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# The Windsor-Essex Parkway Project

# Geotechnical Investigation and Design Report – Bridge B-11

## (Highway 3 Underpass East of Montgomery Drive)

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

## 1.2 Report Introduction

This report presents the geotechnical design of Bridge B-11(Highway 3 Underpass East of Montgomery Drive), located in the LaSalle sector of the proposed Windsor-Essex Parkway (WEP) project. Bridge B-11 carries traffic of Highway 3 over Highway 401. The report includes the results of geotechnical investigation carried out to support the design, which were available at the time of preparation of the report and other relevant background information, and addresses review comments from peer reviews. This is the final report and is issued for construction (IFC).

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

Bridge B-11 is a twin bridge, four-span underpass which carries the traffic of Highway 3 over Highway 401 between Sta. 13+060L and Sta. 13+300L (Highway 3 Station 41+408 to 41+643 ) and near Montgomery (Drawing 285380-03-060-WIP1-1101).

The proposed Bridge B-11 will consist of twin multi-lane post-tensioned cast-in-place concrete box girder structures located on a curved alignment over Highway 401. The bridge structures comprise true abutments made with cast-in-place concrete and piers supported on deep end-bearing vertical and batter HP 310×110 steel piles. The abutments between the North and South bridges are staggered, which requires a Reinforced Soil Structure (RSS) retaining wall to be constructed between the North and South bridges at both the East and West abutments of the bridge.

The geotechnical design presented in this report was generally advanced from the preliminary geotechnical design developed for the WEMG proposal in June 2010 (ref. R-43)[1].

The report includes the results of geotechnical investigations carried out to support the design and other relevant background information.

The geotechnical design has been developed through interactive collaboration of the geotechnical, structural, other design disciplines, and the Parkway Infrastructure Constructors (PIC).

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

The design of Bridge B-11 complies with the requirements of execution version of the Project Agreement (PA) Schedule 15-2 Part 2, Article 5.

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[1] References are listed in Section 9.

# 2 Background Information

## 2.1 Geological Setting

The WEP project site is located within the Essex Clay Plain, (a part of the St. Clair Clay Plain physiographic region) (ref. R-16, R-18, and R-25). The Essex Clay Plain was deposited during the retreat of the late Pleistocene Era ice sheets, when a series of glacial lakes inundated the area. The ice sheets generally deposited materials with a glacial till-like gradation in the Windsor area. Depending on the locations of the glacial ice sheets and depths of water in the ice-contact glacial lakes, the materials may have been directly deposited at the contact between the ice sheet and bedrock or, as the lake levels rose and the ice sheets retreated and floated, the soil and rock debris within and at the base of ice may have been deposited through the lake water (i.e., lacustrine environment). It is considered that unlike typical till deposits (that have undergone consolidation and densification under the weight of the ice sheet), the majority of the “glacial till” soils in the Windsor and Detroit area were deposited through water and have a soft to firm consistency below a surficial crust layer that has become stiff to hard due to weathering and desiccation. Geologically, the deposit in the project area is considered to be slightly over-consolidated, having experienced no major overburden stresses in excess of the existing stresses.

The overburden in the St. Clair Clay Plain has variously been described as clayey silt till, silty clay till and glacio-lacustrine clay. Hudec (ref. R-25) summarized the overburden geology in Windsor as consisting of the following successive strata: desiccated lacustrine clay, normally consolidated lacustrine clay, silty Tavistock till, glacio-lacustrine clay and coarse Catfish Creek till. A distinct change in overburden deposits occurs in the east-west direction along a boundary located generally along Huron Church Road. The eastern part of Windsor is underlain by firm to stiff, glacio-lacustrine silts and clays with upper deposits of stiff sandy to silty weathered clay and a hard to stiff lacustrine clay-silt crust. The western part of Windsor is characterized by a thin surficial granular deposit underlain by a thin crust layer underlain by soft to firm glacio-lacustrine 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 weathering and desiccation. Up to 2 m thick surficial layers of lacustrine silty clay or silt and sand are also encountered in the western sector of the project. A 1 m to 6 m thick, very dense or hard basal glacial till or dense silty sand may be found directly overlying the bedrock surface. The bedrock at the project area consists of limestone, dolostone and shale comprising the Devonian Dundee Formation of the Hamilton Group Formation underlain by the Devonian Lucas Formation of the Detroit River Group Formation.

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

## 2.2 Site Seismic Background

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

In accordance with the CHBDC, and based on a series of cross-hole tests completed during the background investigation program (ref. R-18), 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). The above noted cross-hole tests were completed during the background investigation program (ref. R-21) at locations distributed along the project alignment between Howard Avenue (east end) and Matchette Road (west end). The measured velocities of the shear waves were consistently over 200 m/s, with the bulk of results ranging between 200 and 300 m/s.

## 2.3 Site Conditions

Bridge B-11 site is situated in the western half of LaSalle segment of the Parkway. The bridge structure will be constructed under WEP Phase I development and will be used to carry Highway 3 traffic over Highway 401. An east bound and west bound ramp will also be within the general area to carry traffic onto Highway 401 from Highway 3.

The topography of the lands immediately adjacent to Bridge B-11 is generally flat with elevations around 185.5[2] . Adjacent land use is typically residential (see Appendix G for selected site photos).

## 2.4 Frost Depth

In accordance with MTO–SDO-90-01, “Pavement Design and Rehabilitation Manual” (ref. R-37) and OPSD 3090.101, the frost depth in the Windsor area is estimated to be 1.0 m below the ground surface. [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.

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

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[2] Elevations are in meters 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) as part of background studies for development of the WEP proposal designs. Additional geotechnical investigation was carried out to supplement the previously obtained (pre-bid) subsurface soil data and support the detailed design development of the WEP embankment and structures. The subsurface exploration program at and around the proposed location of Bridge B-11 comprised a total of 7 boreholes, 2 Nilcon vanes, 4 cone penetration tests (CPT B11-1, CPT 48-RW, CPT 49-RW and CPT 50-RW) and 1 Flat Blade Dilatometer (DMT B11-1**)** test. Table 3-1lists the test holes advances 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-11 Site**

| Reference | Boreholes | Nilcon Vane Tests | CPT | DMT |
|---|---|---|---|---|
| Additional Investigation (2011) | BH B11-1 | NIL B11-2 | CPT B11-1 | DMT B11-1 |
| | BH B11-2 | NIL B11-6 | CPT 48-RW | |
| | BH B11-3 | | CPT 49-RW | |
| | BH B11-4 | | CPT 50-RW | |
| | BH B11-5 | | | |
| | BH B11-6 | | | |
| | BH B11-7 | | | |
| Previous Studies (2007-09) | BH-109 | | CPT-04 | |
| | | | BH/CPT-108 | |
| | | | BH/CPT-307 | |
| | | | BH/CPT-309 | |

Drawing 285380-04-090-WIP1-1101 shows the locations of the test holes and an interpreted soil stratigraphic profile along the WEP centreline for the general area at and around Bridge B-11. The test hole locations and stratigraphic sections at the bridge location are illustrated on Drawings 285380-04-090-WIP1-1102 and 285380-04-090-WIP0-1103.

## 3.2 Fieldwork

The boreholes were advanced using track-mounted CME 55 auger rigs, owned and operated by Marathon Drilling Co. Ltd., under contract to AMICO and under technical observation by AMEC engineers and technicians. Boreholes were generally advanced using 215 mm OD hollow stem augers, followed by wash boring with NW casing. The depth at which the drilling methods transition occurred is noted on the borehole logs.

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

Standard Penetration Tests (SPT, ASTM D1586) were carried out in conjunction with split spoon sampling. Field vane tests (using conventional vanes) were carried out in between split spoon sampling at selected depths. The Nilcon vane tests listed in Table 3-1 were carried out adjacent to the borehole locations. 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.

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

| Borehole | Location[5] | Overburden Thickness, m | Test Name & Elevation | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Rock Coring | Nilcon Vane | S-Piez. | VWP | MHSG | IN |
| BH B11-1 | N 4678221<br>E 334583 | 37 | 148.4 to 145.3 | | | 177.2 & 168.4 | | X |
| BH B11-2 | N 4678193<br>E 334624 | 36.7 | 148.9 to 147.1 | 180.4 to 162.4 | | | | |
| BH B11-3 | N 4678180<br>E 334655 | 36.9 | 148.5 to 145.2 | | | | | |
| BH-B11-4 | N 4678196<br>E 334679 | 38.3 | 146.7 to 145.5 | | | 175, 166.7 & 149 | 176.6 & 166.7 | |
| BH-B11-5 | N 4678173<br>E 334734 | 37.4 | 148.0 to 145.8 | | | | | |
| BH-B11-6 | N 4678154<br>E 334772 | 37.1 | 148.7 to 147.2 | 180.9 to 160.9 | 181.1 | | | |
| BH-B11-7 | N 4678126<br>E 334810 | 35.4 | 150 to 146.9 | | | 175.5 & 166.4 | | X |
| CPT B11-1 | N 4678195<br>E 3345956 | >5.0 | | | | | | |
| DMT B11-1 | N 4678224<br>E 334579 | >5.0 | | | | | | |

| | | |
|---|---|---|
| Legend: | S-Piez. | Screen elevations for Standpipe Piezometer |
| | VWP | Sensor elevation for Vibrating Wire Piezometer |
| | MHSG | Magnet Heave/Settlement Gauge |
| | IN | Slope Inclinometer |

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[4] Advanced lab tests (consolidation, direct shear and triaxial tests) were carried out in AMEC's Scarborough lab.

[5] Location coordinates are in UTM-NAD 83 (Zone 14).

Borehole logs illustrating the interpreted soil conditions, field test results and laboratory index test results are included in Appendix A and B. Laboratory test results are presented on figures included in Appendix C.

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

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

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 the CPT B11-1 at 12.50 m below grade.

The Nilcon blade was pushed in ground using the hydraulic ram of the drill rig. The shear vane tests were conducted in general accordance with ASTM D2573-01.

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

The locations of boreholes, Nilcon tests, and CPT executed during the most recent 2011 investigation, and the inferred soil stratigraphy at and around the Bridge B-11 area, are shown on Drawings 285380-04-091-WIP1-1101, 285380-04-090-WIP1-1102 and 285380-04-091-WIP1-1103. Borehole and CPT logs from the additional 2011 investigation are included in Appendix A. Relevant borehole logs from earlier investigations are included in Appendix B.

## 3.3 Geotechnical and Analytical Laboratory Testing

All recovered soil samples and rock cores were examined in the field and the AMEC geotechnical laboratory. Natural moisture content tests were carried out on most of the recovered samples. Grain size distribution and Atterberg limit tests were carried out on selected representative samples. Three representative soil samples were selected for one oedometer (1-D consolidation) test and one consolidated-undrained triaxial test (CIUC).

Selected samples of the silty clay to clayey silt obtained from Boreholes B11-1 to B11-7 were sent to the ALS Environmental Analytical Laboratory in London, Ontario to determine the pH, redox potential, resistivity, sulphide and sulphate content of the soil to assess corrosion potential.

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

## 3.4 Instrumentation

Geotechnical instruments were installed at designated locations upon completion of boreholes to monitor pore water pressure and deformation behaviour of the soil strata during and after construction. An instrumentation location plan is provided in Figure J-1 in Appendix J (Instrumentation Location Plan). A brief description of these instruments is as follows:

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

Vibrating Wire Piezometers (VWP): VWP transducers (RST Model VW2100, 0.35 MPa for shallow to mid-depth and 0.7 MPa for deep installations) were installed at the designated depths and electrical wires extended to the monitoring station located at the ground surface (outside the parkway footprint area). The 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.

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

Inclinometers (IN): Snap-seal 2.75 inch inclinometer casings with groves were installed in selected boreholes (Table 3-2) to measure the lateral movement of the soil. The boreholes were backfilled with bentonite-cement grout to ground surface.

## 3.5 Data Interpretation

***Field Vane Test Data Correction**:* The chart (Figure 3-1[6]) developed initially by Bjerrum (1972) and updated subsequently by Ladd et al (1977) ) based on circular arc failure analyses of embankment failures suggest correction by multiplying the field vane data by 1.05 to 1.10 for soils with plasticity index of about 15 (ref. R-6 and R-32). However, based on re-evaluation of the Bjerrum chart by Aas et al. (1986), the Canadian Foundation Engineering Manual suggests that the vane test data for clays with PI<20 should not be corrected (ref. R-1 and R-8, Figure 3-2 ). Therefore, the field vane test data (from conventional and Nilcon vane tests) at this site were not corrected for PI. The undrained shear strength ($S_u$) profiles inferred from the DMTs and the $S_u$ values obtained from the conventional field vane tests in boreholes were consistently higher than the Nilcon vane test values.

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

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

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

Where:

| | |
|---|---|
| $S_{uCPT}$ | is the undrained shear strength estimated from the CPT test; |
| $Q_t$ | is the corrected total cone tip resistance; |
| $\sigma_{vo}$ | is the total vertical stress at the corresponding depth of measurement of the Qt value; and |
| $N_{kt}$ | is an empirical factor, depends on soil type & test arrangement, typically between 8 & 20. |

The CPT based $S_u$ profiles were developed to achieve a general agreement with the nearby Nilcon vane test profiles by modifying the $N_{kt}$ factor values used to calibrate the CPT strength profiles varied slight for different segments of the WEP and the soil strata. Thus, 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 15-16, and 12-13[7], respectively. In CPTs indicating pore pressures higher than cone tip resistance (e.g., soft clay stratum in CPT B11-1), the undrained shear strength was estimated from the excess pore pressures (using the $N_u$ method).

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

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

Where:

| | |
|---|---|
| $S_u$ | is the undrained shear strength; |
| $\sigma'_{vo}$ | is the vertical effective stress; |
| $\sigma'_p$ | is the pre-consolidation pressure (also referred as maximum past pressure); |
| $S$ | is the normalized strength ratio ($S_u/\sigma'_{vo}$) of normally consolidated soil; |

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[7] $N_{kt}$ values for upper silty clay 16 and for lower clayey silt 12 (for 10+400L to 11+000L).

*OCR* is the over-consolidation ratio; and

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

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

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

***Flat Blade Dilatometer (DMT) Test Data:*** DMT tests were conducted following the ASTM D6635-01(2007) method.

The soil properties from the results of these tests were developed in general using the guidelines layout in ISSMGE, 2001 (ref. R-26), except that the undrained shear strength values for the clay deposits were estimated using the relationship $S_u = S\ \sigma'_{vo}\ (0.5\ K_d)^{1.25}$, where $S = 0.18$. $K_d$ is the horizontal stress index obtained from DMT reading and is defined by:

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

Where:

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

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

The undrained shear strength ($S_u$), pre-consolidation pressure ($\sigma_p'$), natural water content ($w_N$) and compression index ($C_c$) profiles based on field and laboratory testing from boreholes, CPTs and DMT carried out in the general area of Bridge B-11 (WEP segment between Sta. 12+800L to 13+400L) are presented in Figure 3-3. Also included on these figures are $0.18\sigma\times\ {}_{vo}'$ curve (representing undrained strength for OCR=1 condition) and simplified soil stratigraphic deposits to facilitate correlation of soil properties to the individual soil units. The constant 0.18 for $S_u\ \ {}_{vo}$ for OCR=1 curve is based on average plasticity index of the silty clay to clayey silt stratum and published relationships (ref. R-11).

# 4 Subsurface Conditions

The general soil stratigraphy at the borehole locations consists of the following successive strata: surficial layers of occasional fills, topsoil, and upper granular deposit; extensive clayey silt to silty clay deposit below about elevation 184.7, and a lower granular deposit below about elevation 154, overlying limestone and dolostone bedrock below about elevation 148.5. The thickness of the clayey silt to silty clay deposit varies between about 26 m and 30.5 m. The lower granular deposit (sandy silt / silty sand / sand and gravel) varied in thickness between 4.9 to 5.5 m. The bedrock was encountered at depths ranging from about 35.4 m to 38.3 m below the ground surface.

## 4.1 Surficial Fills, Topsoil and Upper Granular Deposit

Boreholes BH B11-1 and B11-6 was advanced through existing pavement and encountered between 25 to 225 mm thick asphalt layer overlaying sand fill which extended to between 0.3 to 0.8 m below existing grade. Boreholes BH B11-2 and B11-3 encountered between 0.6 to 0.8 m of silty sand and gravel fill (crushed limestone) overlying a silty clay mixed with topsoil which extended to of up to 2.1 m below existing grade. Boreholes BH B11-4, B11-5 and B11-7 encountered up to 0.5 m thick layer of brown to black topsoil. Some clay was present in the organics in B11-7.

The topsoil and underlying fill are expected to vary in quality and thickness throughout the project area.

## 4.2 Silty Clay to Clayey Silt Stratum

The cohesive silty clay stratum was encountered directly beneath the surficial topsoil or fill/granular deposit. The encountered depth was from 0.3 to 2.1 m below existing ground surface corresponding to elevation 185.5 to 183.3. Based on the gradation, in-situ moisture content, and strength characteristics, the stratum may be divided into 4 layers as follows: brown desiccated stiff to very stiff clay crust, transition zone, upper grey silty clay to clayey silt deposit (referred to hereafter as upper silty clay), and then a generally coarser lower grey clayey silt deposit (referred to hereafter as lower clayey silt). The natural water content, Atterberg limits and total unit weight properties of the clay sub-strata are summarized in Table 4-1.

The undrained shear strength (Su) of the crust, transition zone, upper silty clay and lower clayey silt layers generally varied from 189 to 128 kPa, 55 to 107 kPa, 38 to 95 kPa and 47 to 156 kPa, respectively. As shown on Figure 3.3, the undrained strength decreased gradually from the crust layer to the bottom of the upper silty layer.

## Table 4-1: Index Properties of Silty Clay to Clayey Silt Strata

| Property | Clay Crust | Transition | Grey Silty Clay | Clayey Silt |
|---|---|---|---|---|
| Elevation Range, m | 185 to 179 | 179 to 175 | 175 to 163 | 163 to 156 |
| Natural Water Content, $w_N$, % | 11 to 25 | 14 to 17 | 10 to 36[1] | 10 to 29 |
| Liquid Limit, $w_L$, % | 22 to 29 | 22 to 24 | 21 to 39 | 22 to 31 |
| Plastic Limit, $w_P$, % | 12 to 18 | 13 to 15 | 12 to 21 | 14 to 17 |
| Plasticity Index, PI | 8 to 13 | 7 to 11 | 8 to 18 | 7 to 14 |
| Liquidity Index, LI | <0.35 | 0.12 to 0.27 | 0.08 to 1.71 | < 0.86 |
| Unit Weight, $\gamma$, kN/m$^3$ | 20.3 to 23.5 | 21.4 to 22 | 19.0 to 21.8 | 19.4 to 22.5 |

1) The overall and general ranges of moisture content are 8% to 42% and 10% to 36%, respectively..

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

- Crust layer: > 100 kPa
- Transition layer: 100 kPa to 65±15 kPa
- Upper silty clay: 65±15 kPa to 50±10 kPa
- Lower clayey silt: 60 ±10 kPa to >75 kPa

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

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

$$C_c = 0.0086w_N - 0.0086)$$

$$C_r = 0.11C_c$$

$$C_s = 0.25C_c$$

$$C_\alpha = 0.028C_c$$

The interpreted average values used for the clay substrata for the Bridge B-11 site are summarized in Table 4-2.

### Table 4-2: Clay Interpreted Compressibility Properties

| Property | Clay Crust | Transition | Grey Silty Clay | Clayey Silt |
|---|---|---|---|---|
| Average Natural Water Content, $w_N$, % | 14 | 16 | 18 to 24 | 19 |
| Average Total Unit Weight, kN/$m^3$ | 22 | 21.5 | 21 to 20.5 | 22 |
| Pre-consolidation Pressure, kPa | 550 | 550 to 340 | 340 to 280 to 315 | 500 |
| Virgin Compression Index, $C_c$ | 0.11 | 0.12 | 0.14 to 0.19 | 0.15 |
| Recompression Index, $C_r$ | 0.017 | 0.019 | 0.022 to 0.029 | 0.023 |
| Swelling Index, $C_s$ | 0.028 | 0.032 | 0.036 to 0.048 | 0.039 |
| Secondary Compression Index, $C_\alpha$ | 0.003 | 0.004 | 0.004 to 0.005 | 0.004 |

Oedometer testing carried out on samples in the upper grey silty clay from Borehole BH B11-6 TW16 (16.8 m depth) indicated the following compressibility indexes: $C_c = 0.144$, $C_r = 0.0288$, $C_s = 0.0375$, which are within the range of compressibility characteristics presented in Table 4-2.

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). For the unweathered portion of the silty clay stratum the empirical relationship were used based on average shear strength profiles for the material, as follows:

$$E_u = 300\ S_u$$

$$E' = 0.9E_u$$

### Table 4-3: Clay Interpreted Elastic Moduli Properties

| Soils Stratigraphy | Elastic Modulus-Undrained, MPa | Poisson’s Ratio-Undrained (*) | Elastic Modulus-Drained, MPa | Poisson’s Ratio-Drained (*) |
|---|---|---|---|---|
| Clay Crust | 35 | 0.49 | 31.5 | 0.35 |
| Transition | 22.5 to 19.5 | | 20.2 to 17.6 | |
| Grey Silty Clay | 15.9 to 15.8 | | 14.3 to 14.2 | |
| Clayey Silt | 28.2 | | 25.4 | |

(*) Assumed values

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

### Table 4-4: Effective shear strength Properties

| | |
|---|---|
| Apparent cohesion, c’ | 0 kPa |
| Angle of internal friction, f’ | 30° |
| Residual angle of internal friction, $f_r$’ | 27° |
| Friction angle at critical state, $\Phi_c$ * | $25^0$ - $26^0$ |

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

Consolidated Undrained Triaxial Compression (CIUC) tests carried out on a clayey silt sample obtained from approximately 13.7 m from Borehole B11-6 indicated an effective friction angle of 30 degrees.

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

## 4.3 Lower Granular Deposit

Underlying the silty clay to clayey silt stratum and overlying the bedrock was a discontinuous and heterogeneous non-cohesive material (varying from silty sand, sand and gravel, and silts with sand). Based on SPT N-values ranging generally from 4 to greater than 100, this material is considered to be in a compact to dense state. This layer was approximately 3.9 to 7.7 m thick and varies significantly throughout the Bridge site. Indications of presence of cobble and boulders within deposit were noted in BH B11-4, 5 and 7.

## 4.4 Bedrock

Where rock coring was undertaken, a grey, limestone bedrock was encountered. The bedrock was generally fresh, medium strong, thinly laminated, fine grained, faintly to moderately porous and highly fractured. Bedrock was encountered at elevations ranging from 146.8 to 151.9 in the vicinity of Bridge B-11. The Rock Quality Designation (RQD) of the recovered rock varied between 0 to 100 per cent. Except for a few low numbers, the RQD values generally ranged from 38% to 100% with most of the values greater than 50% indicating fair to excellent quality. Photographs of rock cores recovered from the additional investigation are provided in Appendix H.

Based on this core logging the rock mass classification was estimated to range from 2.8 to 5 for the Q-System (Barton, ref. R-3) and 53 to 58 for the Rock Mass Rating (RMR) based on Bieniawski (ref. R-5) and indicates that the rock mass can be considered as a Fair quality rock mass based on the later system.

It was found during the preliminary pre-bid investigation report (ref. R-16) that little variation in the strength of the rock mass conditions was identified from site to site. For this reason in order to obtain a reasonable statistical sample, the density, unit weight and uniaxial compressive strength of the samples from all of the key sites have been grouped and are summarised in (Table 4-5). The average strength of the limestone is determined to be 85.5 MPa and is ‘strong rock’ based on the ISRM (ref. 27). Additionally, based on the coefficient of variation, enough tests have been performed to characterise the compressive strength.

### Table 4-5: Summary of Intact Rock Properties

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

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

## 4.5 Groundwater Conditions

A shallow standpipe piezometer and a set of shallow and deep vibrating wire piezometers were installed in selected boreholes to measure the stabilized water levels within overburden and bedrock. The latest available readings are summarized in Table 3-2.

The piezometric levels in the upper part of the upper silty clay were generally around 182.4 to 184.1. The piezometric levels in the sand and gravel deposit overlying the bedrock were 176.9 and 177.8. The highest piezometric levels within the overburden and the bedrock were reported to be at elevations 184.9 and 177.8, respectively. These observations suggest a downward gradient between the overburden and the bedrock. Nevertheless, given the experience at other locations in the Windsor area, and based on heaving sand and gravel in the augers encountered in the Boreholes B11-1, 2 and 4 when drilling within the lower granular deposit, local occurrence of artesian condition in bedrock cannot be ruled out.

### Table 4-6: Summary of Stabilized Measured Piezometric Water Levels

| Piezometer in Borehole | Surface El, m | Piezo. Type | Screen / Sensor El, m | Stratum at Screen / Sensor Depth | Measured Water level | |
|---|---|---|---|---|---|---|
| | | | | | Date | El, m |
| BH B11-1 | 185.4 | VWP | 177.0 | Silty Clay | Aug. 29, 2011 | 183.7 |
| | | VWP | 167.8 | Silty Clay | Aug. 29, 2011 | 182.8 |
| BH B11-4 | 185.0 | VWP | 175.43 | Silty Clay | Aug. 18, 2011 | 182.4 |
| | | VWP | 167.14 | Clayey Silt | Aug. 18, 2011 | 181.4 |
| | | VWP | 148.84 | Sand and Gravel | Aug. 18, 2011 | 176.9 |
| BH B11-6 | 185.8 | S-Piez | 182.6 to 181.1 | Silty Clay | July 29, 2011 | 184.9 |
| BH B11-7 | 185.4 | VWP | 175.88 | Clayey Silt | Aug. 29, 2011 | 184.1 |
| | | VWP | 166.76 | Silty Clay | Aug. 29, 2011 | 183.8 |
| BH-109 | 185.3 | S-Piez | 176.2 | Clayey Silt | Jan. 28, 2009 | 183.5 |
| | | S-Piez | 151.1 to 149.6 | Sand and Gravel | July 10, 2011 | 177.8 |

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

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

## 4.6 Subsurface Gases

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

The $H_2S$ gas can frequently be detected by odour at concentrations on the order of 0.5 ppm and can be corrosive at concentrations of about 2 ppm to 3 ppm in the groundwater. The gas odour was not detected during the drilling at the Bridge B-11 site.

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

**Table 4-7: Pumping Tests Data**

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

The understanding of the engineering behaviour (related to the impact on design and construction) of the gassy soils is rather limited. In the case of low permeability cohesive soils it is known that these soils may experience rapid drop in undrained shear strength during unloading. Due to the relatively high compressibility of the pore water fluid in gassy soils, the immediate pore water pressure response ($\Delta U$) to total stress changes can be very low. This phenomena leads to reduction in effective stress and hence shear strength (ref. R-24 and R-41). It is, therefore, recommended that the design and construction methodologies should be developed in consideration of the potential presence of these gases (ref. R-14). Air quality and subgrade pore pressure monitoring should be carried out during construction. The equipment operating in confined spaces should be selected to safely operate in a potentially gaseous environment. Excavation layers should be decided in consideration of the pore pressure monitoring data and the potential ground softening.

# 5 Development of Geotechnical Design

## 5.1 Bridge Configuration

Bridge B-11 is a twin bridge, four-span underpass near Montgomery Drive which will carry the traffic of Highway 3 over Highway 401 between Sta. 13+060L and Sta. 13+300L (Highway 3 Station 41+408 to 41+643 , Drawing 285380-03-060-WIP1-1101).

The proposed Bridge B-11 will consist of twin multi-lane post-tensioned cast-in-place concrete box girder structures along a curved alignment over Highway 401. The bridge structures comprise true abutments made with cast-in-place concrete and piers supported on deep end-bearing vertical and batter HP 310×110 steel piles. The abutments between the North and South bridges are staggered, which requires a Reinforced Soil Structure (RSS) retaining wall to be constructed between the North and South bridge at both the East and West abutments of the bridge.

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

**Table 5-1: Summary of Control Elevations at Abutments**

| Bridge | Location | Existing Ground Surface[1] | Top of Deck El, m | Top of Pile Cap El, m | Bottom of Pile Cap El, m | HWY 401 Subgrade El, m |
|---|---|---|---|---|---|---|
| North | West Abutment | 185.5 | 186.2 | 179.0 | 177.5 | Varies 177.5 near West end to 180 near East end |
| | Pier 1 | 185.5 | 186.8 | 177.3 | 175.3 | |
| | Pier 2 | 185 | 188.0 | 177.3 | 175.3 | |
| | Pier 3 | 185.5 | 188.6 | 179.0 | 177.0 | |
| | East Abutment | 185.5 | 188.7 | 180.5 | 179.0 | |
| South | West Abutment | 185.6 | 187.2 | 179.2 | 177.7 | |
| | Pier 1 | 185.5 | 187.6 | 177.6 | 174.6 | |
| | Pier 2 | 185.6 | 188.4 | 177.6 | 175.6 | |
| | Pier 3 | 185.6 | 188.6 | 179.3 | 177.3 | |
| | East Abutment | 185.8 | 188.7 | 181.1 | 179.6 | |

1 – Approximate – estimated from neighbouring borehole logs.

