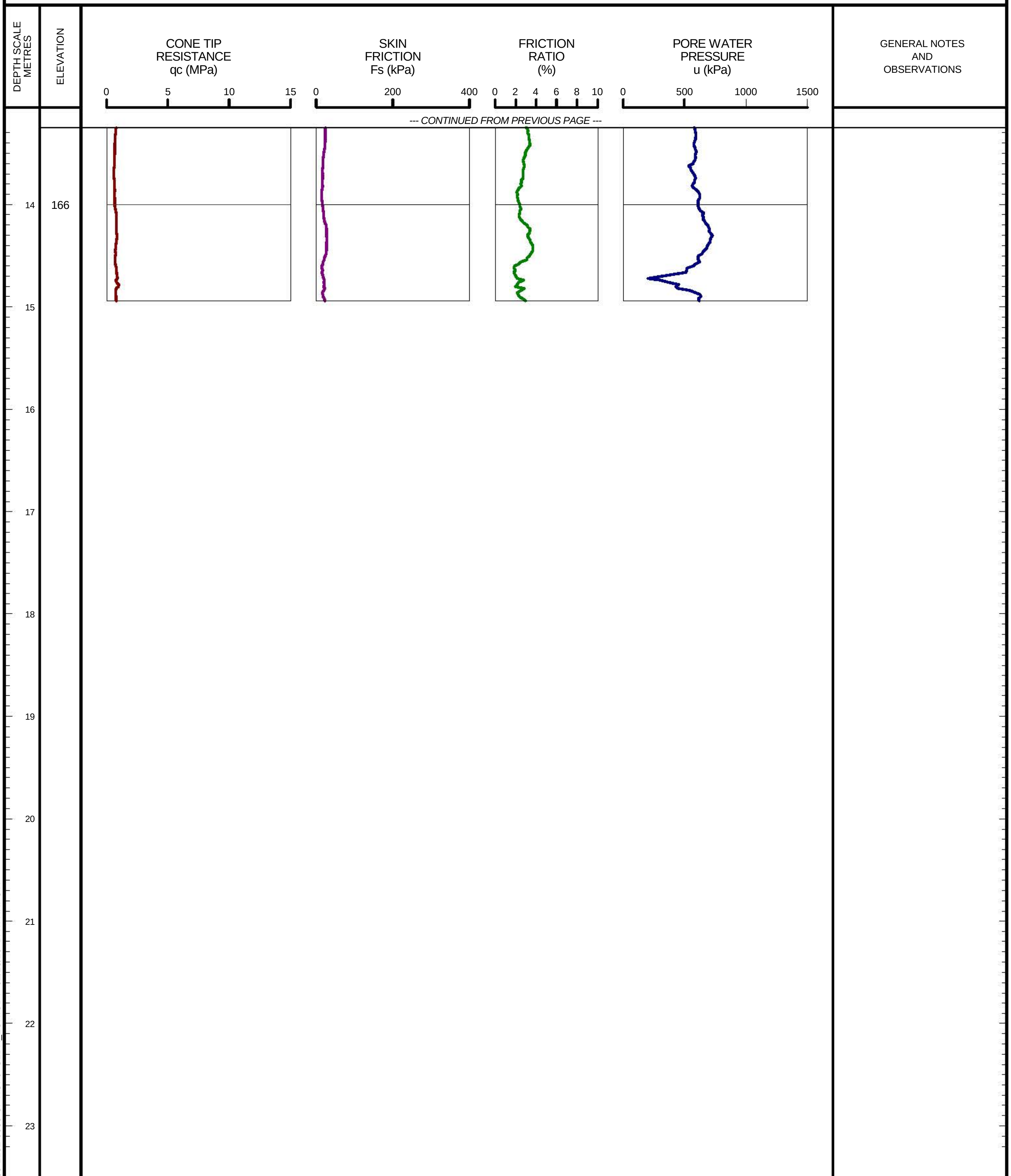
RECORD OF CONE PENETRATION TEST CPT-339

LOCATION: N 4682147.4 ;E 329635.6

TEST DATE: December 8, 2009 - December 9, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.53m PREDRILL DEPTH: 3.25m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 50

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

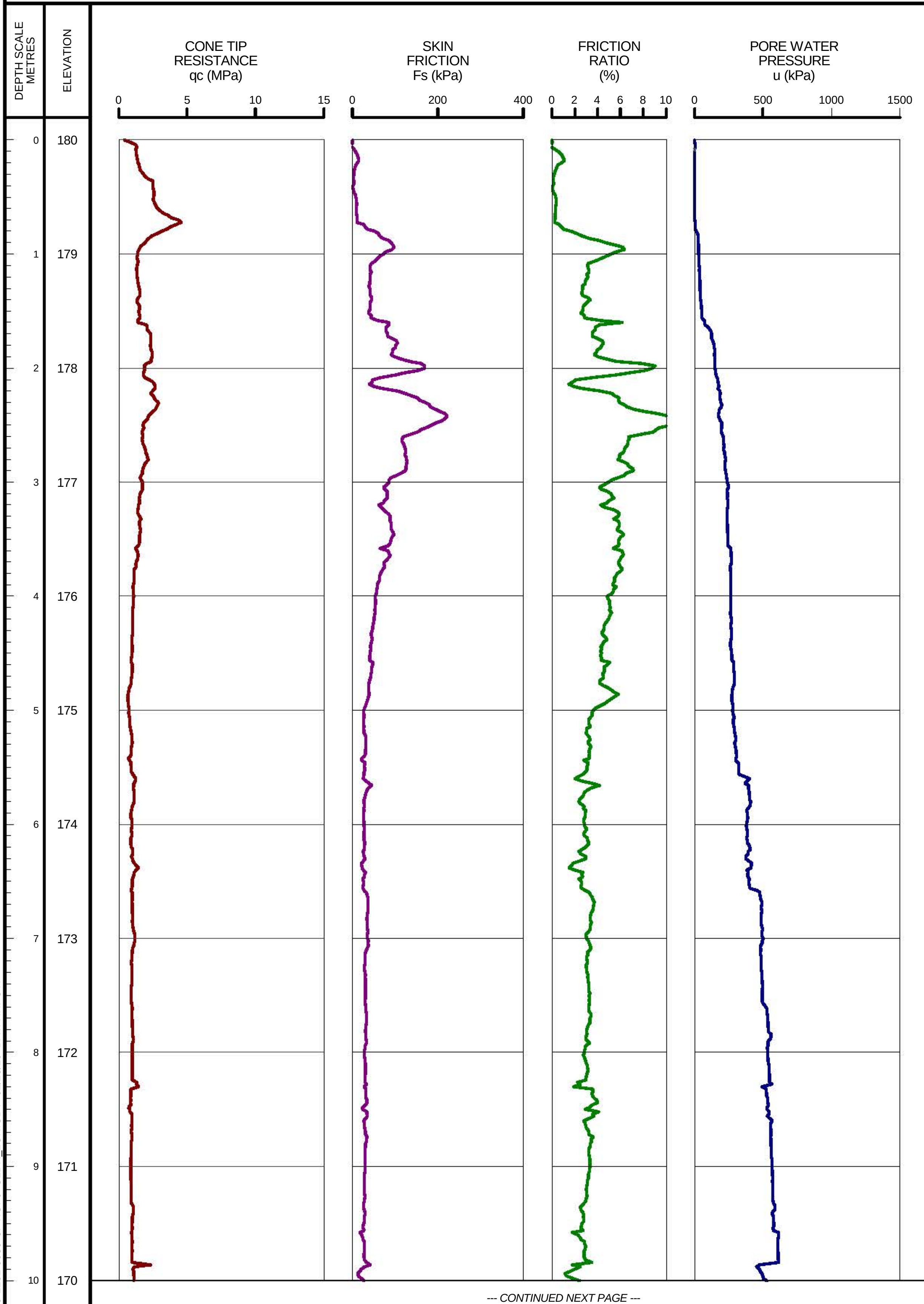
RECORD OF CONE PENETRATION TEST CPT-339A

LOCATION: N 4682149.4 ;E 329635.6

TEST DATE: December 9, 2009 - December 10, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.53m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



GENERAL NOTES AND OBSERVATIONS

CPT PUSHED FROM SURFACE ADJACENT TO CPT-339.
DISSIPATION TESTS WERE CONDUCTED AT THE FOLLOWING DEPTHS: 15.05 m and 19.60 m.

--- CONTINUED NEXT PAGE ---

DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

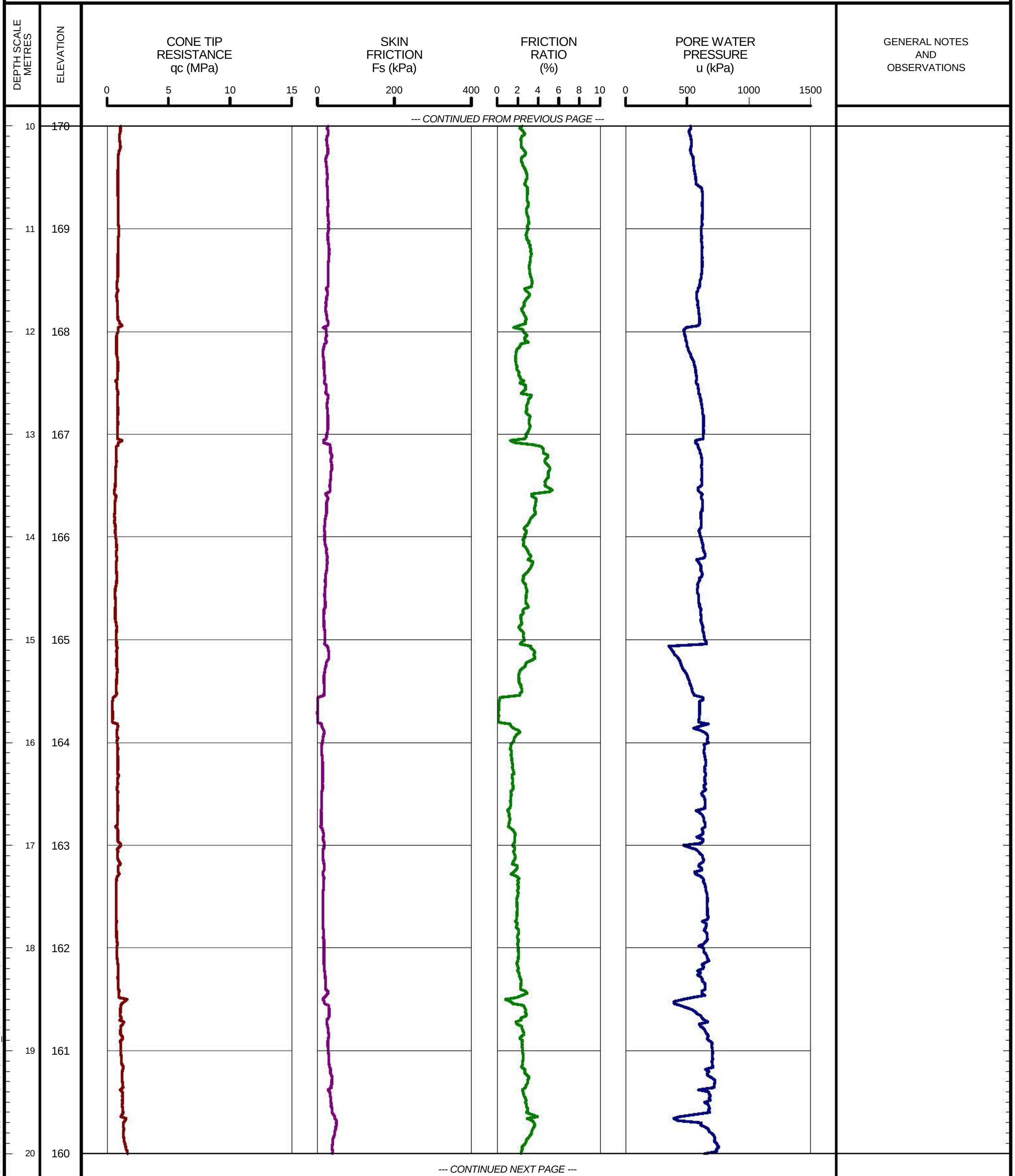
RECORD OF CONE PENETRATION TEST CPT-339A

LOCATION: N 4682149.4 ;E 329635.6

TEST DATE: December 9, 2009 - December 10, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.53m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

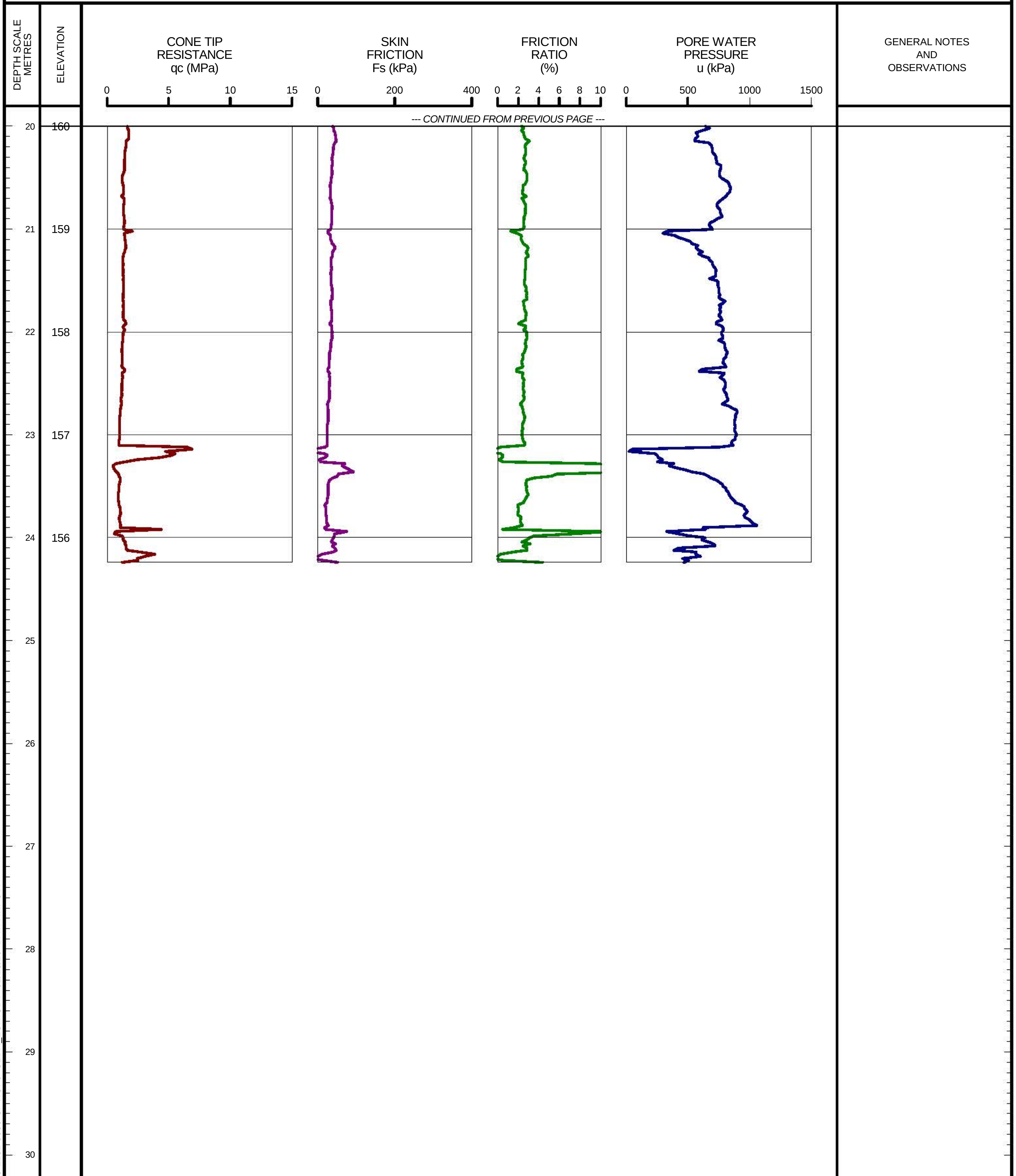
RECORD OF CONE PENETRATION TEST CPT-339A

LOCATION: N 4682149.4 ;E 329635.6

TEST DATE: December 9, 2009 - December 10, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.53m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 50

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

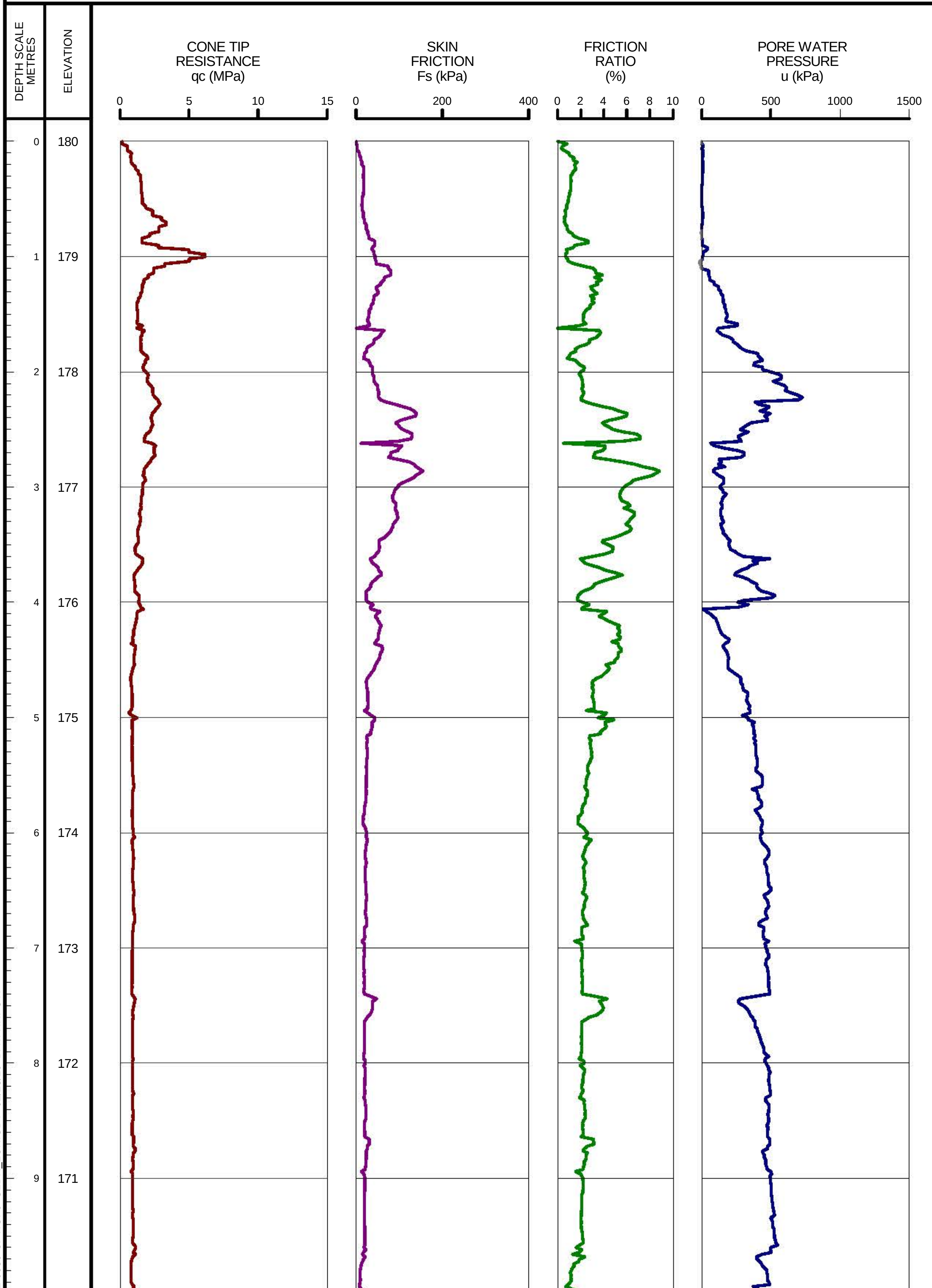
RECORD OF CONE PENETRATION TEST CPT-340

LOCATION: N 4682203.2 ;E 329538.7

TEST DATE: December 10, 2009 - December 15, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.58m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



GENERAL NOTES AND OBSERVATIONS

CPT PUSHED FROM SURFACE.
FOR SOIL PROFILE, SEE RECORD OF ADJACENT BOREHOLE No. CPT-340.
DISSIPATION TESTS WERE CONDUCTED AT THE FOLLOWING DEPTHS: 4.35 m, 7.40 m, 12.75 m, 15.08 m and 19.59 m.

--- CONTINUED NEXT PAGE ---

DEPTH SCALE

1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

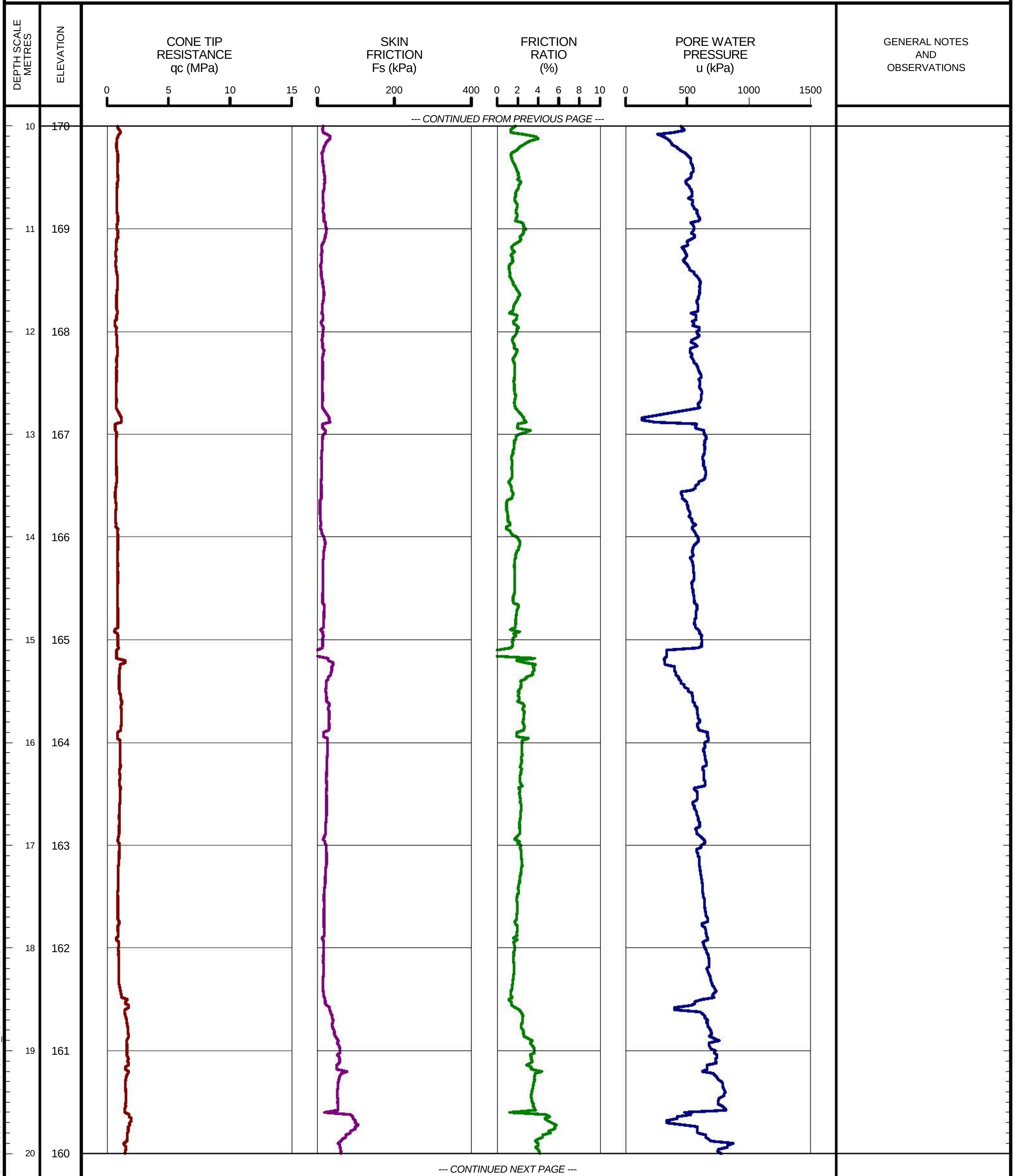
RECORD OF CONE PENETRATION TEST CPT-340

LOCATION: N 4682203.2 ;E 329538.7

TEST DATE: December 10, 2009 - December 15, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.58m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 50

OPERATOR: TA

CHECKED:

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-340

LOCATION: N 4682203.2 ;E 329538.7

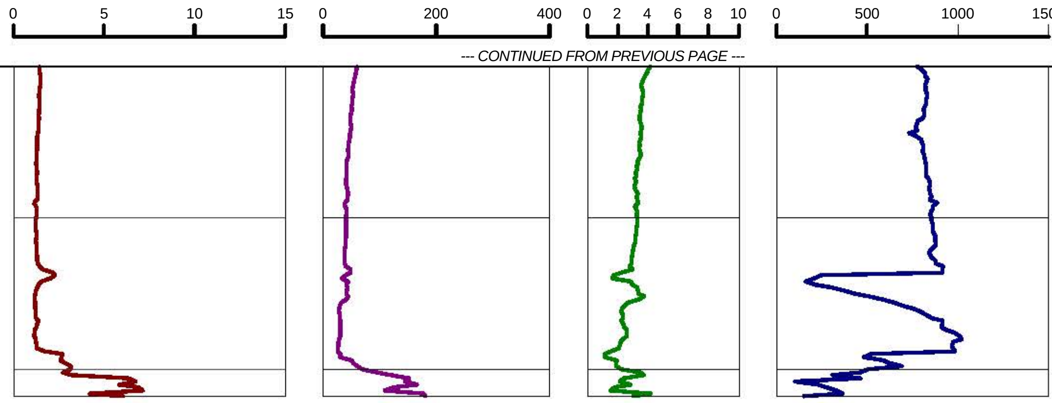
TEST DATE: December 10, 2009 - December 15, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.58m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012