## 5.2 Geotechnical Design Criteria and Considerations

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

Working Stress Design (WS Method) was employed for global stability of the earthworks and soil mass containing earth retaining structures. The stability of the soil mass containing the bridge end abutments was checked for all potential surfaces of sliding.

As per the design criteria revision approved in December 2011 and CFEM guidelines, Working Stress method was also employed for the external design of the RSS walls.

For the purposes of the geotechnical analyses it is considered that Bridge B-11 construction will involve the following main earthwork, design elements and loading stages:

- Bulk excavations for the depressed corridor of Highway 401 (approximate elevations 180 to 177.5);
- Temporary excavation at east and west abutments, and bridge piers to elevations down to 175 m approximately;
- Installation of piles (HP310×110) for all bridge supports;
- Construction of the bridge abutments and piers;
- Backfilling of piers;
- Installation of Reinforced Granular Mats (RGM) foundation under the RSS walls between the north and south approaches behind the east and west abutments;
- Construction of the RSS walls and associated permanent sub-drainage works, and approved backfill behind the RSS structure typically up to the seat levels;
- Construction of bridge deck;
- Completion of approach fills including conventional fill, Ultra Light Weight Fill (LWF), and grading; and
- Completion of pavements over Highway 3 and Highway 401.

## 5.3 Design Soil Properties

The design soil properties for the silty clay to clayey silt deposit were interpreted from the Nilcon vane test, CPT and DMT profiles and the laboratory test results. The undrained shear strength, $S_u$ profiles were estimated from the DMT and CPT based on the calibrations described in Section 3.5. The $S_u$ profiles inferred from the investigation data at the Bridge B-11 are shown in Figure 3.3. Selected typical design values obtained from the profiles are summarized in Table 5-2. Effective cohesion (where present) in the upper clay crust and transition zone has been neglected due to long term weathering, moisture ingress, and fissuring effects.

### Table 5-2: Interpreted Design Strength Properties of Silty Clay Stratum

| Clay Substratum | Elevation, m | Undrained Shear Strength, $(S_u)$, kPa | Effective Stress Parameters | Pre-consolidation Pressure $(\sigma'_p)$, kPa | OCR |
|---|---|---|---|---|---|
| Clay Crust | > 179 | 75 (*) | Cohesion, c' = 0<br>Friction Angle, ǿ = 30° | 550 | 7 |
| Transition | 179 to 175 | 75 to 55 | | 550 to 340 | 4.5 |
| Upper Silty Clay-1 | 175 to 168 | 55 to 51 | | 340 to 305 | 2 |
| Upper Silty Clay-2 | 168 to 163 | 50 to 58 | | 280 to 315 | 1.1 |
| Lower Clayey Silt-1 | 163 to 156 | 58 to 100 | | 315 to 500 | 1.5 |

(*) Applicable for global stability verifications

The design values of the coefficient of horizontal permeability ($k_h$) and the hydraulic conductivity anisotropy ratio ($A=k_h/k_v$) used for the analysis of stress and deformation response of the soils are provided in Table 5-3. These are considered to be within range of precision of the measurements (Figure 4.5).

### Table 5-3: Other Interpreted Design Parameters for Clay

| Clay Substratum | Horizontal Permeability, m/days | Anisotropy ratio, $k_h/k_v$ | Initial Void Ratio, $e_0$ |
|---|---|---|---|
| Clay Crust | $6.9 \times 10^{-7}$ | 1 | 0.37 |
| Transition | $3.9 \times 10^{-7}$ | 2 | 0.42 |
| Upper Silty Clay | $1.1 \times 10^{-7}$ | 2 | 0.47 to 0.63 |
| Lower Clayey Silt | | | 0.5 |

For design purposes the initial groundwater level in the overburden was considered at elevation 185.

The following material properties were assumed for the fill materials (Table 5-4 and Table 5-5).

### Table 5-4: Assumed Proprietary Product Properties

| Material | Unit weight, $kN/m^3$ | Limit Equilibrium Analyses | | Stress Deformation Analyses | |
|---|---|---|---|---|---|
| | | Friction Angle, ǿ° | Apparent Cohesion, kPa | Modulus of Elasticity, E, MPa | Poisson's ratio, ☐ |
| RSS | 21 | 35 | 50 | 40 | 0.35 |
| RGM | 21 | 35 | 0 | 60 | 0.35 |

### Table 5-5: Assumed Backfill Material Properties

| Backfill Material | Unit weight, $kN/m^3$ | Undrained Shear Strength, kPa | Drained Strength Parameters | Modulus of Elasticity, E, MPa | Drained Poisson's ratio, $\mu$ |
|---|---|---|---|---|---|
| Compacted Clay Fill | 21 | 50 | =0 kPa, ǿ=30° | 20 | 0.35 |
| Light Weight Fill (LWF) | 12 | N/A | =0 kPa, ǿ=35° | 30 | 0.35 |
| Roadway Backfill | 22 | N/A | =0 kPa, ǿ=35° | 50 | 0.35 |

## 5.4 Excavations and Temporary Cut Slopes

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

Excavations are expected to encounter surficial fills and topsoil and will be extended up to 10 m below existing grade (elevation 185 and 185.8) to about elevation 175.2 for North Pier No. 1. Excavations for the West and East abutments will extend to about elevation 179.2 and 179.0 respectively.

Basal hydrostatic uplift was calculated based on the highest measured water level in the rock (elevation 178.1), anticipated deepest excavation depth (Pier#1 underside of pile cap at elevation 175.3), and a silty clay to clayey silt layer thickness of 18.1 m (Borehole BH-109) below the deepest excavation. The factor of safety against hydrostatic uplift was 1.7 based on the weight only of the clay cap.

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

## 5.5 Stability of Abutments

### 5.5.1 Global Stability

Slope stability analyses (Limit Equilibrium) were carried out using SLOPE/W Version 2007 and the Morgenstern-Price method of analysis. As the maximum excavation occurs at North Pier No. 1 (10.0 m), this section has been analyzed as the critical section for the piers. Other sections analyzed and considered to be representative for global stability verification were the southwest approachway along transversal direction just behind the concrete abutment, southwest approachway and abutment in longitudinal direction, and northeast approachway in transverse direction. The analyzed sections are shown of Figure E-1.

Sections across retaining structures MSEW-34R and HRW 24L are being addressed in the reports for these structures.

Each section was analyzed for (a) the short-term (temporary – undrained conditions) condition involving completed approachway without the final Highway 401 pavement and including presence of a tension crack; (b) the short-term – end of construction loading condition; (c) and the long-term (drained properties) considering fully completed works . The results of the analyses are summarized in Table 5-6. The limit equilibrium stability analyses are presented on the referenced figures in Appendix E.

**Table 5-6: Summary of Global Abutment Stability**

| Structure | Loading Condition | Minimum Calculated Factor of Safety[1] | Figure |
|---|---|---|---|
| North Pier No. 1 – Section A-A | Short – Term - Undrained | 1.43(1.28) | E-2 |
| | End of Construction - Undrained | 1.62 (1.42) | E-3 |
| | Long-Term – Steady State | 1.72 (1.64) | E-4 |
| Southwest Abutment – Longitudinal - Section B-B | Short – Term - Undrained | 1.60 (1.50) | E-5 |
| | End of Construction - Undrained | 1.72 (1.61) | E-6 |
| | Long-Term – Steady State | 2.29 (2.14) | E-7 |
| Southwest Abutment – Transverse – Section C-C | Short – Term - Undrained | 1.39 (1.25) | E-8 |
| | End of Construction - Undrained | 1.61 (1.45) | E-9 |
| | Long-Term – Steady State | 1.82 (1.73) | E-10 |
| Northeast Abutment – Transverse – Section D-D | Short – Term - Undrained | 1.44 (1.25) | E-11 |
| | End of Construction - Undrained | 1.65 (1.41) | E-12 |
| | Long-Term – Steady State | 1.96 (1.79) | E-13 |

Notes: 1) Values in brackets present optimized values.

### 5.5.2 RSS Wall External Stability

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

For the east and west abutments, an RGM base along with select use of light weight fill (LWF) (12 kN/m$^3$) material were required to satisfy design criteria for these failure modes. Conceptual wall configurations established to meet the external stability requirements are shown in Appendix I on Figure I.1 and the dimensions are summarized in Table 5-7.

The following RSS wall dimensions were determined to meet the above conditions:

### Table 5-7: Summary of RSS Wall Configuration (4)

| RSS Wall | Preliminary RSS Structure Size, Width x Height (m) Total Height, m (1) (3) | RGM Size, Thickness x Width, m (2) | LWF, thickness, m |
|---|---|---|---|
| West RSS Wall<br>Section A | 5.0 x 5.0 | 1 x 7.0 | |
| Section B | 6.0 x 6.5 | 1.5 x 9 | None |
| Section C1 | 8.0 x 7.7 | 1.5 x 11 | 2 |
| Section C | 8.0 x 8.0 | 1.5 x 11 | 4 |
| East RSS Wall<br>Section A | 8.0 x 8.1 | 1.5 x 11 | 4 |
| Section A1 | 8.0 x 8.3 | 1.5 x 11 | 2.5 |
| Section B1 | 6.5 x 7.4 | 1.5 x 9.5 | None |
| Section C | 6 x 6.2 | 1.5 x 9 | None |

(1) Height measured from anticipated base of RSS wall to top of RSS wall on Highway 3.
(2) The RSS supplier may require wider structures to meet the internal design requirement.
(3) Wall height varies – maximum wall height given.

(4) Final design configuration and dimensions may vary from the listed values subject to detailed design

The design of the RGM may be based on a subgrade net ultimate bearing capacity of 300 kPa for the undrained (short-term) case and the maximum unfactored top loads presented in Table 5-8.

### Table 5-8: Summary of Average Loads on RGM

| RSS Wall | Assumed RSS grade Elevation (m) | Avg. Unfactored Top Loads s (kPa) |
|---|---|---|
| West<br>Section A | 181.0 | 160 |
| Section B | 179.8 | 200 |
| Section C1 | 178.6 | 200 |
| Section C | 178.6 | 195 |
| East Section A | 180.1 | 200 |
| Section A1 | 180.1 | 200 |
| Section B | 181.3 | 215 |
| Section C | 182.5 | 190 |

The above resistances and loads are applicable in conjunction with the RSS wall configurations described above. Based on the above, a maximum tensile unfactored load of 65 kN/m of RGM along the toe of the RSS facing was calculated. For cost estimates, this tensile load can be accommodated by 5 layers of UX1000HS, or equivalent.

Backfilling along highway must be completed prior to RSS completion.

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

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

Where:

A' ($m^2$) = effective contact area of the base;
c' (kPa) = cohesion/adhesion at sliding interface;
$\delta$ (°) = friction angle at sliding interface;
V (kN) = vertical force (kN); and
$H_f$ (kN) = design horizontal load.

Allowance for buoyancy should be made, where applicable.

The following soil properties (Table 5-9) can be used in the design at the interfaces between the RSS, RGM and silty clay subgrade:

**Table 5-9: Soil Properties for use at Sliding**

| **Interface** | **Undrained (Short-Term)** | | **Drained (Long-Term)** | |
|---|---|---|---|---|
| | **δ, degrees** | **c, kPa** | **δ′, degrees** | **c', kPa** |
| RSS to RGM | 30 | 0 | 30 | 0 |
| RGM to Silty Clay | 0 | 75 | | |

### 5.5.3 Stress Deformation Analysis Models

Finite element stress-deformation analyses (SDA) were carried out using the SIGMA/W software Version 2007. The main focus of the SDA was to assess the ground deformations (heave, settlements, lateral movement) due to excavations and backfilling. The output from the analysis was also used to evaluate the potential skin friction on the piles due to transient settlement/heave of the surrounding soil.

SDA has been carried out for two sections as shown on Figure F.1: 1) a longitudinal section along the south bridge that captures the entire bridge configuration and is presented in plan view on Figure F.2, and 2) a transverse section through South Pier Number 3 and is presented in plan view on Figure F.3. The SDA model was done using slightly different elevations that listed in the Table 5-1 based on the pile cap elevations provided by HMM on October 23, 2011. The effect of the changes in elevations are considered minimal and within the accuracy of deformation estimates.

The models are based on the following construction sequence:

(a) Generation of the initial (in-situ) stress condition for level ground assuming an average bulk soil unit weight of 21 kN/m$^3$, Poisson' Ratio of 0.334 and an at-rest earth pressure coefficient $K_0$ of 0.75 (based on published information, ref R-42, and local experience based on DMT) for the soil deposit;

(b) Temporary excavation to at Highway 401 subgrade level (elevation 179.5 to 178), followed by H-pile installation at the bridge abutments and piers;

(c) Construction of the reinforced concrete pile caps, abutment/retaining walls and piers, and completion of the fill behind abutment and retaining walls; and

(d) Completion of the pavement structure for Highway 401, and dissipation of excess pore pressure.

The stratigraphy and selection of the soil properties was based on the design soil properties discussed in Section 5.3.

The SDA was carried out for drained (effective stress long-term) soil behaviour. The negative pore pressure generation was simulated during fill removal and redistribution of pore pressures allowed versus time based on diffusion analysis and the defined permeability and volume change parameters for the soil. This is termed as coupled hydro-mechanical analysis. Modified Cam-Clay constitutive models were considered for the unweathered clayey silt below the transition zone, and the elastic-plastic Mohr-Coulomb model for the remaining soil layers (Crust, Transition, and Backfill). The drained Modified Cam-Clay model required the following inputs: critical state friction angle, pre-consolidation pressure, initial void ratio, primary compression and unloading compression indices. The latter was selected as the rebound compression index given in Table 4-2. The drained elastic-plastic Mohr-Coulomb model required as input the peak friction angle, the drained initial Young's modulus, and used an effective Poisson's ratio.

### 5.5.4 SLS Performance

Ground deformations (i.e., heave/settlement, horizontal displacement, etc.) and stress distributions were estimated for the following elapsed times (days) [years]:

| | |
|---|---|
| 0 | In Situ condition |
| 1 – 90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 [0.5 to 27.9] | Dissipation |
| 10180 [27.9] | Long term, after complete pore pressure dissipation |

The Serviceability Limit States (SLS) performance was assessed on the basis of the SDA model described above in Section 5.5.3.

Contours of computed cumulative vertical displacement (heave and settlements) at the end of construction for the longitudinal section are shown in Figure F-4. The cumulative computed deformation in the long-term condition is shown in Figure F-5. The calculated cumulative vertical displacements for the transverse section for the end of construction and long-term condition are presented in Figure F-6 and Figure F-7.

The SIGMA/W model used for stress-deformation analysis is a plane strain model and does not capture the 3-D excavation effects involved around the bridge. Based on 3-D elastic stress distribution theory, it is estimated that the actual 3-D effects may reduce the calculated settlements/heaves to about 2/3 of the estimated value at the central pier and near the abutment.

Charts of calculated heave at the ground surface progressing from South Pier No 3 to South Pier No 1 along Highway 401 are shown in Figure F-8. Plots of ground settlements versus elevation at the abutment approach way locations for various construction stages are shown in Figure F-9. Table 5-10 and Table 5-11 summarizes the representative cumulative deformation response obtained for the south bridge B-11 along the longitudinal sections. The results are anticipated to be similar for the North Bridge.

**Table 5-10: Summary of Calculated Cumulative Settlements Southeast Abutment**

| Loading Stage | Settlements at Various Distances from the Bridge Abutment Along the Highway 3 Approachway (mm) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 0 m | 10 m | 20 m | 30 m | 50 m | 75 m | >100 m |
| End of Bridge Construction | (+) 39 | (+) 44 | (+) 34 | (+) 28 | (+) 25 | (+) 25 | <(+) 25 |
| Long-term | (+) 57 | (+) 65 | (+) 59 | (+) 57 | (+) 56 | (+) 57 | < (+)57 |

Notes: (+) indicates settlement, (-v) indicates heave.

**Table 5-11: Summary of Calculated Cumulative Settlements Southwest Abutment**

| Loading Stage | Settlements at Various Distances from the Bridge Abutment Along the Highway 3 Approachway (mm) | | | | | | |
|---|---|---|---|---|---|---|---|
| | 0 m | 10 m | 20 m | 30 m | 50 m | 75 m | >100 m |
| End of Bridge Construction | (+) 27 | (+) 83 | (+) 80 | (+) 81 | < (+) 45 | < (+) 45 | < (+) 45 |
| Long-term | (+) 49 | (+) 88 | (+) 112 | (+) 117 | < (+) 75 | < (+) 75 | < (+) 75 |

Note: Distances measured along center line of the approachway

It should be noted that the ground settlements at the end of construction are actually corrected during construction of the backfill and pavement. The anticipated post-construction deformations are represented by the difference between the values reported at the end of construction and those reported for the long-term.

The movements presented in the Tables are based on the 2-D model only. The rigidity of the proposed RSS and retaining walls will have a minor influence on the amount of movement.

Approximately 90% of the ground movements generated by the construction loads are anticipated to occur within 4 years following completion of construction. Figure F-10 presents heave/settlement rates for the southeast and southwest abutments, and along Highway 401 East and West Bound Lanes. Highway 401 may anticipate close to 30 mm of post-construction heave while approximately 10 mm of post-construction settlement may be expected at the abutments.

Contours of stabilised long-term porewater pressures are presented in Figures F-11 and F-12.

Figure F-13, F-14, F-15 and F-16 shows the net lateral ground movements and vertical soil movements versus elevation at various times acting on the South Pier 3 piles. Figures F-17 and F-18 show the net lateral and vertical ground movements at the Southwest Abutment. It should be noted that “Interior” piles refer to those within the abutment and “Exterior” piles refer to piles outside of the abutment fill zone. These lateral soil movements and the effect of bending is discussed further in Section 5.6.3.

All ground movement and deformations discussed above were estimated 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/or refined based on performance monitoring in the field.

The settlements discussed above do not include deformations caused by seasonal temperature and moisture variations. Also, they do not include the effects of the long-term compression of the backfill materials which should be minor. Stringent compaction control should be carried out to minimize these risks.

## 5.6 Pile Foundations

### 5.6.1 ULS and SLS Resistance to Axial Loads

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

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

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

Based on the available borehole data at this structure, the bedrock surface varies between elevations 147 and 150, where the tips of piles are anticipated to be set.

Based on the boreholes at this site, very stiff clayey silt changing to dense granular till were encountered below approximate elevation 160. Indications of the presence of cobbles and boulders were noted in some of the boreholes. Accordingly, "hard driving" conditions may be encountered at this site. In cases where some of the piles cannot be driven to bedrock due to presence of dense till lying immediately above the bedrock and a perceived risk of damaging the piles by overdriving is apparent, consideration should be given to supplementing the field testing to prove the actual mobilized resistance. If lower mobilized pile resistances are proven, options based on the most economical approaches may be considered (e.g., changes to the driving method and equipment, or addition of more piles).

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

The following general pile installation recommendations should be considered:

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

### 5.6.2 ULS and SLS Resistance to Lateral Loads

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

The SLS geotechnical resistance to lateral loads is dependent on the acceptable levels of the lateral pile deflections under the design loads and should be obtained on the basis of field load tests. In the absence of field tests, the preliminary design can be based on a conventional SLS resistance of 65 kN along the strong axis and 45 kN along the weak axis of the HP310×110. This conventional SLS resistance represents the lateral shear force applied on a free-head pile at the level of the ground surface that causes a lateral deflection of 10 mm measured at the ground surface or 50% of ULS (whichever gives the lesser lateral load).

The ULS lateral resistance is defined as the lateral force applied to the pile shaft causing unstabilised pile displacements due to soil failure or pile structural failure. In the absence of field tests, the ULS lateral resistance based on pile structural failure can be assumed to be 185 kN and 85 kN along the strong axis and weak axis, respectively. The above estimates were based on a pile model assumed embedded within firm to stiff silty clay. The estimates were carried out 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 Reese “Stiff-Clay without free water” model in conjunction with the soil parameters described in Table 5-13.

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 the induced maximum bending moment.

It should be noted that during driving significant soil disturbance and damage occur around the pile shaft forming sizeable gaps between the pile and the surrounding soils. These gaps cause severe 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 should be undertaken (typically, the voids are grouted using non-shrink fills).

The design of piles to lateral loads may be carried out by one of the methods described below.

*Coefficient of Horizontal Subgrade Reaction Method:*

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

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

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

Where:

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

$n_h$ (MPa/m) = Soil coefficient

$S_u$ (MPa) = Undrained shear strength

z (m) = Depth below finished grade

d (m) = Pile diameter/width

The recommended ranges of soil parameters are presented in Table 5-12-.

**Table 5-12: Soil Parameters for Lateral Load Resistance Calculations**

| Soils Around the Piles | Elevation Range | $n_h$, MPa/m | Undrained Shear Strength, $S_u$ (kPa) |
|---|---|---|---|
| Native Silty Clay Crust | Above 177 | - | 75 |
| Native Transition Clay | 177 to 175 | - | Decreases linearly with depth from 75 to 55 |
| Upper Silty Clay - 1 | 175 to 166 | - | Decreases linearly with depth from 55 to 50 |
| Upper Silty Clay – 2 | 166 to 163 | - | Increase linearly with depth from 50 to 58 |
| Native Lower Clayey Silt - 1 | 163 to 161 | - | Increases linearly with depth from 58 to 100 |
| Lower Clayey Silt - 2 | 161 | - | 100 |
| Lower Granular Deposit | Below 151 | 10 to 15 | - |

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 (CFEM) (ref.6)

Significant lateral loads should be resisted fully or partially by the use of battered piles. Batter piles are considered to be more effective in resisting horizontal loads, as a part of lateral load is converted into axial loads. For ease of construction and to limit the loss of hammer energy delivered to the pile during driving, batters are usually limited to no steeper than 1H:5V. However, larger batter may be considered, but not greater than 1H:3V.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, as indicated in Table 5-13. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed this table.

### Table 5-13: Lateral Resistance Reduction Factor for Pile Groups for 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).

The pile spacing in the direction of loading under the abutments and piers is between 10 and 5 times pile diameter, which result in a reduction factor of 1 (i.e. no reduction) and 0.55 reduction in the lateral resistance, respectively.

*Alternative Nonlinear 'p-y' Curve Method:*

The p-y curve represents the total lateral soil reaction pressure 'p' (kPa) to the pile lateral deflection 'y' (m) relative to the surrounding soil mass at a particular section of the pile shaft in contact with the surrounding soils. 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 (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. 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 5-14. "Stiff clay" p-y curves as given in the LPile manual should be developed appropriate for either static or cyclic loading conditions in absence of free water. For p-y curves below the water table, submerged unit weights in the soil mass shall be used.

### Table 5-14: Soil Parameters for p-y Curve Calculation

| Soils Around the Piles | Elevation Range | Design Bulk Unit Weight (kN/m$^3$) | Undrained Shear Strength, $S_u$ (kPa) | $\varepsilon_{50}$ |
|---|---|---|---|---|
| Native Silty Clay Crust | Above 177 | 22 | 75 | 0.005 |
| Native Transition Clay | 177 to 175 | 21.5 | Decreases linearly with depth from 75 to 55 | 0.007 |
| Upper Silty Clay - 1 | 175 to 166 | 21 | Decreases linearly with depth from 55 to 50 | 0.010 |
| Upper Silty Clay – 2 | 166 to 163 | 20.5 | Increase linearly with depth from 50 to 58 | 0.010 |
| Lower Clayey Silt - 1 | 163 to 161 | 22 | Increases linearly with depth from 58 to 100 | 0.007 |
| Lower Clayey Silt - 2 | 161 | 22 | 100 | 0.005 |

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

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

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

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

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

where :

$\beta_{ki}$ = the influence factor of pile ‘k’ in the group on pile ‘i’, with $k \neq i$, and is calculated with one of the following expressions in Table 5-15- depending on the relative position of pile ‘k’ in the group with respect to pile ‘i’.

### Table 5-15: Lateral Load Capacity Reduction Factor For Pile Groups for p-y Method

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

The modifier factor applies to the “p” values.

The space between the piles under the abutments varies from 4250 mm (Drawing 285380-03-061-SWIP-1106) to 1100 mm (Drawing 285380-03-061-WIP-1104. At the piers a closer spacing of approximately 1200 mm is anticipated. Group reduction factors will apply for lateral pile loadings.

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.

### 5.6.3 Soil Pile Interaction Assessment

*Downdrag Loads (Negative Skin Friction – NSF):*

Potential for downdrag loads on piles was considered in conjunction with the anticipated ground movements (rebound and settlements) that are assumed to occur during and following excavation of the overburden of up to 9 m to accommodate the future depressed highways, followed by partial re-placement of fills to construct the bridge abutments.

Soil stress-deformation analyses described in Section 5.5.3 were conducted using the SIGMA/W software. The net estimated ground vertical movement (settlement/heave) after excavation in the vicinity of the pile shaft at representative stages: after completion of the backfill and in long-term (dissipation), and associated ground movements are presented in Figures F-12 to F-17.

The analyses indicate that under and near abutments ground settlements are expected to occur around the piles located in the immediate vicinity of the approachway fill. These settlements are associated with the backfilling and the long-term post-construction pore water pressure dissipation. Table 5-16 presents the resulting analyses:

**Table 5-16: Calculated Downdrag at South Pier 3 and Southwest Abutment Piles**

| **Location** | **Unfactored Transient Downdrag (kN)** | **Unfactored Long Term Downdrag (kN)** | **Unfactored Design Dead Load (kN) (*)** | **Foundation areas with calculated similar downdrag** |
|---|---|---|---|---|
| South Pier 3 – Interior Battered Piles | < 200 | 700 | N/A. | North Pier 1 – Interior Battered Piles |
| South Pier 3 – Exterior Battered Piles | <200 | <200 | N/A | North Pier 1 - Exterior Battered Piles |
| South Pier 3 - Vertical | <200 | 760 | 920 | North Pier 1 – Vertical Piles |
| SW Abutment – Interior Battered | 740 | 775 | 735 | All Other Abutments - Interior Battered Piles |
| SW Abutment – Exterior Battered | < 200 | 250 | 735 | All Other Abutments – Exterior Battered Piles South Pier 1 & North Pier 3 Piles. |

(*) Provided by HMM, Battered pile load has been calculated to act along pile axis.

The larger downdrag was estimated for the piles battered at 3V:1H toward the approachway embankment. A reduction of the downdrag may be obtained by a reduction of the batter.

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).

All the pier piles not mentioned in the above table are not subjected to downdrag.

*Shaft Bending:*

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

- The ground lateral movement along the pile shaft anticipated to occur after the installation of the piles was estimated using the stress-deformation analysis described in Section 5.5.3.
- 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,R-15). The “p-y” curves were generated using the Reese method (for Stiff Clay without free water) described in the Technical manual for LPile, using undrained shear strength of 50 kPa and submerged unit weight of 10 kN/m$^3$.

Based on the above approach and anticipated lateral ground displacement, the estimated maximum unfactored bending moment in the shaft was 55 kN-m for the strong axis pile loadings for a free head condition and 135 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 (65 kN) was 70 kN-m for the strong axis pile loadings. Accordingly, a potential combination of the maximum bending stresses from pile head shear force and ground displacement field would lead to a maximum bending moment of 125 kN-m for the free-head condition and 205 kN-m for a fixed-head condition. 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 himself with these findings.

### 5.6.4 Earth Pressures on Abutment Walls

Adequate width/thickness of non-frost susceptible and free draining granular fill should be placed behind the abutment and wing walls to prevent excessive deformation and damage (ref.R-9).

The granular backfill should be compacted in maximum 200 mm thick loose lifts in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to ensure positive drainage of the backfill. Other aspects of the abutment backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 and 3190.100.

Heavy compaction equipment should not be used adjacent to the walls of the structure, where the backfill should be placed in maximum 100 mm thick loose lifts and compacted with small compactors. Effects of backfill compaction activities should be simulated as live load over and above the static lateral earth pressure for structural design in accordance with the CHBDC.

For retained backfill that is placed and compacted in layers, the lateral force caused by compaction should be considered. In the absence of detailed analysis, the total lateral pressure due to soil weight and compactive effort should not be less than 12 kPa in any section of the wall.

Earth pressures on abutment walls may be calculated on the basis of the following parameters:

**Table 5-17: Soil Parameters for Earth Pressure Calculations**

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

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

Group III soil backfilled can be used as general backfill within approved areas only.

## 5.7 Flood Events

Based on the estimated elevation of 177.5 for the 100year flooding event and 177.9 for the regional storm event from Pump Station 6 in the vicinity of Bridge B-11, flooding of the roadway in this bridge will not occur during a 100-year storm event. As such, submergence of the LWF material is not anticipated to occur in the area of Bridge B-11.

# 6 Construction Requirements

## 6.1 General Construction Requirements

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

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

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

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

## 6.2 Backfilling

Behind the concrete abutment and wing walls, non-frost susceptible and free draining Granular fill should be placed in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC). Alternatively, a synthetic insulation with drainage blanket and site generated clay fill behind the walls may be considered.

It is understood that the native silty clay to clayey silt from the crust zone is being considered for backfill material, where appropriate. The clay crust material is considered suitable for re-use as engineered fill but may require moisture conditioning. Well graded, 75 mm minus sand and gravel (Granular B Type 1 or approved equivalent) can also be considered for use as engineered fill since such materials are less sensitive to moisture content increases. The fill materials should not contain deleterious material such as construction debris or organics. Geotechnical engineering input is required in order to assess the suitability of fill materials for the use intended.

The fill should be placed in loose lifts not exceeding 200 mm in accordance with SP 105S10. Fill in the vicinity of the structural walls should be placed in 100 mm thick loose lifts. 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 and 3190.100.

Backfill shall be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD) under wing wall footings or 98% SPMDD when used as backfill behind abutment retaining walls or wing walls. Heavy compaction equipment should not be employed near structural walls. Fill should be placed at moisture contents within ±2 percent of the Optimum Moisture Content. Lift thicknesses can be adjusted once the compaction equipment has been selected.

Qualified geotechnical personnel should monitor the placement and quality of the fill soils. Fill placement and compaction should be monitored by field density testing at regular frequencies. The recommended minimum test frequency should be one field density test per 500 $m^2$ for each lift of fill.

Heavy compaction equipment should not be used immediately adjacent to the walls of the structure. 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 CAN/CSA-S6-06.

Fill placement and compaction during the winter months is not recommended since the required degree of compaction cannot be attained using frozen clay or granular fills.

## 6.3 Dewatering

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

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

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

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

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

## 6.4 Instrumentation and Monitoring during Construction

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

Details of the permanent works instrumentation and monitoring plan, recommendations for alert levels and contingencies are provided in the document 285380-04-118-0001.

The Contractor is responsible for installation and maintenance of all 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.

## 6.5 Corrosion Potential

Analytical testing was carried out on samples of the silt and clay obtained in boreholes BH B11-1 to B11-7. Table 6-1 provides the results of various analyses carried out on the soil samples to assess the potential for corrosion on concrete.

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

| Location of Soil Samples | Elevation of Soil Sample | pH | Redox Potential, mV | Resistivity, ohm.cm | Sulphide, mg/kg | Sulphate, mg/kg |
|---|---|---|---|---|---|---|
| Borehole BH B11-1 (Sample 9) | 177.5 | 7.86 | 121 | 4170 | <0.20 | 218 |
| Borehole BH B11-2 (Sample 25) | 153.4 | 8.05 | 115 | 1710 | <0.20 | 745 |
| Borehole BH B11-3 (Sample 10) | 176 | 7.83 | 136 | 3680 | <0.20 | 106 |
| Borehole B11-4 (Sample 27) | 149.8 | 7.86 | 154 | 1790 | <0.20 | 608 |
| Borehole B11-5 (Sample 29) | 150.0 | 7.83 | 119 | 5680 | <0.20 | 52 |
| Borehole B11-6 (Sample 10) | 177.9 | 7.94 | 122 | 2780 | <0.20 | 549 |
| Borehole B11-7 (Sample 10) | 176.0 | 7.80 | 98.0 | 4080 | <0.20 | 120 |

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

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

A corrosion specialist should review the test result and provide recommendations to address corrosion concerns.

## 6.6 Construction Quality Control

To ensure that construction is carried out in a manner consistent with the intent of the recommendations set forth in this report, a program of geotechnical inspection and testing should be developed and implemented throughout the construction phase. In addition, related laboratory testing should be carried out in conjunction with the field work 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 structure 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, and CPT probing. The boundaries typically represent a transition from one soil type to another and are not intended to define exact planes of geological change. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. Thus, unsuitable foundation soils may be encountered at the foundation grade requiring extra sub-excavations, subgrade improvement, and/or changes to the design. It is important that the AMEC geotechnical design engineer be involved during construction throughout the WEP project site to confirm that the subsurface conditions do not deviate materially from those encountered in test holes, and that any material deviations, if encountered, do not adversely affect the geotechnical design.

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

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

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

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

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

# 8 Closure

The design for Bridge B-11 was developed by Mr. Tommi Leinala, P.Eng. and checked by Dr. Dan Dimitriu, P.Eng. (Lead designer). The project was executed under the technical direction of Dr. Narendra S. Verma, P.Eng., who also reviewed the report. Mr. Matt Oldewening, P.Eng. managed the geotechnical investigation and Mr. Brian Lapos, P.Eng. is the project manager.

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

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

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

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(Designated MTO RAQS Contact)

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# Drawings

| | | | |
|---|---|---|---|
| **Project:** | Windsor-Essex Parkway | **Date:** | April/2012 |
| **Document:** | Geotechnical Investigation and Design Report – Bridge B-11<br>(Highway 3 Underpass East of Montgomery Drive) | **Rev:** | 0 |
| **Doc No.:** | 285380-04-119-0027 (Geocres No. 40J3-9) | **Page No.:** | Drawings |

# METRIC

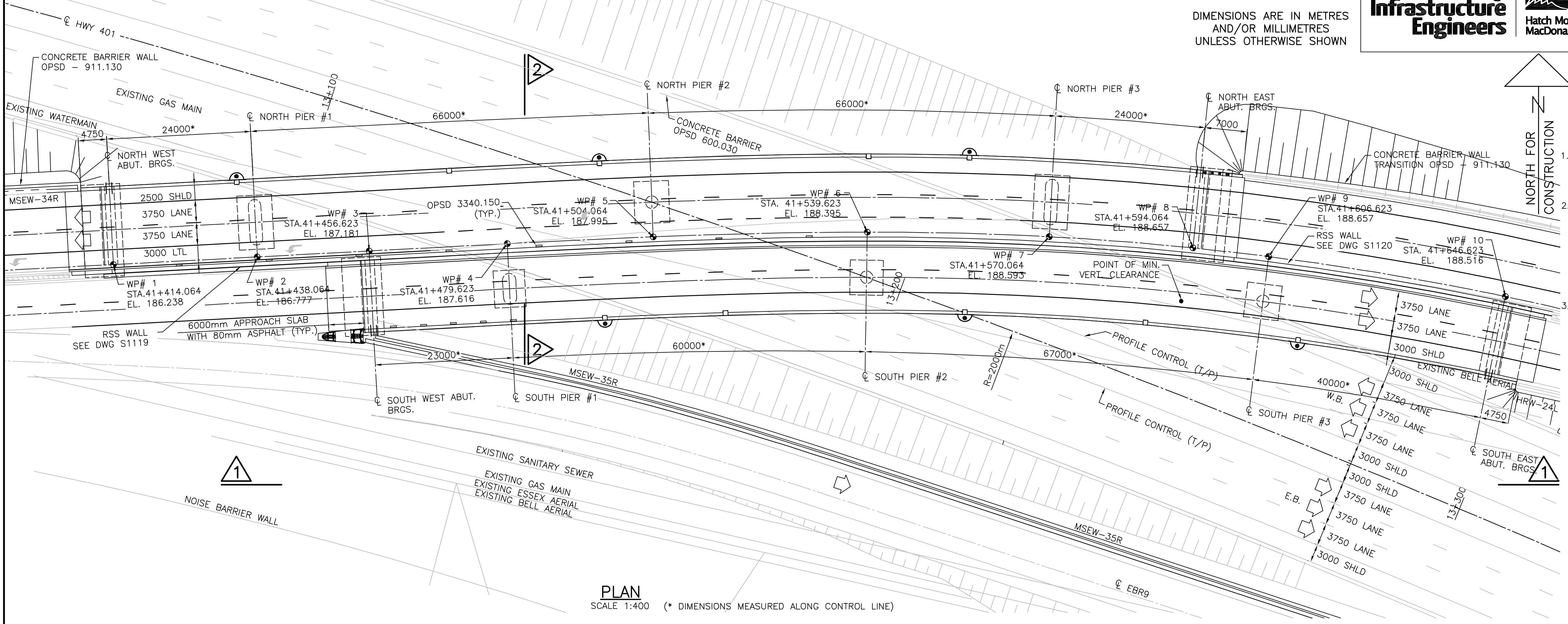
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

**Parkway Infrastructure Engineers**
Hatch Mott MacDonald

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

NEW CONSTRUCTION
BRIDGE B–11
HWY 3 UNDERPASS NEAR MONTGOMERY
GENERAL ARRANGEMENT

SHEET
S1101
Phase 1
90% Sub



PLAN
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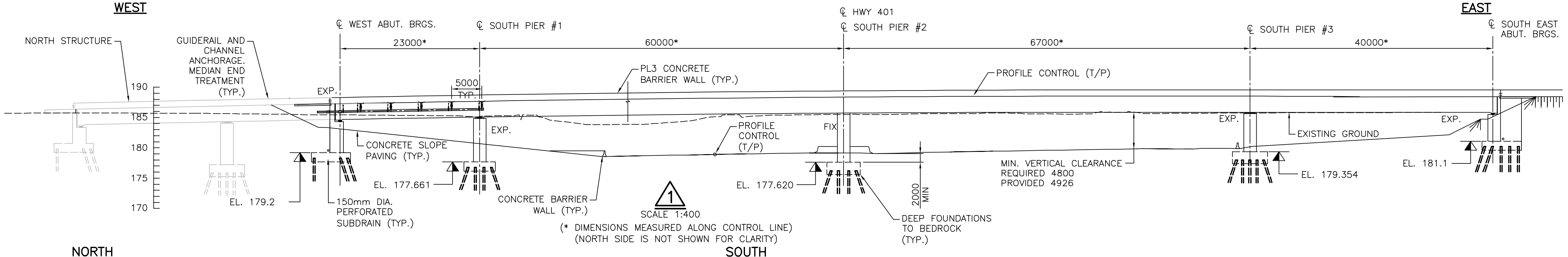

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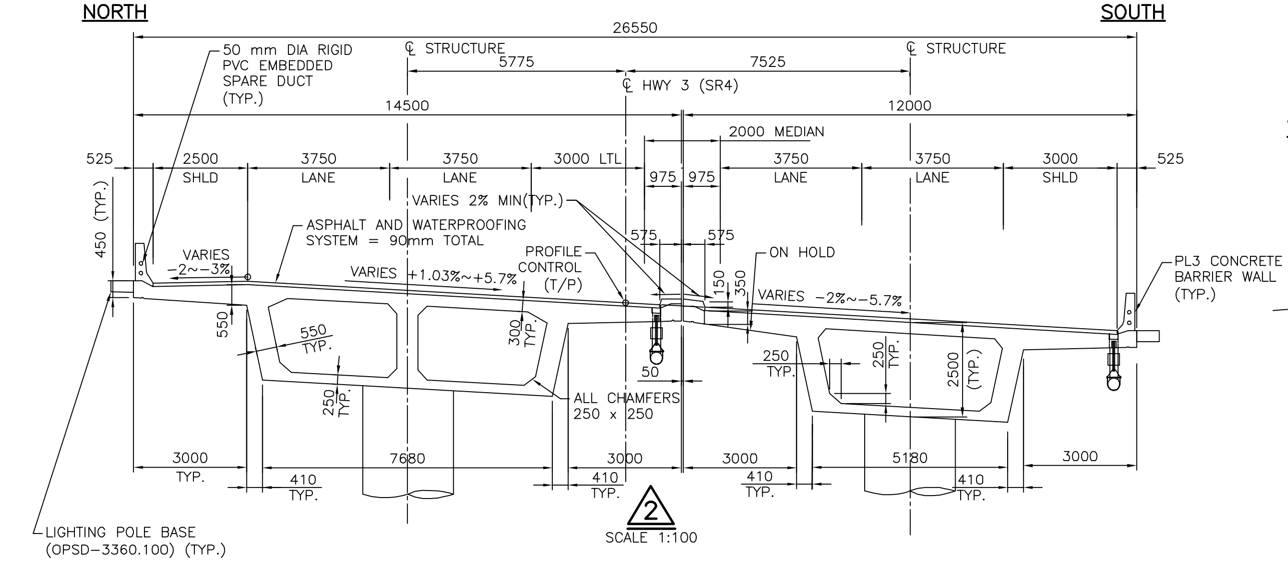

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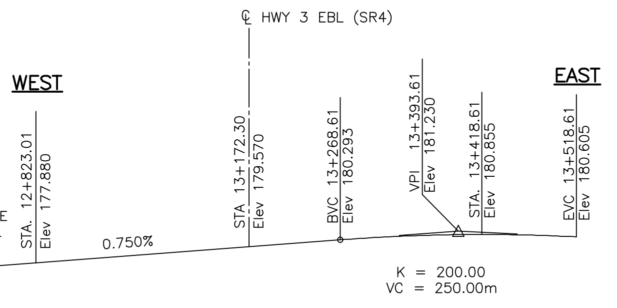
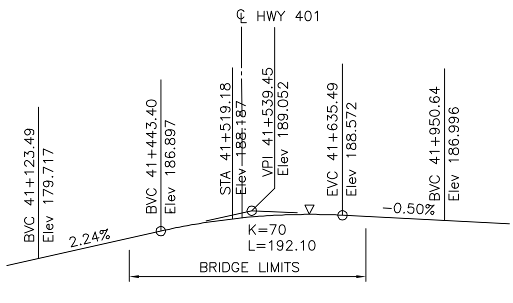


PROFILE OF HWY 401
N.T.S.



PROFILE OF HWY3 (SR4)
N.T.S.

## GENERAL NOTES:

1. CLASS OF CONCRETE:
   - DECK....................50 MPa
   - MASS CONCRETE....................20 MPa
   - REMAINDER....................30 MPa
2. CLEAR COVER TO REINFORCING STEEL:
   - FOOTING....................100 ± 25
   - DECK : TOP SLAB TOP....................70 ± 20
     TOP SLAB BOT....................50 ± 10
     BOTTOM SLAB TOP....................40 ± 10
     BOTTOM SLAB BOT....................60 ± 10
     WEBS....................60 ± 10
   - REMAINDER....................70 ± 20
     UNLESS OTHERWISE NOTED.
3. REINFORCING STEEL:
   - REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
   - STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 OR TYPE XM–28 AND HAVE MINIMUM YIELD STRENGTH OF 500MPa, UNLESS OTHERWISE SPECIFIED.
   - BAR MARKS WITH PREFIX 'C' DENOTE COATED BARS.
   - BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
   - UNLESS SHOWN OTHERWISE TENSION LAP SPLICES SHALL BE CLASS B.
   - BARS HOOKS SHALL HAVE STANDARD HOOK DIMENSION USING MINIMUM BEND DIAMETER, 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 THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
2. THE CONTRACTOR IS FULLY RESPONSIBLE FOR PROTECTION OF ALL EXISTING UTILITIES DURING CONSTRUCTION OPERATIONS UNLESS THE EXISTING UTILITIES ARE TO BE RELOCATED.
3. THE CONTRACTOR IS FULLY RESPONSIBLE FOR GROUNDWATER CONTROL ON TIMING OF CONSTRUCTION AND PREVAILING WEATHER CONDITIONS.
4. FOR ALL HIGHWAY WORKS REFER TO HIGHWAY NEW CONSTRUCTION DRAWINGS.
5. FOR ALL ELECTRICAL AND ATMS WORKS REFER TO ELECTRICAL AND ATMS NEW CONSTRUCTION DRAWINGS.
6. FOR ALL UTILITY WORKS REFER TO UTILITY NEW CONSTRUCTION DRAWINGS.

## LIST OF ABBREVIATIONS

| | | | |
|---|---|---|---|
| ABUT. | ABUTMENT | OPENG. | OPENING |
| BOT | BOTTOM | O.F. | OUTSIDE FACE |
| BRGS. | BEARINGS | OPP. | OPPOSITE |
| C.J. | CONSTRUCTION JOINT | RD | ROAD |
| DIA. | DIAMETER | RTL | RIGHT TURN LANE |
| DWG. | DRAWING | RW | RETAINING WALL |
| E.B. | EASTBOUND | SCL | SPEED CHANGE LANE |
| E.F. | EACH FACE | SE | SOUTH EAST |
| EL. | ELEVATION | SIM. | SIMILAR |
| EQ.SP. | EQUAL SPACE | STA. | STATION |
| EXP. | EXPANSION | SHLD | SHOULDER |
| HORIZ. | HORIZONTAL | SW | SOUTH WEST |
| HWY | HIGHWAY | T/P | TOP OF PAVEMENT |
| I.F. | INSIDE FACE | THK. | THICK |
| MIN. | MINIMUM | TYP. | TYPICAL |
| NE | NORTH EAST | VERT. | VERTICAL |
| N.T.S. | NOT TO SCALE | W.B. | WESTBOUND |
| NW | NORTH WEST | WP | WORKING POINT |

## 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 |

NOT FOR CONSTRUCTION

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

| REVISIONS | DATE | REV. | BY | DESCRIPTION |
|---|---|---|---|---|
| | 17–JAN–12 | B3 | PP | 90% SUPERSTRUCTURE FINAL IDR SUBMISSION |
| | 8–DEC–11 | B2 | PP | 90% SUBSTRUCTURE FINAL IDR SUBMISSION |
| | 17–NOV–11 | B1 | PP | SUBSTRUCTURE ICT SUBMISSION |
| | 18–AUG–11 | A | PP | 60% MTO SUBMISSION |

| | | | |
|---|---|---|---|
| DESIGN RF | CHK BR | CODE CAN/CSA S6–06 | LOAD CL 625–ONT |
| DRAWN YZ | CHK PP | SITE 6–611 | DATE 18–JUL–11 |

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

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AND/OR MILLIMETRES
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Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

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

| REVISIONS | | | |
|---|---|---|---|
| | | | |
| | | | |
| 22–MAR–12 | 0 | TL | SUBSTRUCTURE IFC SUBMISSION |
| DATE | BY | REV. | DESCRIPTION |

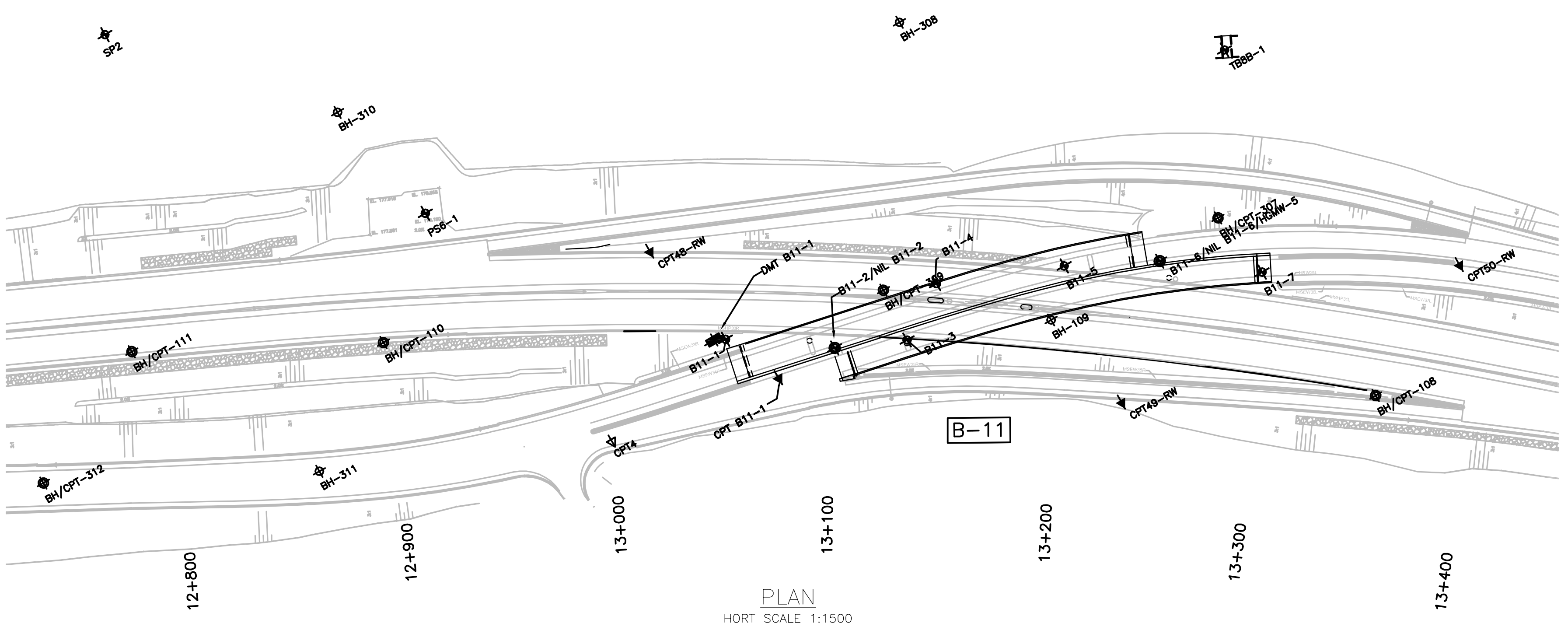
DESIGN TL | APR NSV | DATE MAY 30/11

## LOCATION PLAN & INTERPRETED STRATIGRAPHIC PROFILE

STA 12+800L TO STA 13+400L

SHEET
G1101

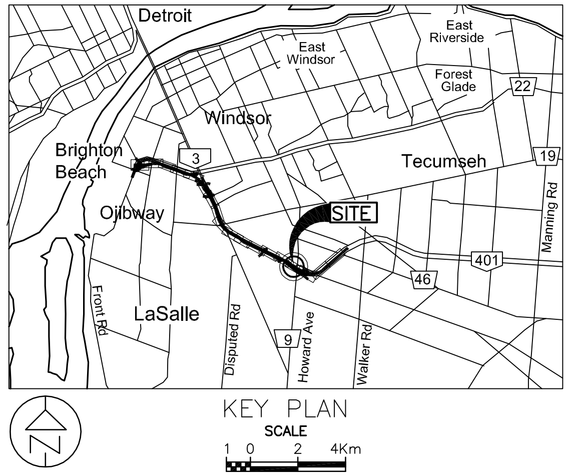
Phase 1
IFC



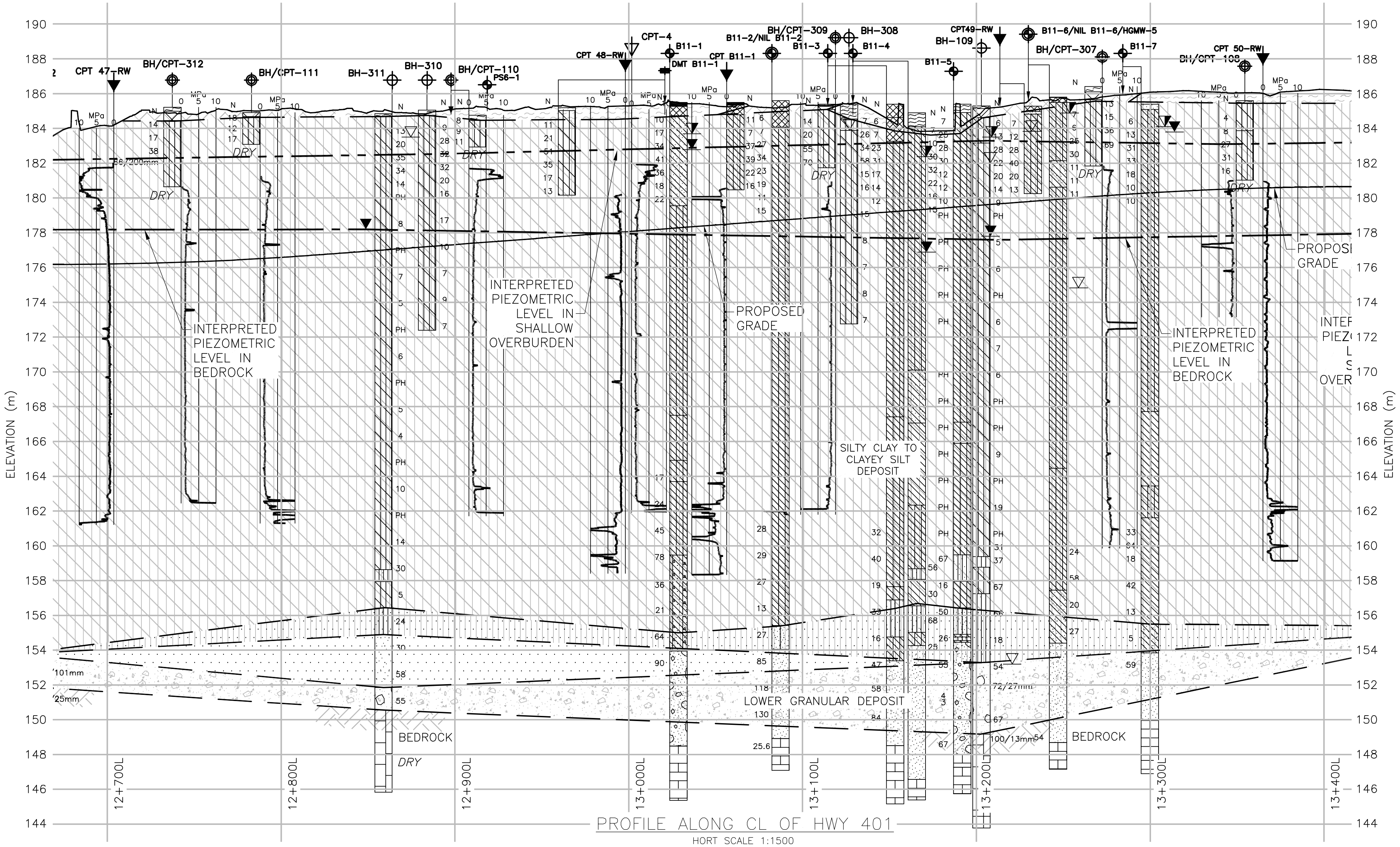
PLAN
HORT SCALE 1:1500

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



KEY PLAN
SCALE
1 0 2 4Km



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

## LEGEND

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

## NOTES

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

SCALES
15 0 30m Horizontal
1.5 0 3 Vertical

DATE PLOTTED: 3/22/2012 9:50:29 AM
FILE LOCATION: C:\pwworking\hmmg_285380\stephen.labute@amec.com\dms09837\285380-04-090-WIP1-1101.dwg

# METRIC

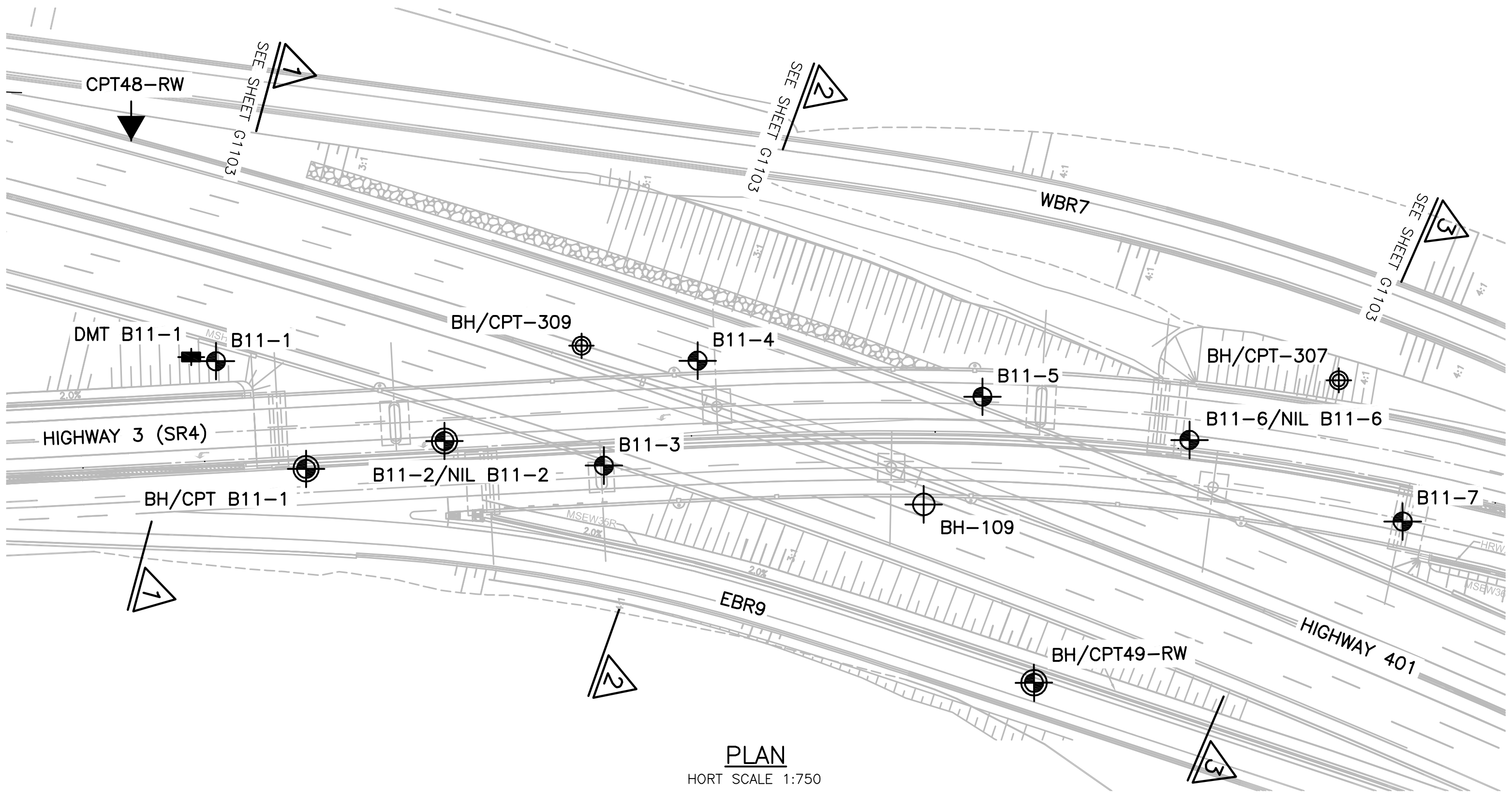
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