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

--- CONTINUED FROM PREVIOUS PAGE ---



DEPTH SCALE METRES	ELEVATION
20	160
21	159
22	158
23	
24	
25	
26	
27	
28	
29	
30	

DEPTH SCALE

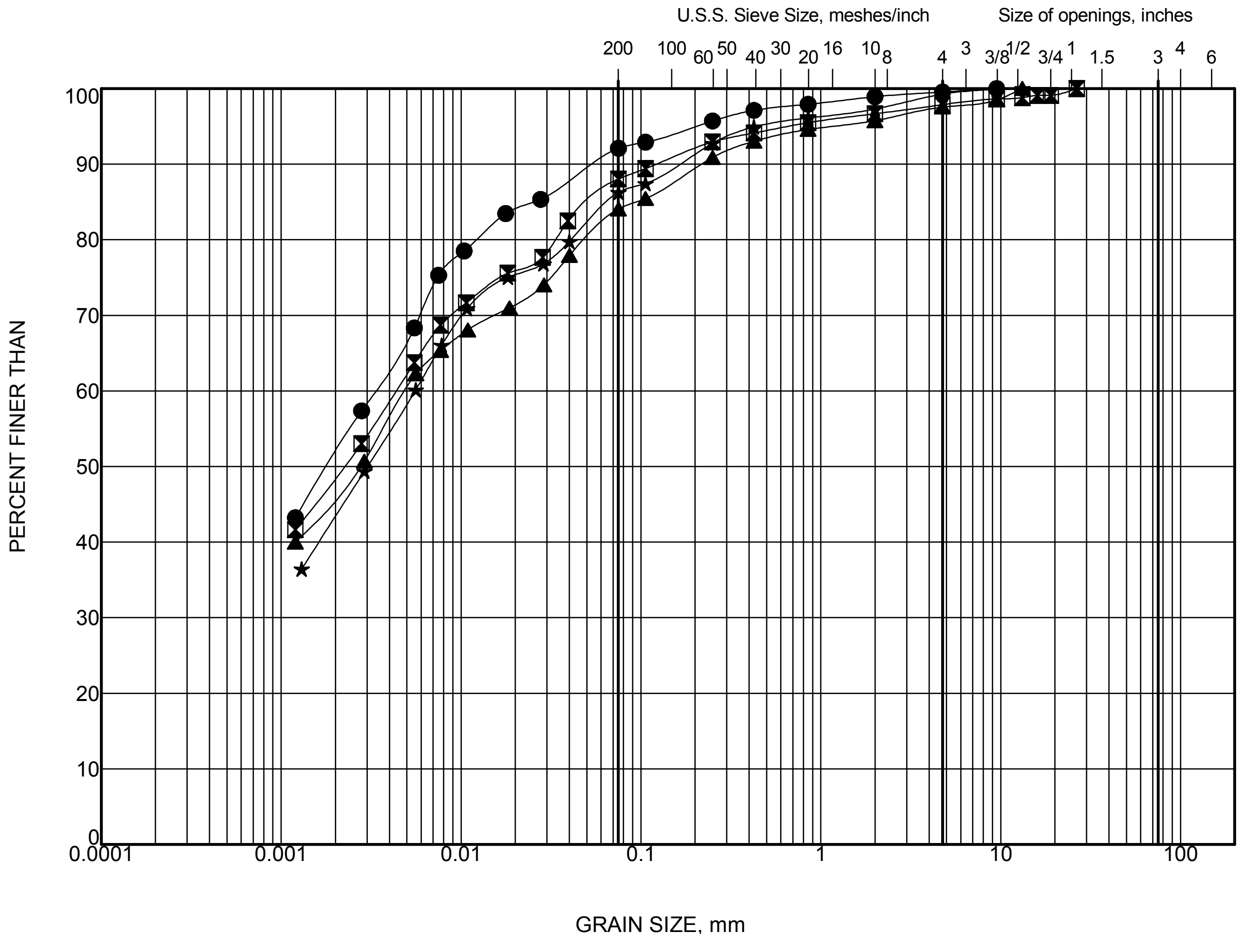
1 : 50

Golder Associates

OPERATOR: TA

CHECKED:

Appendix C Geotechnical Laboratory Test Results



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

LEGEND:

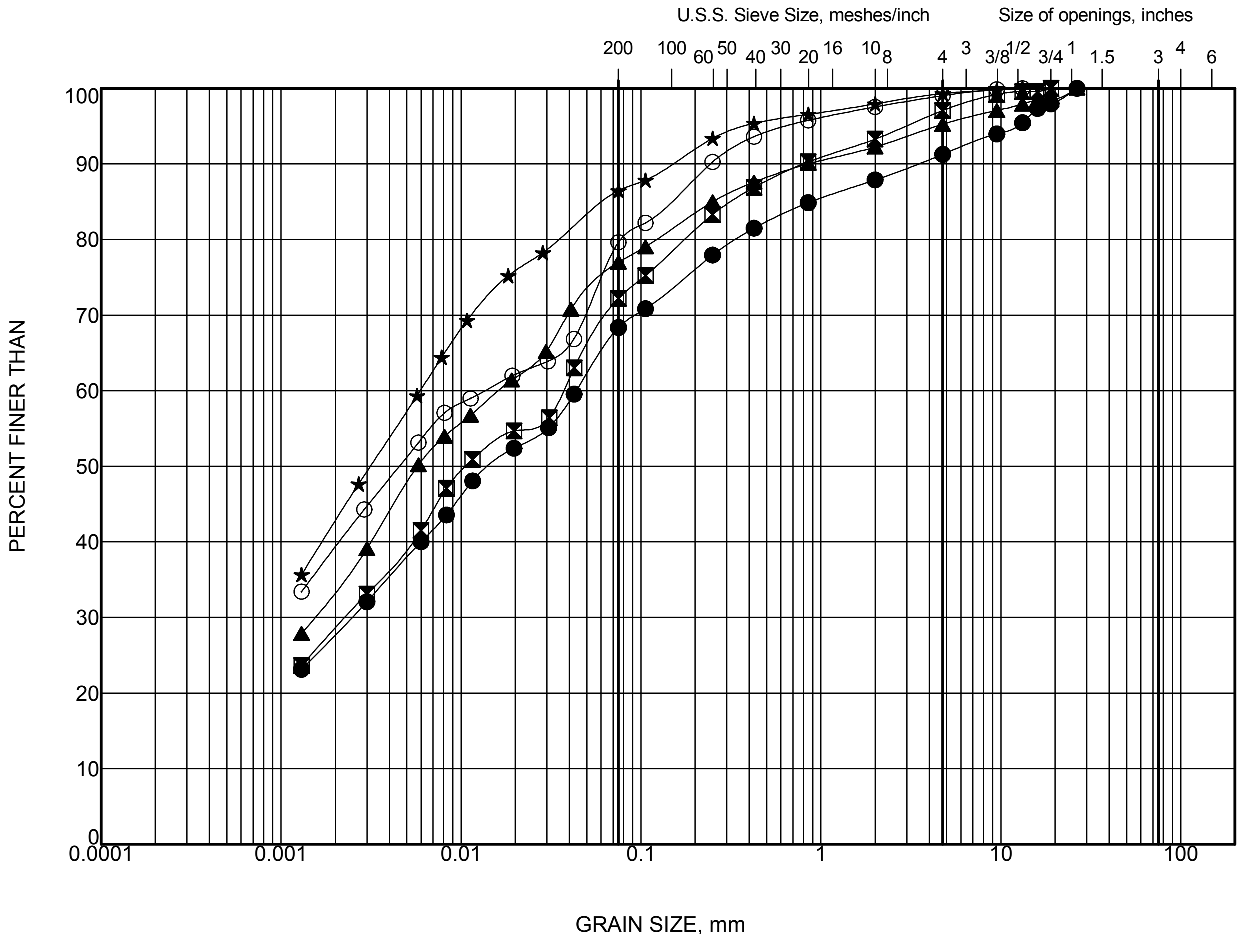
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B3-1	8	6.1
⧗	B3-1	12	12.2
▲	B3-2	6	4.6
★	B3-2	10	9.1

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION
SILTY CLAY

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.	
		SCALE	REV.
DRAWN	EA	FIGURE C.1	
CHECK	MSO		



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

LEGEND:

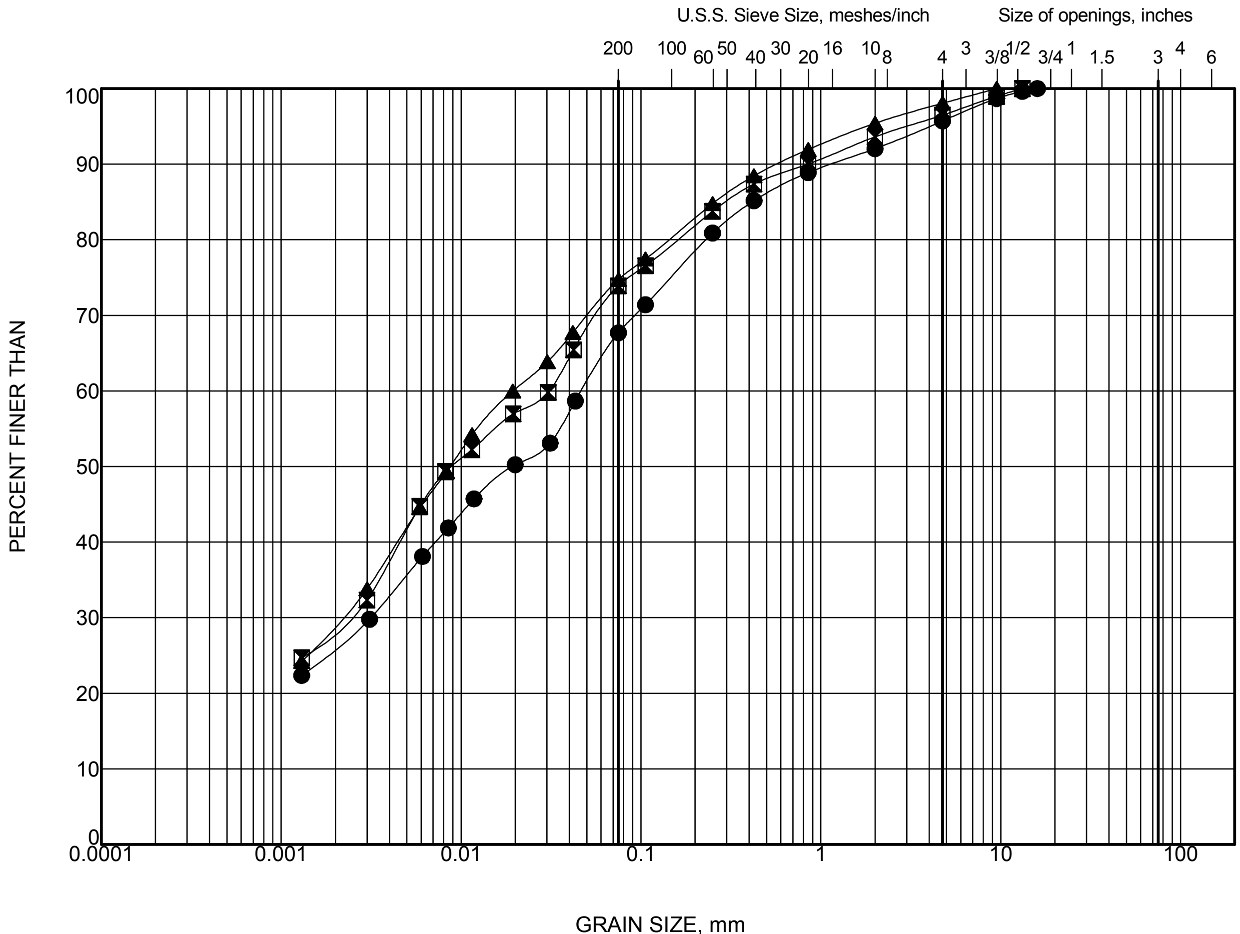
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B3-1	16	18.3
⊠	B3-2	14	15.2
▲	B3-2	16	18.3
★	B3-3	9	7.6
⊙	B3-3	11	10.7

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION
CLAYEY SILT

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.	
		SCALE	REV.
DRAWN	EA	FIGURE C.2	
CHECK	MSO		



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

LEGEND:

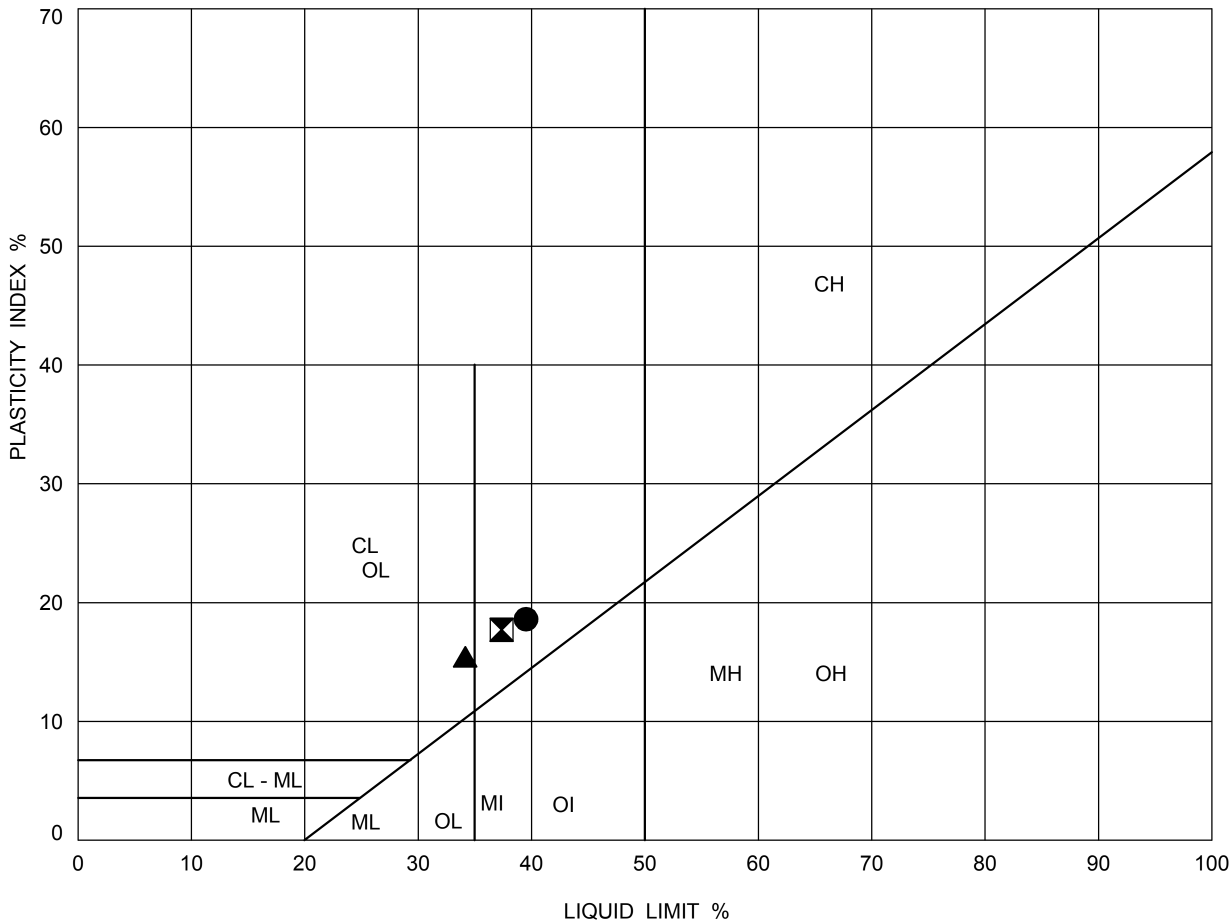
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	B3-3	13	13.7
⧗	B3-3	15	16.8
▲	B3-3	17	19.8

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
GRAIN SIZE DISTRIBUTION
CLAYEY SILT

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

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



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B3-1	8	6.1	**40**	**21**	**19**
⧗	B3-1	12	12.2	**37**	**20**	**17**
▲	B3-2	10	9.1	**34**	**19**	**15**

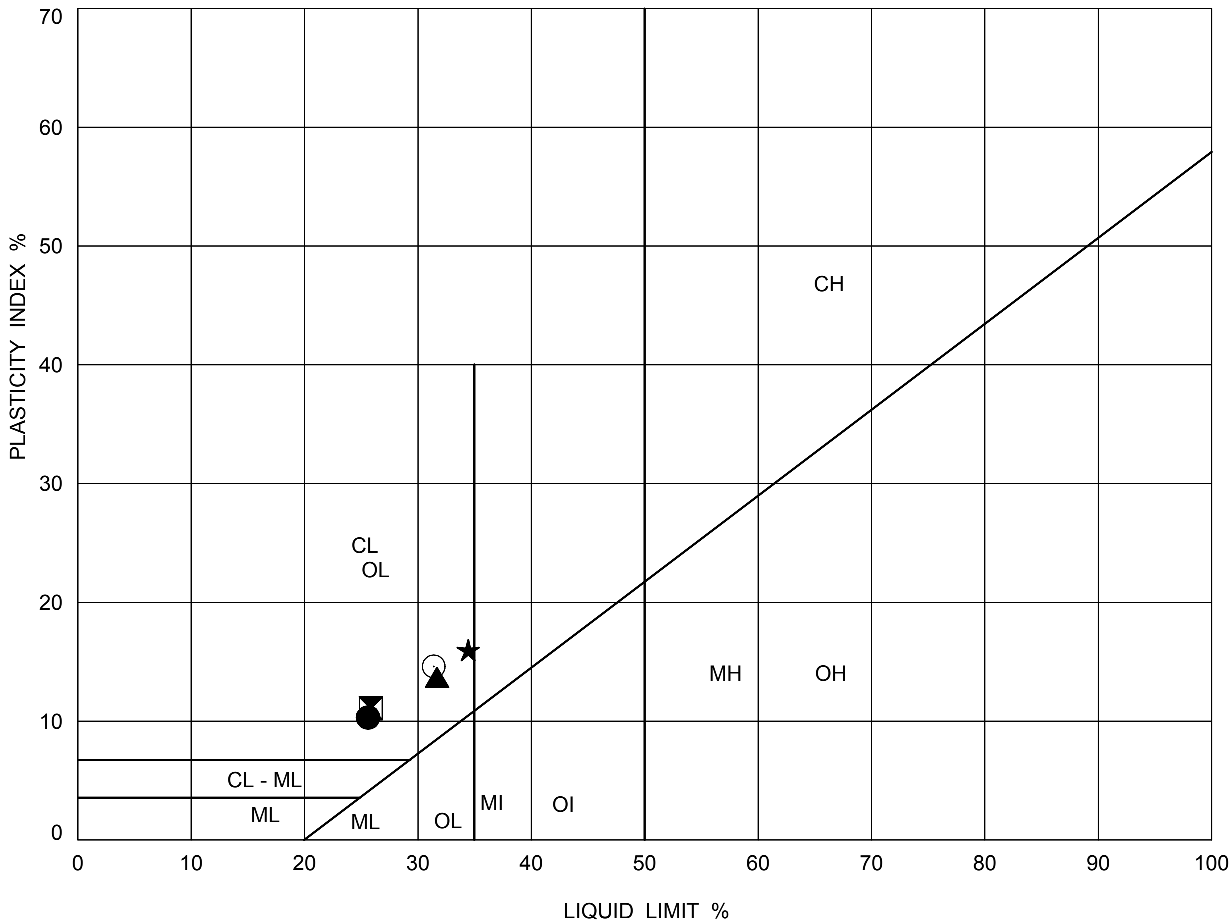
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
PLASTICITY CHART
SILTY CLAY

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PROJECT No. SW8801.1004.101		FILE No.	
		SCALE	REV.
DRAWN	EA		
CHECK	MSO	FIGURE **C.4**	

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 16/04/12



SOIL TYPE

C = Clay
M = Silt
O = Organic

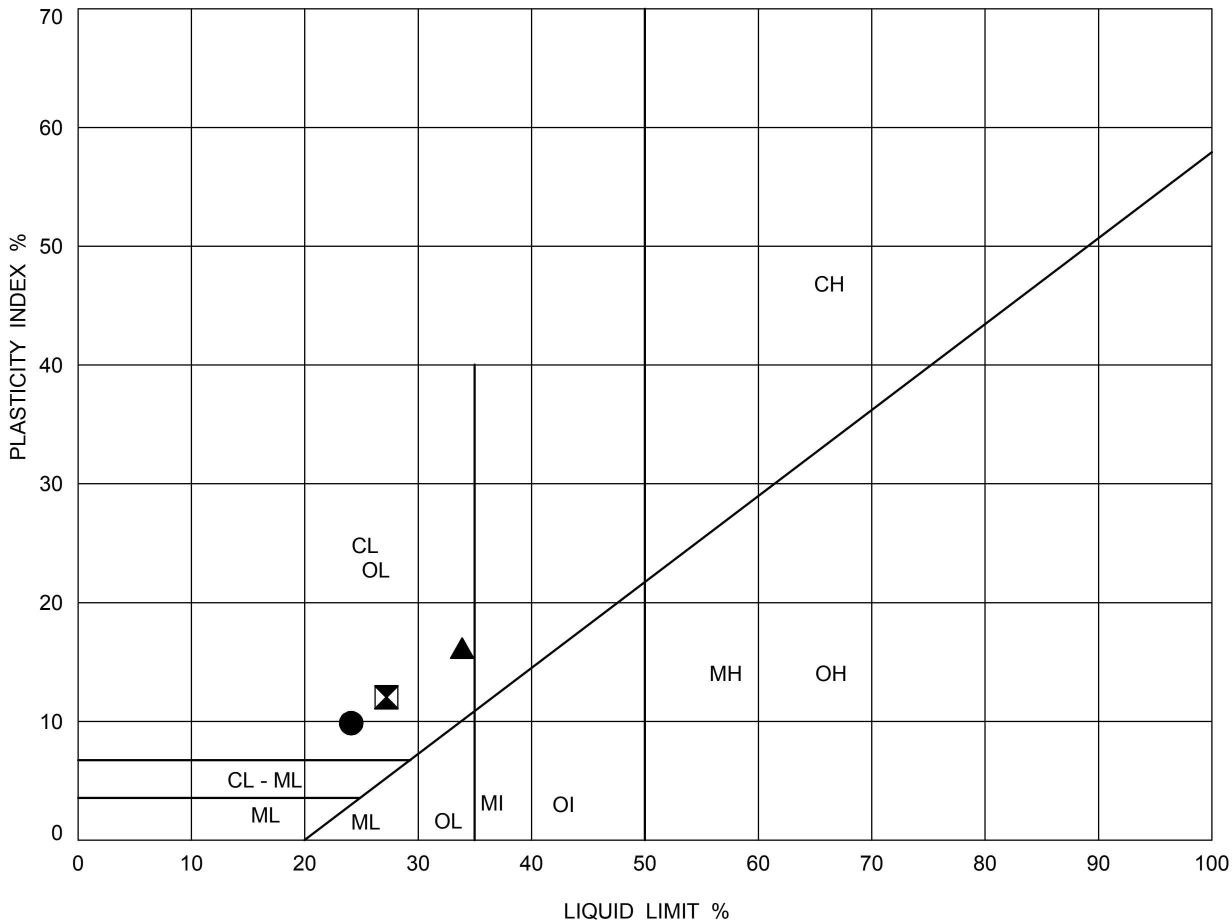
PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B3-1	16	18.3	**26**	**15**	**11**
⧗	B3-2	14	15.2	**26**	**15**	**11**
▲	B3-2	16	18.3	**32**	**18**	**14**
★	B3-3	9	7.6	**34**	**18**	**16**
⊙	B3-3	11	10.7	**31**	**17**	**14**

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario		
TITLE	PLASTICITY CHART **CLAYEY SILT**		
Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PROJECT No. SW8801.1004.101	FILE No.	
		SCALE	REV.
	DRAWN EA	FIGURE **C.5**	
	CHECK MSO		



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

LEGEND:

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)	LL(%)	PL(%)	PI
●	B3-3	13	13.7	**24**	**14**	**10**
⧗	B3-3	15	16.8	**27**	**15**	**12**
▲	B3-3	17	19.8	**34**	**18**	**16**

PROJECT	Windsor Essex Parkway (WEP) Windsor, Ontario
TITLE	PLASTICITY CHART **CLAYEY SILT**
Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PROJECT No. SW8801.1004.101 DRAWN EA CHECK MSO FILE No. SCALE REV. FIGURE **C.6**

Appendix D Analytical Laboratory Test Results

AMEC EARTH & ENVIRONMENTAL
ATTN: Brian Lapos
11865 County Road 42
TECUMSEH ON N8N 2M1

Date Received: 30-JUN-11
Report Date: 08-JUL-11 07:11 (MT)
Version: FINAL

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: **L1025369**
Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 092732-4

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1025369-1				
	Description	SOIL				
	Sampled Date	29-JUN-11				
	Sampled Time					
	Client ID	B3-1 SA#18 72'				
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	14.3				
	pH (pH units)	7.87				
	Redox Potential (mV)	195				
	Resistivity (ohm cm)	2170				
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20				
Anions and Nutrients	Sulphate (mg/kg)	246				

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
MOISTURE-WT	Soil	% Moisture	Gravimetric: Oven Dried
PH-WT	Soil	pH	MOEE E3137A

Soil samples are mixed in the deionized water and the supernatant is analyzed directly by the pH meter.

ALS Test Code	Matrix	Test Description	Method Reference**
REDOX-POTENTIAL-WT	Soil	Redox Potential	APHA 2580
RESISTIVITY-WT	Soil	Resistivity	MOEE E3137A
SO4-WT	Soil	Sulphate	EPA 300.0
SULPHIDE-WT	Soil	Sulphide	APHA 4500S2D

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

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

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

Chain of Custody Numbers:

092732-4

GLOSSARY OF REPORT TERMS

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

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

Quality Control Report

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

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT	**Soil**							
Batch R2212765								
WG1305352-2 LCS								
% Moisture			92		%		70-130	30-JUN-11
WG1305352-1 MB								
% Moisture			<0.10		%		0.1	30-JUN-11
PH-WT	**Soil**							
Batch R2214528								
WG1307906-1 CVS								
pH			100		%		80-120	06-JUL-11
RESISTIVITY-WT	**Soil**							
Batch R2215155								
WG1308646-1 CVS								
Resistivity			100		%		70-130	07-JUL-11
SO4-WT	**Soil**							
Batch R2213607								
WG1306314-3 LCS								
Sulphate			101		%		60-140	04-JUL-11
WG1306314-1 MB								
Sulphate			<20		mg/kg		20	04-JUL-11
SULPHIDE-WT	**Soil**							
Batch R2213798								
WG1307079-1 CVS								
Sulphide			79		%		50-120	05-JUL-11
WG1307075-1 MB								
Sulphide			<0.20		mg/kg		0.2	05-JUL-11

Quality Control Report

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential							
	1	29-JUN-11	07-JUL-11 17:07	24	197	hours	EHTL
Resistivity							
	1	29-JUN-11	07-JUL-11 17:04	7	8	days	EHT

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 L1025369 were received on 30-JUN-11 11:00.