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

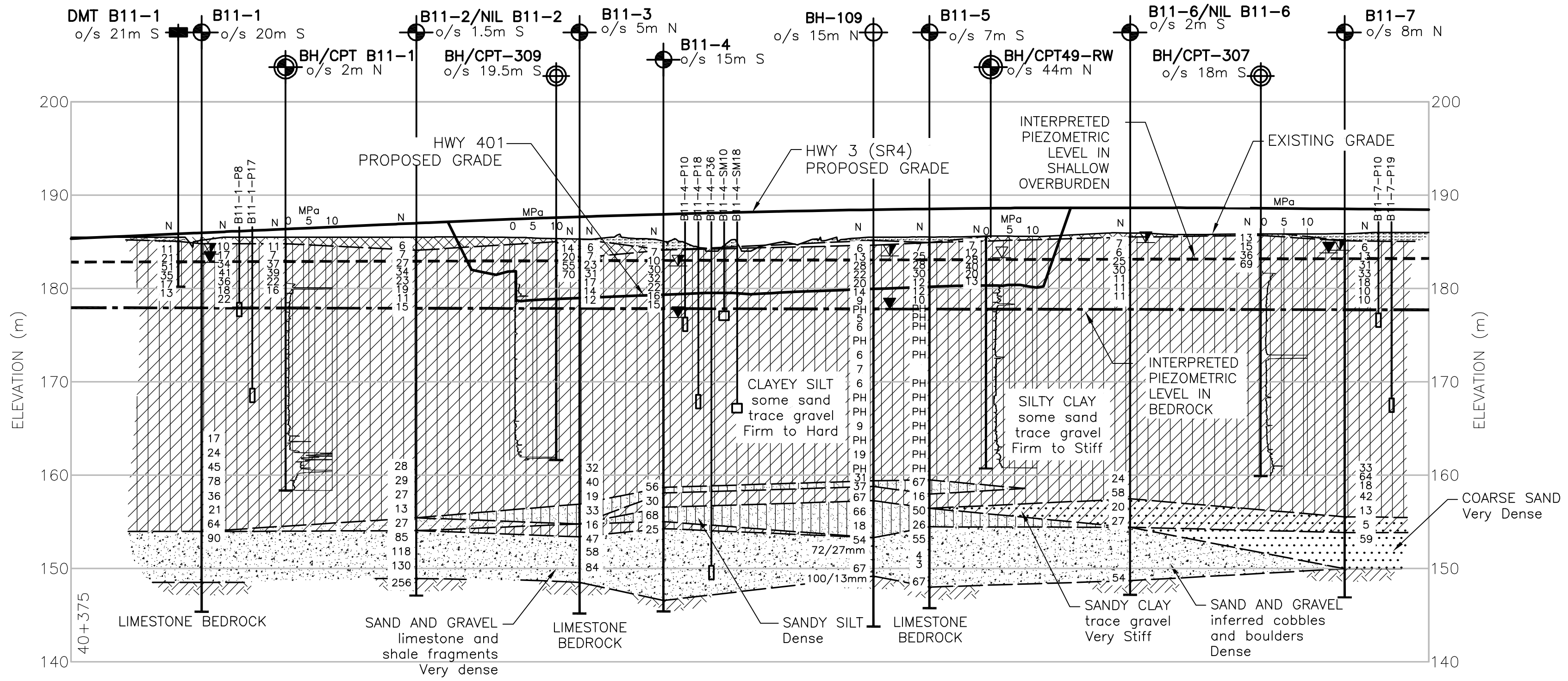
NEW CONSTRUCTION
BRIDGE B–11
HWY 3 UNDERPASS NEAR MONTGOMERY
BORINGHOLE LOCATIONS & SOIL STRATA

SHEET
G1102
Phase 1
IFC



## PLAN

HORT SCALE 1:750



## PROFILE ALONG CL OF HIGHWAY 3 (SR4)

HORT SCALE 1:750
VERT SCALE 1:375

| | READY FOR ISSUE | |
|---|---|---|
| | SUBMISSION: SUBSTRUCTURE IFC SUBMISSION | |
| | NAME (PRINT) | DATE |
| ORIGINATOR | T. LEINALA | 20–MAR–12 |
| CHECKER | D. DIMITRIU | 20–MAR–12 |
| REVIEWER | N. S. VERMA | 20–MAR–12 |

| | READY FOR CHECK | |
|---|---|---|
| | SUBMISSION: SUBSTRUCTURE IFC SUBMISSION | |
| | NAME (PRINT) | DATE |
| CADD TECHNICIAN | S. LABUTE | 20–MAR–12 |
| ORIGINATOR | T. LEINALA | 20–MAR–12 |

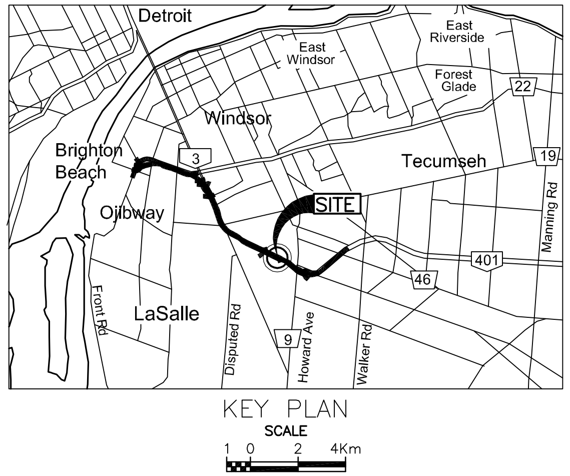
LIST OF ABBREVIATIONS

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

MATERIAL LEGEND

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

| No. | ELEVATION | CO–ORDINATES (UTM, NAD 83 ZONE 17) | |
|---|---|---|---|
| | | NORTHING | EASTING |
| AMEC BOREHOLES | | | |
| B11–1 | 185.4 | 4678221.4 | 334583.9 |
| B11–2/NIL B11–2 | 185.6 | 4678193.3 | 334624.8 |
| B11–3 | 185.4 | 4678179.9 | 334655.1 |
| B11–4 | 185.0 | 4678195.6 | 334679.3 |
| B11–5 | 185.4 | 4678173.2 | 334733.7 |
| B11–6/NIL B11–6 | 185.8 | 4678153.5 | 334772.4 |
| B11–7 | 185.4 | 4678125.8 | 334810.2 |
| BH/CPT B11–1 | 185.4 | 4678195.2 | 334595.9 |
| BH/CPT49–RW | 185.6 | 4678107.8 | 334725.3 |
| DMT B11–1 | 185.1 | 4678223.6 | 334579.2 |
| PREVIOUS BOREHOLES | | | |
| BH–109 | 185.3 | 4678155.0 | 334716.3 |
| BH/CPT–307 | 186.4 | 4678157.2 | 334805.1 |
| BH/CPT–309 | 185.3 | 4678204.8 | 334657.1 |



KEY PLAN
SCALE
1 0 2 4Km

## LEGEND

- ⊕ BOREHOLE CURRENT INVESTIGATION
- ⊕ BOREHOLE AND NILCON VANE CURRENT INVESTIGATION
- ⊕ SW/SP HOLE (HYDROGEOLOGY) CURRENT INVESTIGATION
- ◆ NILCON VANE CURRENT INVESTIGATION
- ▼ CPT – CURRENT INVESTIGATION
- ▬ DMT – CURRENT INVESTIGATION
- ⊕ BOREHOLE PREVIOUS INVESTIGATION
- ⊕ BOREHOLE, CPT AND NILCON VANE PREVIOUS INVESTIGATIONS
- ▽ CPT –PREVIOUS INVESTIGATION
- N SPT N–VALUE
- 16 BLOWS/0.3m UNLESS OTHERWISE STATED (STD. PEN. TEST, 475 J/BLOW)
- PH – SAMPLE OBTAINED UNDER HYDRAULIC PRESSURE
- ☐ 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

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

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

| REVISIONS | | | |
|---|---|---|---|
| 20–MAR–12 | 0 | TL | SUBSTRUCTURE IFC SUBMISSION |
| DATE | REV. | BY | DESCRIPTION |

| | | | |
|---|---|---|---|
| DESIGN WH | CHK NSV | CODE CAN/CSA S6–06 | LOAD CL–625–ON |
| DRAWN MM | CHK DD | SITE 6–611 | DATE 18–JUL–11 |

# METRIC

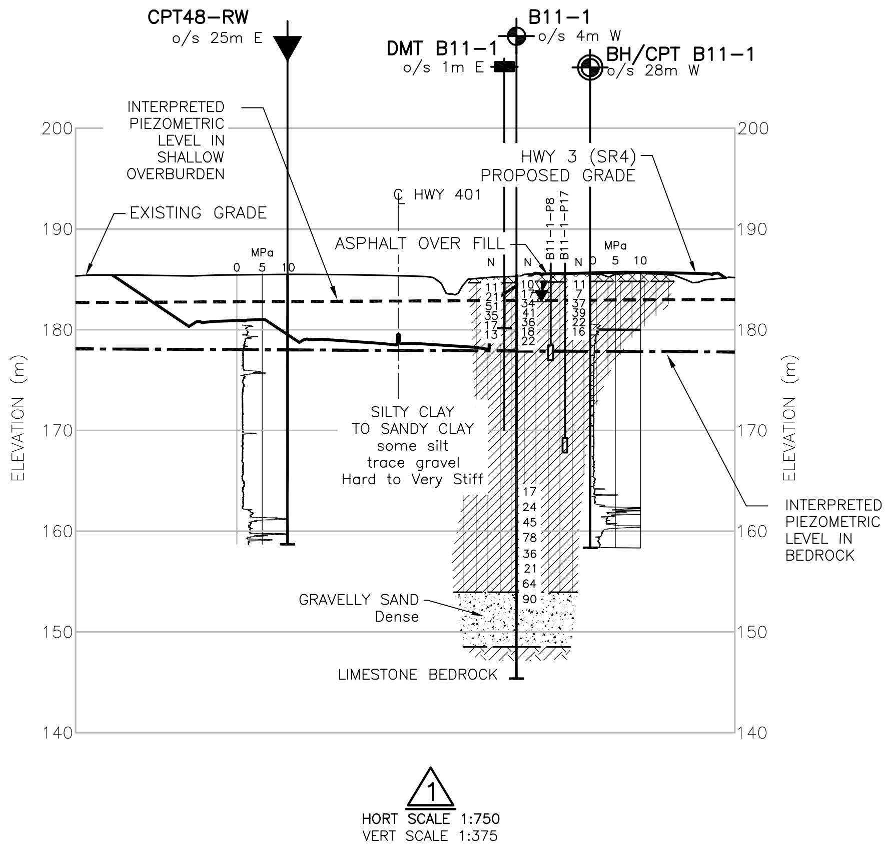
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

**Parkway Infrastructure Engineers** amec Hatch Mott MacDonald

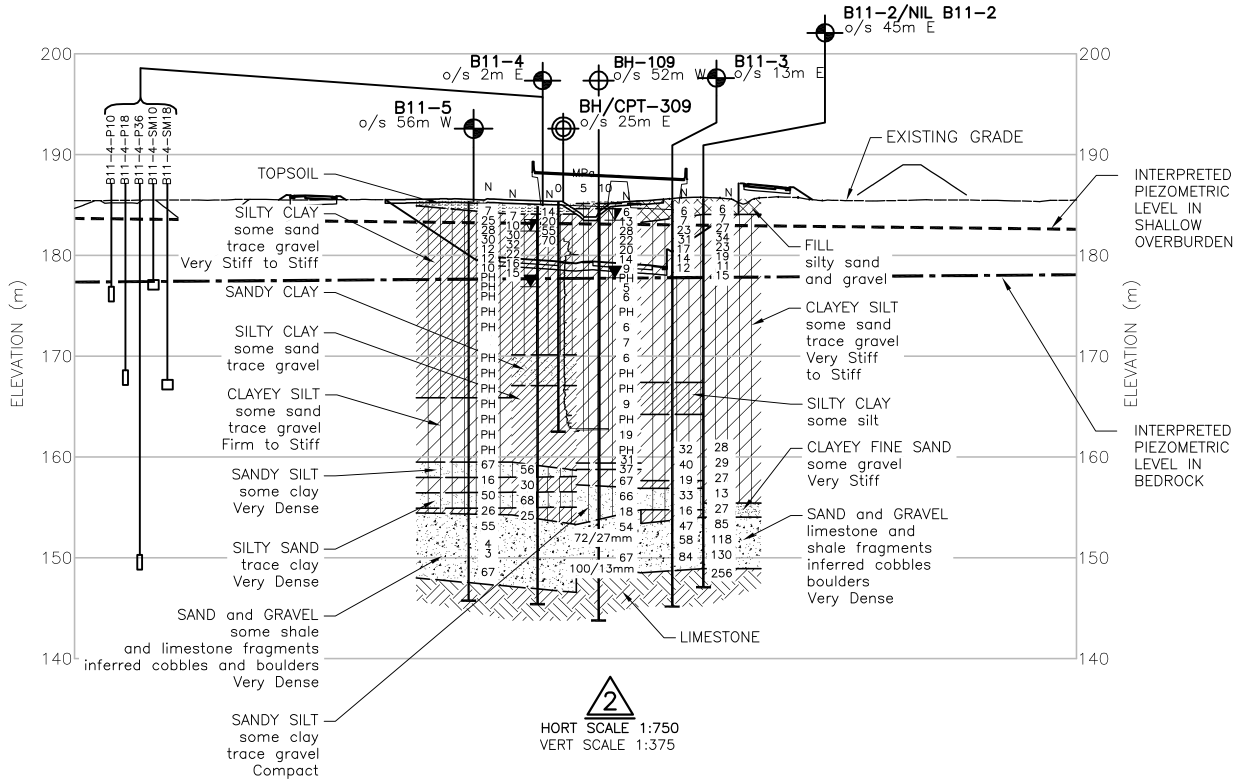
Windsor-Essex
Parkway Project
RFP No. 09-54-1007

NEW CONSTRUCTION
BRIDGE B-11
HWY 3 UNDERPASS NEAR MONTGOMERY
SOIL STRATIGRAPHY

SHEET
G1103
Phase 1
IFC



1
HORT SCALE 1:750
VERT SCALE 1:375



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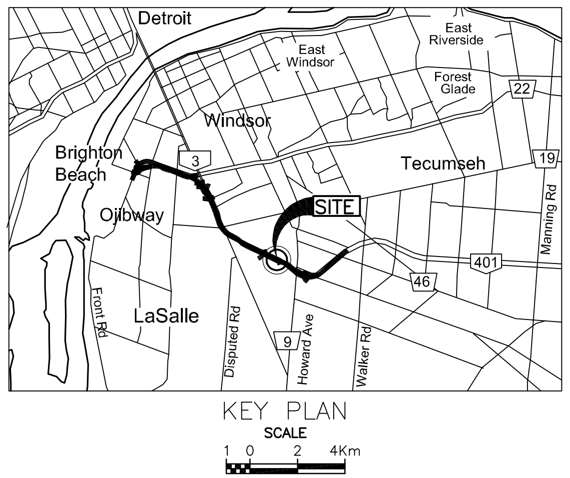
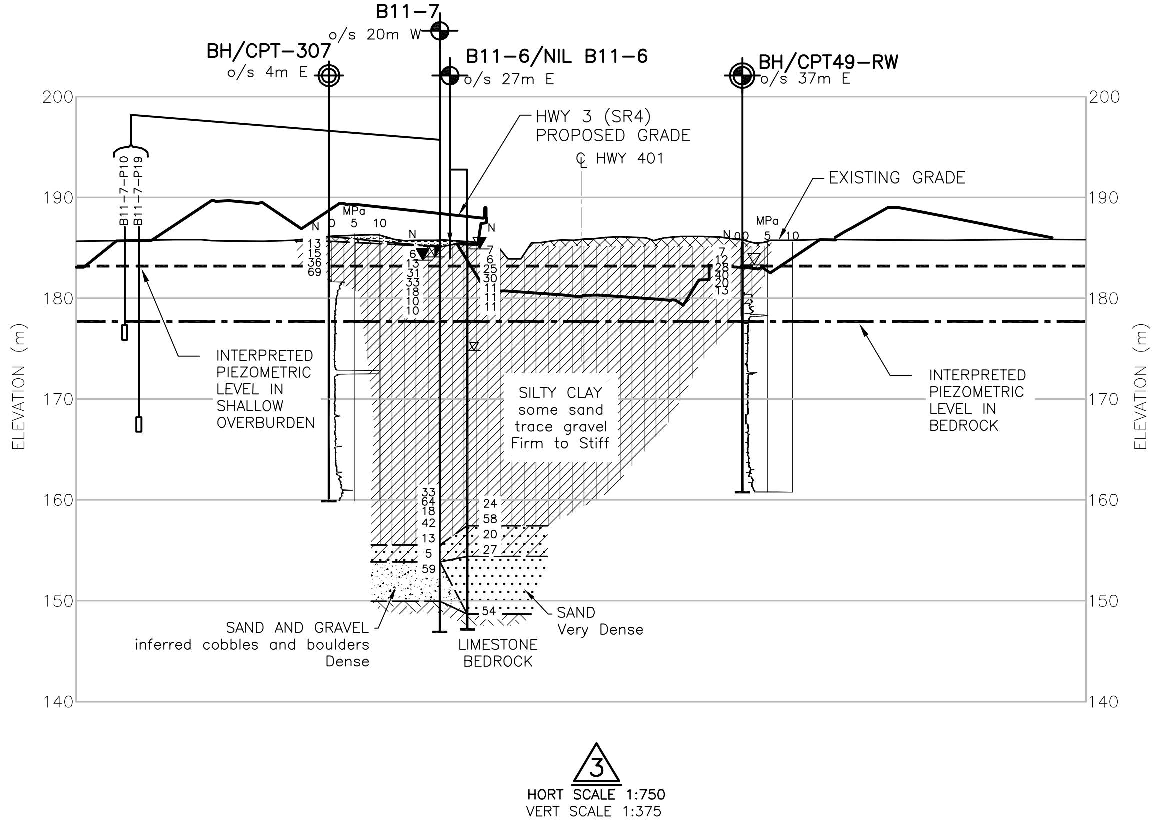
HORT SCALE 1:750
VERT SCALE 1:375



KEY PLAN
SCALE
1 0 2 4Km



3
HORT SCALE 1:750
VERT SCALE 1:375

## LEGEND

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- *DRY* BOREHOLE DRY DURING DRILLING
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MATERIAL LEGEND

| | |
|---|---|
| ▨ TOPSOIL/ ORGANICS | ▥ SILT |
| ☒ FILL | ▨ SANDY SILT |
| ⁚ SAND | ▨ CLAYEY SILT |
| ▨ SILTY CLAY | ▨ SAND AND GRAVEL |
| ▨ SILTY SAND | ▨ SILTY SAND AND GRAVEL |
| | ▨ LIMESTONE DOLOSTONE /BEDROCK |

| | READY FOR ISSUE | |
|---|---|---|
| | SUBMISSION: SUBSTRUCTURE IFC SUBMISSION | |
| | NAME (PRINT) | DATE |
| ORIGINATOR | T. LEINALA | 20-MAR-12 |
| CHECKER | D. DIMITRIU | 20-MAR-12 |
| REVIEWER | N. S. VERMA | 20-MAR-12 |

| | READY FOR CHECK | |
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| CADD TECHNICIAN | S. LABUTE | 20-MAR-12 |
| ORIGINATOR | T. LEINALA | 20-MAR-12 |

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| B11-7 | 185.4 | 4678125.8 | 334810.2 |
| BH/CPT B11-1 | 185.4 | 4678195.2 | 334595.9 |
| BH/CPT49-RW | 185.6 | 4678107.8 | 334725.3 |
| DMT B11-1 | 185.1 | 4678223.6 | 334579.2 |
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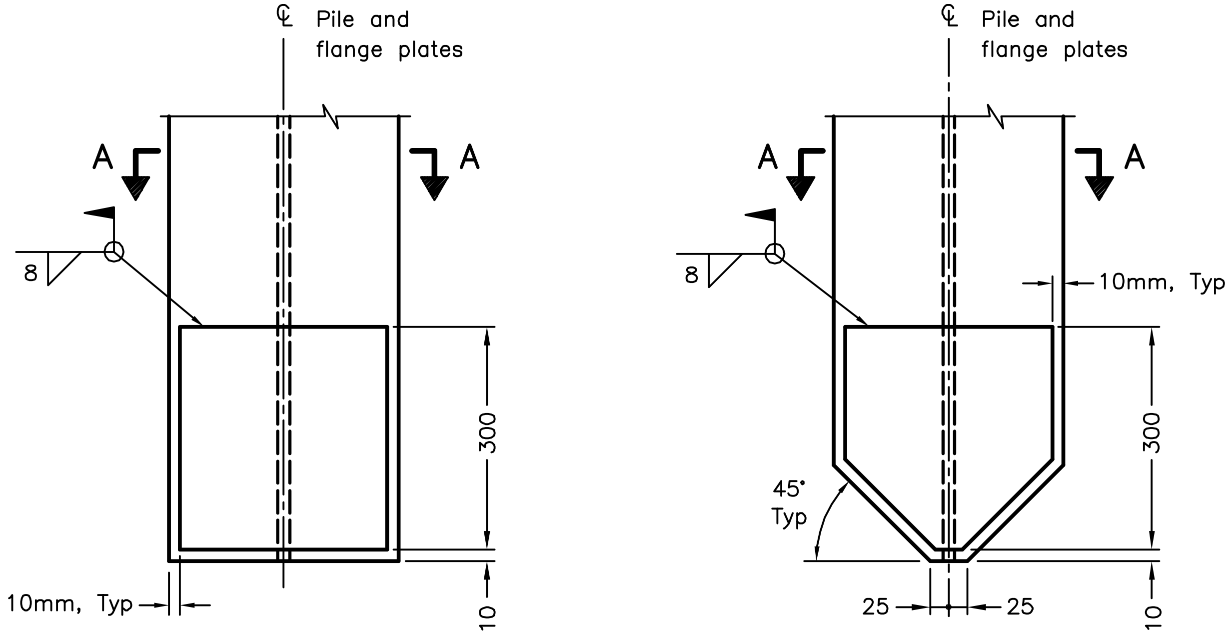
| BH-109 | 185.3 | 4678155.0 | 334716.3 |
| BH/CPT-307 | 186.4 | 4678157.2 | 334805.1 |
| BH/CPT-309 | 185.3 | 4678204.8 | 334657.1 |

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

| REVISIONS | | | |
|---|---|---|---|
| 20-MAR-12 | 0 | TL | SUBSTRUCTURE IFC SUBMISSION |
| DATE | REV. | BY | DESCRIPTION |

| | | | |
|---|---|---|---|
| DESIGN WH | CHK NSV | CODE CAN/CSA S6-06 | LOAD CL-625-ON |
| DRAWN MM | CHK DD | SITE 6-611 | DATE 18-JUL-11 |

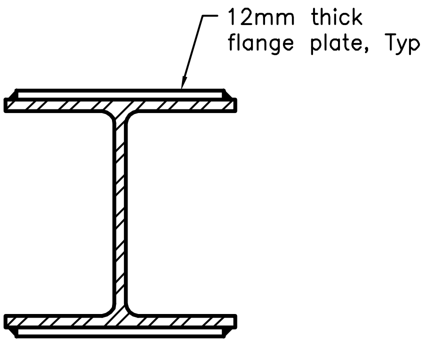
# OPSDs



**TYPE I**

**TYPE II**

## ELEVATION



**PILE DRIVING SHOE**

## SECTION A–A

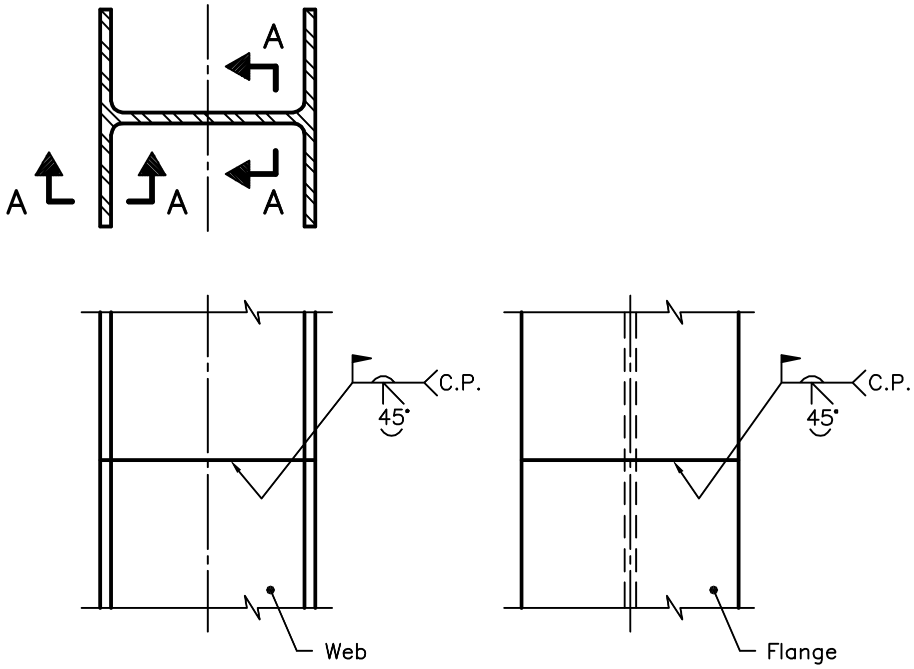
NOTES:

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

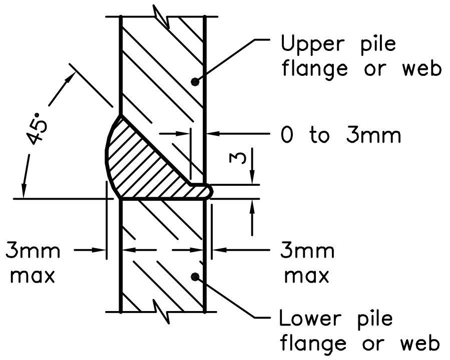
ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2010 | Rev 2

**FOUNDATION**
**PILES**
**STEEL H–PILE DRIVING SHOE**

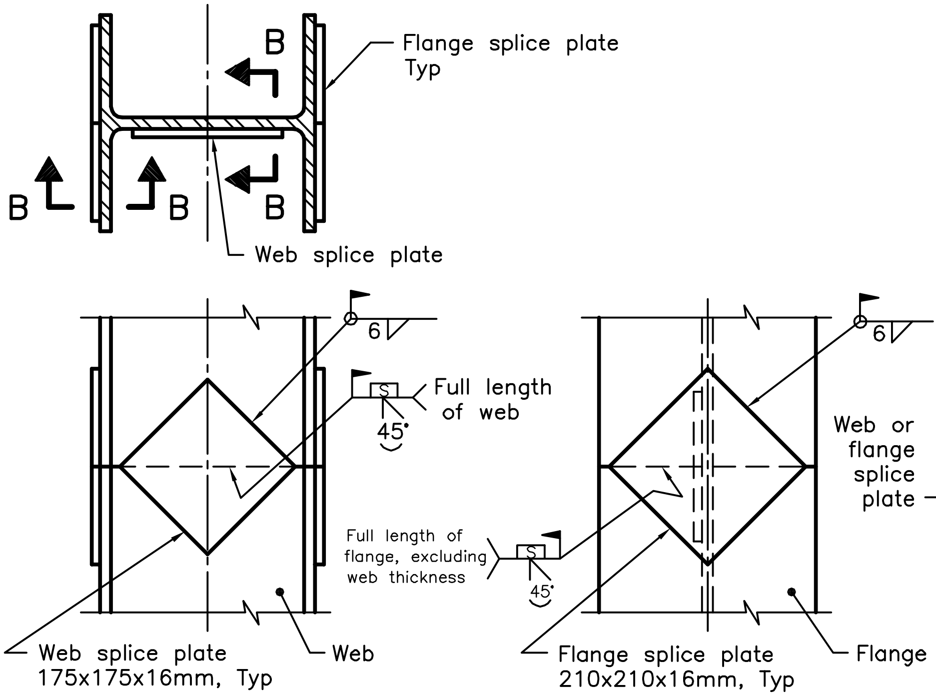
**OPSD 3000.100**



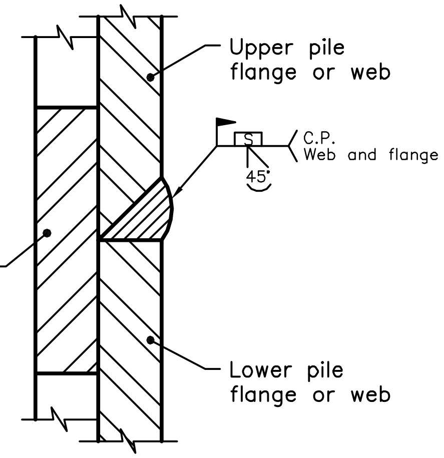
## BUTT WELD



## SECTION A–A



## BUTT WELD WITH SPLICE PLATES



## SECTION B–B

NOTES:

A The pile splice shall be perpendicular to the centreline of pile.

B Splice plates shall be according to CSA G40.20/G40.21, Grade 300W.

C Welding shall be according to CSA W59.

D Splice plate alternative is only applicable to H–pile sizes HP310x79, HP310x110, and HP310x132.

E All dimensions are in millimetres unless otherwise shown.

| ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2010 | Rev 1 |
|---|---|---|
| **FOUNDATION**<br>**PILES**<br>**STEEL H–PILE SPLICE** | **OPSD 3000.150** | |

## NOTES:

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

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

C Values between contours should be interpolated. If interpolation is not possible, use the adjacent contour with the greater depth.

D Frost penetration depths are in metres.

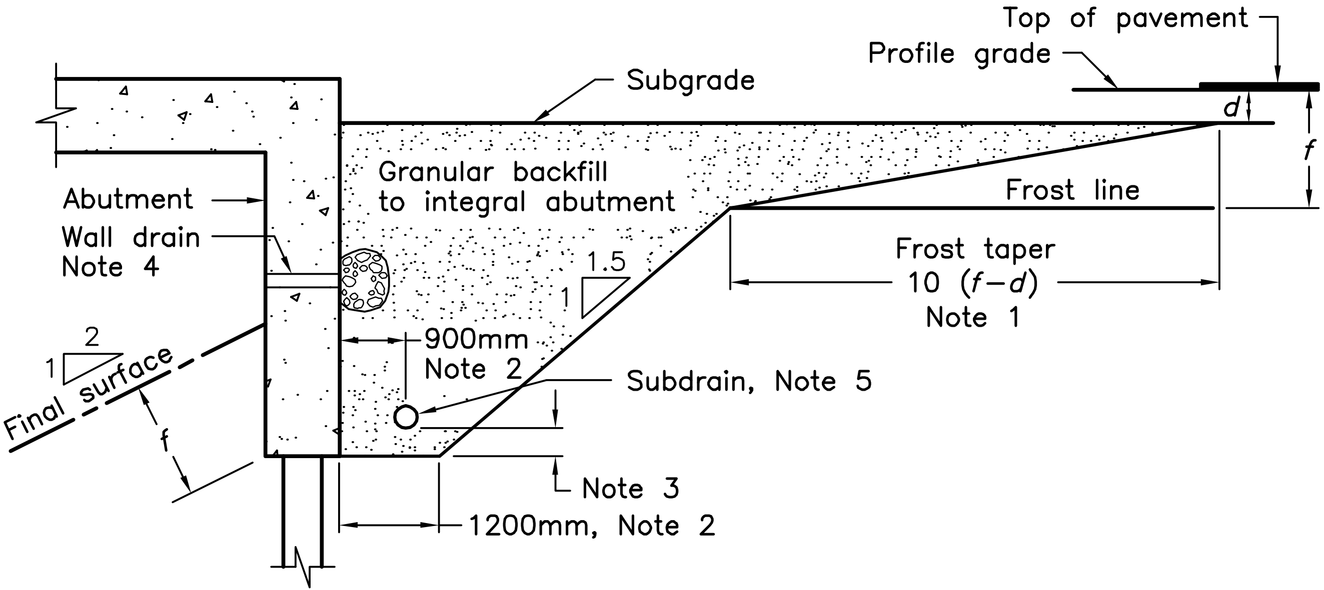
ONTARIO PROVINCIAL STANDARD DRAWING

# FOUNDATION
# FROST PENETRATION DEPTHS
# FOR SOUTHERN ONTARIO

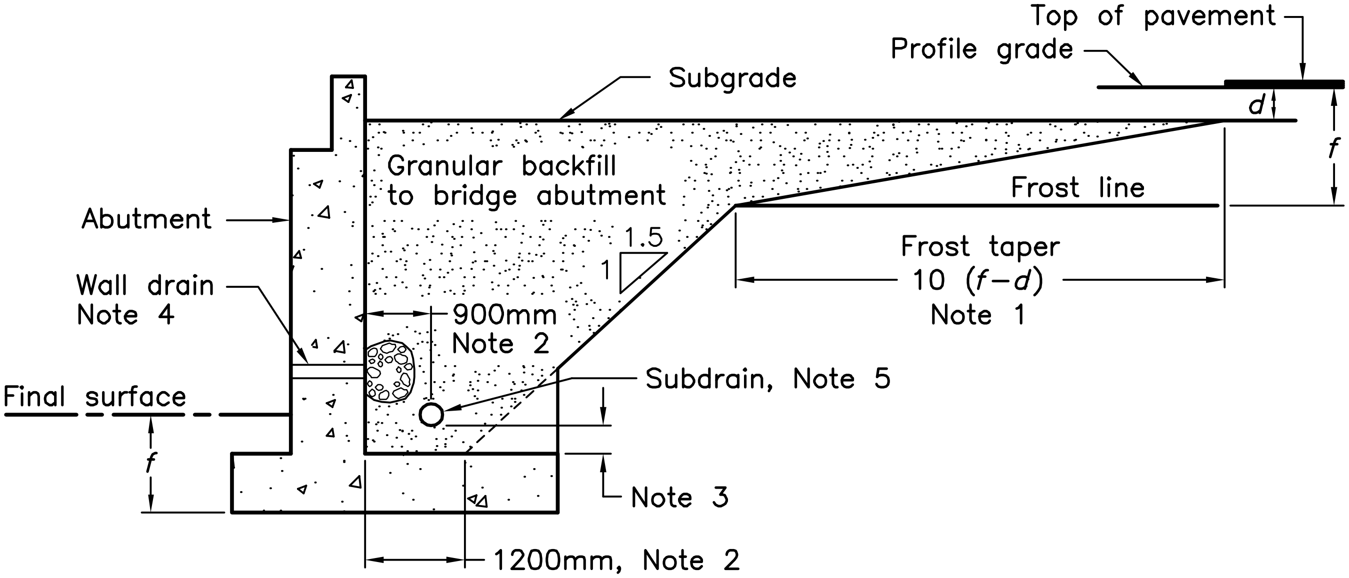
Nov 2010

Rev 1

OPSD 3090.101



**INTEGRAL ABUTMENT**



**ABUTMENT**

NOTES:

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

ONTARIO PROVINCIAL STANDARD DRAWING
Nov 2010 Rev 1

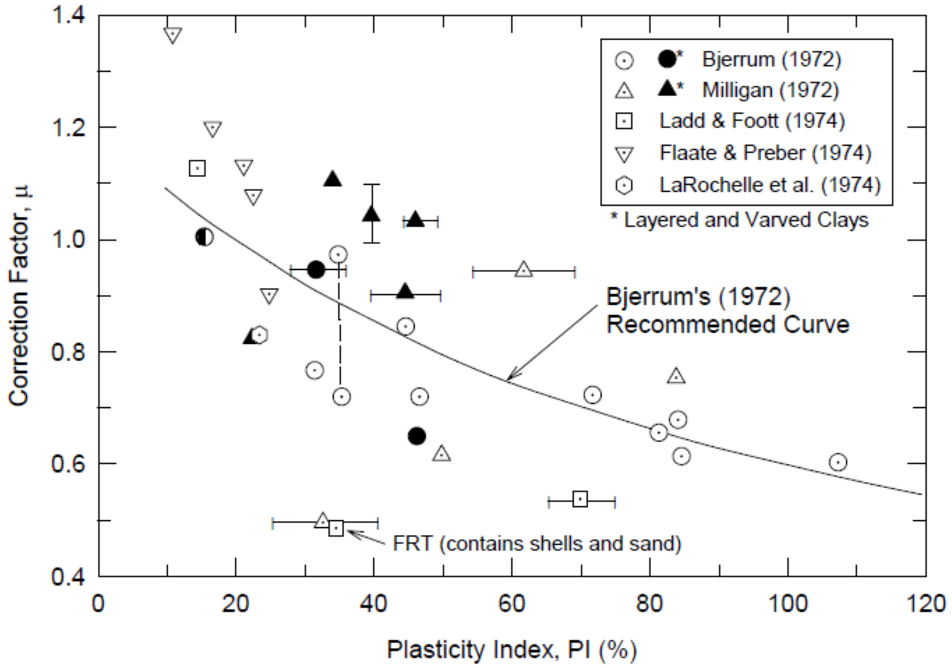
WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD 3101.150

# Figures

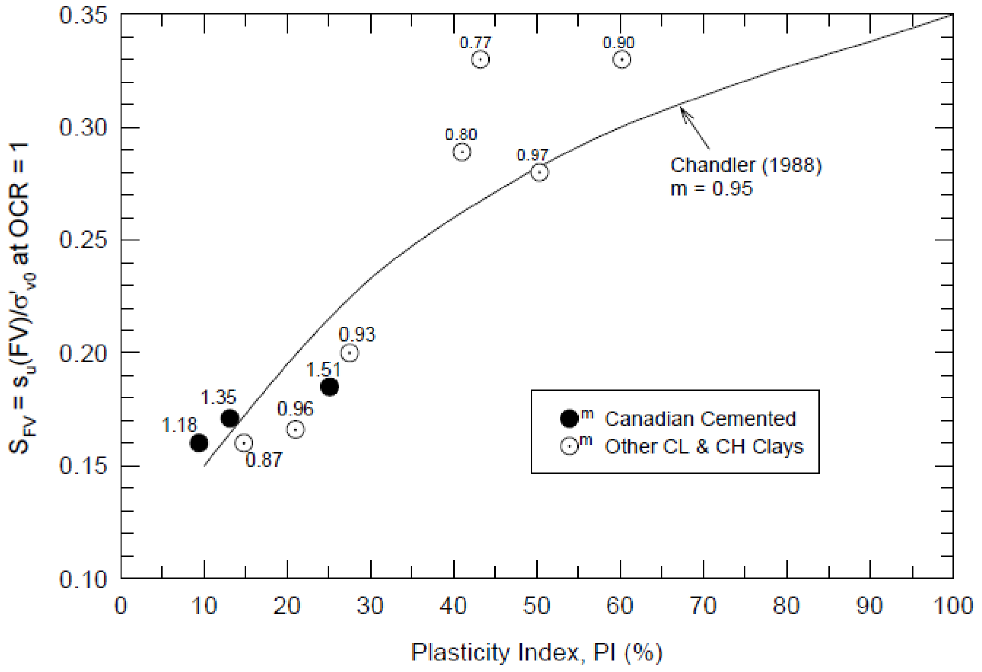
## Figure 3-1: Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures

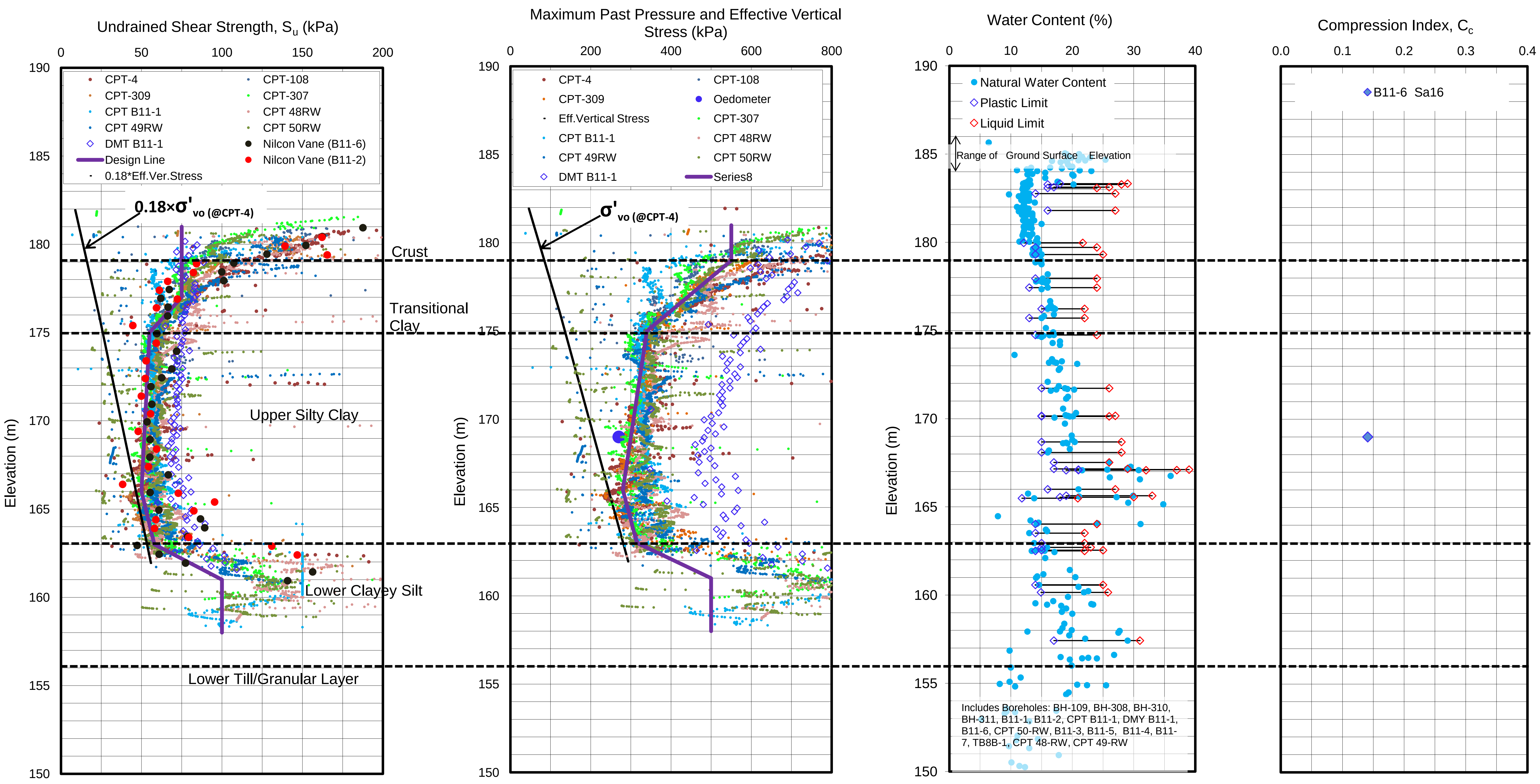
(Figure 5.1, Ladd & DeGroot, 2004, ref. R-30)



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

(Figure 5.2, Ladd & DeGroot, 2004, ref. R-30)





Notes:

1. Shear strength profiles were estimated from CPT data using the equation $S_u = (q_t - \sigma_{VO}) / N_{KT}$. The cone factor $N_{KT}$ was estimated by comparing the CPT profiles with a nearby Nilcon Vane profile.
2. Maximum past pressure profiles estimated using SHANSEP method. OCR = $[(S_u/\sigma'_V)/S]^{1/m}$
3. Data from geotechnical investigation by AMEC and historic investigtions by Golder Associates

amec Earth & Environmental

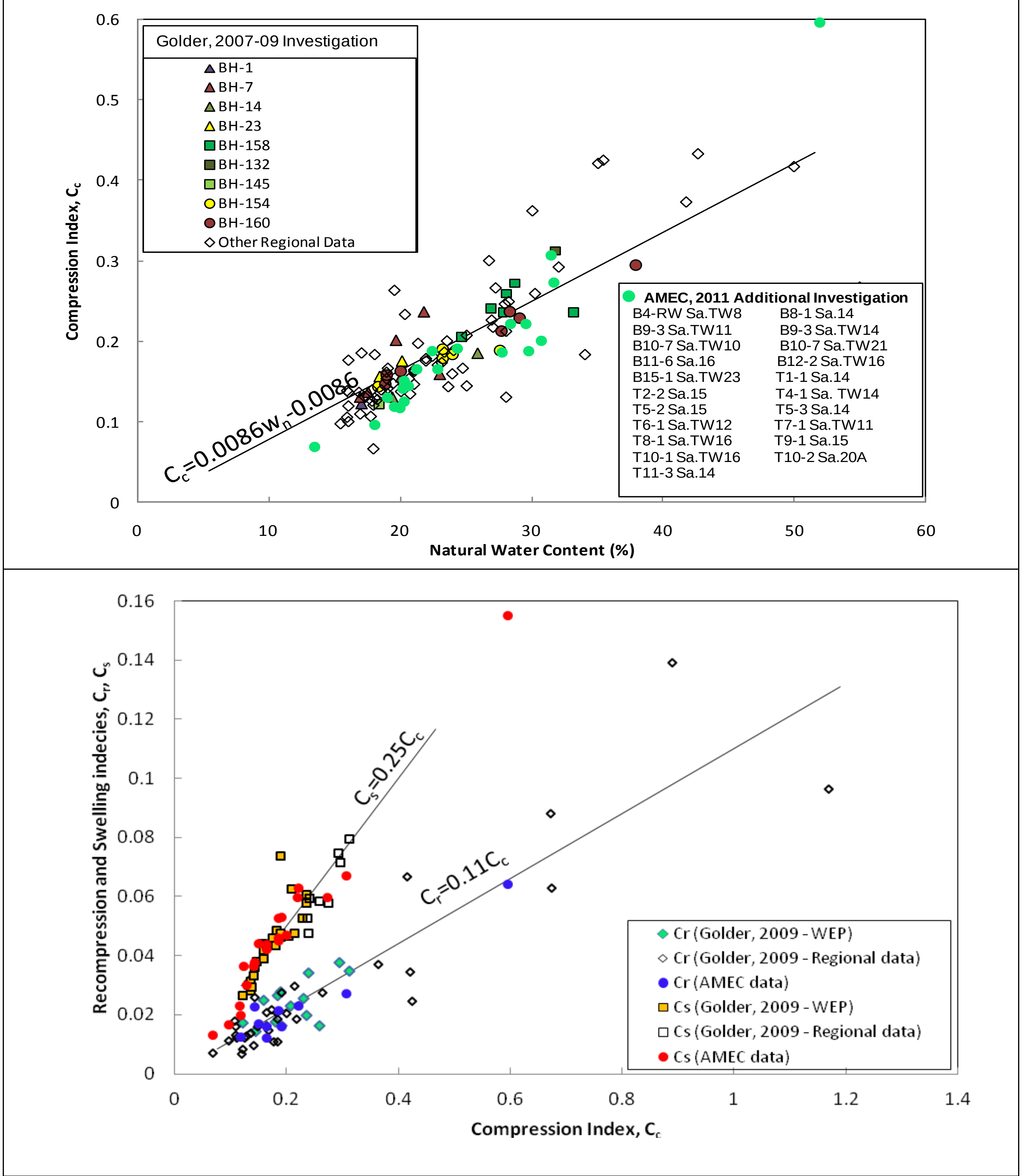
CLIENT :

PROJECT: **WINDSOR ESSEX PARKWAY**

TITLE: SOIL PROPERTIES PROFILES
STA.12+800L TO 13+400L

| DATE: | JOB NO.: | CAD FILE: | FIGURE NO.: | REV. |
|---|---|---|---|---|
| Mar 2012 | SW8801.1002 | | 3. | |

Figure 4-1: Compressibility Parameters at WEP



# Figure 4-2: Cc versus Cα Relationship at WEP

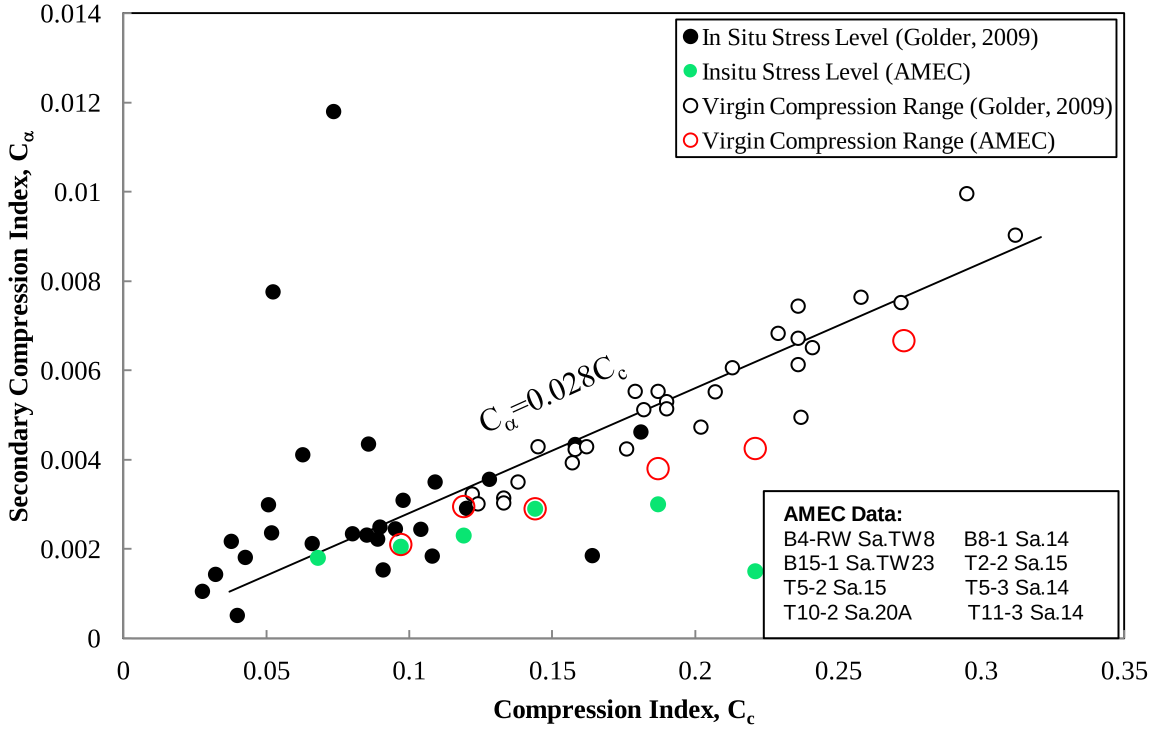


Figure 4-3: Effective Friction Angle (ϕ') for Silty Clay to Clayey Silt Stratum at WEP

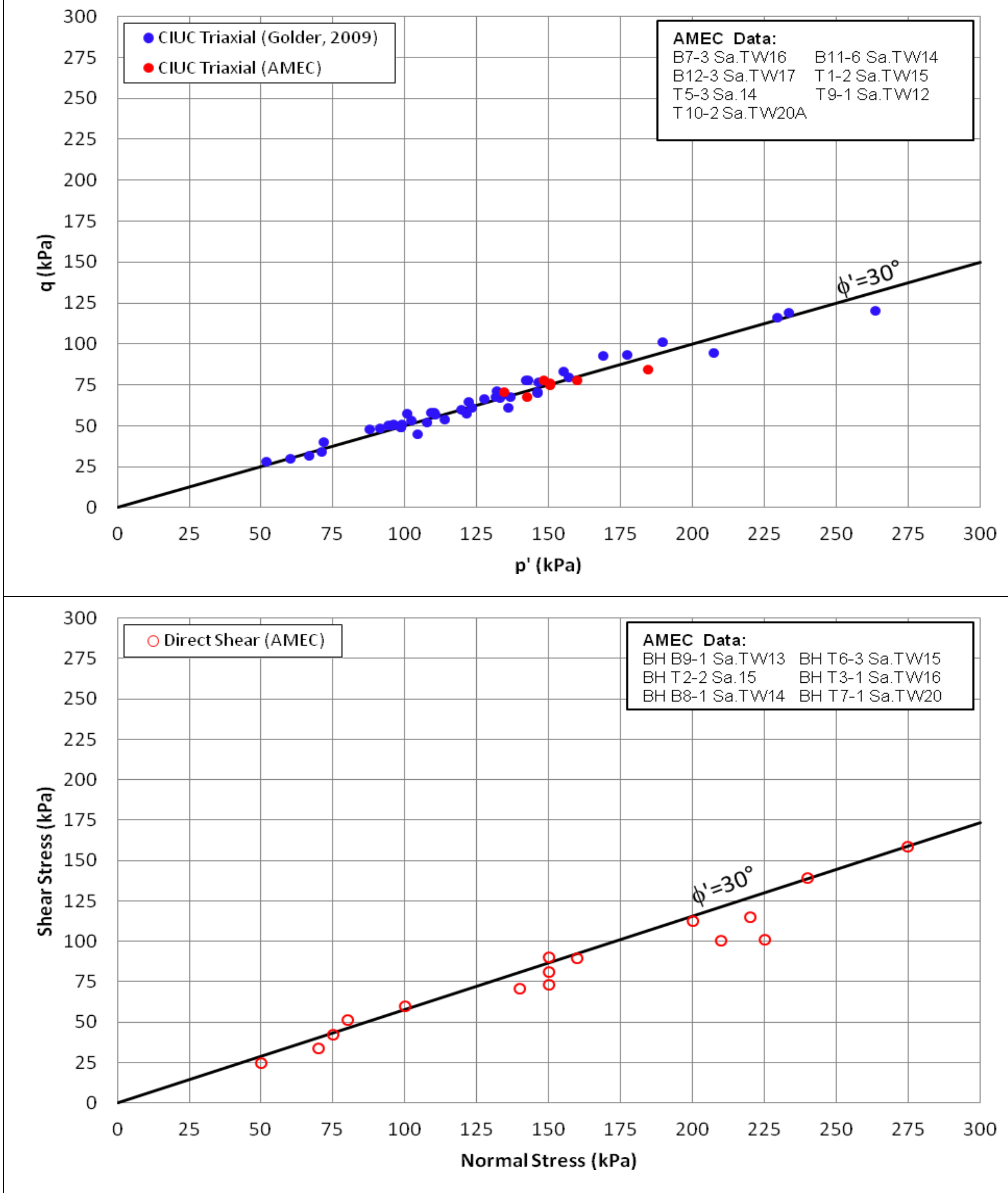
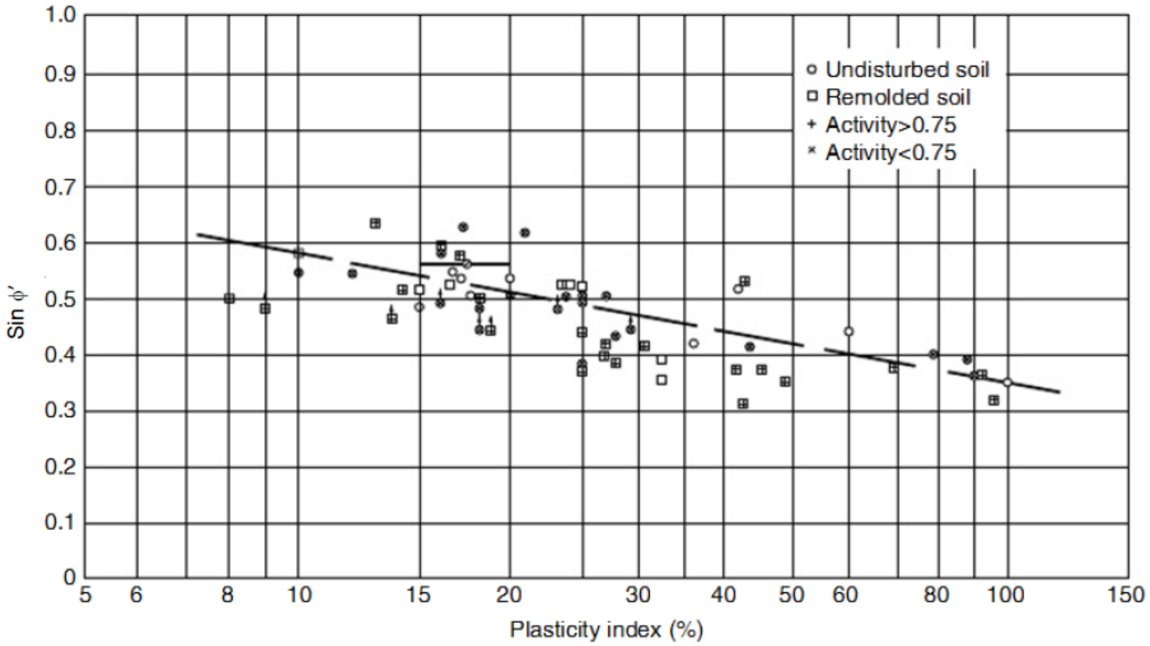
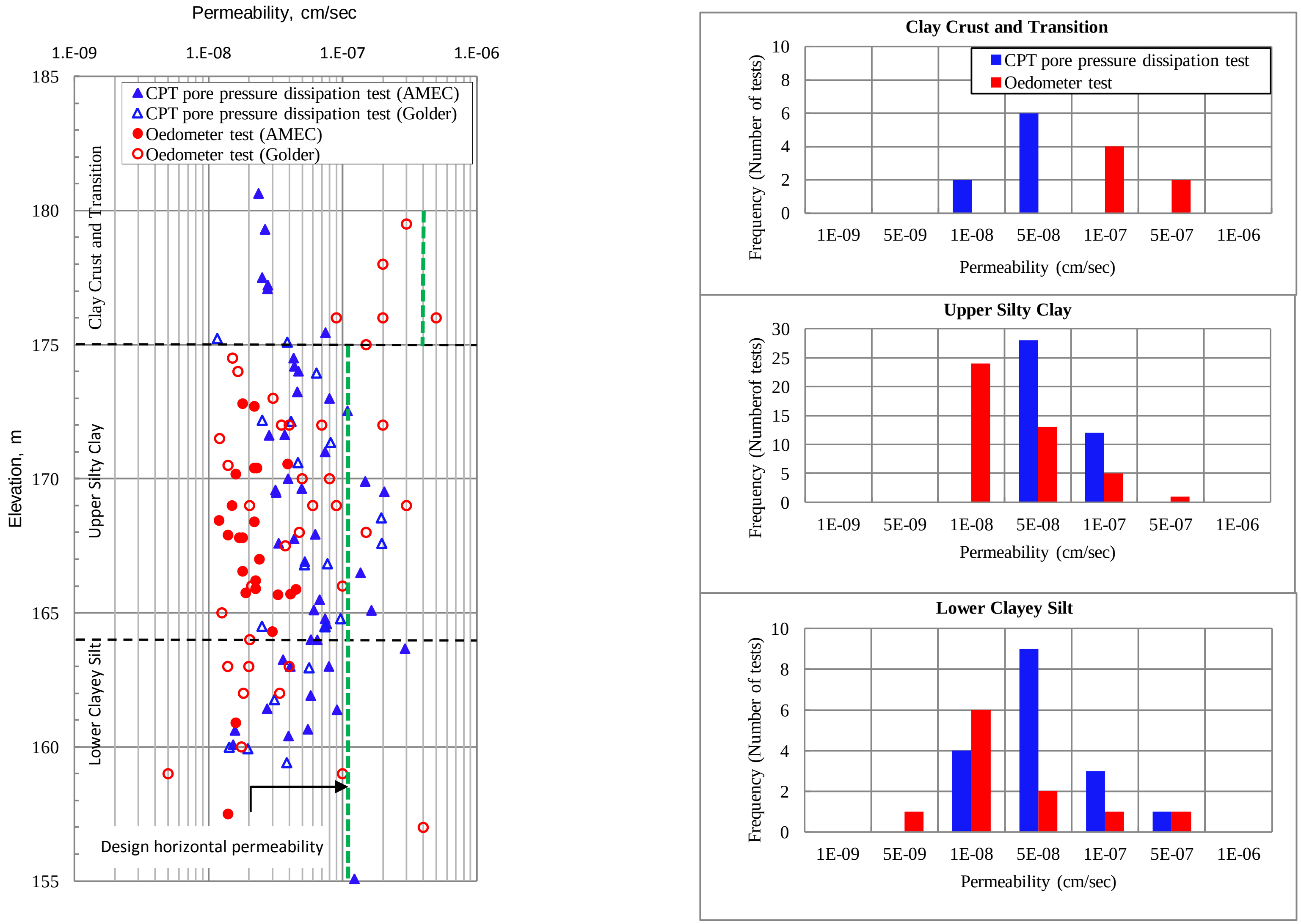