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.

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

Date Received: 29-JUL-11
Report Date: 05-AUG-11 07:45 (MT)
Version: FINAL REV. 2

Client Phone: 519-735-2499

Certificate of Analysis

Lab Work Order #: L1037976
Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 112836

Comments:

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

Gayle Braun
Senior Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1037976-1	L1037976-2			
	Description	SOIL	SOIL			
	Sampled Date	28-JUL-11	28-JUL-11			
	Sampled Time					
	Client ID	B3-2,SA#16@60',SILTY CLAY,GREY	B3-3,SA#1@2.5',SILTY CLAY,BROWN			
Grouping	**Analyte**					
SOIL						
Physical Tests	% Moisture (%)	16.9	19.1			
	pH (pH units)	7.65	7.70			
	Redox Potential (mV)	158	147			
	Resistivity (ohm cm)	2580	7410			
Leachable Anions & Nutrients	Sulphide (mg/kg)	<0.20	<0.20			
Anions and Nutrients	Sulphate (mg/kg)	449	<20			

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
MOISTURE-WT	Soil	% Moisture	Gravimetric: Oven Dried
PH-WT	Soil	pH	MOEE E3137A

Soil samples are mixed in the deionized water and the supernatant is analyzed directly by the pH meter.

ALS Test Code	Matrix	Test Description	Method Reference**
REDOX-POTENTIAL-WT	Soil	Redox Potential	APHA 2580
RESISTIVITY-WT	Soil	Resistivity	MOEE E3137A
SO4-WT	Soil	Sulphate	EPA 300.0
SULPHIDE-WT	Soil	Sulphide	APHA 4500S2D

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

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

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

Chain of Custody Numbers:

112836

GLOSSARY OF REPORT TERMS

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

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

Quality Control Report

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

Contact: SHANE MACLEOD

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
MOISTURE-WT	**Soil**							
Batch 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
WG1323310-3 DUP		**L1037976-1**						
Resistivity		2580	2700		ohm cm	4.7	25	04-AUG-11
SO4-WT	**Soil**							
Batch R2229091								
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

Legend:

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

Sample Parameter Qualifier Definitions:

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

Quality Control Report

Hold Time Exceedances:

ALS Product Description	Sample ID	Sampling Date	Date Processed	Rec. HT	Actual HT	Units	Qualifier
Physical Tests							
Redox Potential							
	1	28-JUL-11	04-AUG-11 07:35	24	164	hours	EHTL
	2	28-JUL-11	04-AUG-11 07:36	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 L1037976 were received on 29-JUL-11 10:13.

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 Rock Core Photographs

Photo 1: Borehole B3-1 - Rock Core. Elevation 157.6 meters to 154.7 meters

Photo 2 Borehole B3-2 - Rock Core. Elevation 156.0 meters to 153.6 meters

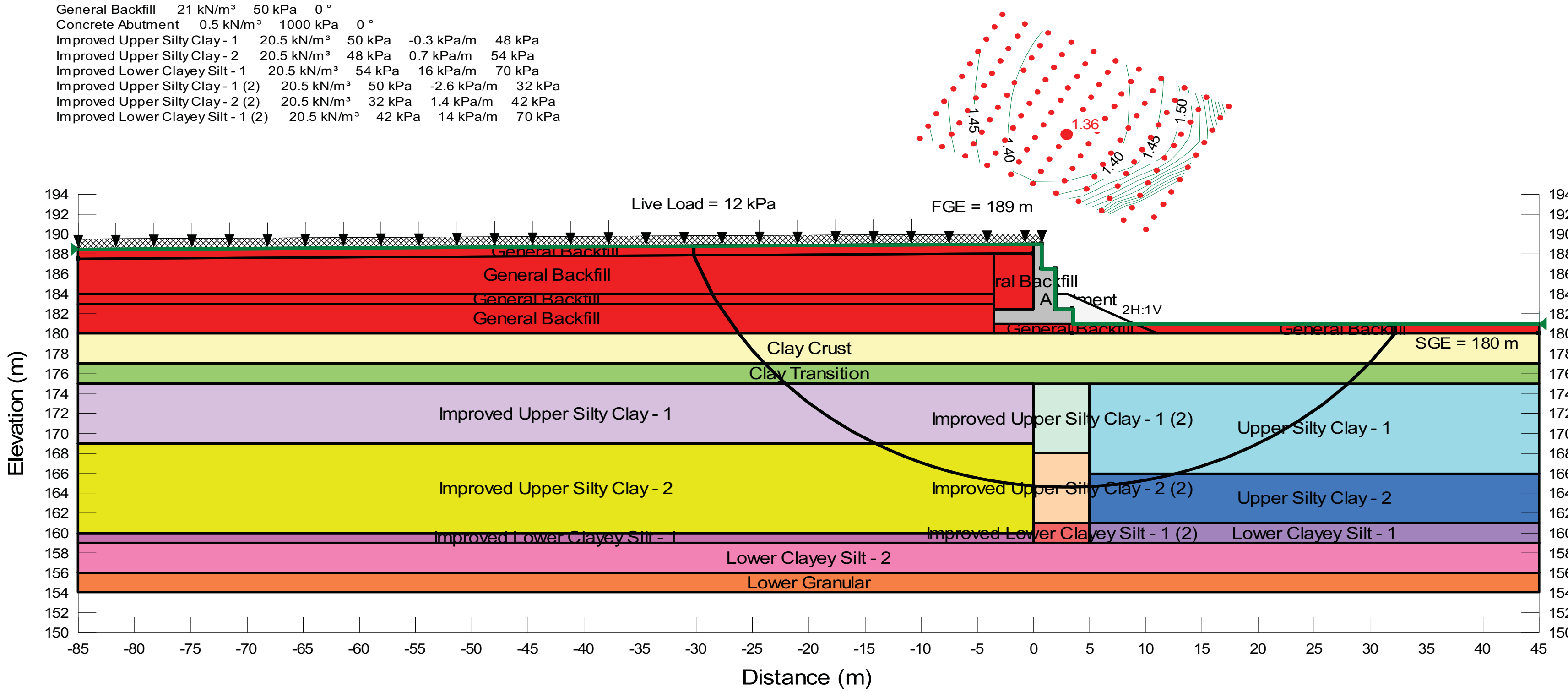
Photo 3   Borehole B6-3 - Rock Core. Elevation 153.7 meters to 152.2 meters

Appendix F Slope Stability Analyses Results

Bridge B-3-West Abutment-Undrained-Rev7.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 60 kPa 0 °
Clay Transition 22 kN/m³ 60 kPa -5 kPa/m 50 kPa
Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -2.8 kPa/m 25 kPa
Upper Silty Clay - 2 20.5 kN/m³ 25 kPa 2.4 kPa/m 37 kPa
Lower Clayey Silt - 1 20.5 kN/m³ 37 kPa 16.5 kPa/m 70 kPa
Lower Clayey Silt - 2 20.5 kN/m³ 70 kPa 0 °
Lower Granular 22 kN/m³ 0 kPa 30 °
General Backfill 21 kN/m³ 50 kPa 0 °
Concrete Abutment 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -0.3 kPa/m 48 kPa
Improved Upper Silty Clay - 2 20.5 kN/m³ 48 kPa 0.7 kPa/m 54 kPa
Improved Lower Clayey Silt - 1 20.5 kN/m³ 54 kPa 16 kPa/m 70 kPa
Improved Upper Silty Clay - 1 (2) 20.5 kN/m³ 50 kPa -2.6 kPa/m 32 kPa
Improved Upper Silty Clay - 2 (2) 20.5 kN/m³ 32 kPa 1.4 kPa/m 42 kPa
Improved Lower Clayey Silt - 1 (2) 20.5 kN/m³ 42 kPa 14 kPa/m 70 kPa



Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT: WINDSOR ESSEX PARKWAY

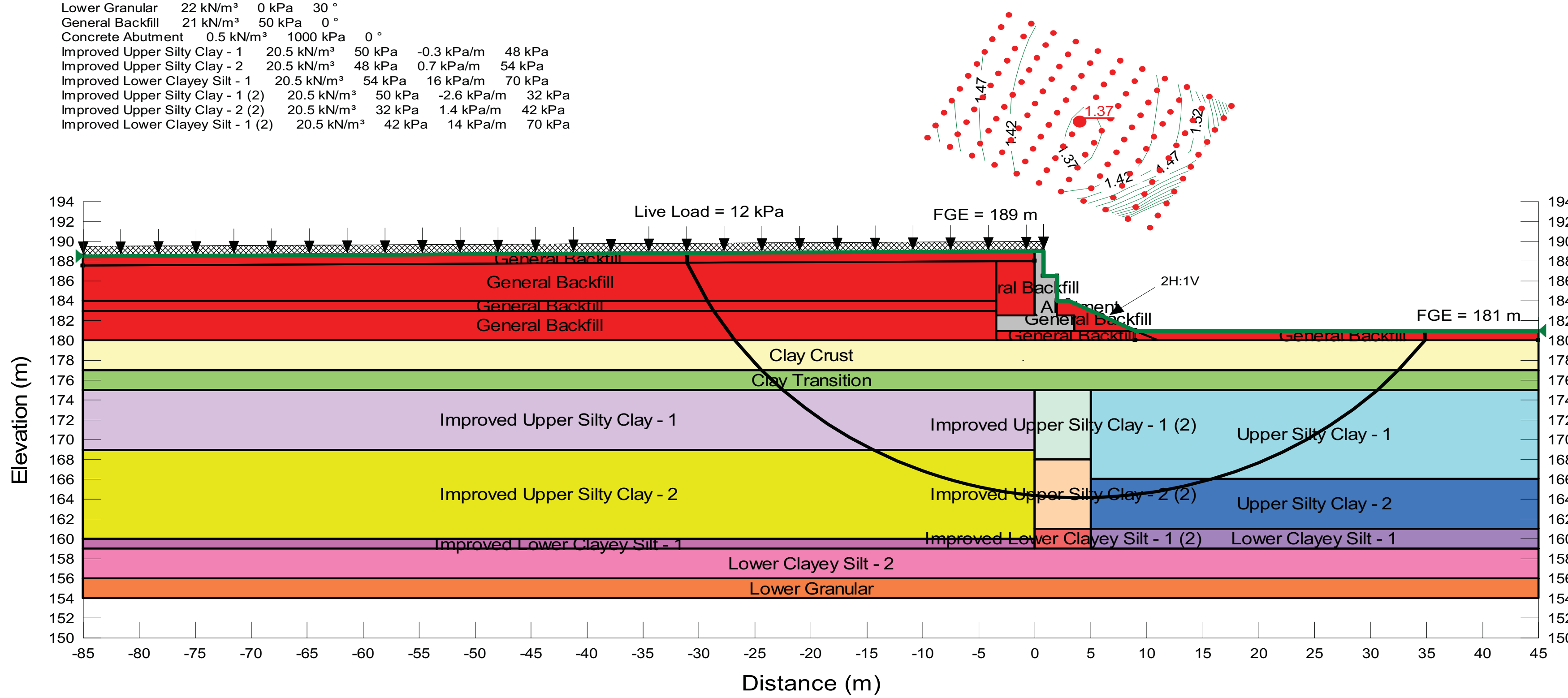
TITLE: SHORT-TERM (UNDRAINED) STABILITY ANALYSES - WEST ABUTMENT BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.:	REV.
Jun 2012			F.1	

Bridge B-3-West Abutment-Undrained-Rev7.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 60 kPa 0 °
Clay Transition 22 kN/m³ 60 kPa -5 kPa/m 50 kPa
Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -2.8 kPa/m 25 kPa
Upper Silty Clay - 2 20.5 kN/m³ 25 kPa 2.4 kPa/m 37 kPa
Lower Clayey Silt - 1 20.5 kN/m³ 37 kPa 16.5 kPa/m 70 kPa
Lower Clayey Silt - 2 20.5 kN/m³ 70 kPa 0 °
Lower Granular 22 kN/m³ 0 kPa 30 °
General Backfill 21 kN/m³ 50 kPa 0 °
Concrete Abutment 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -0.3 kPa/m 48 kPa
Improved Upper Silty Clay - 2 20.5 kN/m³ 48 kPa 0.7 kPa/m 54 kPa
Improved Lower Clayey Silt - 1 20.5 kN/m³ 54 kPa 16 kPa/m 70 kPa
Improved Upper Silty Clay - 1 (2) 20.5 kN/m³ 50 kPa -2.6 kPa/m 32 kPa
Improved Upper Silty Clay - 2 (2) 20.5 kN/m³ 32 kPa 1.4 kPa/m 42 kPa
Improved Lower Clayey Silt - 1 (2) 20.5 kN/m³ 42 kPa 14 kPa/m 70 kPa



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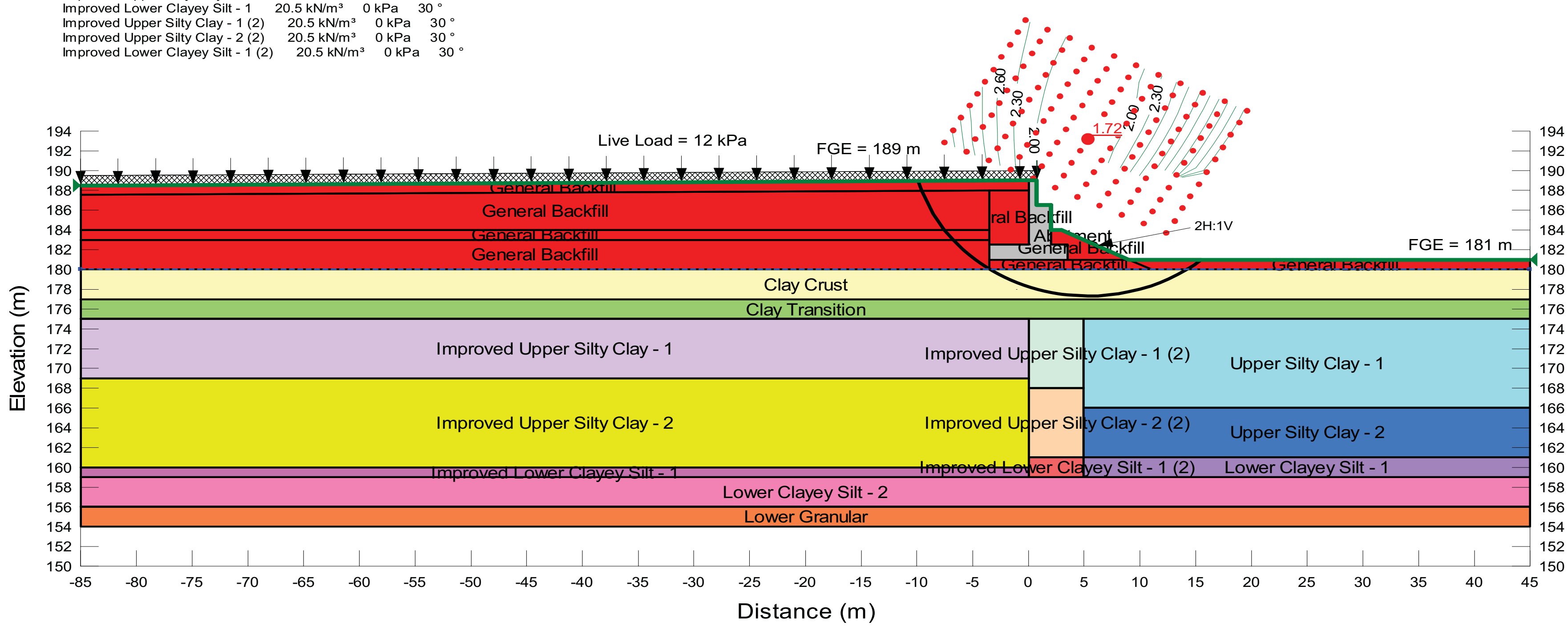
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: END OF CONSTRUCTION (UNDRAINED) STABILITY ANALYSES - WEST ABUTMENT BRIDGE B-3				
DATE: Jun 2012	JOB NO.:	CAD FILE:	**FIGURE NO.: F.2**	REV.

Bridge B-3-West Abutment-Drained-Rev7.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 0 kPa 30 °
Clay Transition 22 kN/m³ 0 kPa 30 °
Upper Silty Clay - 1 20.5 kN/m³ 0 kPa 30 °
Upper Silty Clay - 2 20.5 kN/m³ 0 kPa 30 °
Lower Clayey Silt - 1 20.5 kN/m³ 0 kPa 30 °
Lower Clayey Silt - 2 20.5 kN/m³ 0 kPa 30 °
Lower Granular 22 kN/m³ 0 kPa 30 °
General Backfill 21 kN/m³ 0 kPa 30 °
Concrete Abutment 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 0 kPa 30 °
Improved Upper Silty Clay - 2 20.5 kN/m³ 0 kPa 30 °
Improved Lower Clayey Silt - 1 20.5 kN/m³ 0 kPa 30 °
Improved Upper Silty Clay - 1 (2) 20.5 kN/m³ 0 kPa 30 °
Improved Upper Silty Clay - 2 (2) 20.5 kN/m³ 0 kPa 30 °
Improved Lower Clayey Silt - 1 (2) 20.5 kN/m³ 0 kPa 30 °



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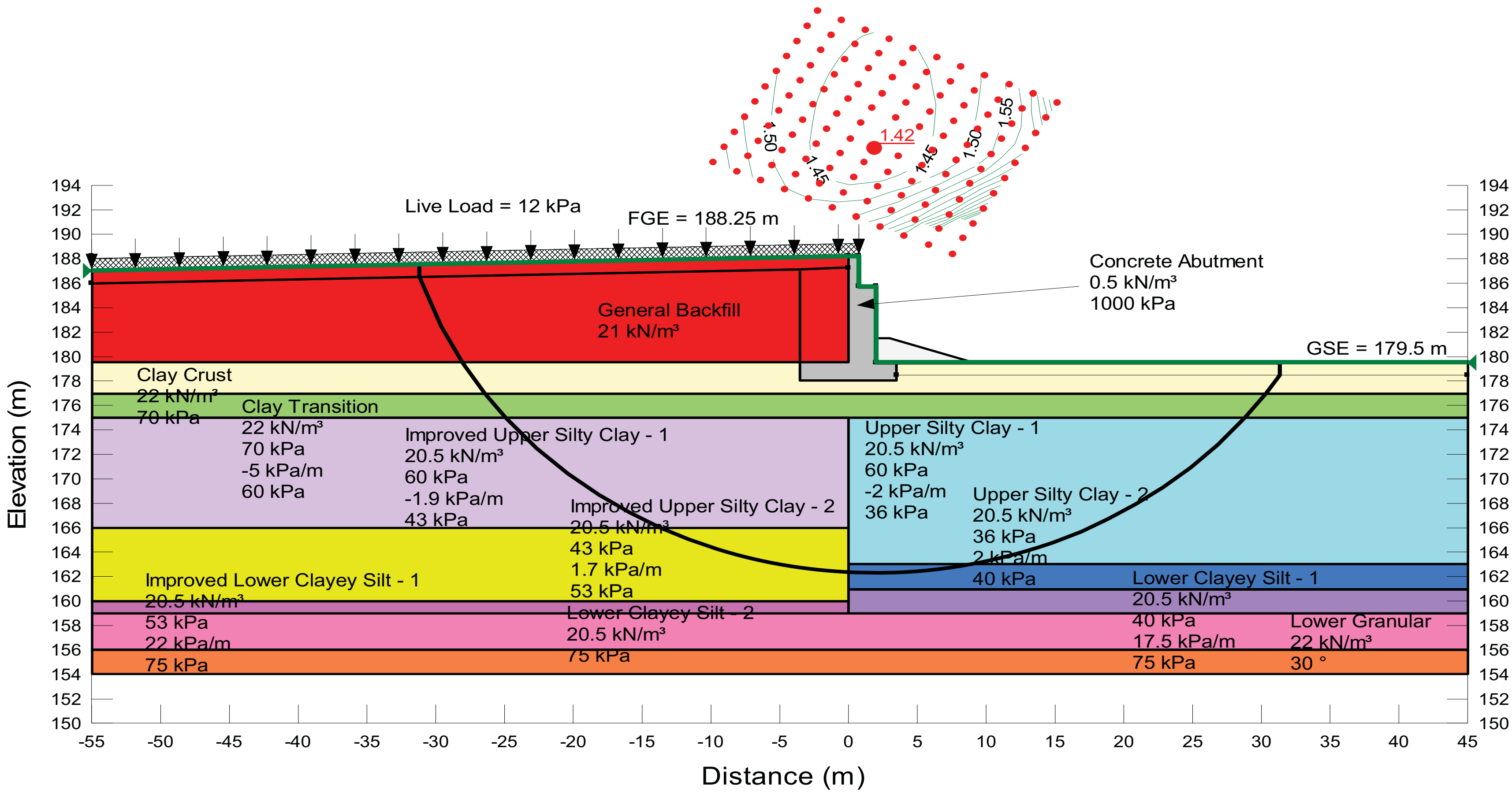
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	LONG-TERM (DRAINED) STABILITY ANALYSES - WEST ABUTMENT BRIDGE B-3
DATE: Jun 2012	JOB NO.:
CAD FILE:	FIGURE NO.: F.3
REV.:	

Bridge B-3-East Abutment-Undrained-Rev6.gsz

WEP SW8801.1002.101



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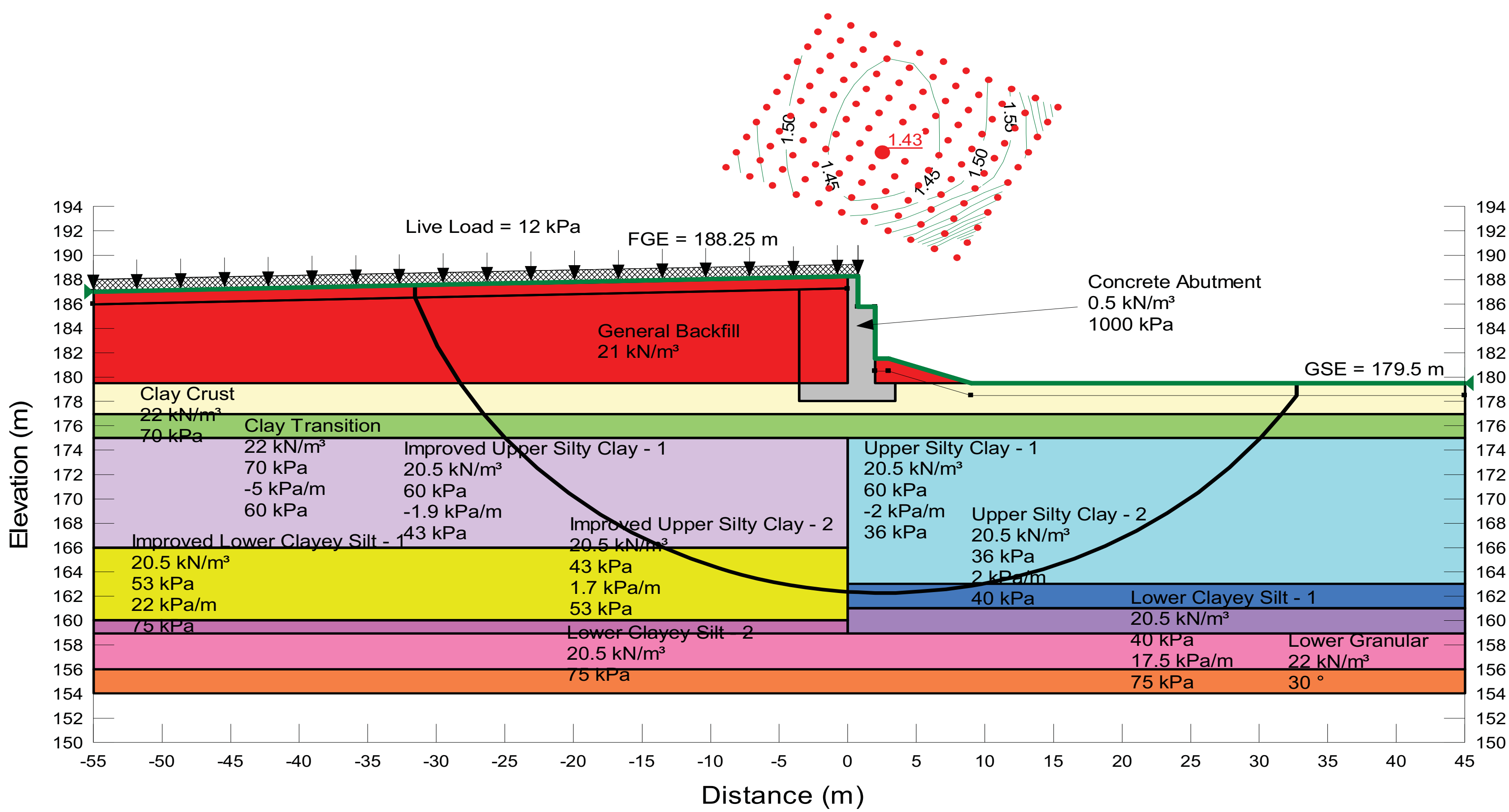
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: SHORT-TERM (UNDRAINED) STABILITY ANALYSES - EAST ABUTMENT BRIDGE B-3				
DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: F.4	REV.

Bridge B-3-East Abutment-Undrained-Rev6.gsz

WEP SW8801.1002.101



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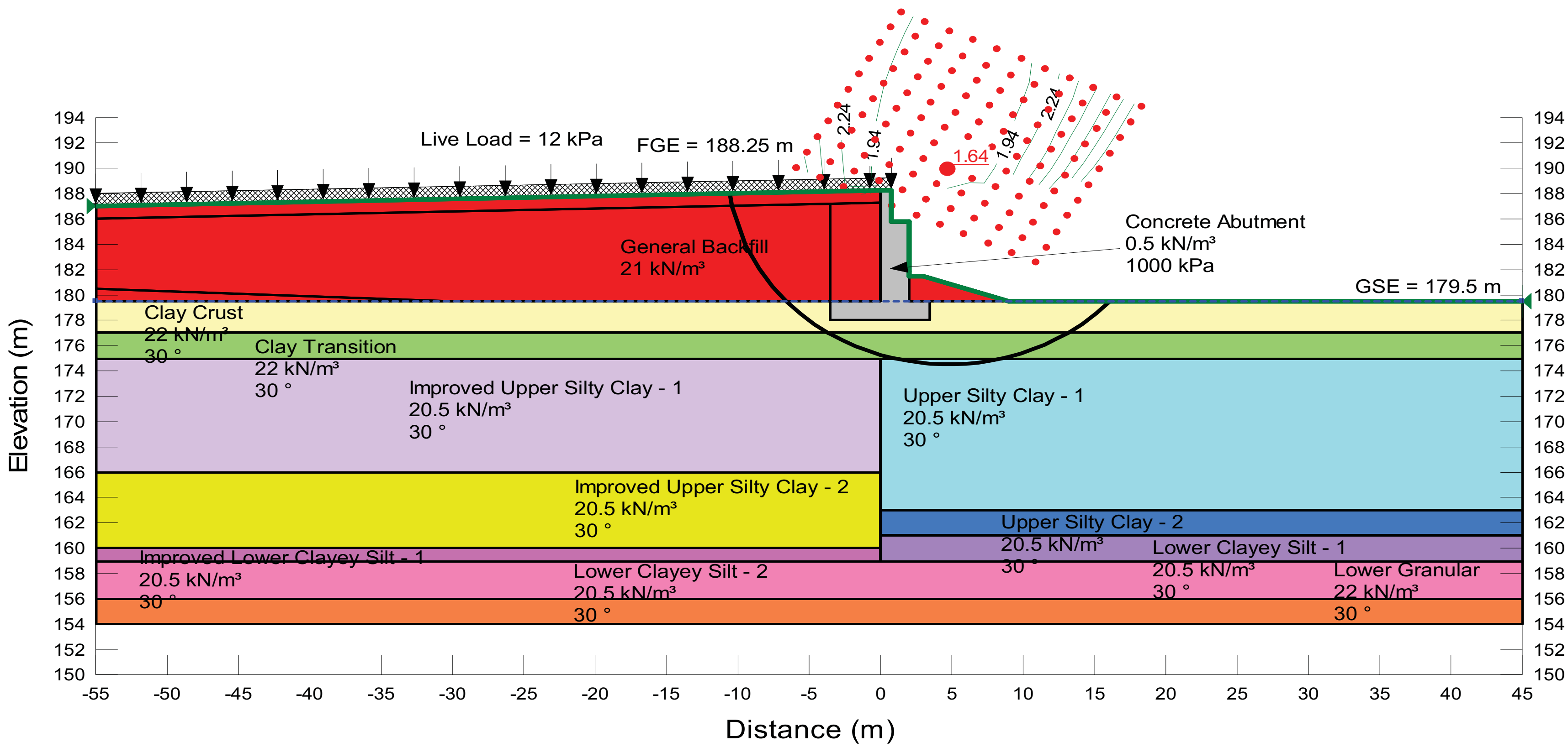
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	END OF CONSTRUCTION (UNDRAINED) STABILITY ANALYSES - EAST ABUTMENT BRIDGE B-3
DATE: Jun 2012	JOB NO.:
CAD FILE:	FIGURE NO.: F.5
REV.	

Bridge B-3-East Abutment-Drained-Rev6.gsz

WEP SW8801.1002.101



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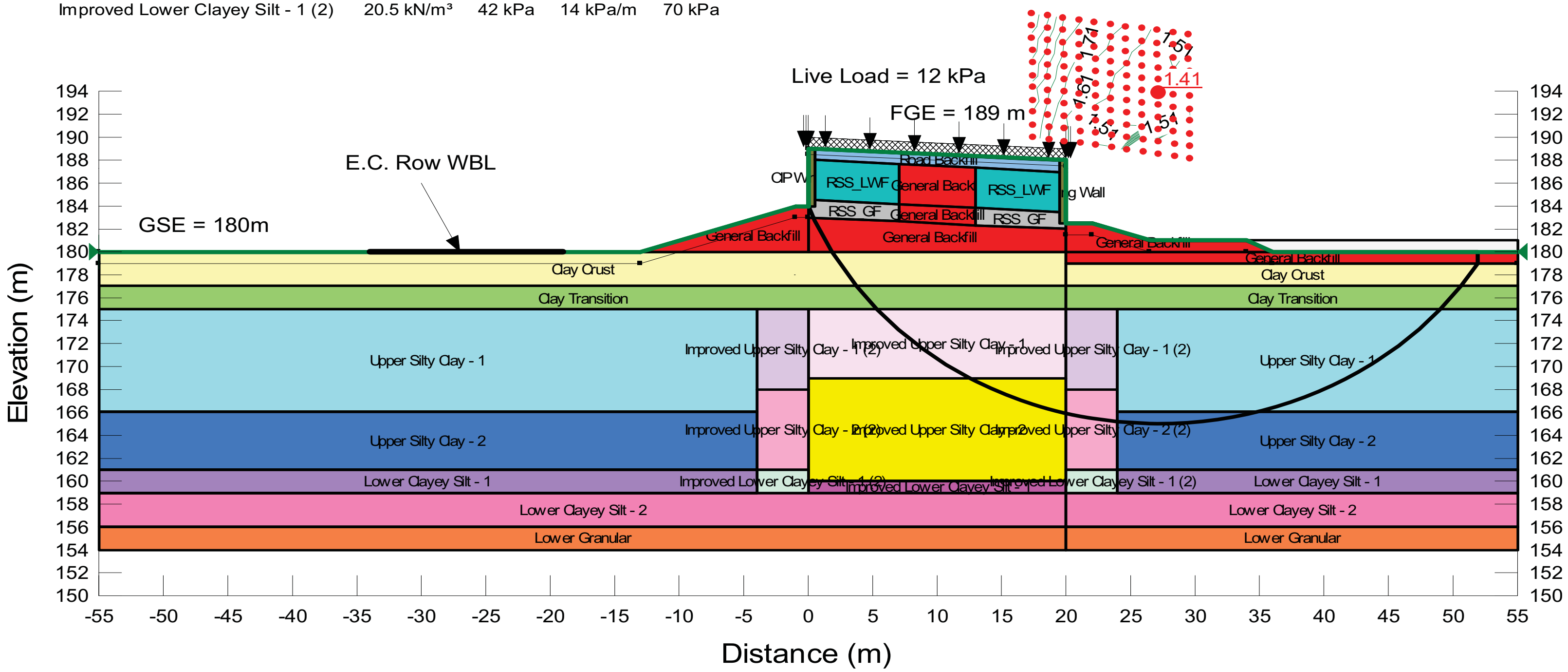
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: LONG-TERM (DRAINED) STABILITY ANALYSES - EAST ABUTMENT BRIDGE B-3				
DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: F.6	REV.

Bridge B-3-West Wing Wall-Undrained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 60 kPa 0 °
Clay Transition 22 kN/m³ 60 kPa -5 kPa/m 50 kPa
Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -2.8 kPa/m 25 kPa
Upper Silty Clay - 2 20.5 kN/m³ 25 kPa 2.4 kPa/m 37 kPa
Lower Clayey Silt - 1 20.5 kN/m³ 37 kPa 16.5 kPa/m 70 kPa
Lower Clayey Silt - 2 20.5 kN/m³ 70 kPa 0 °
Lower Granular 22 kN/m³ 0 kPa 30 °
General Backfill 21 kN/m³ 50 kPa 0 °
RSS_GF 21 kN/m³ 50 kPa 35 °
CIP Wing Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -0.3 kPa/m 48 kPa
Improved Upper Silty Clay - 2 20.5 kN/m³ 48 kPa 0.7 kPa/m 54 kPa
Improved Lower Clayey Silt - 1 20.5 kN/m³ 54 kPa 16 kPa/m 70 kPa
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Improved Upper Silty Clay - 1 (2) 20.5 kN/m³ 50 kPa -2.6 kPa/m 32 kPa
Improved Upper Silty Clay - 2 (2) 20.5 kN/m³ 32 kPa 1.4 kPa/m 42 kPa
Improved Lower Clayey Silt - 1 (2) 20.5 kN/m³ 42 kPa 14 kPa/m 70 kPa



PROJECT: WINDSOR ESSEX PARKWAY

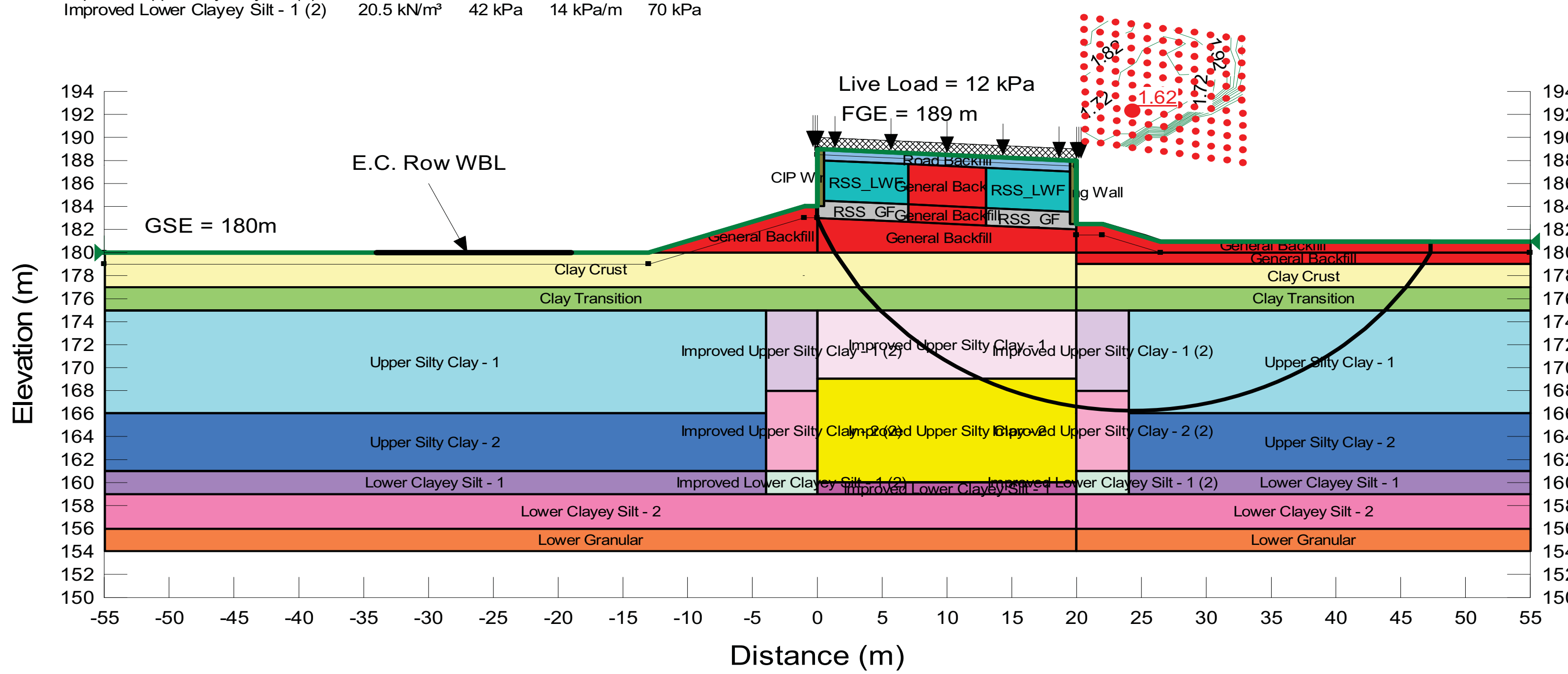
TITLE: SHORT-TERM (UNDRAINED) STABILITY ANALYSES - WEST WING WALL - SOUTH BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.7	REV.
Jun 2012				

Bridge B-3-West Wing Wall-Undrained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 60 kPa 0 °
Clay Transition 22 kN/m³ 60 kPa -5 kPa/m 50 kPa
Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -2.8 kPa/m 25 kPa
Upper Silty Clay - 2 20.5 kN/m³ 25 kPa 2.4 kPa/m 37 kPa
Lower Clayey Silt - 1 20.5 kN/m³ 37 kPa 16.5 kPa/m 70 kPa
Lower Clayey Silt - 2 20.5 kN/m³ 70 kPa 0 °
Lower Granular 22 kN/m³ 0 kPa 30 °
General Backfill 21 kN/m³ 50 kPa 0 °
RSS_GF 21 kN/m³ 50 kPa 35 °
CIP Wing Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -0.3 kPa/m 48 kPa
Improved Upper Silty Clay - 2 20.5 kN/m³ 48 kPa 0.7 kPa/m 54 kPa
Improved Lower Clayey Silt - 1 20.5 kN/m³ 54 kPa 16 kPa/m 70 kPa
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Improved Upper Silty Clay - 1 (2) 20.5 kN/m³ 50 kPa -2.6 kPa/m 32 kPa
Improved Upper Silty Clay - 2 (2) 20.5 kN/m³ 32 kPa 1.4 kPa/m 42 kPa
Improved Lower Clayey Silt - 1 (2) 20.5 kN/m³ 42 kPa 14 kPa/m 70 kPa



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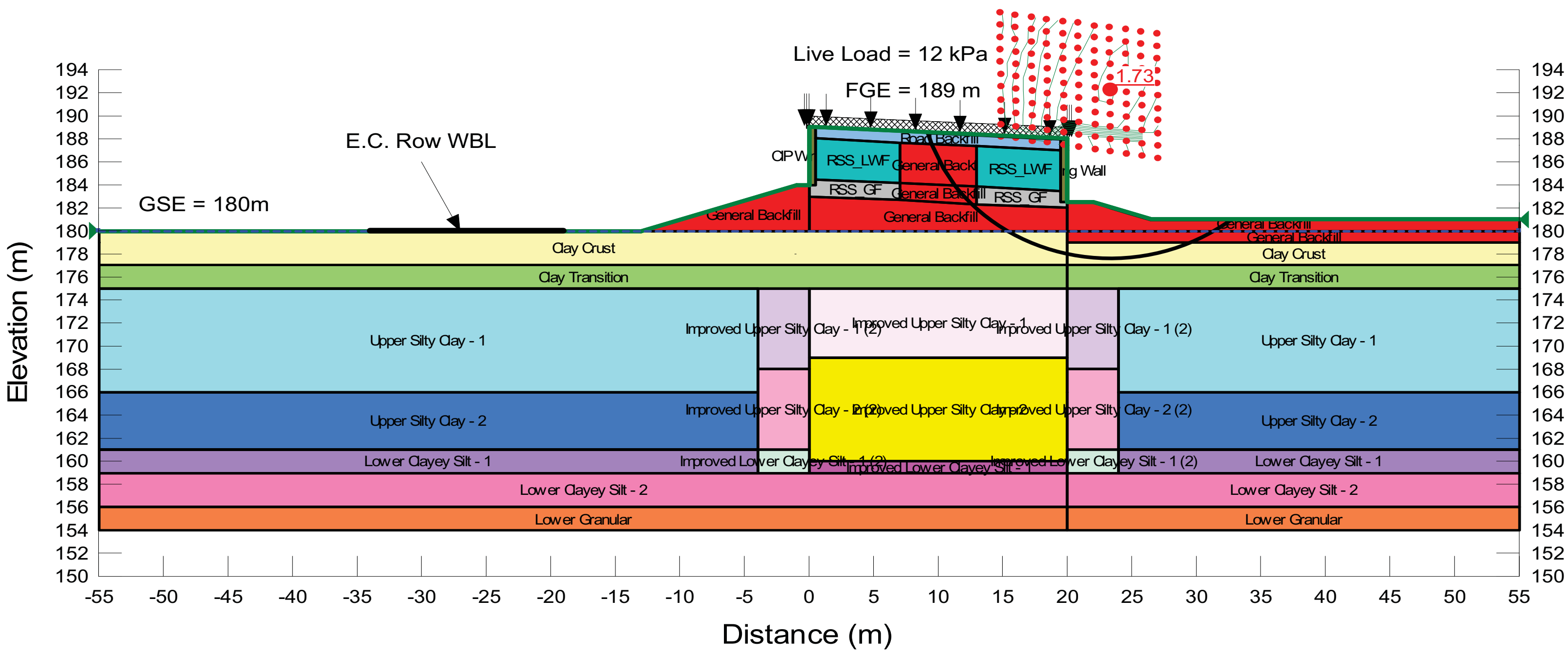
PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	END OF CONSTRUCTION (UNDRAINED) STABILITY ANALYSES - WEST WING WALL - SOUTH BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.8	REV.
Jun 2012				

Bridge B-3-West Wing Wall-Drained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 0 kPa 30 °
Clay Transition 22 kN/m³ 0 kPa 30 °
Upper Silty Clay - 1 20.5 kN/m³ 0 kPa 30 °
Upper Silty Clay - 2 20.5 kN/m³ 0 kPa 30 °
Lower Clayey Silt - 1 20.5 kN/m³ 0 kPa 30 °
Lower Clayey Silt - 2 20.5 kN/m³ 0 kPa 30 °
Lower Granular 22 kN/m³ 0 kPa 30 °
General Backfill 21 kN/m³ 0 kPa 30 °
RSS_GF 21 kN/m³ 50 kPa 35 °
CIP Wing Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 0 kPa 30 °
Improved Upper Silty Clay - 2 20.5 kN/m³ 0 kPa 30 °
Improved Lower Clayey Silt - 1 20.5 kN/m³ 0 kPa 30 °
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Improved Upper Silty Clay - 1 (2) 20.5 kN/m³ 0 kPa 30 °
Improved Upper Silty Clay - 2 (2) 20.5 kN/m³ 0 kPa 30 °
Improved Lower Clayey Silt - 1 (2) 20.5 kN/m³ 0 kPa 30 °



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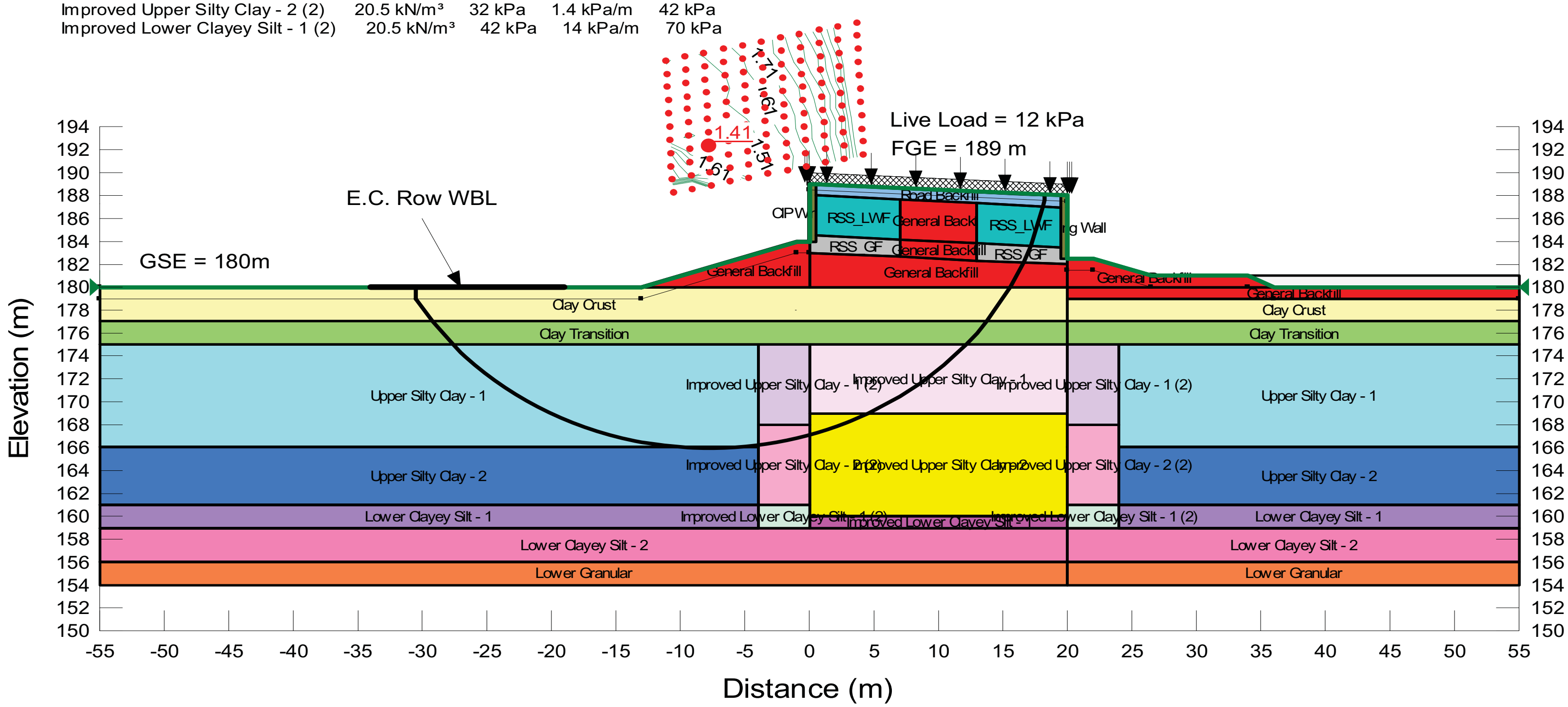
PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	LONG-TERM (DRAINED) STABILITY ANALYSES - WEST WING WALL - SOUTH BRIDGE B-3
DATE: Jun 2012	JOB NO.:
CAD FILE:	FIGURE NO.: F.9
REV.:	

Bridge B-3-West Wing Wall-Undrained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 60 kPa 0 °
Clay Transition 22 kN/m³ 60 kPa -5 kPa/m 50 kPa
Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -2.8 kPa/m 25 kPa
Upper Silty Clay - 2 20.5 kN/m³ 25 kPa 2.4 kPa/m 37 kPa
Lower Clayey Silt - 1 20.5 kN/m³ 37 kPa 16.5 kPa/m 70 kPa
Lower Clayey Silt - 2 20.5 kN/m³ 70 kPa 0 °
Lower Granular 22 kN/m³ 0 kPa 30 °
General Backfill 21 kN/m³ 50 kPa 0 °
RSS_GF 21 kN/m³ 50 kPa 35 °
CIP Wing Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 50 kPa -0.3 kPa/m 48 kPa
Improved Upper Silty Clay - 2 20.5 kN/m³ 48 kPa 0.7 kPa/m 54 kPa
Improved Lower Clayey Silt - 1 20.5 kN/m³ 54 kPa 16 kPa/m 70 kPa
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Improved Upper Silty Clay - 1 (2) 20.5 kN/m³ 50 kPa -2.6 kPa/m 32 kPa
Improved Upper Silty Clay - 2 (2) 20.5 kN/m³ 32 kPa 1.4 kPa/m 42 kPa
Improved Lower Clayey Silt - 1 (2) 20.5 kN/m³ 42 kPa 14 kPa/m 70 kPa



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PROJECT: WINDSOR ESSEX PARKWAY

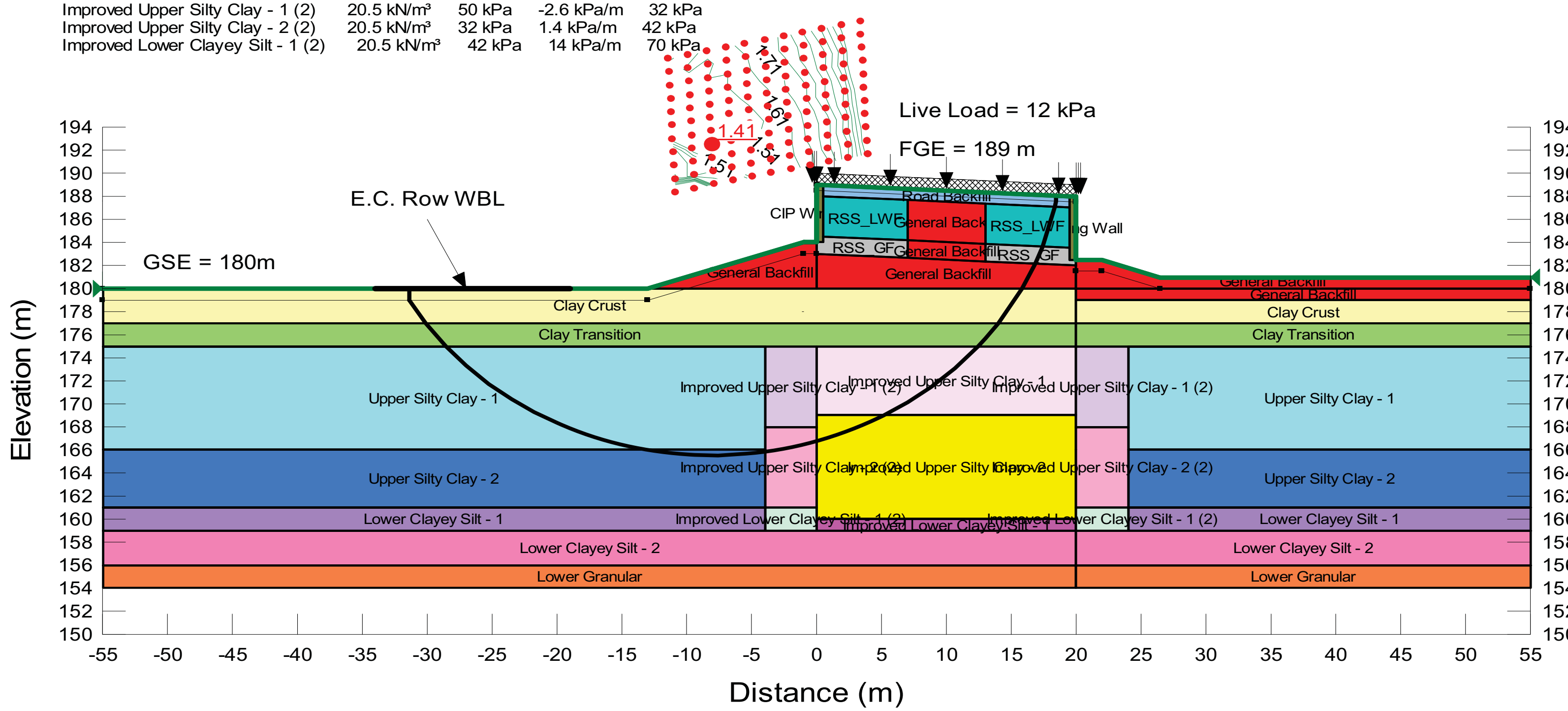
TITLE: SHORT-TERM (UNDRAINED) STABILITY ANALYSES - WEST WING WALL - NORTH BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.10	REV.
Jun 2012				