Figure 4-4: Relationship between sin ϕ' and Plasticity Index for Normally Consolidated Soils

**(Kenney, 1959)**



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



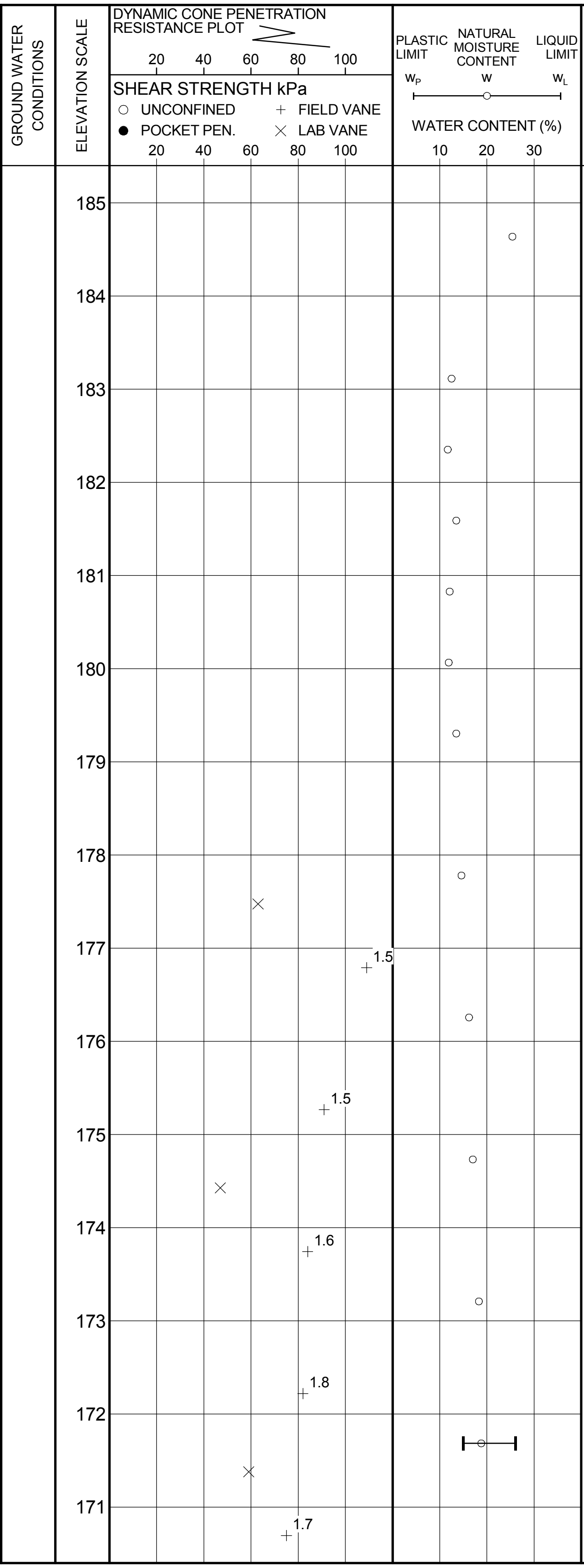
# Appendix A: Borehole and CPT Logs from Additional Geotechnical Investigation

# RECORD OF BOREHOLE No B11-1

1 OF 3 **METRIC**

| | | | |
|---|---|---|---|
| W.P. RFP No. 09-54-1007 | LOCATION N4678221.4, E334583.9 | | ORIGINATED BY TA |
| DIST | HWY WEP | BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY SS |
| DATUM Geodetic | | DATE May 4, 11 - May 7, 11 | CHECKED BY MSO |

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 185.4 | Pavement Surface | | | | | |
| 185.2<br>0.0 | 225mm **ASPHALT** | | | | | -borehole drilled and sampled from surface to 21.8m at (4678221N, 334584E); vibrating wire piezometers (VWP) installed in initial boring; borehole continued at (4678205N, 334573E) by drilling without sampling to 21.8m, then sampling as indicated below. Slope indicator installed in second boring.<br>-no recovery's split spoon plugged by stone |
| 0.2 | Grey **FILL** Crushed limestone sand and gravel | | | | | |
| 184.6<br>0.8 | Mottled Brown-Grey **CLAYEY SILT** to **SANDY SILT** Some clay, trace gravel Stiff | 1 | SS | 10 | | |
| | | 2 | SS | 17 | | |
| | Brown Hard | 3 | SS | 34 | | |
| | | 4 | SS | 41 | | |
| | | 5 | SS | 36 | | |
| | Grey | 6 | SS | 18 | | -end of drilling May 4; restart May 5 |
| | Very stiff | 7 | SS | 22 | | |
| 179.5<br>5.9 | Grey **CLAYEY SILT** Some sand, trace gravel Stiff | 8 | TW | PH | | |
| | | 9 | TW | PH | 21.7 | |
| | | | VT | | | -VWP #P8 installed at 8.23m<br>-switch to wash boring with casing |
| | | 10 | TW | PH | | |
| | | | VT | | | |
| | | 11 | TW | PH | 21.5 | |
| | | | VT | | | |
| | | 12 | TW | PH | | |
| | | | VT | | | |
| | | 13 | TW | PH | 20.9 | 5 26 40 29 |
| | | | VT | | | |



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

# RECORD OF BOREHOLE No B11-1

2 OF 3

**METRIC**

W.P. RFP No. 09-54-1007
LOCATION N4678221.4, E334583.9
ORIGINATED BY TA
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE May 4, 11 - May 7, 11
CHECKED BY MSO

| ELEV / DEPTH | SOIL PROFILE DESCRIPTION | NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | REMARKS | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|
| | Grey **CLAYEY SILT** Stiff | 14 | TW | PH | | | | | | |
| | | 15 | SS | PH | | -no recovery with shelby tube; pushed split spoon for sample<br>-VWP #P17 installed at 17.37m | | | | |
| 167.4 / 18.0 | Grey **SILTY CLAY** And pink clay nodules Stiff | 16 | TW | PH | 19.3 | | | | | |
| | | 17 | SS | PH | | -no recovery with shelby tube; pushed split spoon for sample | | | | |
| 164.8 / 20.6 | Grey **CLAYEY SILT** Some sand, trace gravel Very stiff | | VT | | | | | | | |
| | | 18 | SS | 17 | | | 3 | 26 | 48 | 23 |
| | | | | | | -end of drilling at initial boring location. May 5; restart at second location May 6 | | | | |
| | Very stiff | 19 | SS | 24 | | | | | | |
| | Hard | 20 | SS | 45 | | | | | | |
| 159.4 / 26.0 | Alternating layers of dense to hard glactolacustrine soils from approx. 26.0m to 31.5m<br>Approx. 125mm of grey **SAND** and **GRAVEL,** wet<br>Approx. 75mm of hard, grey **SILTY CLAY,** varved<br>Approx. 125mm of very dense **CLAYEY SAND**<br>Approx. 500mm of hard grey **SILTY CLAY,** varved<br>Approx. 100mm of grey **SILT** with clay | 21 | SS | 78 | | | | | | |
| | | 22 | SS | 36 | | | | | | |
| | Grey **SILTY CLAY,** varved very stiff | 23 | SS | 21 | | | | | | |

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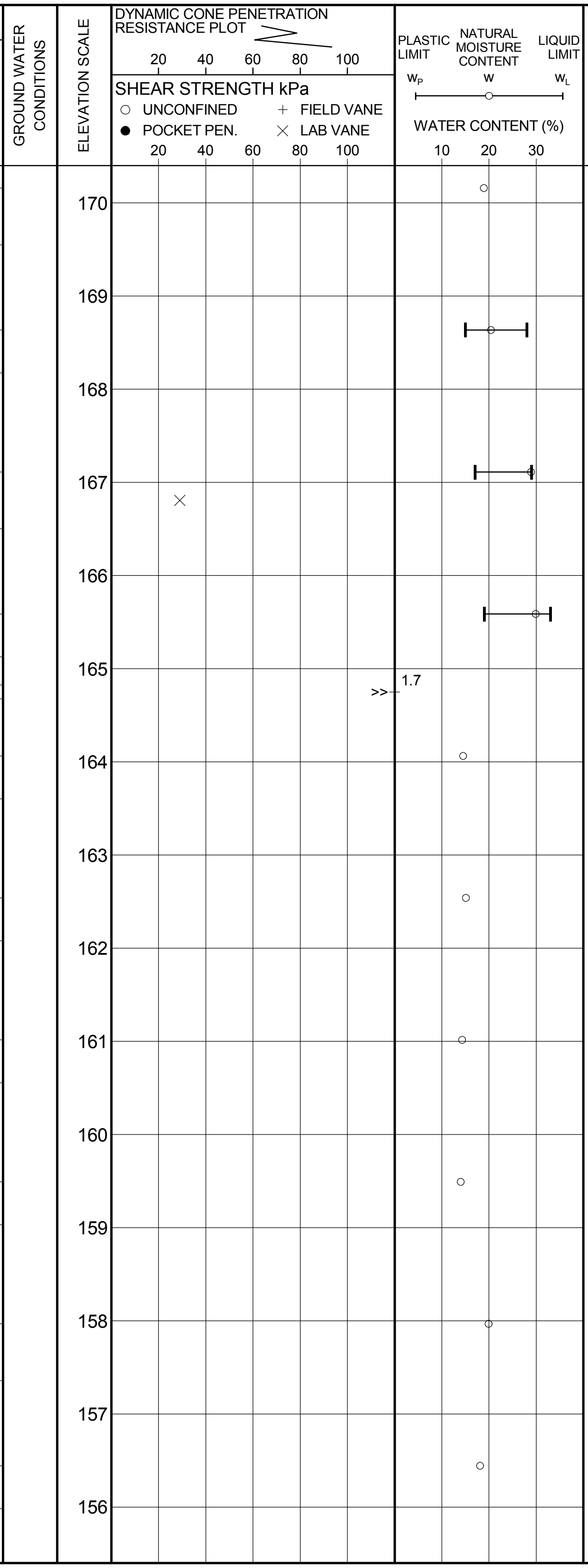


+3, ×3: Numbers refer to Sensitivity
○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B11-1

3 OF 3

**METRIC**

W.P. RFP No. 09-54-1007
LOCATION N4678221.4, E334583.9
ORIGINATED BY TA
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE May 4, 11 - May 7, 11
CHECKED BY MSO

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

| ELEV<br>DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | WATER CONTENT (%) | REMARKS | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|
|  | Alternating layers of dense to hard glactolacustrine soils from approx. 26.0m to 31.5m *(continued)* **SANDY SILT,** with gravel | 24 | SS | 64 | ○ |  |  |  |  |  |
| 153.9<br>31.5 | Grey **GRAVELLY SAND** Very dense | 25 | SS | 90 | ○ |  | 11 | 78 | 11 |  |
|  |  |  |  |  |  | -casing slipped 1.5m; unable to sample |  |  |  |  |
|  |  |  |  |  |  | -sand and gravel flowed up to 32m before sample could be taken |  |  |  |  |
| 148.4<br>37.0 | Light grey, **LIMESTONE,** (fine grained, cherty) bedded, numerous stylolites throughout, faintly porous, pitted between 128'9" to 129'9". Light blue-grey inclusions | 1 | RC |  |  | RQD=53%<br>TCR = 100%<br>SCR = 57% |  |  |  |  |
|  |  | 2 | RC |  |  | RQD=85%<br>TCR = 100%<br>SCR = 85% |  |  |  |  |
| 145.3<br>40.1 | **END OF BOREHOLE** |  |  |  |  |  |  |  |  |  |

Piezometric levels in VWP #P8:
May 12, 2011: EL. 185.1m
July 24, 2011: EL. 183.7m

Piezometric levels in VWP #P17:
May 12, 2011: EL. 185.6m
July 24, 2011: EL. 183.0m

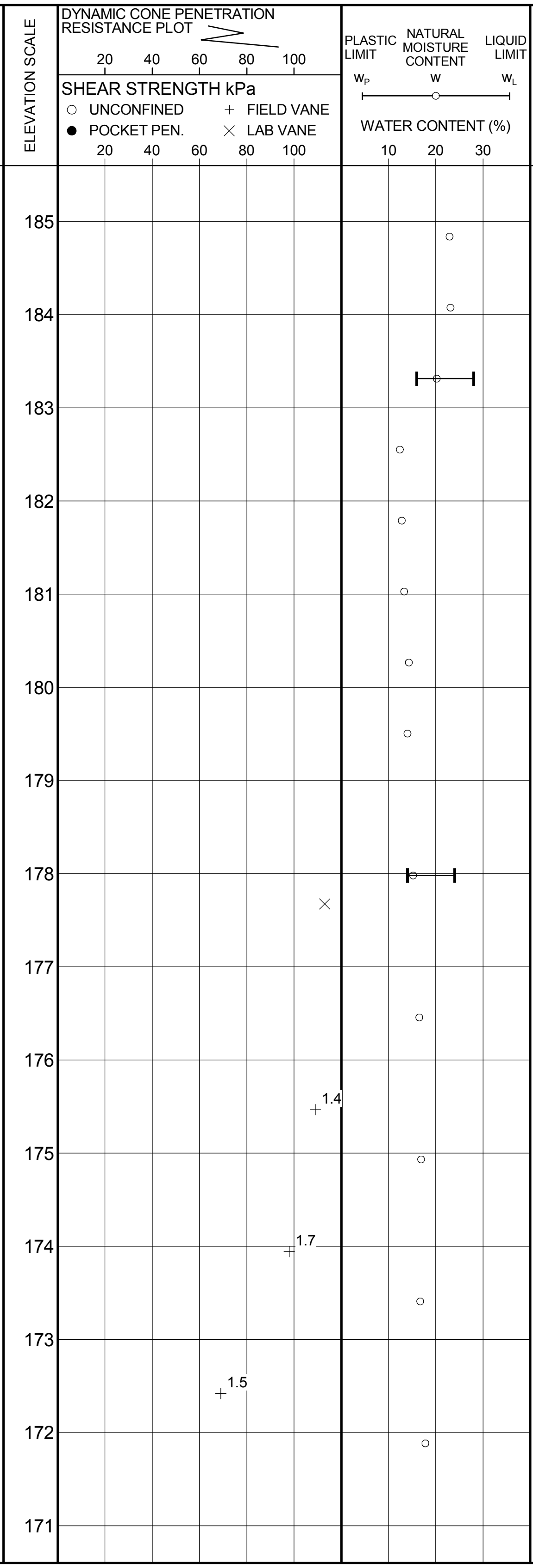
+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B11-2

1 OF 3 **METRIC**

W.P. RFP No. 09-54-1007
LOCATION 4678193.3N, 334624.8
ORIGINATED BY LC
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE Apr 30, 11 - May 3, 11
CHECKED BY MSO

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 185.6 | Road Shoulder Surface | | | | | | | | | | |
| 0.0 | **FILL**<br>Crushed Limestone<br>Silty sand and Gravel | | | | | | | | | | |
| 185.0<br>0.6 | **FILL**<br>Weathered silty clay mixed with topsoil | 1 | SS | 6 | | | | | | | |
| 184.1<br>1.5 | Mottled Brown-Grey<br>**CLAYEY SILT**<br>Some sand, trace gravel<br>Stiff | 2 | SS | 7 | | | | | | | |
| | -Sandy fissures to approx. 3m | 3A,B | SS | 27 | | | 2 | 21 | 49 | 29 | |
| | Brown<br>Hard | 4 | SS | 34 | | | | | | | |
| | Grey<br>Very stiff | 5 | SS | 23 | | | | | | | |
| | | 6 | SS | 19 | | | | | | | |
| | Stiff | 7 | SS | 11 | | | | | | | |
| | | 8 | SS | 15 | | | | | | | |
| | | | | | | | | | | | -switch to wash boring with casing |
| | | 9 | TW | PH | | 21.8 | 2 | 32 | 39 | 27 | |
| | | 10 | TW | PH | | | | | | | |
| | | | VT | | | | | | | | |
| | | 11 | TW | PH | | | | | | | |
| | | | VT | | | | | | | | |
| | | 12 | TW | PH | | | | | | | |
| | | | VT | | | | | | | | -end of drilling April 30; continue May 2 |
| | | 13 | TW | PH | | | | | | | |



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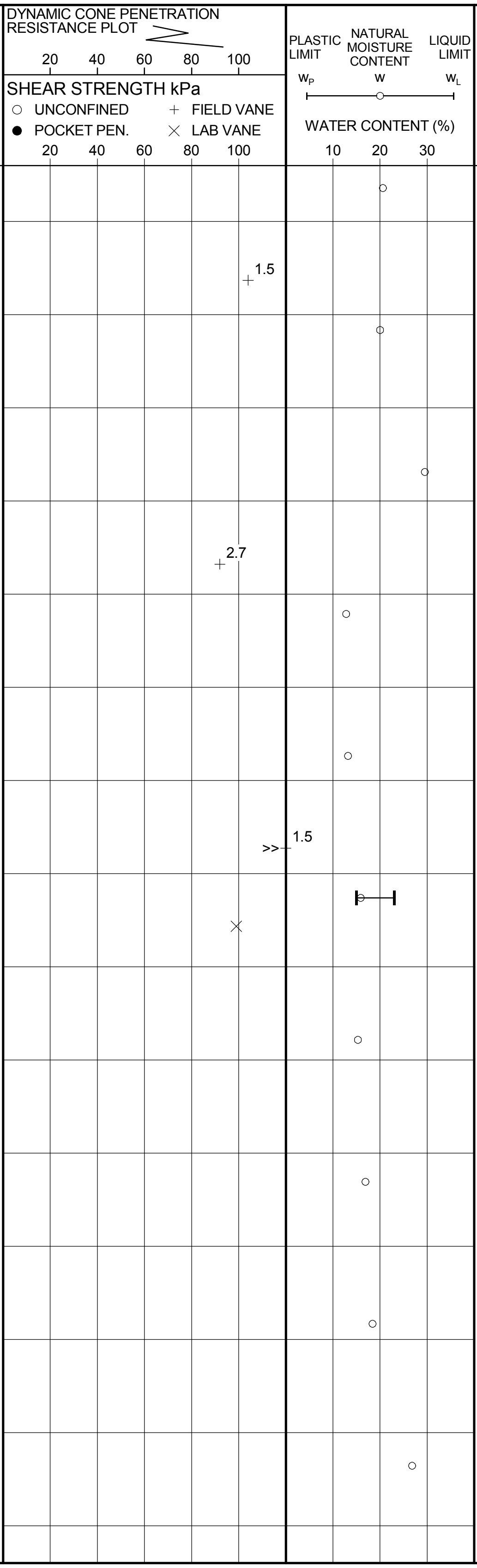
+ 3, × 3: Numbers refer to Sensitivity
○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B11-2

2 OF 3 **METRIC**

W.P. RFP No. 09-54-1007 LOCATION 4678193.3N, 334624.8 ORIGINATED BY LC
DIST HWY WEP BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE Apr 30, 11 - May 3, 11 CHECKED BY MSO

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Mottled Brown-Grey<br>**CLAYEY SILT**<br>Some sand, trace gravel *(continued)* | 14 | SS | PH | 170 | | -no recovery with shelby tube; pushed split spoon for sample | | | | |
| | | | VT | | 169 | | | | | | |
| | | 15 | SS | PH | 168 | | -no recovery with shelby tube; pushed split spoon for sample | | | | |
| | -Numerous thin silt lenses/inclusions | 16 | TW | PH | 167 | | | | | | |
| | | | VT | | 166 | | | | | | |
| | | 17 | TW | PH | 165 | | | | | | |
| | Stiff | 18 | TW | PH | 164 | | | | | | |
| | | | VT | | 163 | | | | | | |
| | | 19 | TW | PH | 162 | 22.2 | | 3 | 26 | 46 | 23 |
| | Very stiff | 20 | SS | 28 | 161 | | | | | | |
| | | 21 | SS | 29 | 160 | | | | | | |
| | | 22 | SS | 27 | 159<br>158 | | | | | | |
| | Stiff | 23 | SS | 13 | 157<br>156 | | | | | | |



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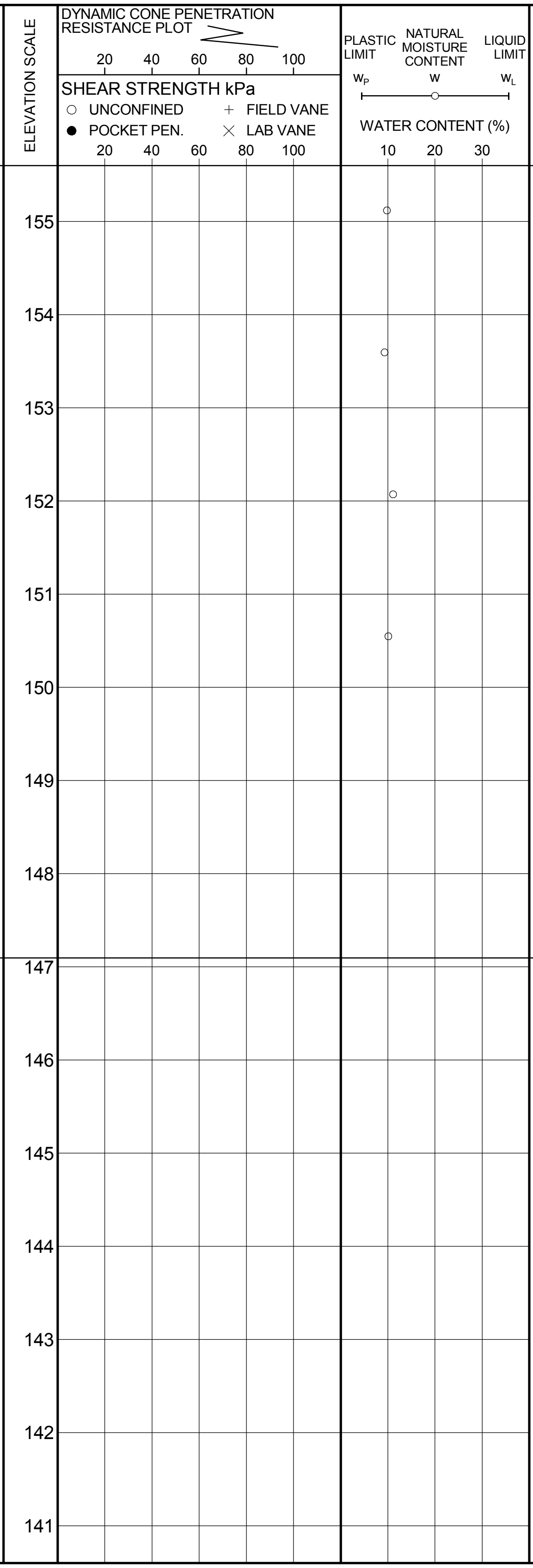
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

## RECORD OF BOREHOLE No B11-2 3 OF 3 METRIC

W.P. RFP No. 09-54-1007
LOCATION 4678193.3N, 334624.8
ORIGINATED BY LC
DIST ____ HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE Apr 30, 11 - May 3, 11
CHECKED BY MSO

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 155.4<br>30.2 | Grey **CLAYEY FINE SAND** Some gravel Very stiff | 24 | SS | 27 | | | -end of drilling May 2; continue May 3 |
| 154.1<br>31.5 | Grey **SAND** and **GRAVEL** Limestone and shale fragments, inferred cobbles, boulders Very dense | 25 | SS | 85 | | | -flushed out casing and drove split spoon but had no penetration; attempted coring but had no recovery; NQ barrel getting jammed in casing; flushed out casing again but getting no return water and pump pressure spiking; remove core barrel and wash casing down to 36.67m; collected wash samples |
| | | 26 | SS | 118 | | | |
| | | 27 | SS | 130 | | | |
| | | 28 | RC | | | | |
| 148.9<br>36.7 | Light Grey **LIMESTONE** Light Grey, fine grained, cherty Highly fractured, numerous stylolites, faintly laminated to bedded, light blue-grey inclusion | 29 | RC | | | | RQD = 17%<br>TCR = 100%<br>SCR = 58% |
| | | 30 | RC | | | | RQD =100%<br>TCR = 100%<br>SCR = 100% |
| 147.1<br>38.5 | **END OF BOREHOLE** | | | | | | |