Bridge B-3-West Wing Wall-Undrained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust	22 kN/m³	60 kPa	0 °	
Clay Transition	22 kN/m³	60 kPa	-5 kPa/m	50 kPa
Upper Silty Clay - 1	20.5 kN/m³	50 kPa	-2.8 kPa/m	25 kPa
Upper Silty Clay - 2	20.5 kN/m³	25 kPa	2.4 kPa/m	37 kPa
Lower Clayey Silt - 1	20.5 kN/m³	37 kPa	16.5 kPa/m	70 kPa
Lower Clayey Silt - 2	20.5 kN/m³	70 kPa	0 °	
Lower Granular	22 kN/m³	0 kPa	30 °	
General Backfill	21 kN/m³	50 kPa	0 °	
RSS_GF	21 kN/m³	50 kPa	35 °	
CIP Wing Wall	0.5 kN/m³	1000 kPa	0 °	
Improved Upper Silty Clay - 1	20.5 kN/m³	50 kPa	-0.3 kPa/m	48 kPa
Improved Upper Silty Clay - 2	20.5 kN/m³	48 kPa	0.7 kPa/m	54 kPa
Improved Lower Clayey Silt - 1	20.5 kN/m³	54 kPa	16 kPa/m	70 kPa
RSS_LWF	12 kN/m³	50 kPa	35 °	
Road Backfill	21 kN/m³	0 kPa	35 °	
Improved Upper Silty Clay - 1 (2)	20.5 kN/m³	50 kPa	-2.6 kPa/m	32 kPa
Improved Upper Silty Clay - 2 (2)	20.5 kN/m³	32 kPa	1.4 kPa/m	42 kPa
Improved Lower Clayey Silt - 1 (2)	20.5 kN/m³	42 kPa	14 kPa/m	70 kPa



PROJECT: WINDSOR ESSEX PARKWAY

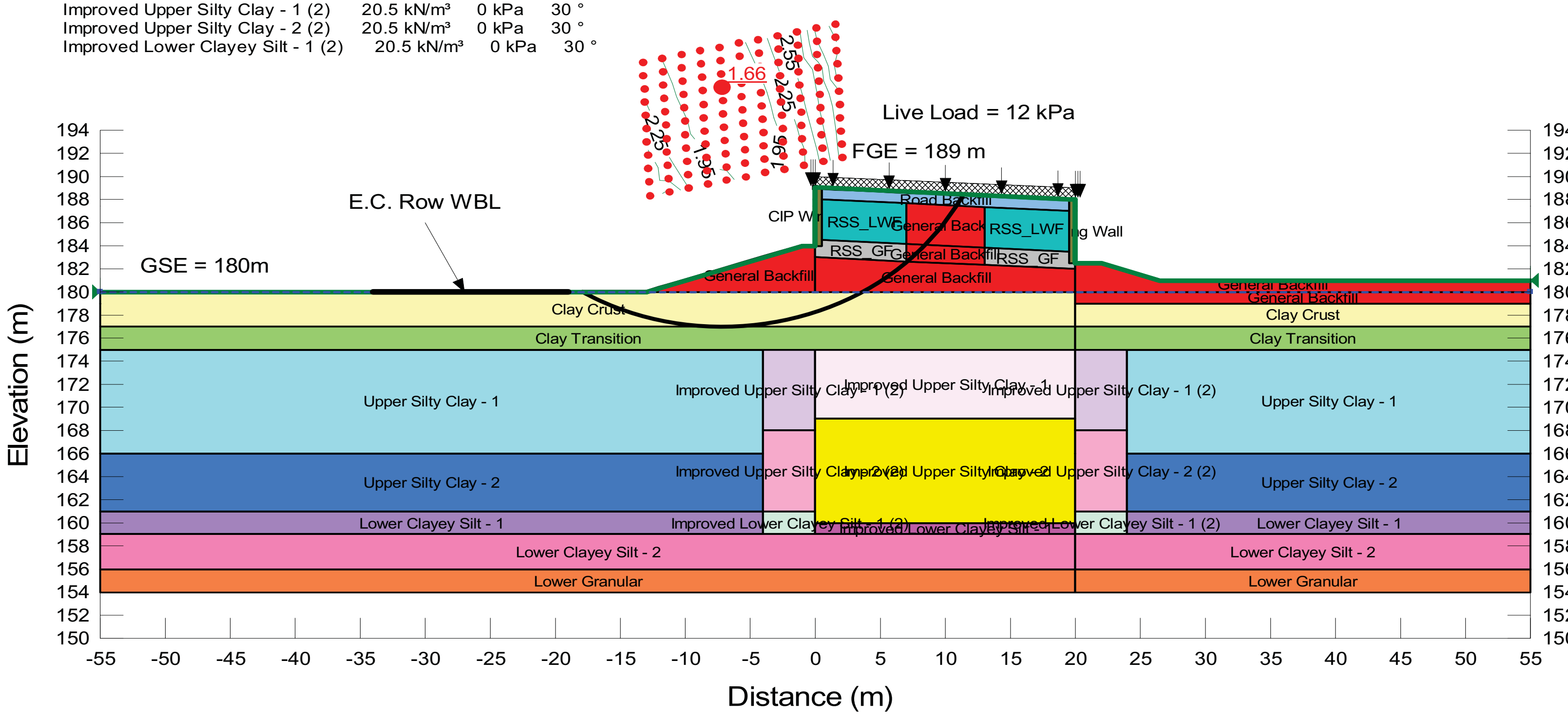
TITLE: END OF CONSTRUCTION (UNDRAINED) STABILITY ANALYSES - WEST WING WALL - NORTH BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.11	REV.
Jun 2012				

Bridge B-3-West Wing Wall-Drained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 0 kPa 30 °
Clay Transition 22 kN/m³ 0 kPa 30 °
Upper Silty Clay - 1 20.5 kN/m³ 0 kPa 30 °
Upper Silty Clay - 2 20.5 kN/m³ 0 kPa 30 °
Lower Clayey Silt - 1 20.5 kN/m³ 0 kPa 30 °
Lower Clayey Silt - 2 20.5 kN/m³ 0 kPa 30 °
Lower Granular 22 kN/m³ 0 kPa 30 °
General Backfill 21 kN/m³ 0 kPa 30 °
RSS_GF 21 kN/m³ 50 kPa 35 °
CIP Wing Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 0 kPa 30 °
Improved Upper Silty Clay - 2 20.5 kN/m³ 0 kPa 30 °
Improved Lower Clayey Silt - 1 20.5 kN/m³ 0 kPa 30 °
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Improved Upper Silty Clay - 1 (2) 20.5 kN/m³ 0 kPa 30 °
Improved Upper Silty Clay - 2 (2) 20.5 kN/m³ 0 kPa 30 °
Improved Lower Clayey Silt - 1 (2) 20.5 kN/m³ 0 kPa 30 °



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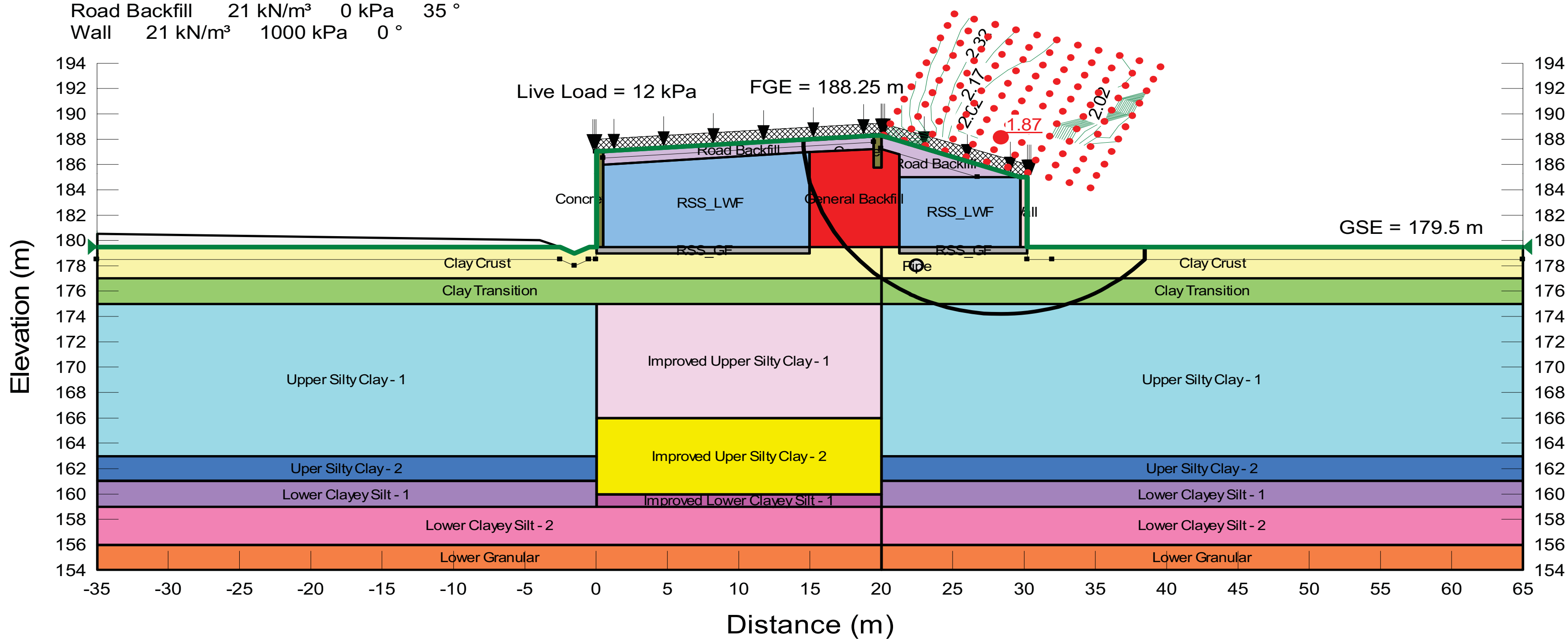
PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	LONG-TERM (DRAINED) STABILITY ANALYSES - WEST WING WALL - NORTH BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.12	REV.
Jun 2012				

Bridge B-3-East Wing Wall-Undrained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 70 kPa 0 °
Clay Transition 22 kN/m³ 70 kPa -5 kPa/m 60 kPa
Upper Silty Clay - 1 20.5 kN/m³ 60 kPa -2 kPa/m 36 kPa
Uper Silty Clay - 2 20.5 kN/m³ 36 kPa 2 kPa/m 40 kPa
Lower Clayey Silt - 1 20.5 kN/m³ 40 kPa 17.5 kPa/m 75 kPa
Lower Clayey Silt - 2 20.5 kN/m³ 75 kPa 0 °
Lower Granular 22 kN/m³ 0 kPa 30 °
RSS_GF 21 kN/m³ 50 kPa 35 °
General Backfill 21 kN/m³ 50 kPa 0 °
Pipe 0.1 kN/m³ 1000 kPa 0 °
Concrete Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 60 kPa -1.9 kPa/m 43 kPa
Improved Uper Silty Clay - 2 20.5 kN/m³ 43 kPa 1.7 kPa/m 53 kPa
Improved Lower Clayey Silt - 1 20.5 kN/m³ 53 kPa 22 kPa/m 75 kPa
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Wall 21 kN/m³ 1000 kPa 0 °



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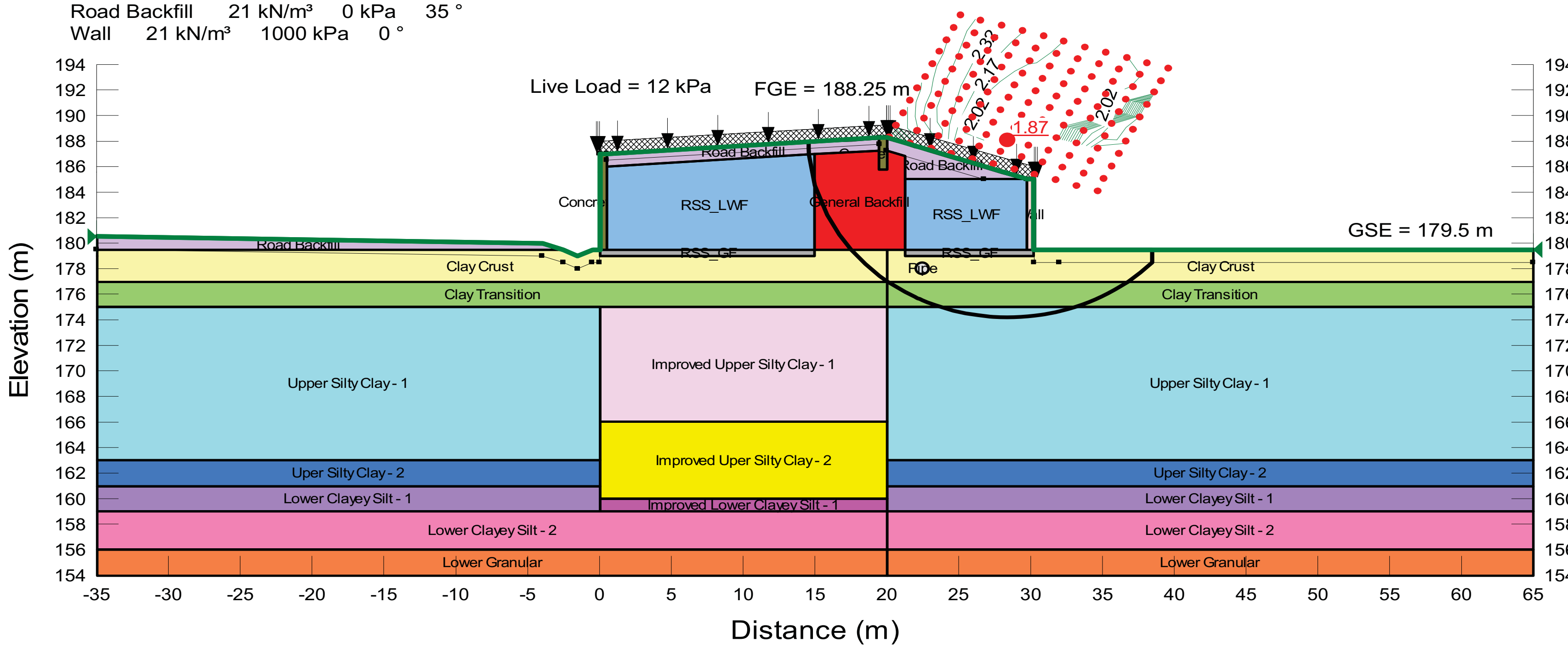
PROJECT: WINDSOR ESSEX PARKWAY
TITLE: SHORT-TERM (UNDRAINED) STABILITY ANALYSES - EAST WING WALL - SOUTH BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.13	REV.
Jun 2012				

Bridge B-3-East Wing Wall-Undrained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 70 kPa 0 °
Clay Transition 22 kN/m³ 70 kPa -5 kPa/m 60 kPa
Upper Silty Clay - 1 20.5 kN/m³ 60 kPa -2 kPa/m 36 kPa
Uper Silty Clay - 2 20.5 kN/m³ 36 kPa 2 kPa/m 40 kPa
Lower Clayey Silt - 1 20.5 kN/m³ 40 kPa 17.5 kPa/m 75 kPa
Lower Clayey Silt - 2 20.5 kN/m³ 75 kPa 0 °
Lower Granular 22 kN/m³ 0 kPa 30 °
RSS_GF 21 kN/m³ 50 kPa 35 °
General Backfill 21 kN/m³ 50 kPa 0 °
Pipe 0.1 kN/m³ 1000 kPa 0 °
Concrete Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 60 kPa -1.9 kPa/m 43 kPa
Improved Uper Silty Clay - 2 20.5 kN/m³ 43 kPa 1.7 kPa/m 53 kPa
Improved Lower Clayey Silt - 1 20.5 kN/m³ 53 kPa 22 kPa/m 75 kPa
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Wall 21 kN/m³ 1000 kPa 0 °



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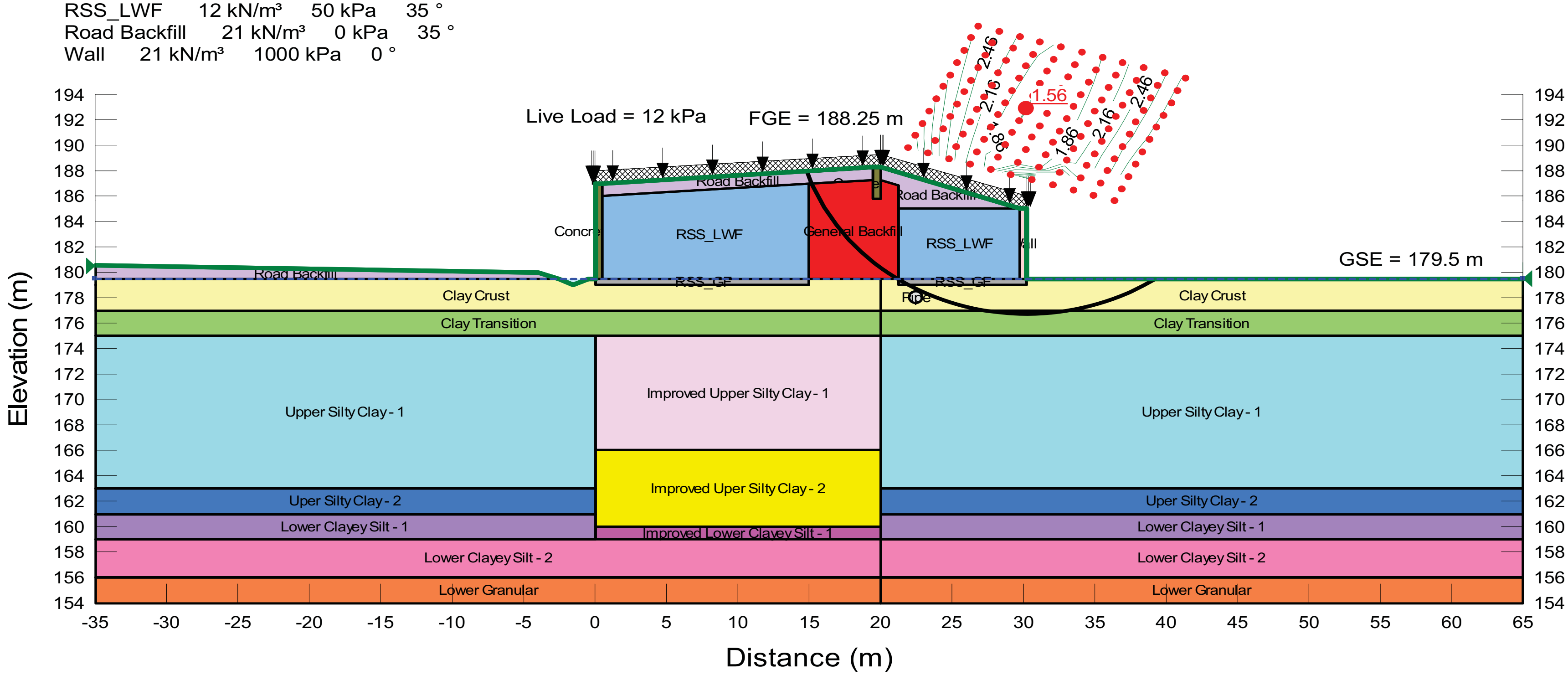
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	END OF CONSTRUCTION (UNDRAINED) STABILITY ANALYSES - EAST WING WALL - SOUTH BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.14	REV.
Jun 2012				

Bridge B-3-East Wing Wall-Drained-Rev1.gsz **WEP SW8801.1002.101**

Clay Crust 22 kN/m³ 0 kPa 30 °
Clay Transition 22 kN/m³ 0 kPa 30 °
Upper Silty Clay - 1 20.5 kN/m³ 0 kPa 30 °
Uper Silty Clay - 2 20.5 kN/m³ 0 kPa 30 °
Lower Clayey Silt - 1 20.5 kN/m³ 0 kPa 30 °
Lower Clayey Silt - 2 20.5 kN/m³ 0 kPa 30 °
Lower Granular 22 kN/m³ 0 kPa 30 °
RSS_GF 21 kN/m³ 50 kPa 35 °
General Backfill 21 kN/m³ 0 kPa 30 °
Pipe 0.1 kN/m³ 1000 kPa 0 °
Concrete Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 0 kPa 30 °
Improved Uper Silty Clay - 2 20.5 kN/m³ 0 kPa 30 °
Improved Lower Clayey Silt - 1 20.5 kN/m³ 0 kPa 30 °
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Wall 21 kN/m³ 1000 kPa 0 °



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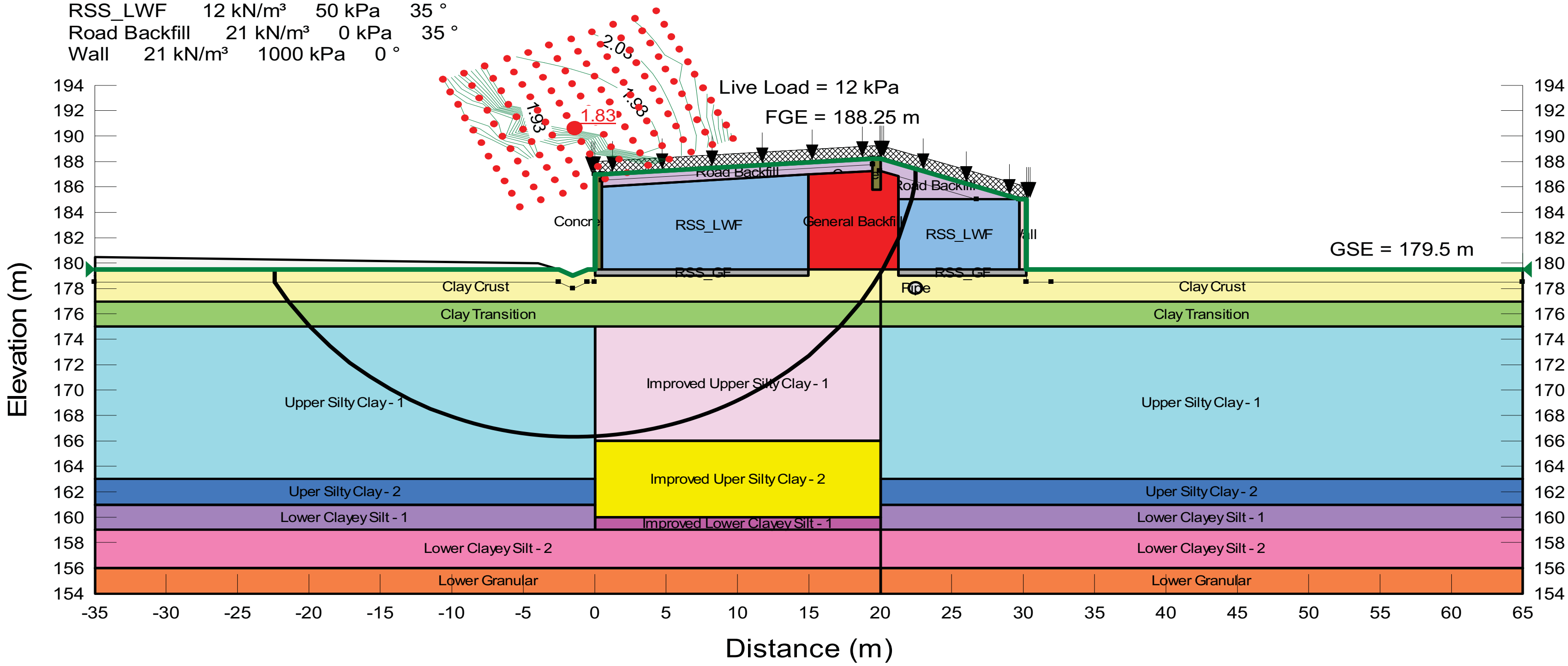
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	LONG-TERM (DRAINED) STABILITY ANALYSES - EAST WING WALL - SOUTH BRIDGE B-3
DATE: Jun 2012	JOB NO.: / CAD FILE: / FIGURE NO.: F.15 / REV.

Bridge B-3-East Wing Wall-Undrained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 70 kPa 0 °
Clay Transition 22 kN/m³ 70 kPa -5 kPa/m 60 kPa
Upper Silty Clay - 1 20.5 kN/m³ 60 kPa -2 kPa/m 36 kPa
Uper Silty Clay - 2 20.5 kN/m³ 36 kPa 2 kPa/m 40 kPa
Lower Clayey Silt - 1 20.5 kN/m³ 40 kPa 17.5 kPa/m 75 kPa
Lower Clayey Silt - 2 20.5 kN/m³ 75 kPa 0 °
Lower Granular 22 kN/m³ 0 kPa 30 °
RSS_GF 21 kN/m³ 50 kPa 35 °
General Backfill 21 kN/m³ 50 kPa 0 °
Pipe 0.1 kN/m³ 1000 kPa 0 °
Concrete Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 60 kPa -1.9 kPa/m 43 kPa
Improved Upper Silty Clay - 2 20.5 kN/m³ 43 kPa 1.7 kPa/m 53 kPa
Improved Lower Clayey Silt - 1 20.5 kN/m³ 53 kPa 22 kPa/m 75 kPa
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Wall 21 kN/m³ 1000 kPa 0 °



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PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

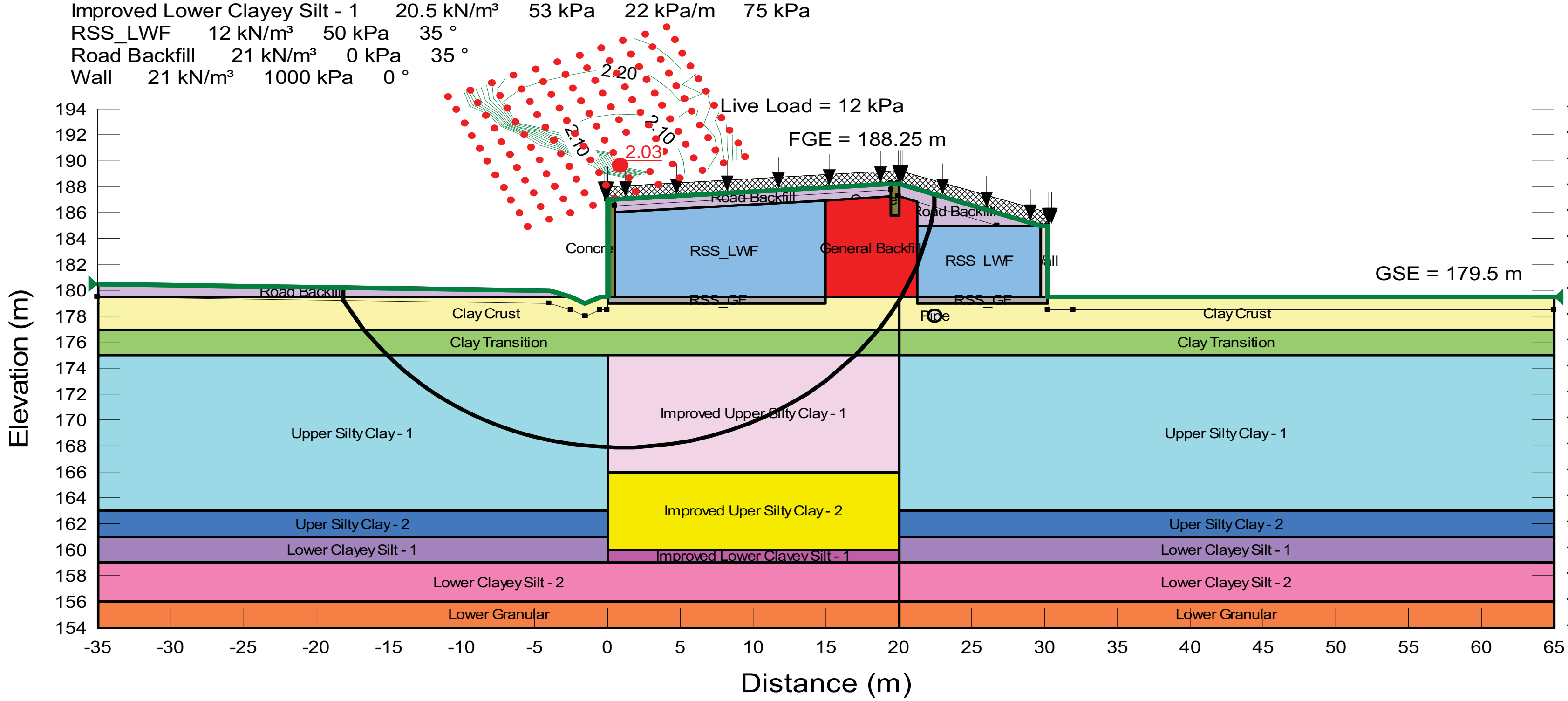
PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	SHORT-TERM (UNDRAINED) STABILITY ANALYSES - EAST WING WALL - NORTH BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.16	REV.
Jun 2012				

Bridge B-3-East Wing Wall-Undrained-Rev1.gsz

WEP SW8801.1002.101

Clay Crust 22 kN/m³ 70 kPa 0 °
Clay Transition 22 kN/m³ 70 kPa -5 kPa/m 60 kPa
Upper Silty Clay - 1 20.5 kN/m³ 60 kPa -2 kPa/m 36 kPa
Uper Silty Clay - 2 20.5 kN/m³ 36 kPa 2 kPa/m 40 kPa
Lower Clayey Silt - 1 20.5 kN/m³ 40 kPa 17.5 kPa/m 75 kPa
Lower Clayey Silt - 2 20.5 kN/m³ 75 kPa 0 °
Lower Granular 22 kN/m³ 0 kPa 30 °
RSS_GF 21 kN/m³ 50 kPa 35 °
General Backfill 21 kN/m³ 50 kPa 0 °
Pipe 0.1 kN/m³ 1000 kPa 0 °
Concrete Wall 0.5 kN/m³ 1000 kPa 0 °
Improved Upper Silty Clay - 1 20.5 kN/m³ 60 kPa -1.9 kPa/m 43 kPa
Improved Uper Silty Clay - 2 20.5 kN/m³ 43 kPa 1.7 kPa/m 53 kPa
Improved Lower Clayey Silt - 1 20.5 kN/m³ 53 kPa 22 kPa/m 75 kPa
RSS_LWF 12 kN/m³ 50 kPa 35 °
Road Backfill 21 kN/m³ 0 kPa 35 °
Wall 21 kN/m³ 1000 kPa 0 °



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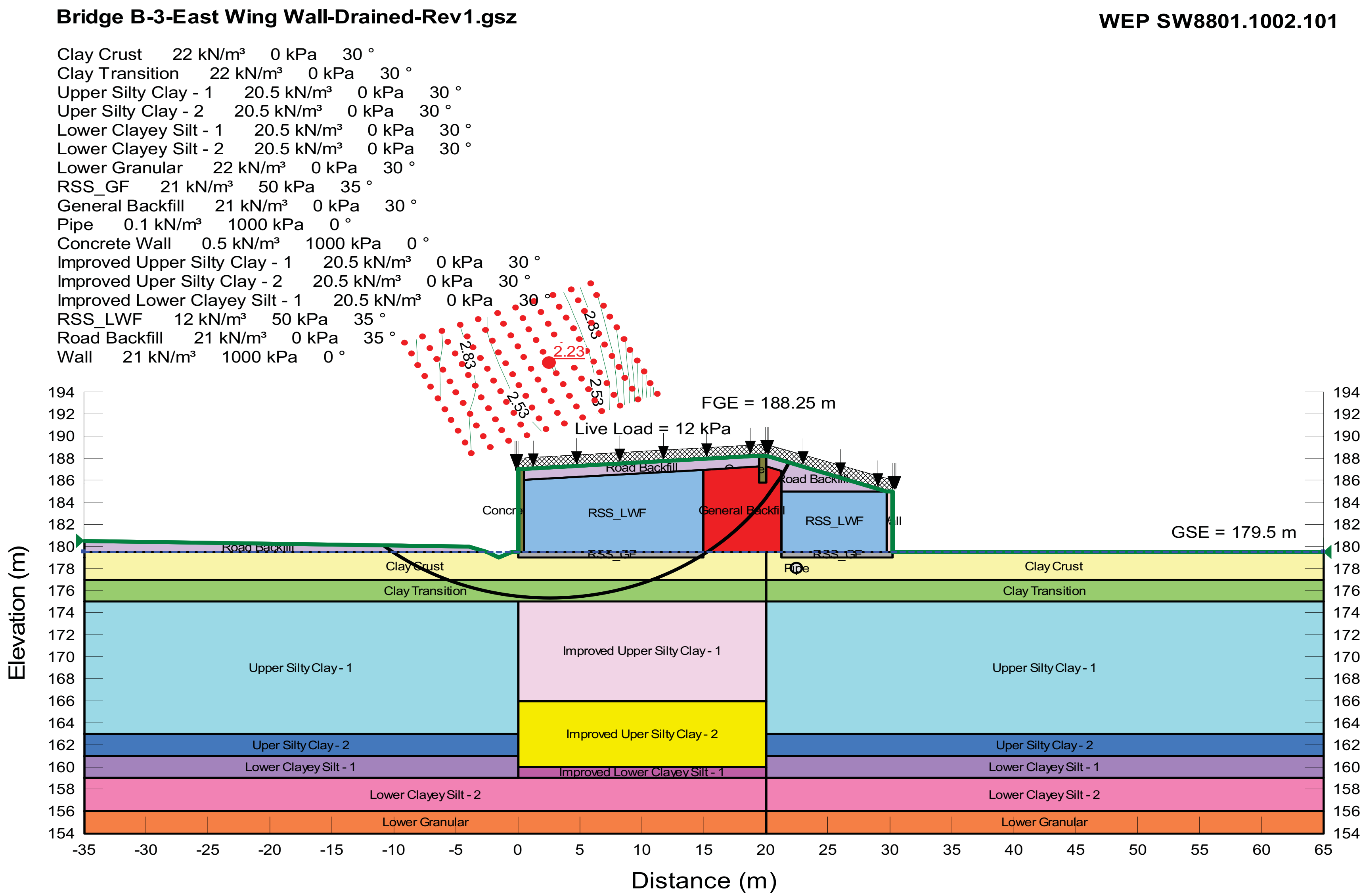
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	END OF CONSTRUCTION (UNDRAINED) STABILITY ANALYSES - EAST WING WALL - NORTH BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.17	REV.
Jun 2012				



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PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	LONG-TERM (DRAINED) STABILITY ANALYSES - EAST WING WALL - NORTH BRIDGE B-3

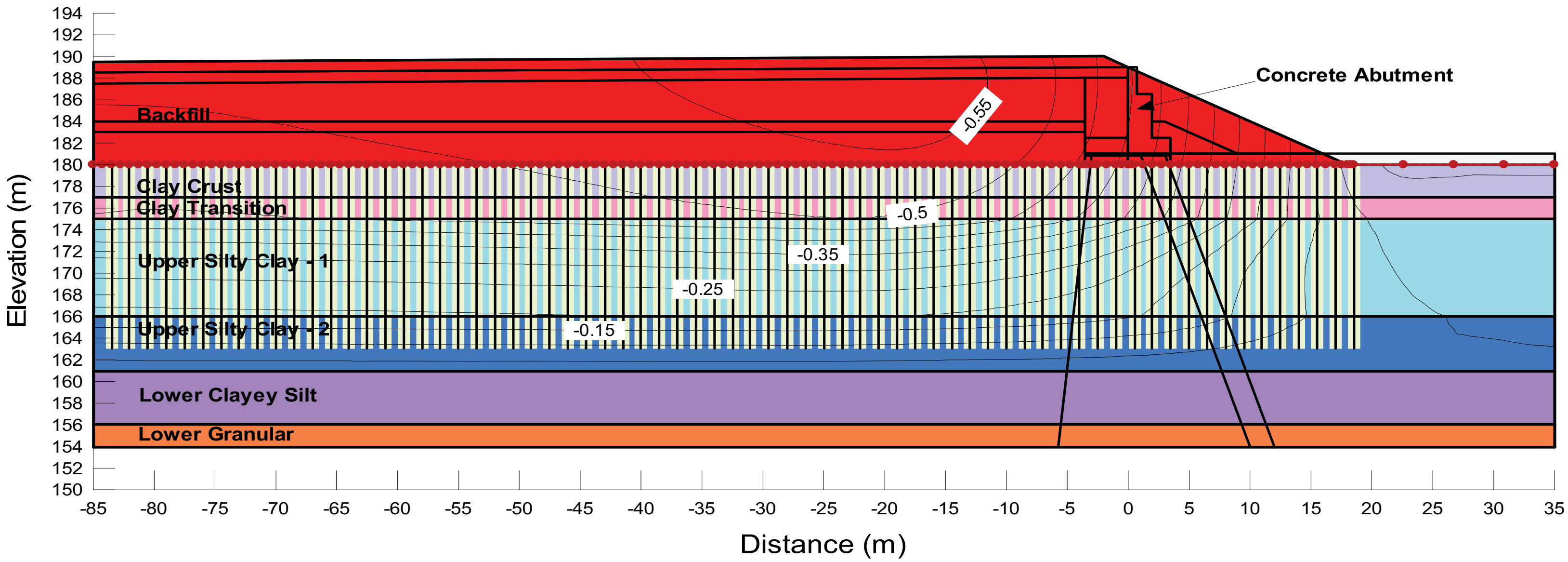
DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: F.18	REV.
Jun 2012				

Appendix G Stress-Deformation Analysis Results

Bridge B-3-Wick-Deformation-Rev6.gsz

WEP SW8801.1002.101

Name: Clay Crust Effective Young's Modulus (E'): 25000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 16000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.0971 Kappa: 0.0107 Initial Void Ratio: 0.73 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.05 Poisson's Ratio: 0.35 Lambda: 0.0896 Kappa: 0.0098 Initial Void Ratio: 0.68 Unit Weight: 20.5 kN/m³ Phi': 25 °
Name: Lower Clayey Silt O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Granular Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³
Name: Wick Drain Effective Young's Modulus (E'): 3000 kPa Poisson's Ratio: 0.35 Unit Weight: 0 kN/m³



Legend:

(-) Sign on Contour Labels = Settlement

No Sign on Contour Labels = Heave

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PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

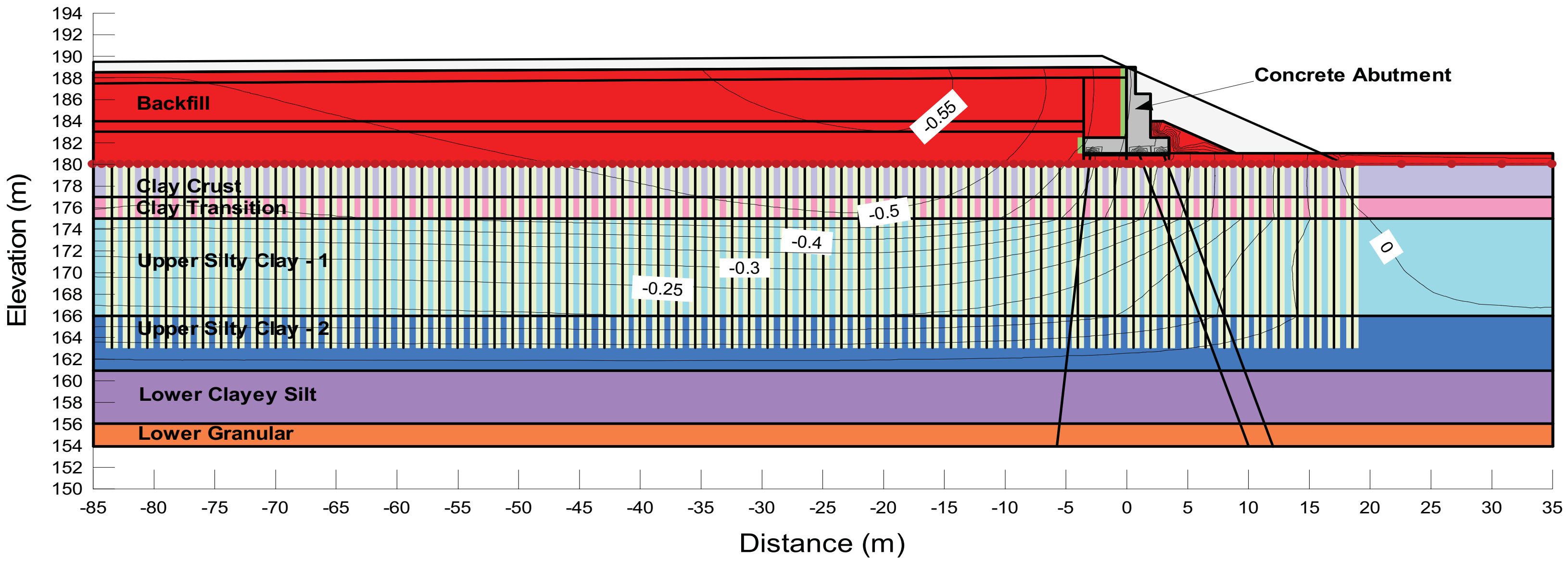
PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	STRESS-DEFORMATION MODEL OF BRIDGE B-3 CUMULATIVE SETTLEMENT/HEAVE CONTOURS AT END OF EMBANKMENT CONSTRUCTION (m)

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: G.1	REV.
Jun 2012				

Bridge B-3-Wick-Deformation-Rev6.gsz

WEP SW8801.1002.101

Name: Clay Crust Effective Young's Modulus (E'): 25000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 16000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.0971 Kappa: 0.0107 Initial Void Ratio: 0.73 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.05 Poisson's Ratio: 0.35 Lambda: 0.0896 Kappa: 0.0098 Initial Void Ratio: 0.68 Unit Weight: 20.5 kN/m³ Phi': 25 °
Name: Lower Clayey Silt O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Granular Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³
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: Slip Material Interface C: 10 kPa Interface Phi: 0 ° G (shear modulus): 10 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35



Legend:
(-) Sign on Contour Labels = Settlement
No Sign on Contour Labels = Heave

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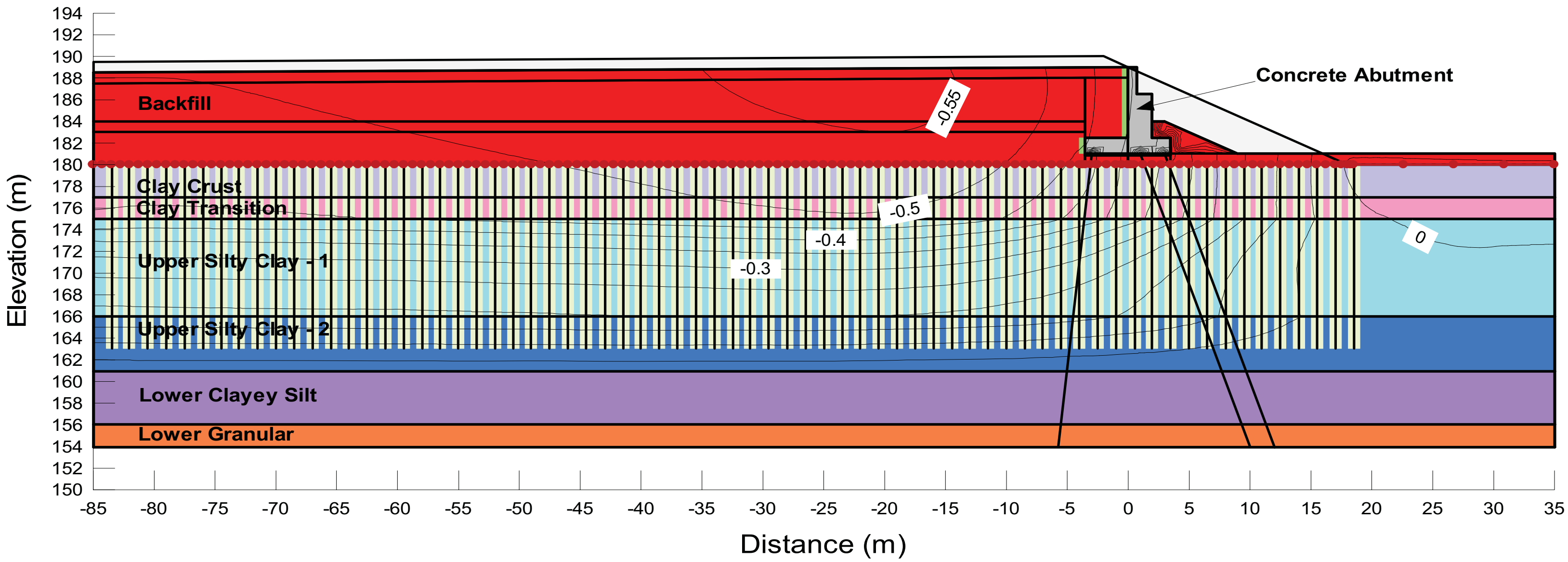
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	STRESS-DEFORMATION MODEL OF BRIDGE B-3 CUMULATIVE SETTLEMENT/HEAVE CONTOURS AT END OF CONSTRUCTION (m)
DATE: Jun 2012	JOB NO.:
CAD FILE:	FIGURE NO.: G.2
REV.:	

Bridge B-3-Wick-Deformation-Rev6.gsz

WEP SW8801.1002.101

Name: Clay Crust Effective Young's Modulus (E'): 25000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 16000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.0971 Kappa: 0.0107 Initial Void Ratio: 0.73 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.05 Poisson's Ratio: 0.35 Lambda: 0.0896 Kappa: 0.0098 Initial Void Ratio: 0.68 Unit Weight: 20.5 kN/m³ Phi': 25 °
Name: Lower Clayey Silt O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Granular Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³
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: Slip Material Interface C: 10 kPa Interface Phi: 0 ° G (shear modulus): 10 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35



Legend:
(-) Sign on Contour Labels = Settlement
No Sign on Contour Labels = Heave

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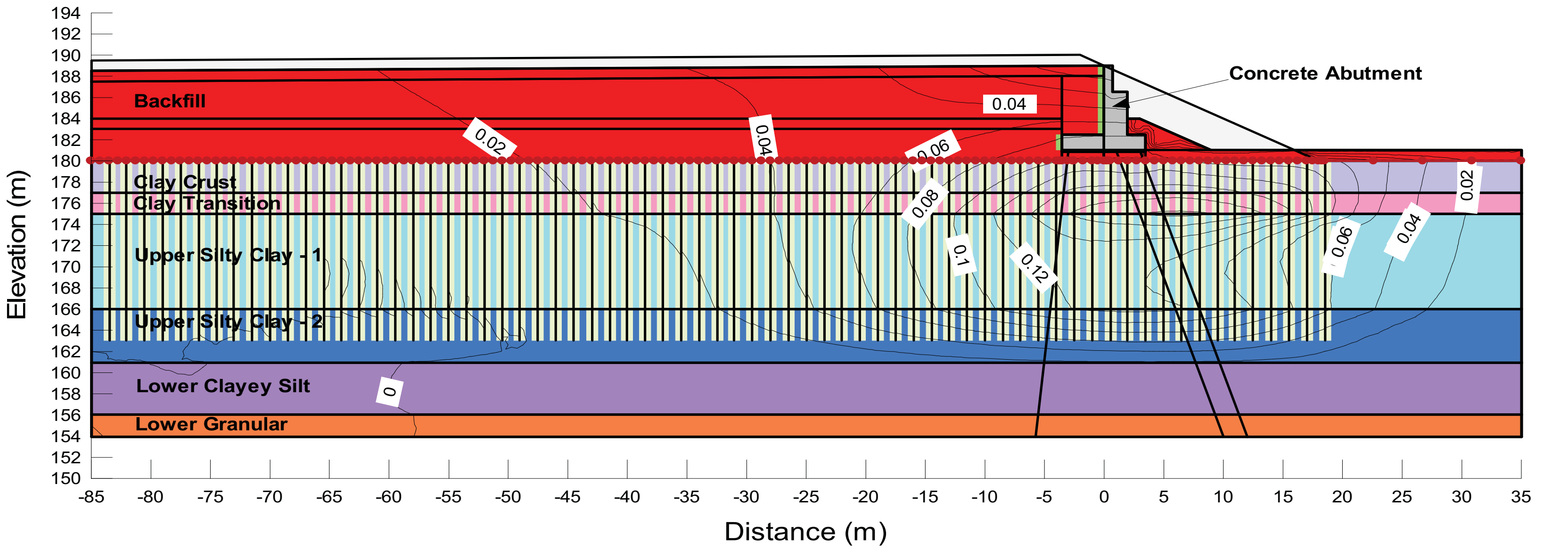
		TITLE: STRESS-DEFORMATION MODEL OF BRIDGE B-3 CUMULATIVE SETTLEMENT/HEAVE CONTOURS IN LONG-TERM (m)			
		DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: G.3	REV.