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

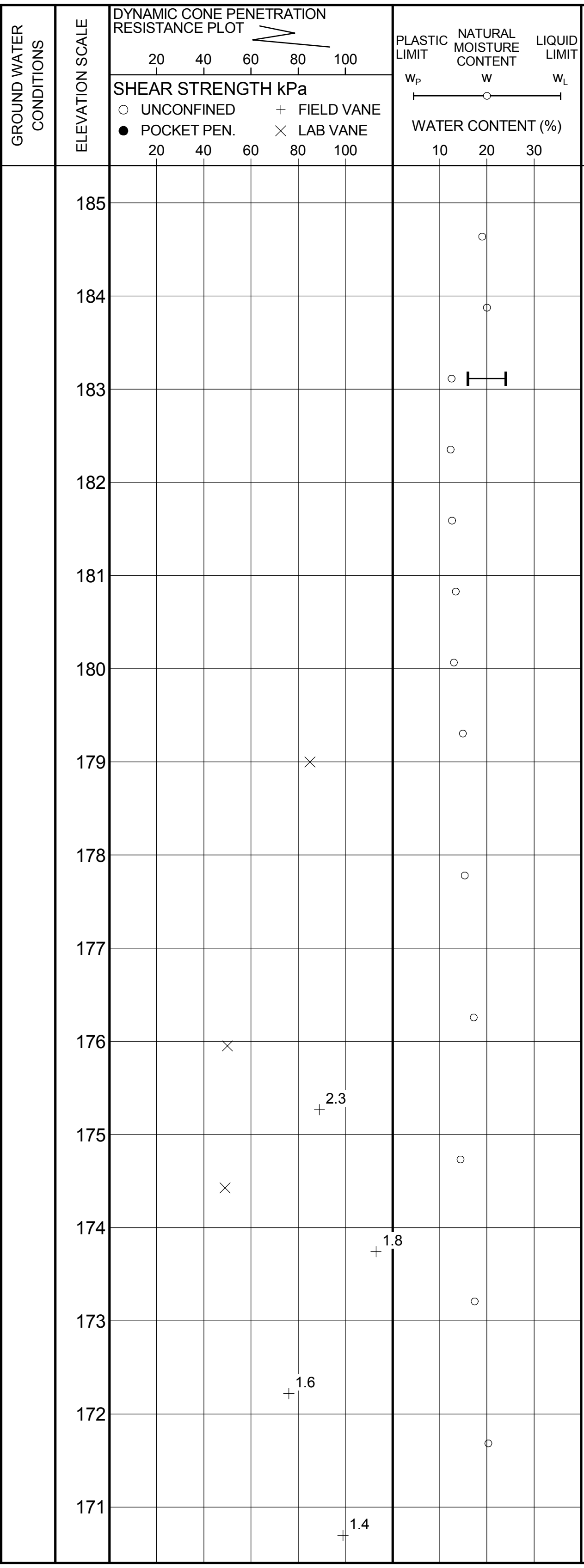
# RECORD OF BOREHOLE No B11-3

1 OF 3

**METRIC**

W.P. RFP No. 09-54-1007
LOCATION 4678179.9N, 334655.1E
ORIGINATED BY TP
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE May 7, 11 - May 9, 11
CHECKED BY MSO

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|
| 185.4 | Road Shoulder Surface | | | | | | | | |
| 0.0 | Grey **FILL** Crushed limestone silty sand and gravel | | | | | | | | |
| 184.6<br>0.8 | Grey-Brown **FILL** Silty Clay with organics | 1A, B | SS | 6 | | | | | |
| | | 2A, B | SS | 7 | | | | | |
| 183.3<br>2.1 | Orange-Brown **CLAYEY SILT** Some sand, trace gravel Very stiff | 3 | SS | 23 | | | | | |
| | Hard | 4 | SS | 31 | | 2 | 33 | 37 | 28 |
| | | 5A, B | SS | 17 | | | | | |
| | Very stiff<br>Grey | | | | | | | | |
| | Stiff | 6 | SS | 14 | | | | | |
| | | 7 | SS | 12 | | | | | |
| | | 8 | TW | PH | 21.9 | | | | |
| | | 9 | TW | PH | | | | | |
| | | 10 | TW | PH | 21.5 | | | | |
| | | | VT | | | | | | |
| | | 11 | TW | PH | | | | | |
| | | | VT | | | | | | |
| | | 12 | TW | PH | 21.4 | | | | |
| | | | VT | | | | | | |
| | | 13 | TW | PH | | | | | |
| | | | VT | | | | | | |



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

# RECORD OF BOREHOLE No B11-3

2 OF 3 **METRIC**

| | | | |
|---|---|---|---|
| W.P. RFP No. 09-54-1007 | LOCATION 4678179.9N, 334655.1E | | ORIGINATED BY TP |
| DIST | HWY WEP | BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY SS |
| DATUM Geodetic | | DATE May 7, 11 - May 9, 11 | CHECKED BY MSO |

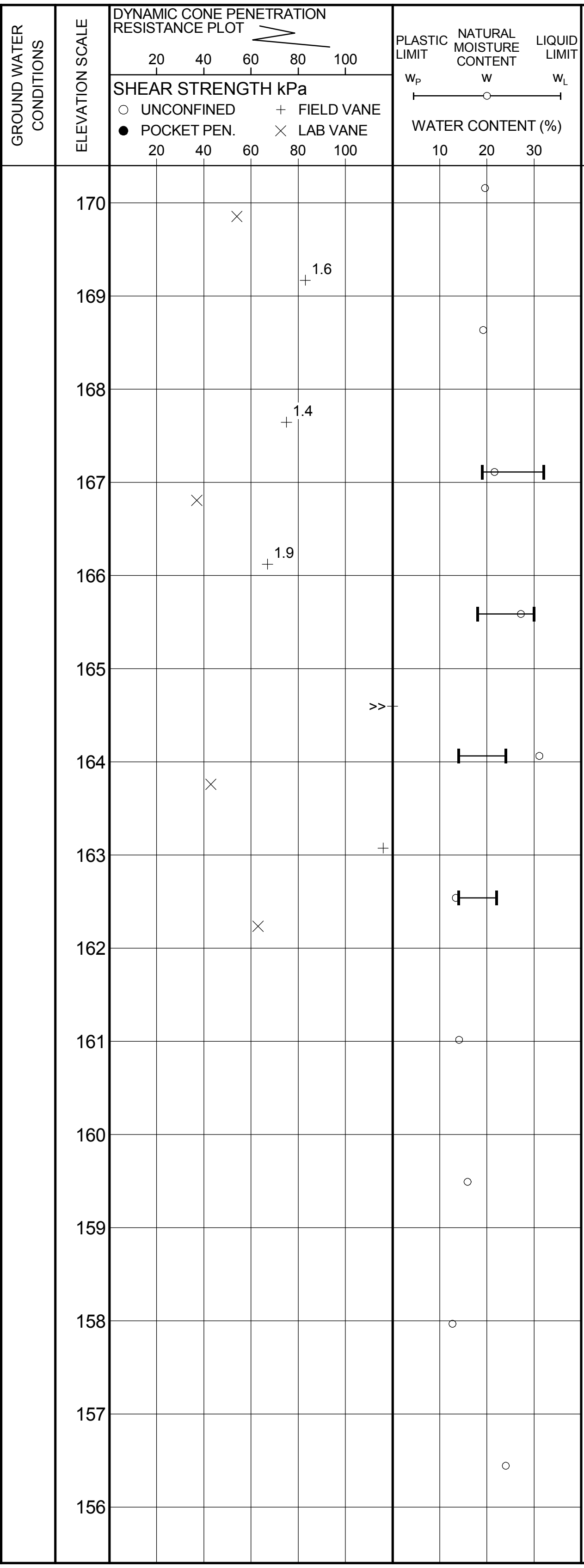
## SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| | Orange-Brown<br>**CLAYEY SILT**<br>Some sand, trace gravel *(continued)* |
| 167.4 / 18.0 | Grey<br>**SILTY CLAY**<br>Pink and black clay nodules |
| 164.2 / 21.2 | Grey<br>**CLAYEY SILT**<br>Some sand, trace gravel<br>-Alternating soft to stiff<br>-75mm dense gravel, some sand to fine sand, dry<br>Very stiff to hard |
| 157.7 / 27.7 | Grey<br>**SILTY CLAY**<br>Some silt, trace gravel<br>Stiff |
| 156.9 / 28.5 | Grey<br>**SANDY SILT**<br>to<br>**FINE SAND**<br>Dense |

Continued Next Page

## SAMPLES

| NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | REMARKS | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|
| 14 | TW | PH | 20.8 | | | | | |
| | VT | | | | | | | |
| 15 | TW | PH | | -switch to wash boring with casing | | | | |
| | VT | | | | | | | |
| 16 | TW | PH | 19.8 | -end of drilling May 8; restart May 9 | | | | |
| | VT | | | | | | | |
| 17 | SS | PH | | | | | | |
| | VT | | | | | | | |
| 18 | TW | PH | 22.4 | | | | | |
| | VT | | | | | | | |
| 19 | TW | PH | 22.3 | | 9 | 26 | 46 | 19 |
| 20 | SS | 32 | | | | | | |
| 21 | SS | 40 | | | | | | |
| 22A, B | SS | 19 | | | | | | |
| 23 | SS | 33 | | | | | | |



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

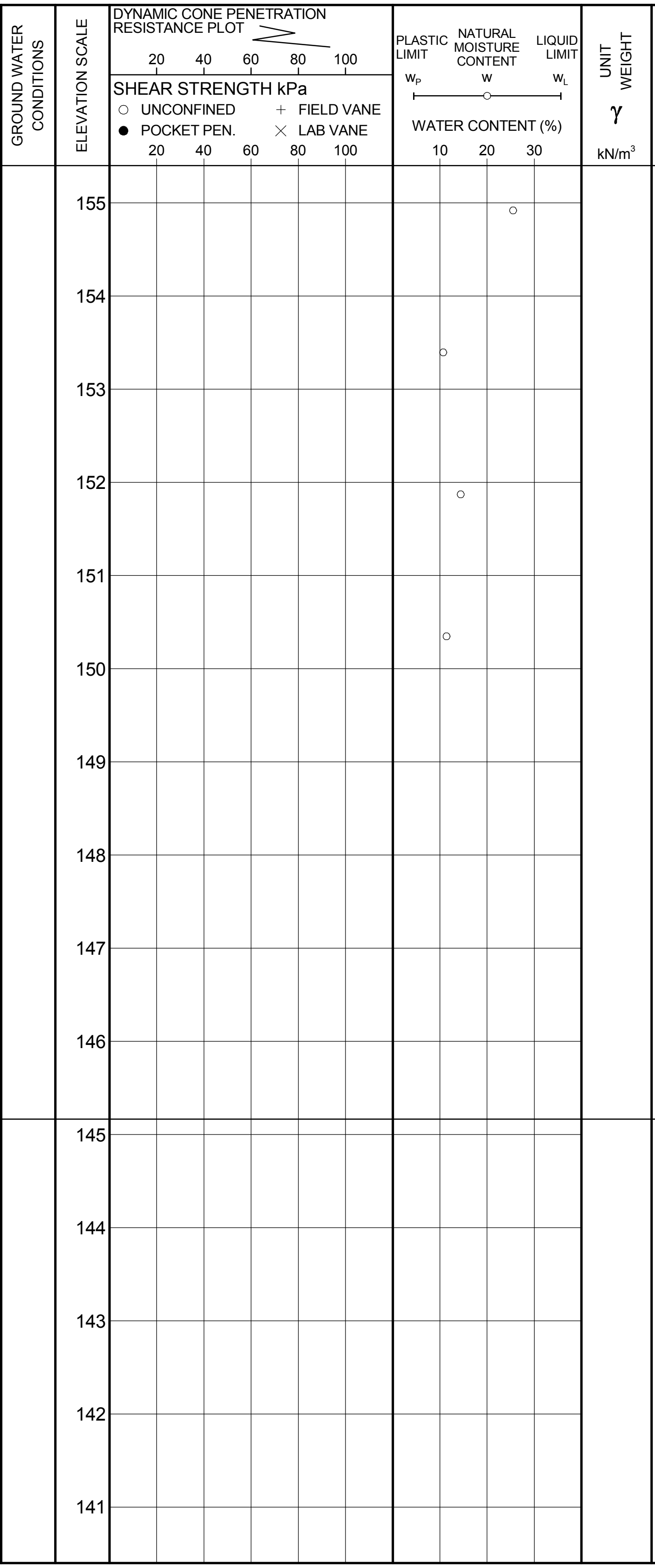
# RECORD OF BOREHOLE No B11-3

3 OF 3

**METRIC**

W.P. RFP No. 09-54-1007
LOCATION 4678179.9N, 334655.1E
ORIGINATED BY TP
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE May 7, 11 - May 9, 11
CHECKED BY MSO

| SOIL PROFILE | | SAMPLES | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GR SA SI CL |
| 154.8<br>30.6 | Grey **SILTY CLAY** Medium Plasticity Some silt, trace gravel | 24A,B,C | SS | 16 | |
| 153.4<br>32.0 | Grey and Black **SAND** and **GRAVEL** and inferred cobbles and boulders Dense to very dense | 25 | SS | 47 | |
| | | 26 | SS | 58 | |
| | | 27 | SS | 84 | 17 26 57 |
| | Limestone and granite gravel (up to 75mm diameter) sampled | 1 | Grab Sample | | |
| | | 2 | Grab Sample | | |
| 148.5<br>36.9 | Light Grey **LIMESTONE** Fine grained, cherty, vuggy with calcite, chalcopyrite and celestite crystals present stylolites throughout, moderately porous, semi-hard trace fossils. Light Blue-grey inclusions | 1 | RC | | RQD = 0%<br>TCR = 57%<br>SCR = 0% |
| | | 2 | RC | | -end of drilling May 9; finish May 10<br>RQD = 80%<br>TCR = 100%<br>SCR = 80% |
| 145.2<br>40.2 | **END OF BOREHOLE** | | | | |



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

# RECORD OF BOREHOLE No B11-4

1 OF 3 **METRIC**

| | | | |
|---|---|---|---|
| W.P. RFP No. 09-54-1007 | LOCATION 4678195.6N, 334679.3E | | ORIGINATED BY TP |
| DIST | HWY WEP | BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Auger | COMPILED BY SS |
| DATUM Geodetic | | DATE May 10, 11 - May 11, 11 | CHECKED BY MSO |

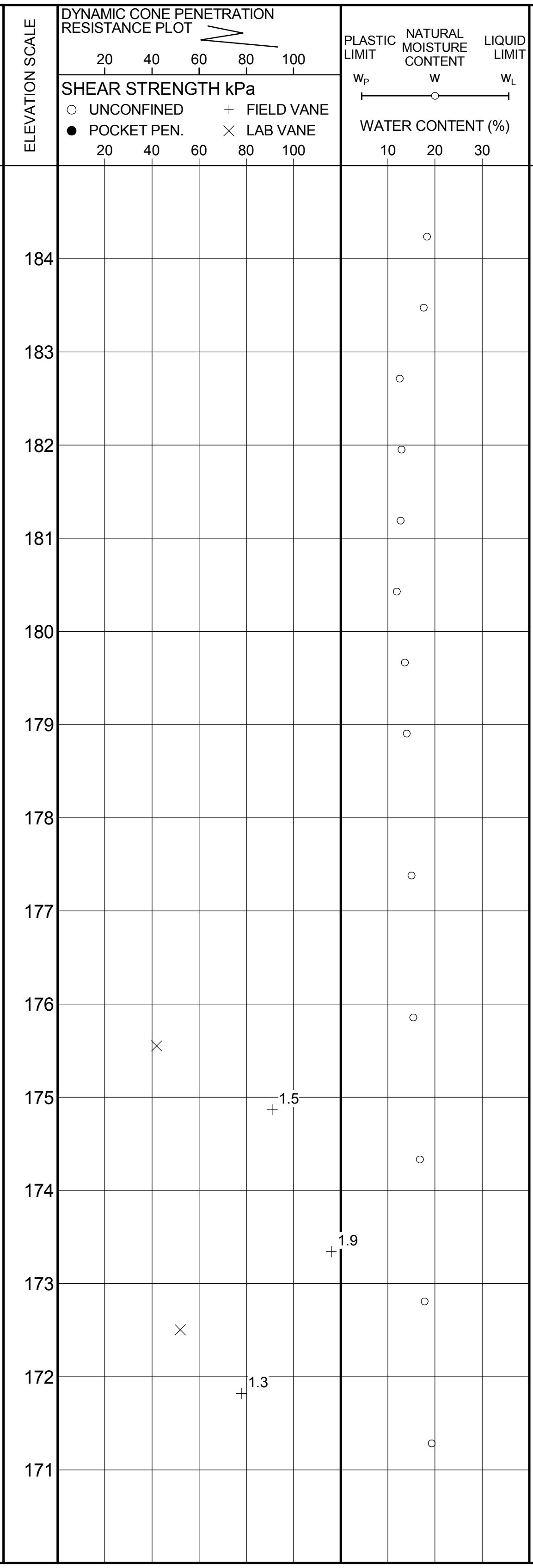
## SOIL PROFILE

| ELEV DEPTH | DESCRIPTION |
|---|---|
| 185.0 | Ground Surface |
| 0.0 | Black<br>**TOPSOIL**<br>Organic Clay |
| 184.2<br>0.8 | Mottled Brown-Orange<br>**CLAYEY SILT**<br>Some sand, trace gravel<br>Stiff<br>Brown<br>Hard<br>Very stiff<br>Stiff |
| 170.2<br>14.8 | |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|
| 1 | SS | 7 | | | | | |
| 2 | SS | 10 | | | | | |
| 3 | SS | 30 | | | | | |
| 4 | SS | 32 | | | | | |
| 5 | SS | 22 | | | | | |
| 6 | SS | 16 | | | | | |
| 7 | SS | 15 | | | | | |
| 8 | TW | PH | 22.0 | 4 | 33 | 41 | 25 |
| 9 | TW | PH | | | | | |
| 10 | TW | PH | 21.7 | | | | |
| | HV | | | | | | |
| 11 | TW | PH | | | | | |
| | HV | | | | | | |
| 12 | TW | PH | 21.3 | | | | |
| | HV | | | | | | |
| 13 | TW | PH | | | | | |

GROUND WATER CONDITIONS: ∇ at approx. elev. 182.2



## REMARKS & GRAIN SIZE DISTRIBUTION (%)

-Deep Vibrating Wire Piezometer (VWP) installed in sampled borehole. Shallow and mid-depth VWP installed in adjacent boring at N 4678197.3, E334679.6; Shallow Spider Magnet (MG) installed in adjacent boring at N4678199, E334679.1; Mid-depth MG installed in adjacent boring at N4678199.8, E334678.1

-MG installed at 8.4m below ground surface

-VWP #P10 installed at 10.0m below ground surface

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

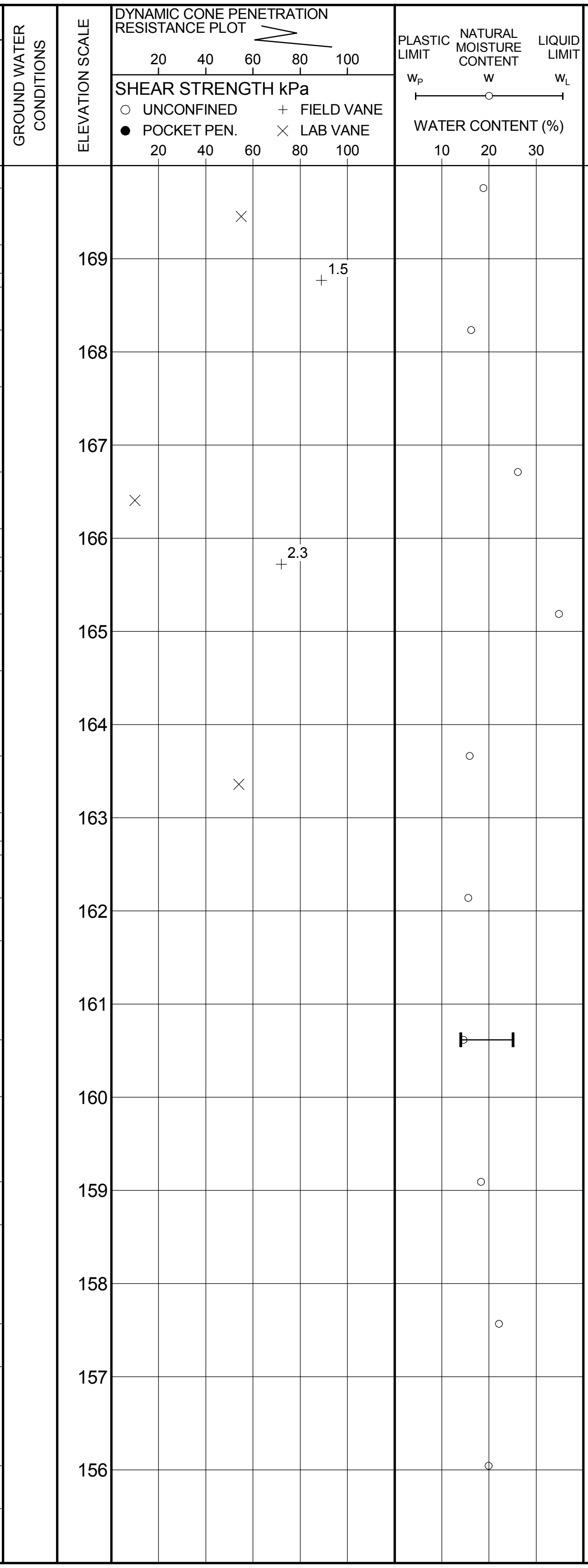
# RECORD OF BOREHOLE No B11-4

2 OF 3

**METRIC**

W.P. RFP No. 09-54-1007
LOCATION 4678195.6N, 334679.3E
ORIGINATED BY TP
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Auger
COMPILED BY SS
DATUM Geodetic
DATE May 10, 11 - May 11, 11
CHECKED BY MSO

| SOIL PROFILE | | SAMPLES | | | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | | GR | SA | SI | CL |
| | Grey **SANDY SILT** Some pink clay nodules *(continued)* | 14 | TW | PH | 21.1 | | | | | |
| | | | HV | | | | | | | |
| | | 15 | TW | PH | | -switch to wash boring with casing | | | | |
| 167.2<br>17.8 | Grey **SILTY CLAY** Some pink clay nodules | | | | | | | | | |
| | | 16 | TW | PH | 19.9 | -VWP #P18 installed at 18.3m below ground surface<br>-MG installed at 18.3m below ground surface<br>-end of drilling May 10; continue May 11 | | | | |
| | | | HV | | | | | | | |
| | | 17 | TW | PH | | | | | | |
| | -Increasing sand content, silt pockets | 18 | TW | PH | 22.1 | | | | | |
| | | | HV | | | | | | | |
| 162.4<br>22.6 | Grey **CLAYEY SILT** Some sand, trace gravel | 19 | SS | PH | | -no recovery with shelby tube (damanged by inferred cobble); retrieved sample by driving split spoon | | | | |
| | | 20 | TW | PH | 21.9 | | 4 | 23 | 43 | 21 |
| | -Laminated with silt and fine sand | | | | | | | | | |
| 158.8<br>26.2 | Grey **SANDY SILT** Some clay Very dense | 21A,B | SS | 56 | | | | | | |
| 158.2<br>26.8 | Grey **SILTY CLAY** Very stiff | 22 | SS | 30 | | | | | | |
| 156.7<br>28.3 | Grey **SILTY SAND** Trace clay Very dense | 23 | SS | 68 | | | | | | |
| 155.1 | | | | | | | | | | |



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

# RECORD OF BOREHOLE No B11-4

3 OF 3

**METRIC**

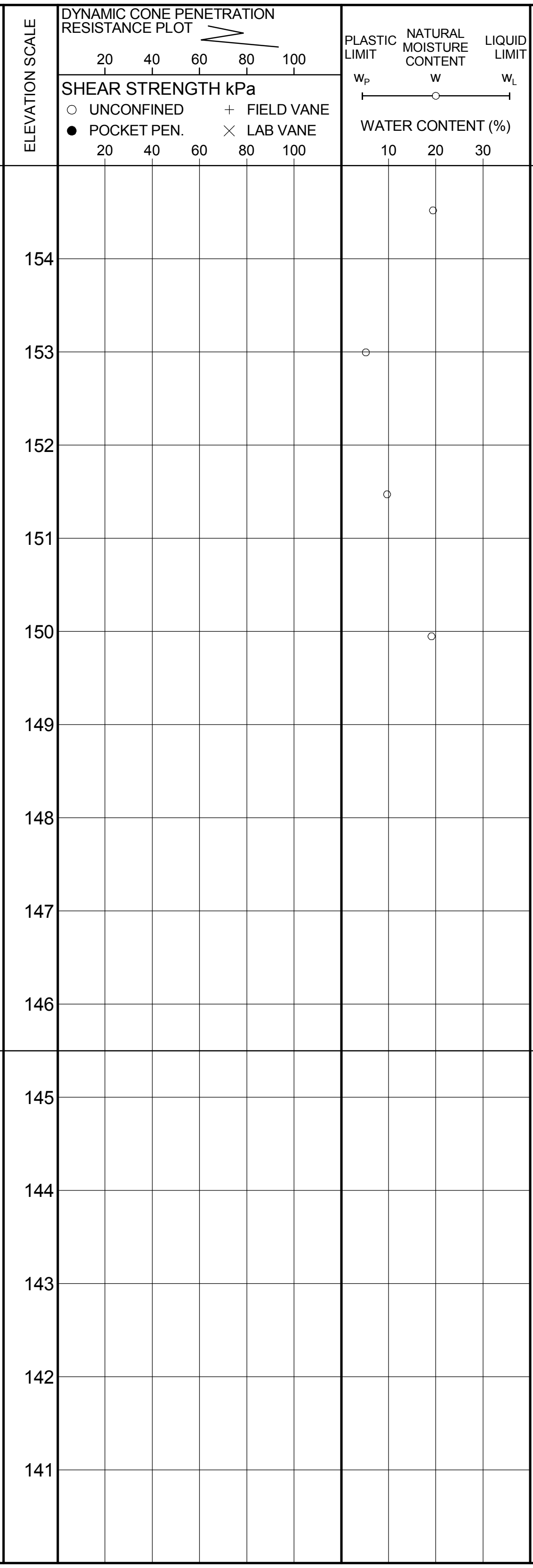
W.P. RFP No. 09-54-1007
LOCATION 4678195.6N, 334679.3E
ORIGINATED BY TP
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Auger
COMPILED BY SS
DATUM Geodetic
DATE May 10, 11 - May 11, 11
CHECKED BY MSO

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | STRAT PLOT | SAMPLES NUMBER | SAMPLES TYPE | SAMPLES "N" VALUES | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|
| 29.9 | Grey<br>**SILTY CLAY**<br>Very stiff *(continued)* | | | | | | | |
| 154.4<br>30.6 | Grey to Black<br>**SAND** and **GRAVEL**<br>Some shale and limestone fragments, inferred cobbles and boulders<br>Very dense | | 24A, B | SS | 25 | | | |
| | | | 25 | SS | 52/ 150mm | | | |
| | | | 26 | SS | 116/ 150mm | | | 0 97 3<br>(gradation completed on sample collected from wash boring) |
| | | | 27 | SS | 100/ 75mm | | | |
| | | | | | | | | -VWP #P36 installed at 36m below ground surface<br>-sand heaved inside casing; unable to sample |
| 146.7<br>38.3 | Light Grey<br>**LIMESTONE**<br>Fine grained, bedded, stylolites present throughout<br>Semi-hard, porous | | 1 | RC | | | | RQD = 91%<br>TCR = 100% |
| 145.5<br>39.5 | **END OF BOREHOLE** | | | | | | | |

Piezometric levels in VWP #P10:
May 24, 2011: EL. 183.4m
June 25, 2011: EL. 183.0m
July 10, 2011: EL. 182.5m
July 24, 2011: EL. 182.1m

Piezometric levels in VWP #P18:
May 24, 2011: EL. 182.2m
June 25, 2011: EL. 181.9m
July 10, 2011: EL. 181.7m
July 24, 2011: EL. 181.4m

Piezometric levels in VWP #P36:
May 24, 2011: EL. 177.3m
June 25, 2011: EL. 176.7m
July 10, 2011: EL. 176.8m
July 24, 2011: EL. 175.8m