Bridge B-3-Wick-Deformation-Rev6.gsz

WEP SW8801.1002.101

Name: Clay Crust Effective Young's Modulus (E'): 25000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 16000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.0971 Kappa: 0.0107 Initial Void Ratio: 0.73 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.05 Poisson's Ratio: 0.35 Lambda: 0.0896 Kappa: 0.0098 Initial Void Ratio: 0.68 Unit Weight: 20.5 kN/m³ Phi': 25 °
Name: Lower Clayey Silt O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Granular Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³
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: Slip Material Interface C: 10 kPa Interface Phi: 0 ° G (shear modulus): 10 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35



Legend:
(-) Sign on Contour Labels = Lateral Deformation opposite to Highway 401 Excavation
No Sign on Contour Labels = Lateral Deformation towards Highway 401 Excavation

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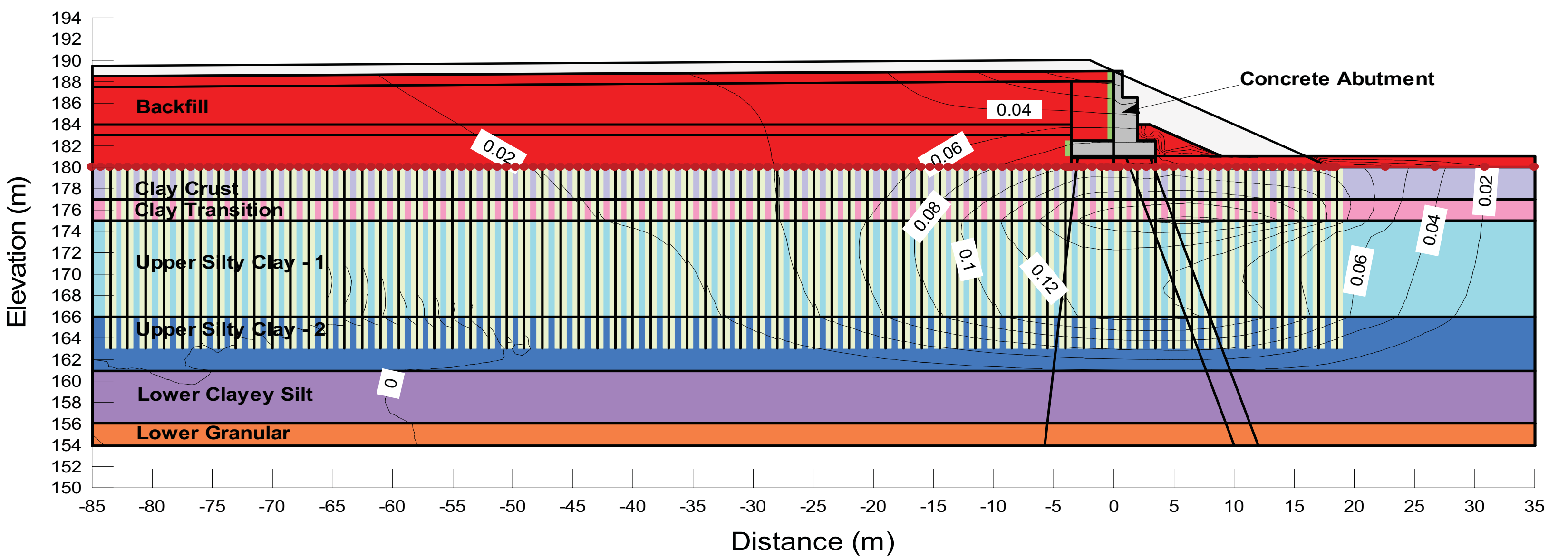
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	STRESS-DEFORMATION MODEL OF BRIDGE B-3 CUMULATIVE LATERAL DEFORMATION CONTOURS AT END OF CONSTRUCTION (m)
DATE: Jun 2012	JOB NO.:
CAD FILE:	FIGURE NO.: G.4
REV.:	

Bridge B-3-Wick-Deformation-Rev6.gsz

WEP SW8801.1002.101

Name: Clay Crust Effective Young's Modulus (E'): 25000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 16000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.0971 Kappa: 0.0107 Initial Void Ratio: 0.73 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.05 Poisson's Ratio: 0.35 Lambda: 0.0896 Kappa: 0.0098 Initial Void Ratio: 0.68 Unit Weight: 20.5 kN/m³ Phi': 25 °
Name: Lower Clayey Silt O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Granular Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³
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: Slip Material Interface C: 10 kPa Interface Phi: 0 ° G (shear modulus): 10 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35



Legend:
(-) Sign on Contour Labels = Lateral Deformation opposite to Highway 401 Excavation
No Sign on Contour Labels = Lateral Deformation towards Highway 401 Excavation

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

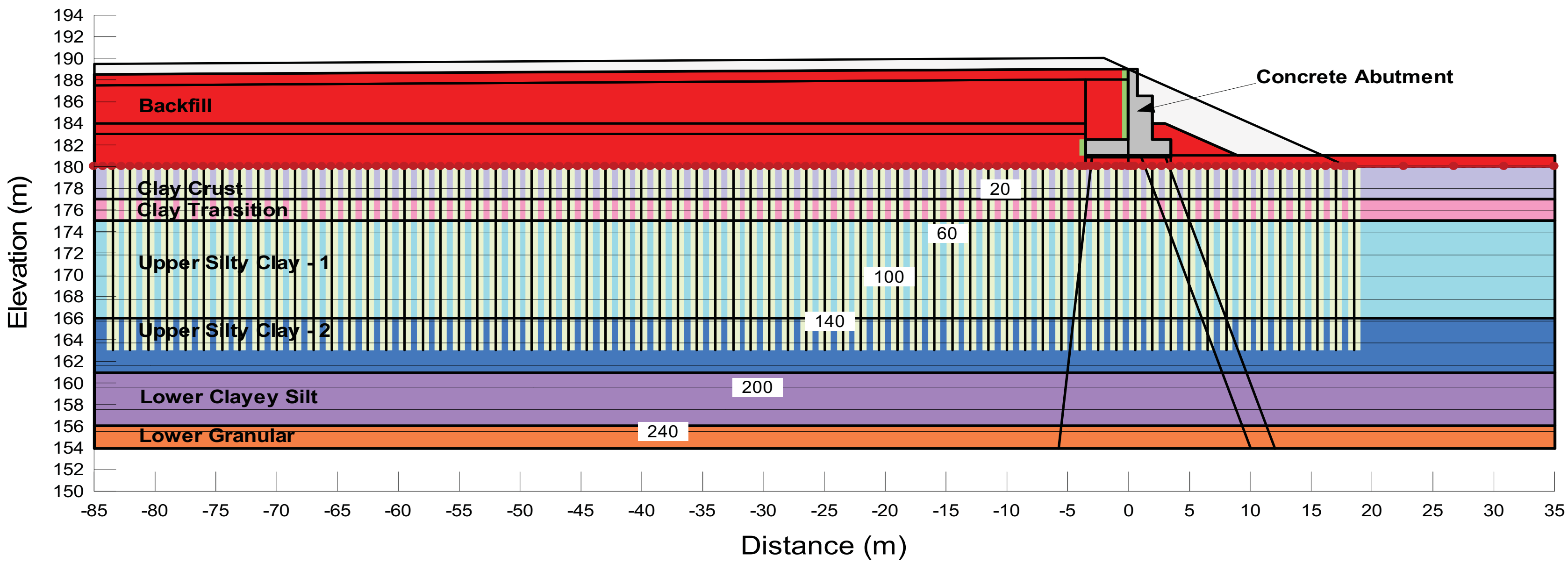
PARKWAY INFRASTRUCTURE CONSTRUCTORS acciona DRAGADOS FLUOR

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: STRESS-DEFORMATION MODEL OF BRIDGE B-3 CUMULATIVE LATERAL DEFORMATION CONTOURS IN LONG-TERM (m)				
DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: G.5	REV.

Bridge B-3-Wick-Deformation-Rev6.gsz

WEP SW8801.1002.101

Name: Clay Crust Effective Young's Modulus (E'): 25000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Transition Effective Young's Modulus (E'): 16000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Upper Silty Clay - 1 O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.0971 Kappa: 0.0107 Initial Void Ratio: 0.73 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Upper Silty Clay - 2 O.C. Ratio: 1.05 Poisson's Ratio: 0.35 Lambda: 0.0896 Kappa: 0.0098 Initial Void Ratio: 0.68 Unit Weight: 20.5 kN/m³ Phi': 25 °
Name: Lower Clayey Silt O.C. Ratio: 2 Poisson's Ratio: 0.35 Lambda: 0.071 Kappa: 0.0078 Initial Void Ratio: 0.54 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Granular Effective Young's Modulus (E'): 32000 kPa Poisson's Ratio: 0.35 Phi': 30 ° Unit Weight: 22 kN/m³
Name: Backfill Young's Modulus (E): 22500 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 21 kN/m³
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: Slip Material Interface C: 10 kPa Interface Phi: 0 ° G (shear modulus): 10 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35



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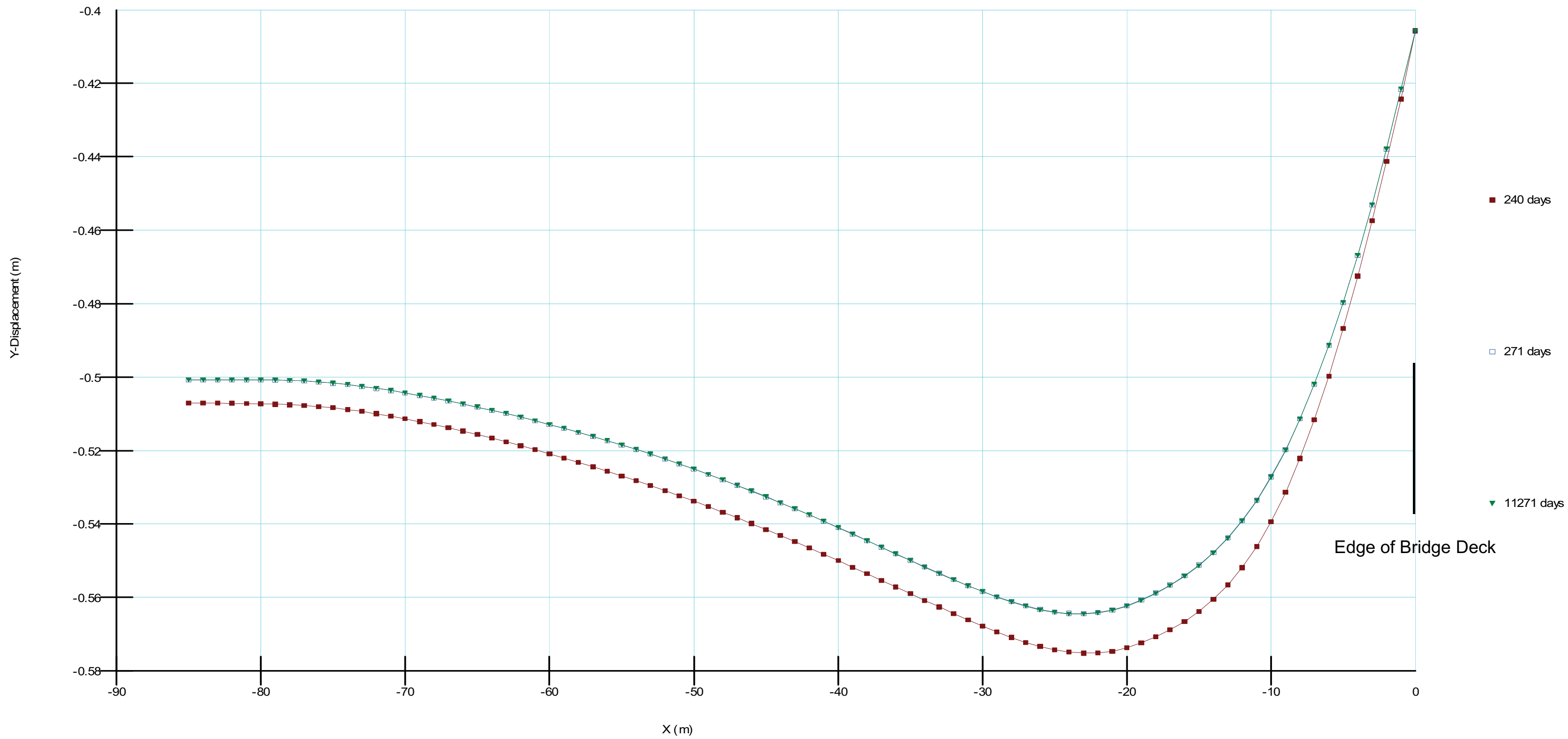
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	STRESS-DEFORMATION MODEL OF BRIDGE B-3 PORE WATER PRESSURE CONTOURS IN LONG-TERM (kPa)
DATE: Jun 2012	JOB NO.: CAD FILE: FIGURE NO.: G.6 REV.

Settlement at Top of Ground



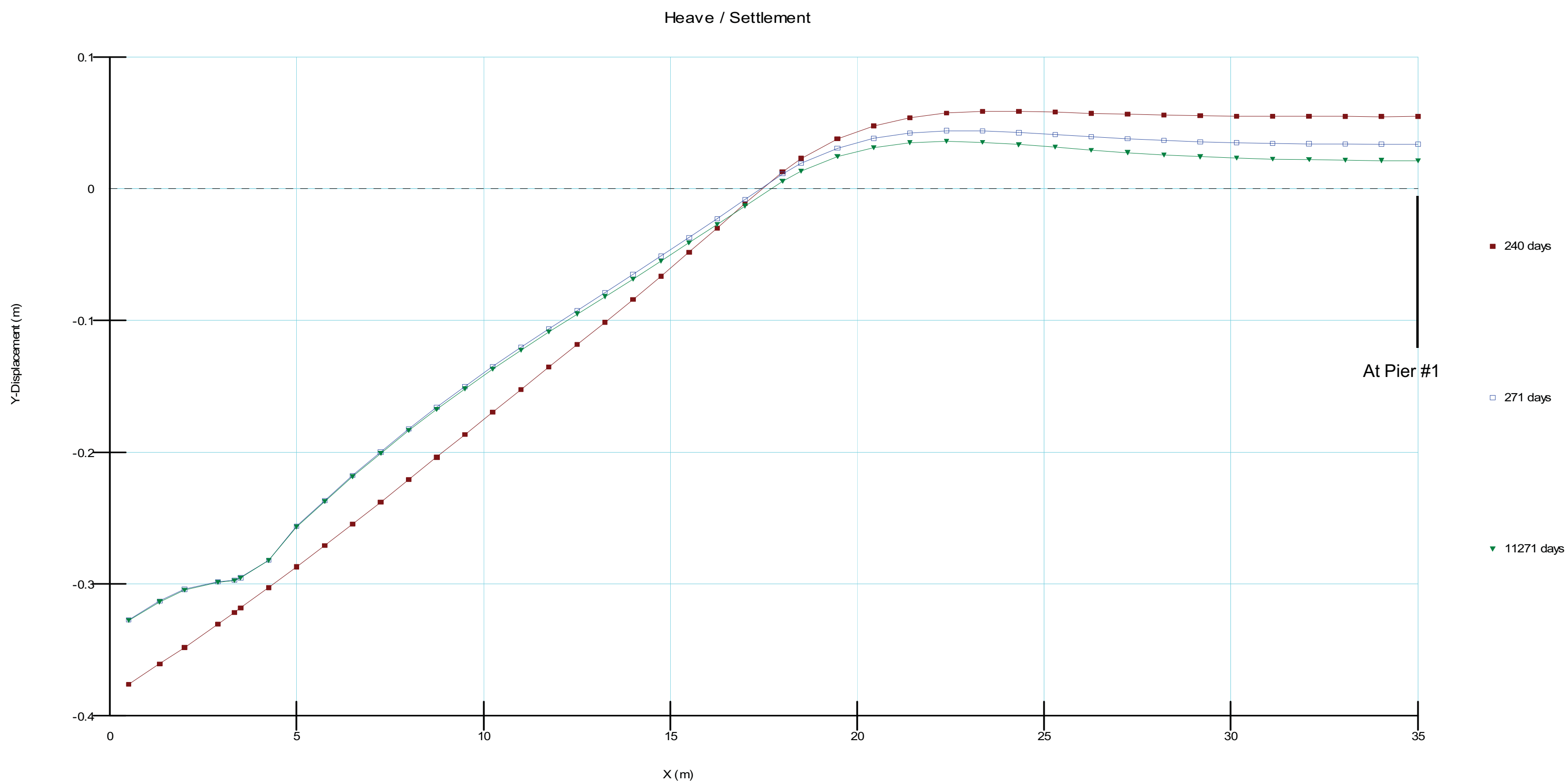
Legend:
240 days = End of Embankment Construction
271 days = End of Construction of Bridge
11271 days = Long-term Condition
(-) Displacement = Settlement
(+) Displacement = Heave

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

PARKWAY INFRASTRUCTURE CONSTRUCTORS acciona DRAGADOS FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	CUMULATIVE GROUND SETTLEMENT AT TOP OF GROUND BRIDGE B-3

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: G.7	REV.
Jun 2012				

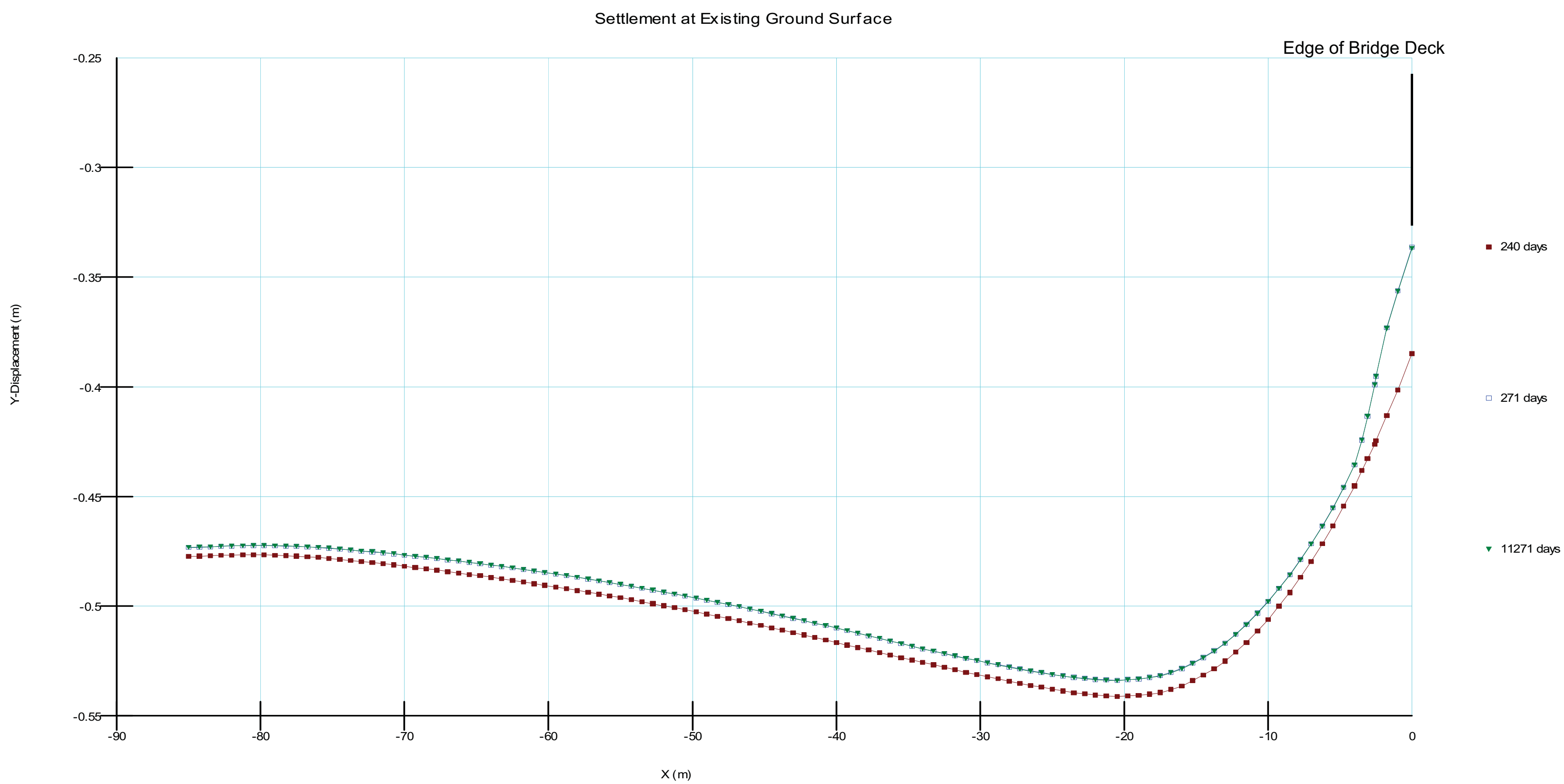


Legend:
240 days = End of Embankment Construction
271 days = End of Construction of Bridge
11271 days = Long-term Condition
(-) Displacement = Settlement
(+) Displacement = Heave

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

PARKWAY INFRASTRUCTURE CONSTRUCTORS acciona DRAGADOS FLUOR

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: CUMULATIVE GROUND HEAVE / SETTLEMENT AT PIER BRIDGE B-3				
DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: G.8	REV.



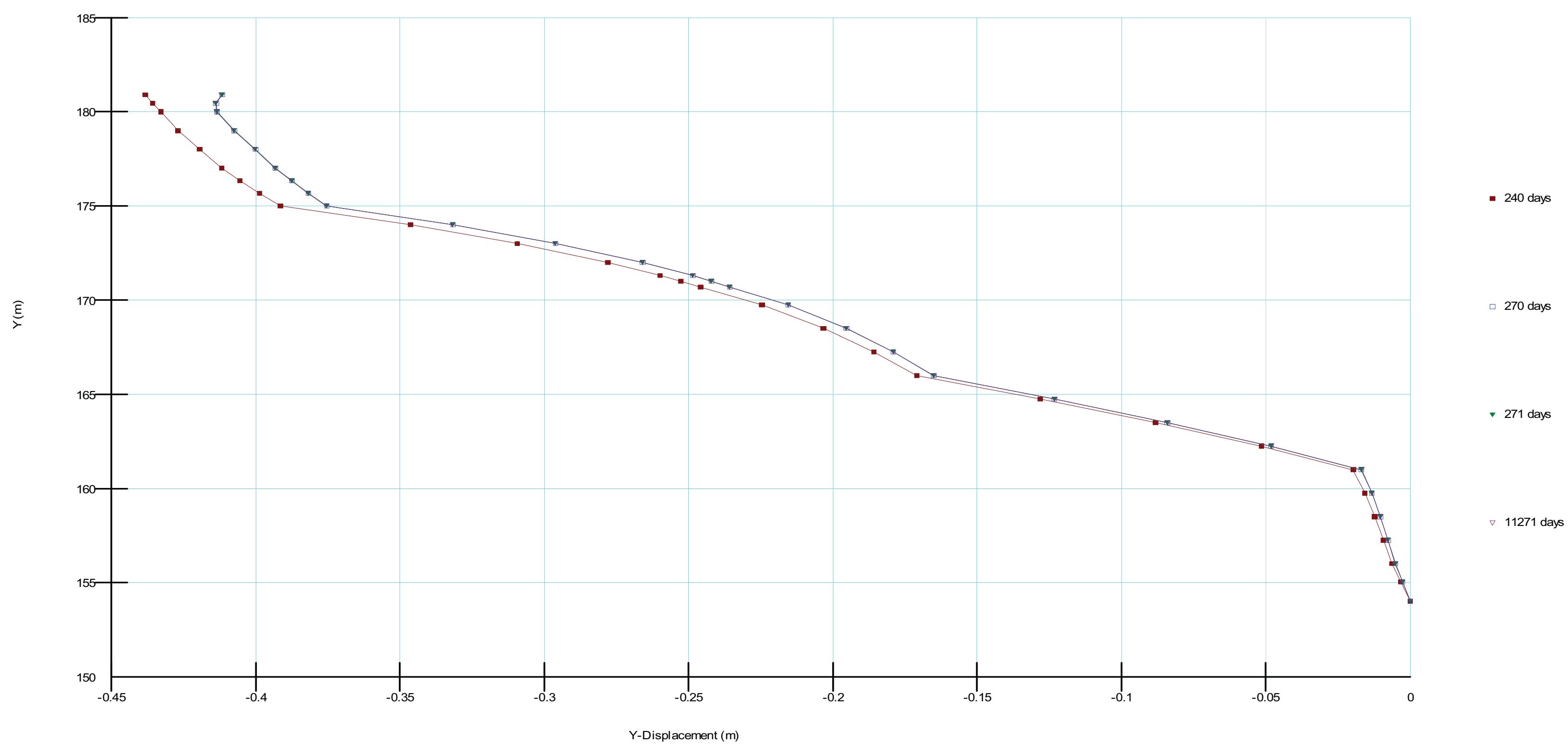
Legend:
240 days = End of Embankment Construction
271 days = End of Construction of Bridge
11271 days = Long-term Condition
(-) Displacement = Settlement
(+) Displacement = Heave

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

PARKWAY INFRASTRUCTURE CONSTRUCTORS acciona DRAGADOS FLUOR

PROJECT:	WINDSOR ESSEX PARKWAY			
TITLE:	CUMULATIVE GROUND SETTLEMENT AT EXISTING GROUND SURFACE BRIDGE B-3			
DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: G.9	REV.

Soil Settlement along Pile Line



Legend:
240 days = End of Embankment Construction
270 days = End of Abutment Construction
271 days = End of Construction of Bridge
11271 days = Long-term Condition
(-) Displacement = Settlement
(+) Displacement = Heave

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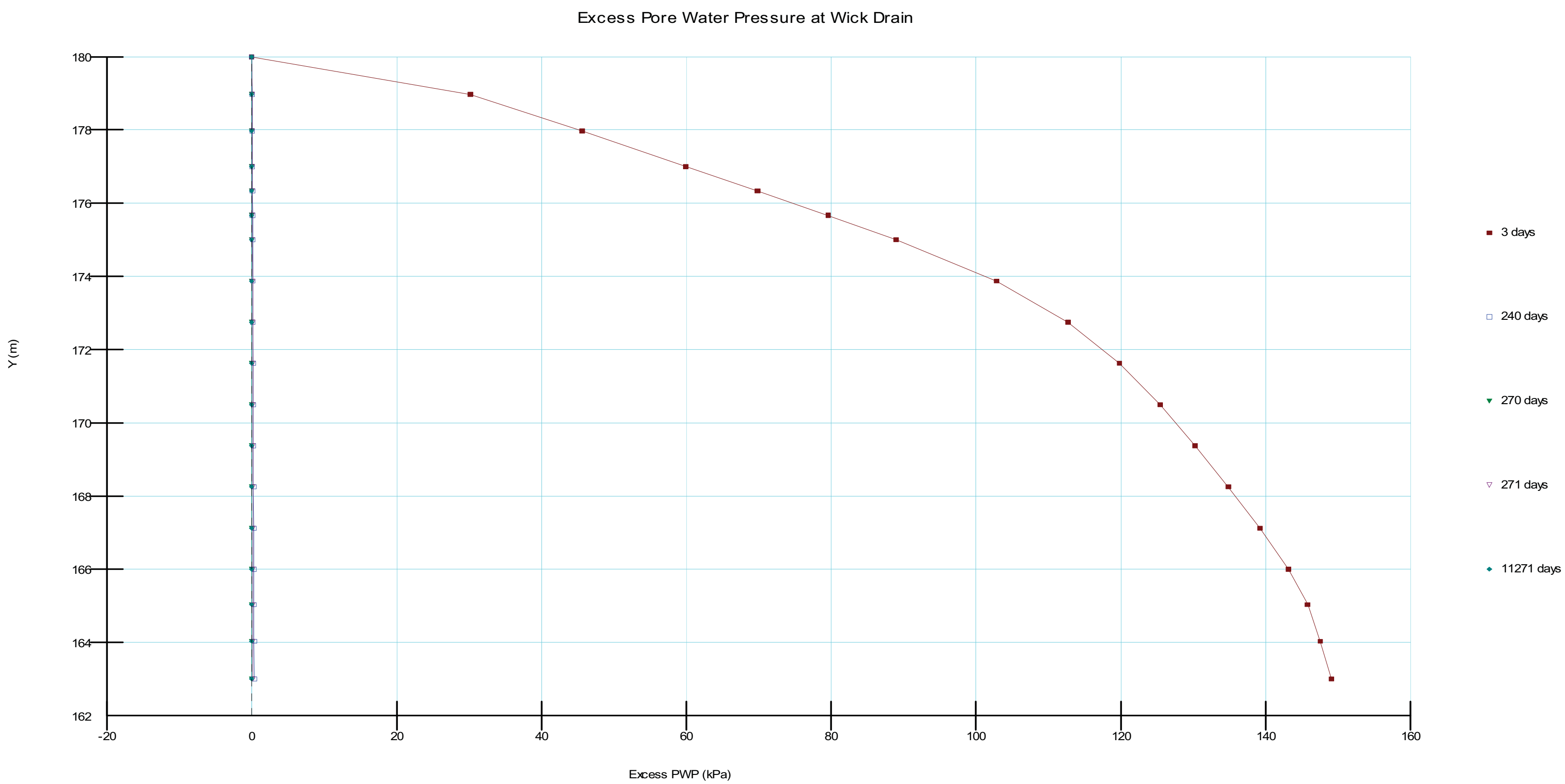
PARKWAY INFRASTRUCTURE CONSTRUCTORS | acciona | DRAGADOS | FLUOR

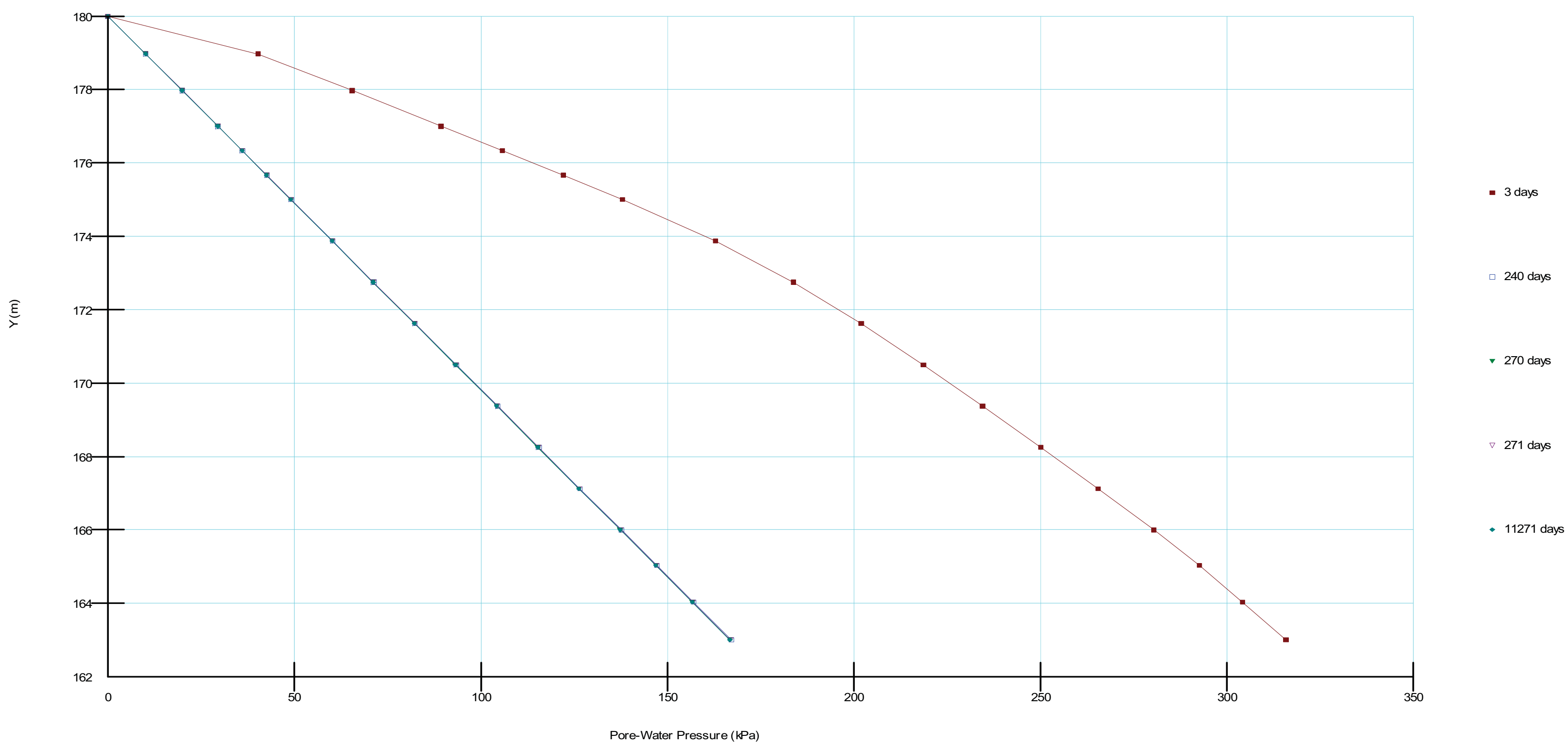
PROJECT:	WINDSOR ESSEX PARKWAY			
TITLE:	CUMULATIVE SOIL SETTLEMENT PROFILE ALONG PILE LINE BRIDGE B-3			
DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: G.10	REV.



Legend:
3 days = 3rd Day of Embankment Construction with Wick Drain
240 days = End of Embankment Construction with Wick Drain
270 days = End of Abutment Construction
271 days = End of Construction of Bridge
11271 days = Long-term Condition

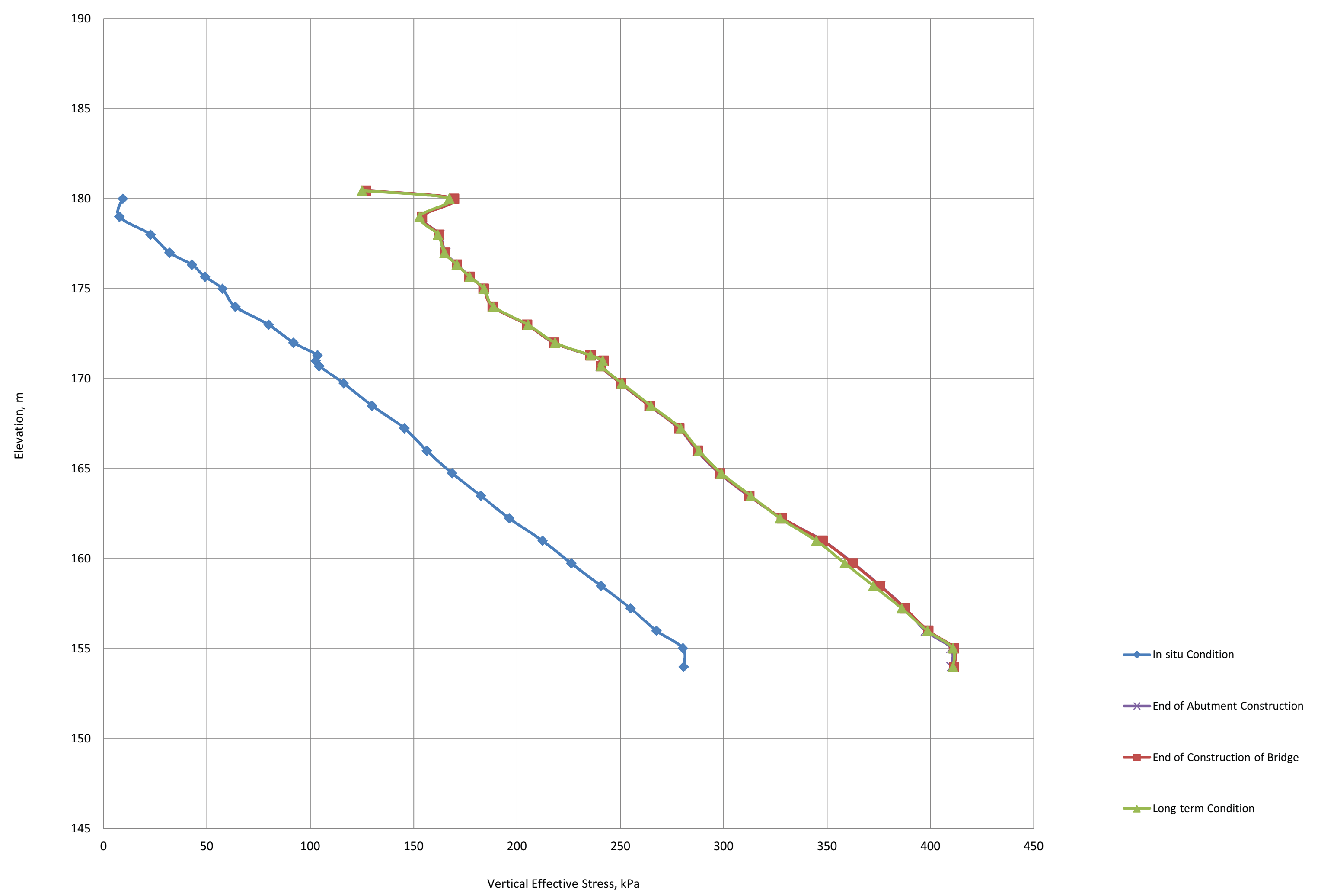
Parkway Infrastructure Engineers / amec / Hatch Mott MacDonald	PARKWAY INFRASTRUCTURE CONSTRUCTORS / acciona / DRAGADOS / FLUOR	PROJECT: WINDSOR ESSEX PARKWAY			
		TITLE: EXCESS PORE WATER PRESSURE AT WICK DRAIN BRIDGE B-3			
		DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: G.11 REV.

Pore Water Pressure at Wick Drain



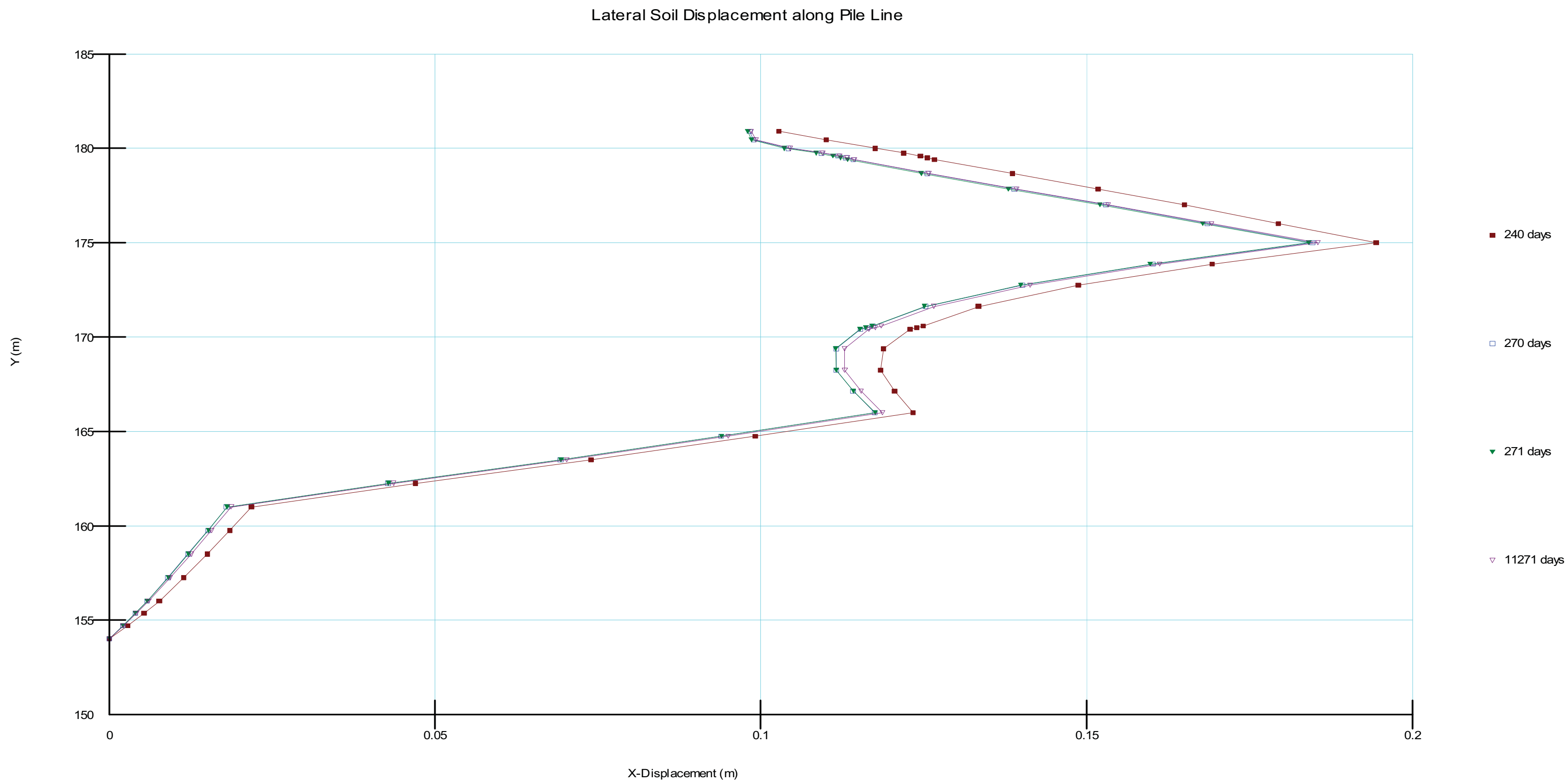
Legend:
3 days = 3rd Day of Embankment Construction with Wick Drain
240 days = End of Embankment Construction with Wick Drain
270 days = End of Abutment Construction
271 days = End of Construction of Bridge
11271 days = Long-term Condition

PROJECT: WINDSOR ESSEX PARKWAY				
TITLE: PORE WATER PRESSURE AT WICK DRAIN BRIDGE B-3				
DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: G.12	REV.



PROJECT:	WINDSOR ESSEX PARKWAY
TITLE:	VERTICAL EFFECTIVE STRESS ALONG PILE LINE BRIDGE B-3

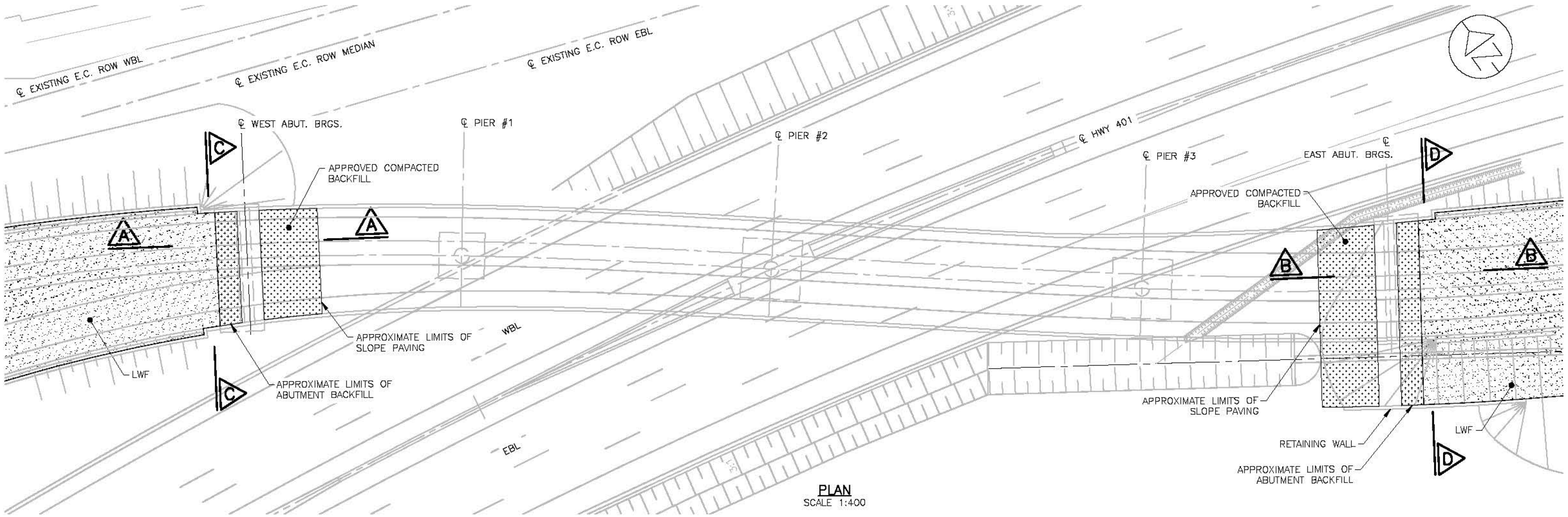
DATE:	JOB NO.:	CAD FILE:	FIGURE NO.: G.13	REV.
Jun 2012				



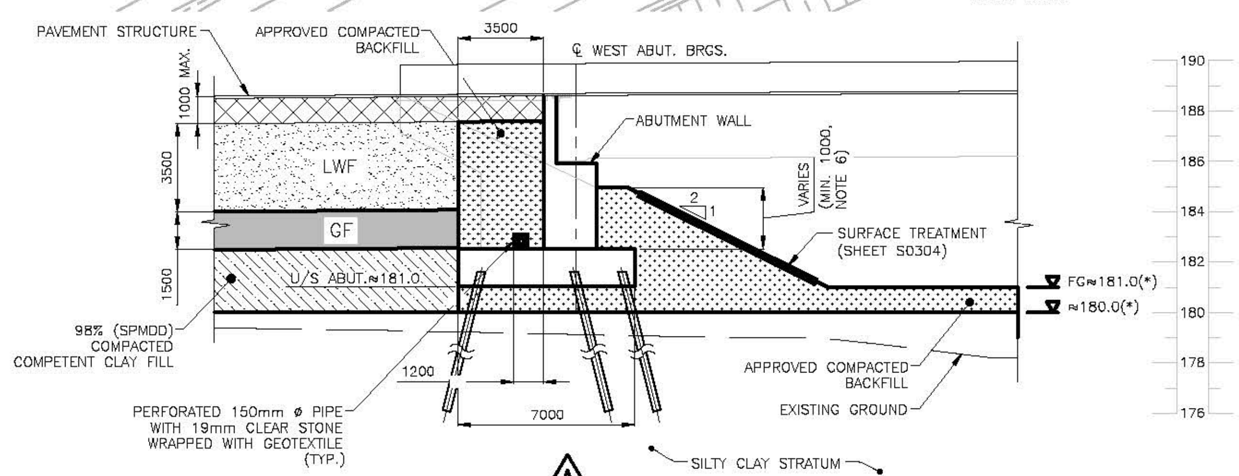
Legend:
240 days = End of Embankment Construction
270 days = End of Abutment Construction
271 days = End of Construction of Bridge
11271 days = Long-term Condition

PROJECT: WINDSOR ESSEX PARKWAY					
TITLE: CUMULATIVE LATERAL SOIL DISPLACEMENT ALONG PILE LINE BRIDGE B-3					
DATE: Jun 2012	JOB NO.:	CAD FILE:	FIGURE NO.: G.14	REV.	

Appendix H Conceptual Drawings

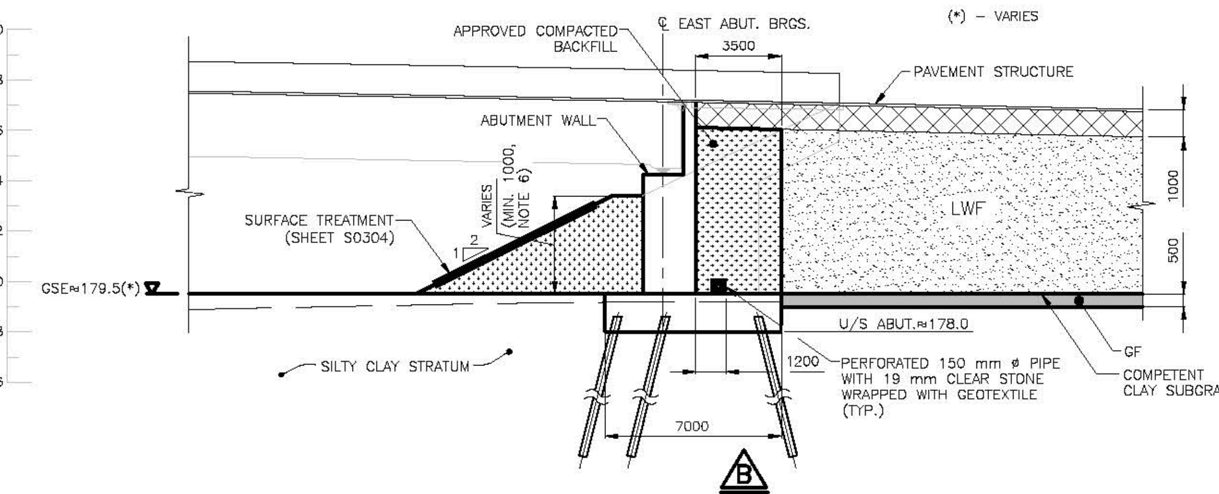


PLAN
SCALE 1:400



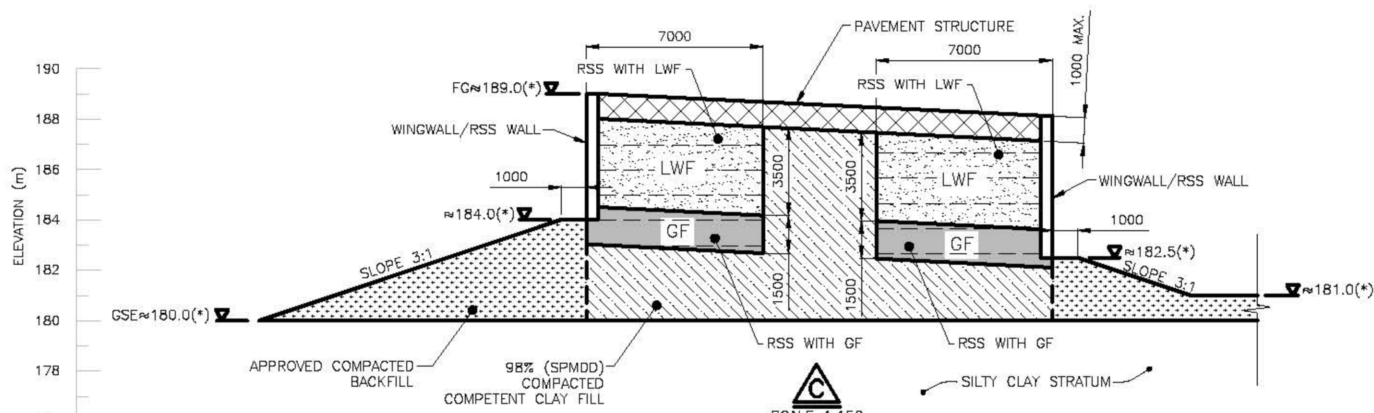
SCALE 1:150

WEST ABUTMENT WALL SECTION A–A



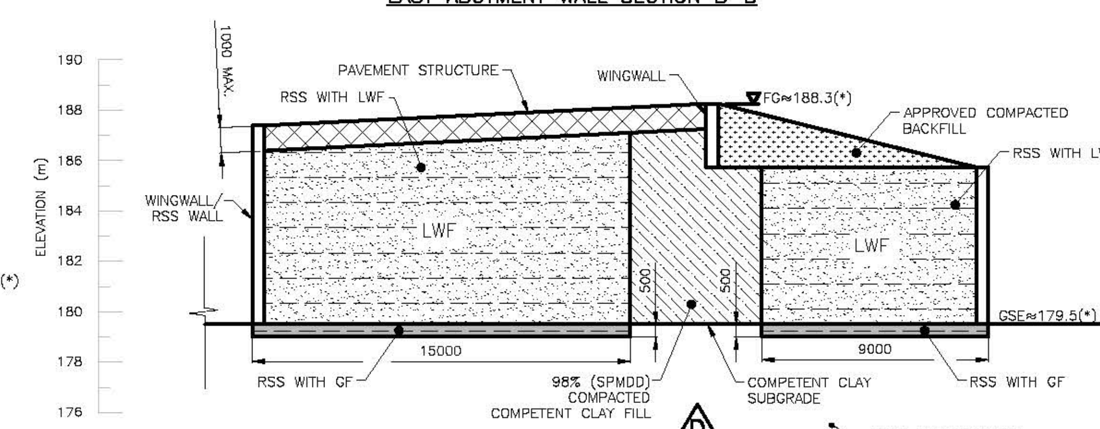
SCALE 1:150

EAST ABUTMENT WALL SECTION B–B



SCALE 1:150

WEST WINGWALL SECTION C–C



SCALE 1:150

EAST WINGWALL SECTION D–D

NOTES:

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
2. THIS DRAWING ILLUSTRATES THE GENERAL BACKFILL ARRANGEMENT AT SELECTED REPRESENTATIVE LOCATIONS OF THE ABUTMENTS OF BRIDGE B–3 BASED ON GEOTECHNICAL DESIGN ANALYSES.
3. ABUTMENT AND APPROACHWAY EMBANKMENT ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGN WERE OBTAINED FROM STRUCTURAL DRAWINGS AVAILABLE IN MAY 2012. ABUTMENT ELEVATIONS VARY ALONG THE APPROACHWAY.
4. 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.
5. 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.
6. BACKFILL IN FRONT OF ABUTMENT WALL SHOULD BE COMPLETED TO AT LEAST 1000MM ABOVE THE PILE CAP BEFORE PLACING BACKFILL BEHIND THE ABUTMENT WALL.
7. GRANULAR BASE UNDER HIGHWAY 401 PAVEMENT MUST BE COMPLETED BEFORE PLACING APPROACHWAY FILL ABOVE THE BRIDGE SEAT LEVEL.
8. GRANULAR BACKFILL BEHIND CONCRETE ABUTMENT AS PER OPSD 3101.150.
9. SEE ACCOMPANYING DRAWINGS FOR APPLICABLE CONSTRUCTION NOTES.
10. RSS STRAP LENGTHS SHOWN ARE PRELIMINARY. FINAL RSS DESIGN WILL BE PERFORMED BY SUPPLIER

LEGEND:

LWF – LIGHT WEIGHT FILL (ULTRALIGHT WATER-COOLED IRON FURNACE SLAG) FULLY WRAPPED IN FILTER FABRIC

GF – GRANULAR FILL

RSS – REINFORCED SOIL STRUCTURE

(*) – VARIES

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

BRIDGE B–3
REALIGNED E.C. ROW–EBL EXPRESSWAY UNDERPASS NEAR MATCHETTE ROAD
PLAN & SECTIONS OF ABUTMENTS AND WINGWALLS

DWG. BY: SJL	CHK. BY: NR	FIGURE NO.: H.1
DATE: June–12	SHEET: 1 OF 1	

DOC: B–3 PLAN W ABMT SECTIONS_FIG H.1