+3, ×3: Numbers refer to Sensitivity

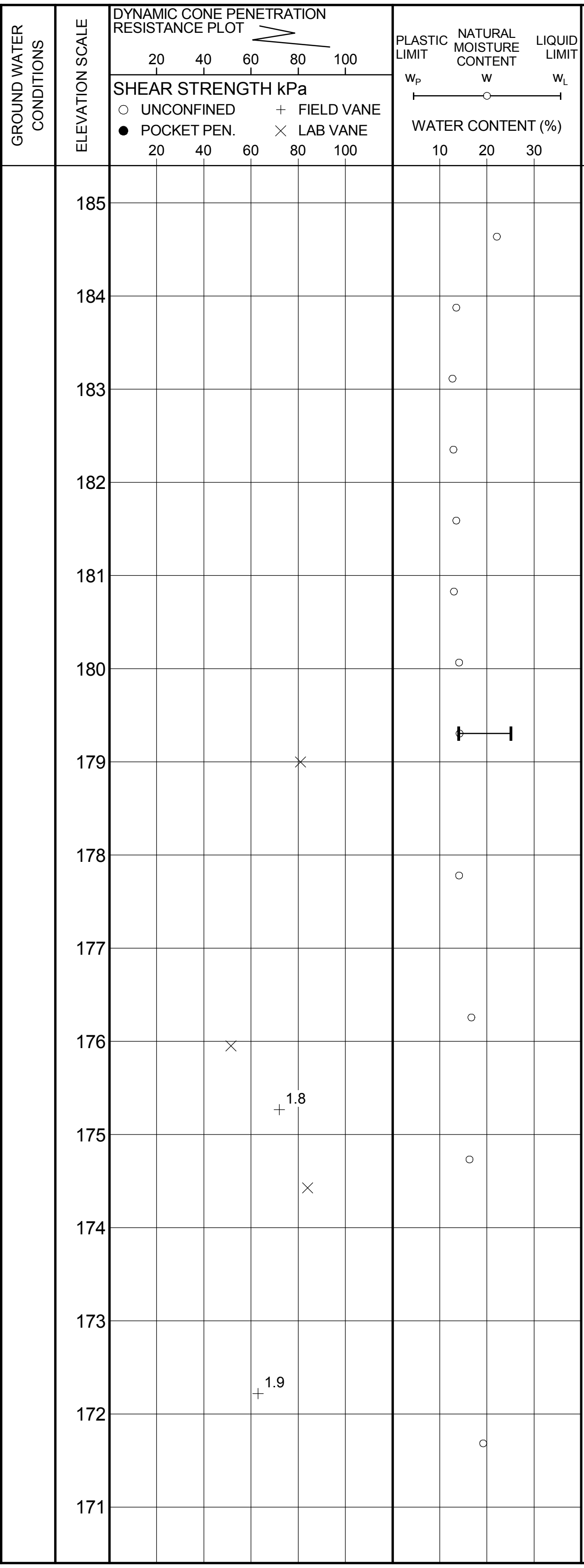
○3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B11-5

1 OF 3 **METRIC**

W.P. RFP No. 09-54-1007
LOCATION 4678173.2N, 334733.7E
ORIGINATED BY TA
DIST ____ HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE May 25, 11 - May 30, 11
CHECKED BY MSO

| ELEV<br>DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 185.4 | Ground Surface | | | | | |
| 0.0 | **TOPSOIL** | | | | | |
| 184.9<br>0.5 | Mottled Brown-Grey<br>**CLAYEY SILT**<br>Some sand, trace gravel<br>Firm | 1 | SS | 7 | | |
| | Brown<br>Trace fissures<br>Very stiff | 2 | SS | 25 | | |
| | | 3 | SS | 28 | | |
| | | 4 | SS | 30 | | |
| | Grey<br>Trace silt layers<br>Stiff | 5 | SS | 12 | | |
| | | 6 | SS | 12 | | |
| | | 7 | SS | 10 | | |
| | | 8 | TW | PH | | 3 32 40 25 |
| | Trace pink clay nodules | | | | | |
| | | 9 | SS | PH | 21.4 | -no recovery with shelby tube; retrieved sample by pushing split spoon |
| | | 10 | TW | PH | 21.6 | |
| | | | VT | | | |
| | | | | | | -end of drilling May 25; continue May 26 |
| | | 11 | TW | PH | 21.3 | |
| | -Horizontal sand and gravel seams encountered | 12 | TW | PH | | -shelby tube damaged; possible cobble |
| | | | VT | | | |
| | | 13 | TW | PH | | |



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

# RECORD OF BOREHOLE No B11-5

2 OF 3

**METRIC**

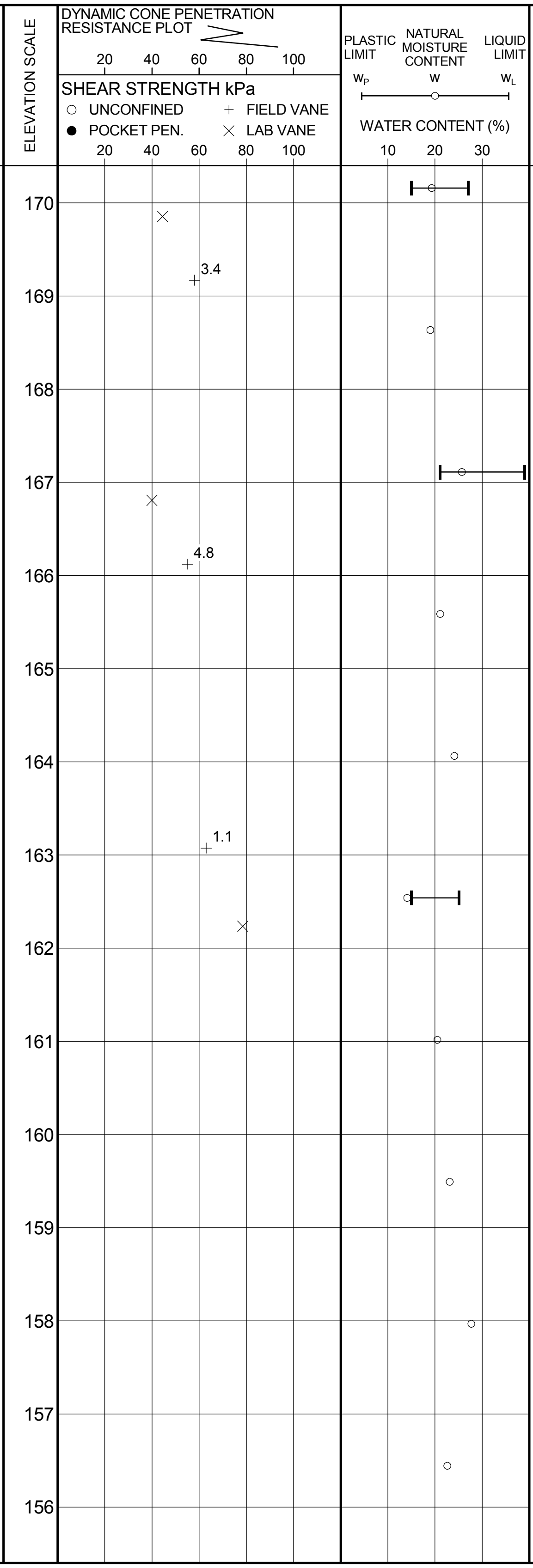
W.P. RFP No. 09-54-1007
LOCATION 4678173.2N, 334733.7E
ORIGINATED BY TA
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE May 25, 11 - May 30, 11
CHECKED BY MSO

## SOIL PROFILE

| ELEV DEPTH | DESCRIPTION |
|---|---|
| | Grey **CLAYEY SILT**<br>-Sand layers |
| 167.1<br>18.3 | Grey **SILTY CLAY** Stiff |
| 165.9<br>19.5 | Grey **CLAYEY SILT** Some sand, trace gravel<br>Firm to stiff |
| 159.5<br>25.9 | Grey **SANDY SILT** Very dense |
| 158.0<br>27.4 | Grey **CLAYEY SILT** Trace sand, interbedded sand layers<br>Very stiff |
| 156.4<br>29.0 | Grey **SILTY FINE SAND** Dense |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL | REMARKS |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | -start wash bore with casing |
| 14 | TW | PH | 20.8 | 3 | 25 | 40 | 33 | |
| | VT | | | | | | | |
| 15 | TW | PH | | | | | | |
| 16 | TW | PH | 19.0 | | 1 | 48 | 51 | |
| | VT | | | | | | | |
| 17 | TW | PH | | | | | | |
| 18 | SS | PH | | | | | | -no recovery with shelby tube; possible cobbles; retrieved sample by pushing split spoon |
| | VT | | | | | | | -erratic torque readings; possible gravel or cobbles |
| 19 | TW | PH | 21.5 | 4 | 24 | 44 | 28 | |
| 20 | TW | PH | | | | | | -end of drilling May 26; continue May 27 |
| 21 | SS | 67 | | | | | | -unable to push shelby tube; sample retrieved by SPT |
| 22 | SS | 16 | | | | | | -end of drilling May 27; continue May 29 |
| 23 | SS | 50 | | | | | | |



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+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B11-5

3 OF 3 **METRIC**

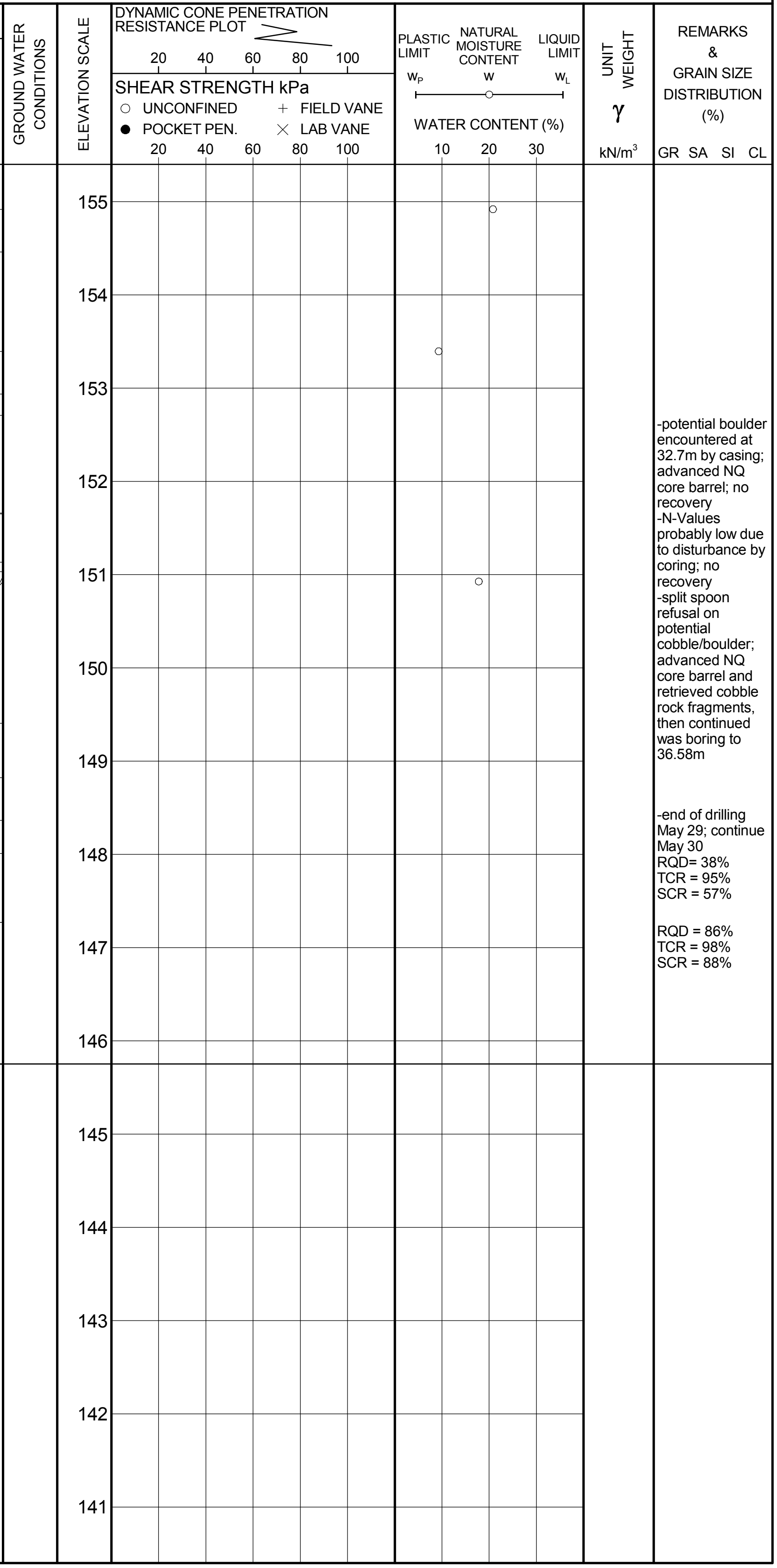
| | | | | | |
|---|---|---|---|---|---|
| W.P. | RFP No. 09-54-1007 | LOCATION | 4678173.2N, 334733.7E | ORIGINATED BY | TA |
| DIST | | BOREHOLE TYPE | CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY | SS |
| HWY | WEP | | | | |
| DATUM | Geodetic | DATE | May 25, 11 - May 30, 11 | CHECKED BY | MSO |

SOIL PROFILE

| ELEV<br>DEPTH | DESCRIPTION |
|---|---|
| 154.9<br>30.5 | |
| 154.8<br>30.6 | Grey **SILTY CLAY** Some sand, trace gravel Very stiff |
| 154.6<br>30.8 | Grey **SILTY SAND** Trace gravel |
| 154.5<br>30.9 | Grey **SILTY CLAY** Trace sand, trace gravel |
| | Grey and Black **SAND** and **GRAVEL** Inferred cobbles and boulders; unknown fine sand/silt/clay content<br>-Cobble and rock fragments |
| 148.0<br>37.4 | White to Grey **LIMESTONE** Laminated, stylolites present, fine grained, pitted between 37.98m and 38.10m |
| 147.3<br>38.1 | Grey to Brown **LIMESTONE** -Fractures are visible between 38.6 and 38.7m |
| 145.8<br>39.6 | **END OF BOREHOLE** |

SAMPLES

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---|---|---|---|
| 24 | SS | 26 | |
| 25 | SS | 55 | |
| 26 | RC | | -potential boulder encountered at 32.7m by casing; advanced NQ core barrel; no recovery<br>-N-Values probably low due to disturbance by coring; no recovery |
| 27 | SS | 4 | |
| 28 | SS | 3 | -split spoon refusal on potential cobble/boulder; advanced NQ core barrel and retrieved cobble rock fragments, then continued was boring to 36.58m |
| 29 | RC | | |
| 30 | SS | 67 | -end of drilling May 29; continue May 30 |
| 31 | RC | | RQD= 38%<br>TCR = 95%<br>SCR = 57% |
| 32 | RC | | RQD = 86%<br>TCR = 98%<br>SCR = 88% |



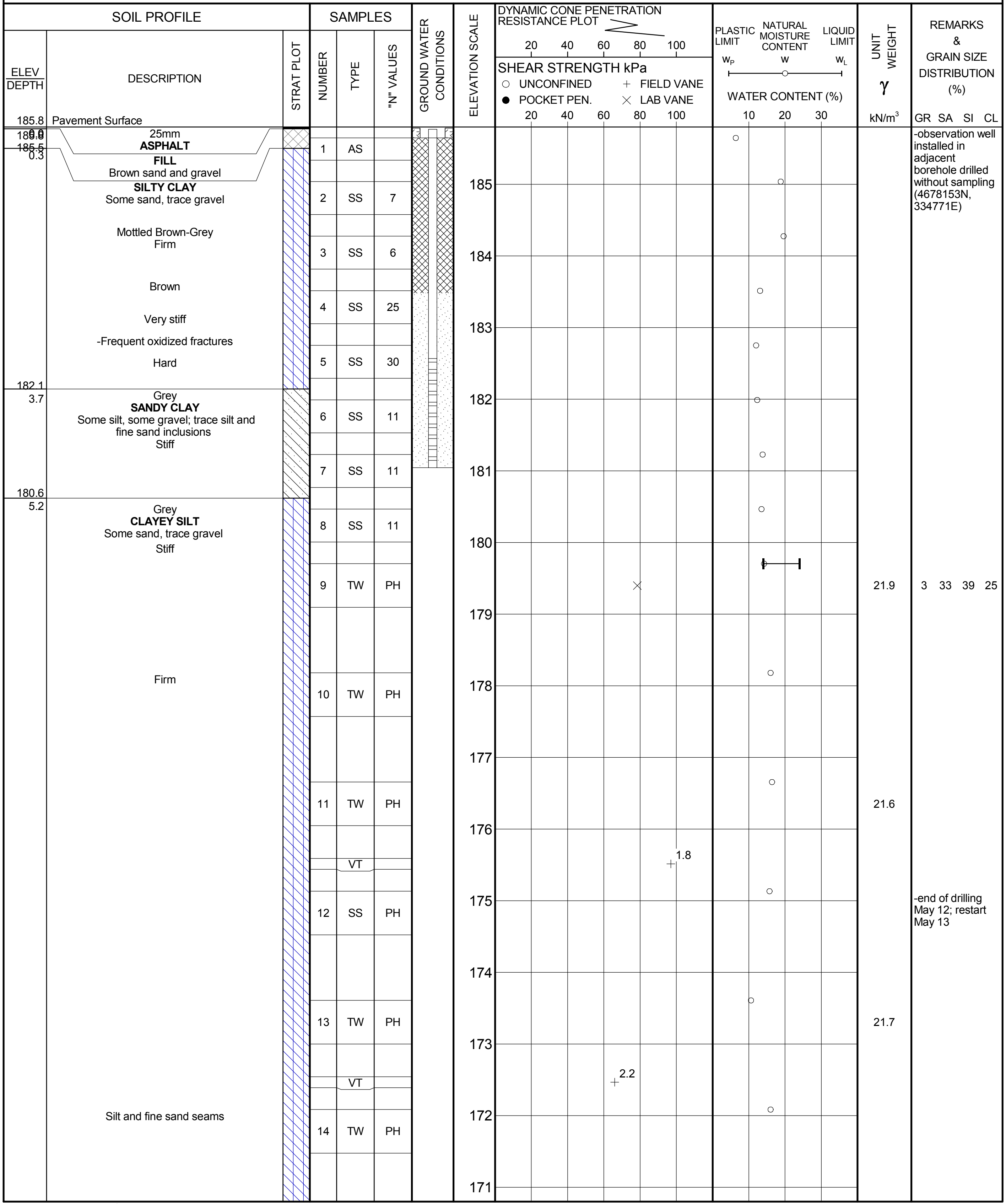
$+^3, \times^3$: Numbers refer to Sensitivity

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

ONTARIO MOT SW8801.1004.101.GPJ ONTARIO MOT.GDT 29/09/11

# RECORD OF BOREHOLE No B11-6/HGMW-05 1 OF 3 METRIC

W.P. RFP No. 09-54-1007
LOCATION 4678153.5N, 334772.4E
ORIGINATED BY NB
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE May 12, 11 - May 14, 11
CHECKED BY MSO



| ELEV DEPTH | DESCRIPTION |
|---|---|
| 185.8 | Pavement Surface |
| 185.8<br>0.0 | 25mm **ASPHALT** |
| 185.5<br>0.3 | **FILL** Brown sand and gravel |
| | **SILTY CLAY** Some sand, trace gravel<br>Mottled Brown-Grey Firm<br>Brown<br>Very stiff<br>-Frequent oxidized fractures<br>Hard |
| 182.1<br>3.7 | Grey **SANDY CLAY** Some silt, some gravel; trace silt and fine sand inclusions Stiff |
| 180.6<br>5.2 | Grey **CLAYEY SILT** Some sand, trace gravel Stiff<br>Firm<br>Silt and fine sand seams |

| NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|
| 1 | AS | | | -observation well installed in adjacent borehole drilled without sampling (4678153N, 334771E) |
| 2 | SS | 7 | | |
| 3 | SS | 6 | | |
| 4 | SS | 25 | | |
| 5 | SS | 30 | | |
| 6 | SS | 11 | | |
| 7 | SS | 11 | | |
| 8 | SS | 11 | | |
| 9 | TW | PH | 21.9 | 3 33 39 25 |
| 10 | TW | PH | | |
| 11 | TW | PH | 21.6 | |
| | VT | | | |
| 12 | SS | PH | | -end of drilling May 12; restart May 13 |
| 13 | TW | PH | 21.7 | |
| | VT | | | |
| 14 | TW | PH | | |

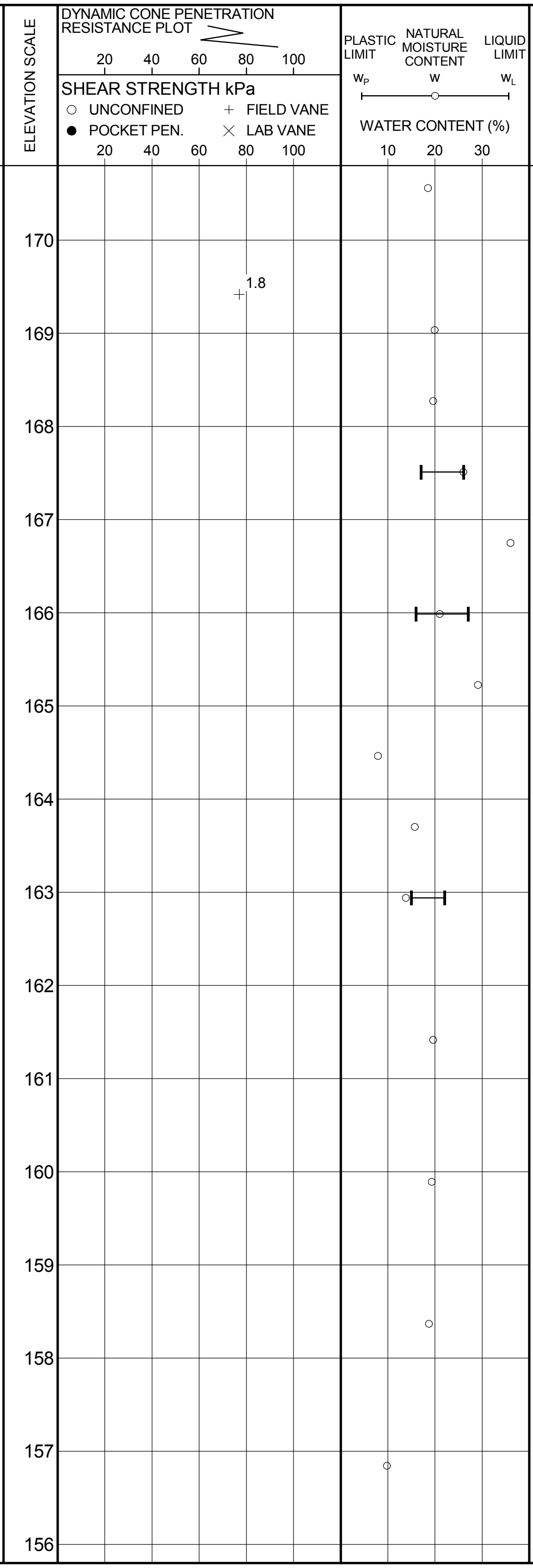
Continued Next Page

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

# RECORD OF BOREHOLE No B11-6/HGMW-05 2 OF 3 METRIC

| | | | | | |
|---|---|---|---|---|---|
| W.P. | RFP No. 09-54-1007 | LOCATION | 4678153.5N, 334772.4E | ORIGINATED BY | NB |
| DIST | | HWY WEP | BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY | SS |
| DATUM | Geodetic | DATE | May 12, 11 - May 14, 11 | CHECKED BY | MSO |

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | REMARKS | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|
| | Grey **CLAYEY SILT** | | | | | | | | | |
| | | 15 | TW | PH | 21.8 | -start wash boring with casing | | | | |
| | | | VT | | | | | | | |
| | | 16 | TW | PH | | | | | | |
| | | 17 | SS | PH | | | | | | |
| | | 18 | TW | PH | 20.3 | | 3 | 18 | 47 | 31 |
| | | 19 | SS | PH | | -no recovery with shelby tube; sample retrieved by pushing split spoon hydraulically | | | | |
| | Dark Grey to Black Clayey till clumps (100mm dia.) embedded | 20 | TW | PH | 19.9 | | 1 | 17 | 48 | 35 |
| | | 21 | SS | PH | | | | | | |
| | Stiff, moist | 22 | SS | PH | | -end of drilling May 13; restart May 14 | | | | |
| | | 23 | SS | PH | | -end of drilling May 13,; restart May 14 | | | | |
| | -Inferred cobbles | 24 | TW | PH | | | 3 | 27 | 48 | 22 |
| | | 25 | SS | PH | | -end of drilling May 13; restart May 14 | | | | |
| | | 26 | SS | 24 | | -unable to push shelby tube | | | | |
| | -Fine sand and silt seams; limestone fragments; wet | 27 | SS | 58 | | | | | | |
| | Light Grey **SANDY CLAY** Trace gravel Very stiff Moist | 28 | SS | 20 | | | | | | |



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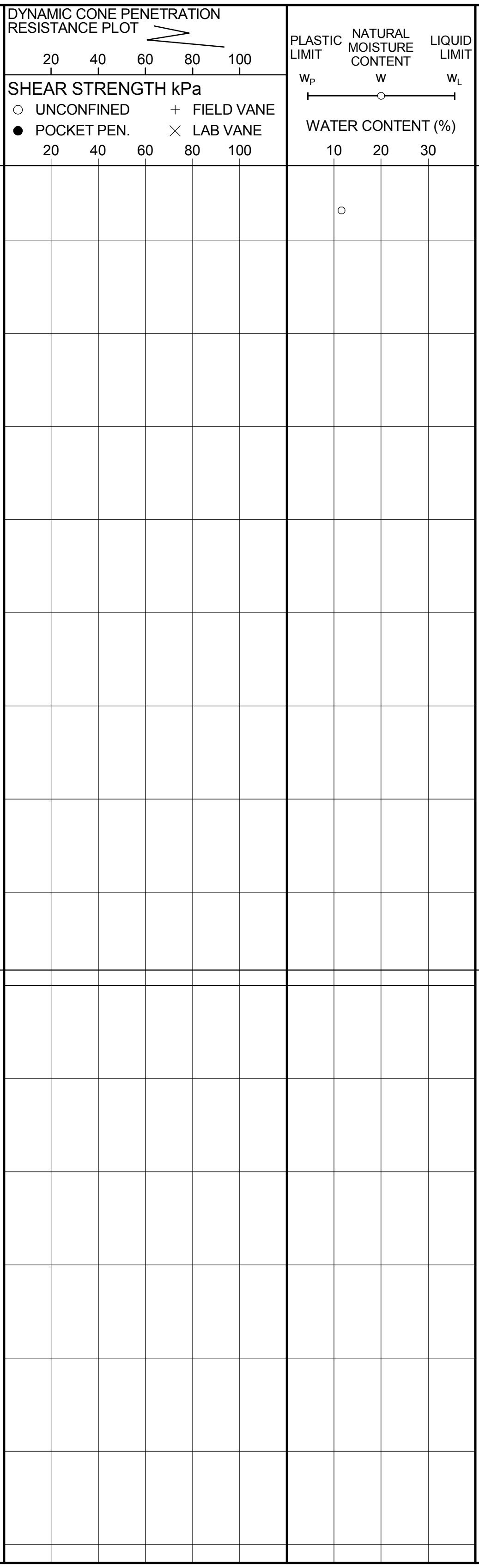
+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B11-6/HGMW-05 3 OF 3 METRIC

| | | | |
|---|---|---|---|
| W.P. RFP No. 09-54-1007 | | LOCATION 4678153.5N, 334772.4E | ORIGINATED BY NB |
| DIST | HWY WEP | BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY SS |
| DATUM Geodetic | | DATE May 12, 11 - May 14, 11 | CHECKED BY MSO |

| ELEV<br>DEPTH | SOIL PROFILE: DESCRIPTION | STRAT PLOT | SAMPLES: NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m$^3$ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|
| | Light Grey<br>**SANDY CLAY**<br>Trace gravel<br>Very stiff<br>Moist *(continued)*<br>-Rock fragments in sample, inferred cobbles and boulders | | 29 | SS | 27 | | 155 | | |
| 154.4<br>31.4 | **SAND** and **GRAVEL**<br>Inferred cobbles and boulders<br>Dense | | | | | | 154 | | |
| | -Grey fine sand observed in wash water | | 30 | SS | | | | | |
| | | | | | | | 153 | | -end of drilling May 14; restart May 16 |
| | -Shale and granite fragments<br>With extensive (inferred) cobbles and boulders<br>-Grey fine sand observed in wash water | | 31 | SS | | | 152 | | |
| | | | | | | | 151 | | |
| | -Retrieved rock fragments in split spoon | | 32 | SS | | | | | |
| | -Rounded coarse gravel, variable mineralogy; retrieved with core barrel | | 33 | WS | | | 150 | | |
| | -Sample included rock fragments | | 34 | SS | 54 | | 149 | | |
| 148.7<br>37.1 | Light Grey<br>**LIMESTONE**<br>Fine grained, laminated, numerous stylolites, faintly porous, semi-hard Light blue-grey nodules. Becoming pitted between 38.56m to 38.68m. | | 35 | RC | | | 148 | | RQD = 62%<br>TCR = 75%<br>SCR = 52% |
| 147.2<br>38.6 | **END OF BOREHOLE**<br>Water levels in observation well:<br>May 24, 2011: EL. 184.9m<br>June 4, 2011: EL. 184.7m<br>June 25, 2011: EL. 184.7m<br>July 10, 2011: EL. 184.6m<br>July 24, 2011: EL. 184.4m<br>July 29, 2011: EL. 184.9m | | | | | | 147 | | |
| | | | | | | | 146 | | |
| | | | | | | | 145 | | |
| | | | | | | | 144 | | |
| | | | | | | | 143 | | |
| | | | | | | | 142 | | |
| | | | | | | | 141 | | |



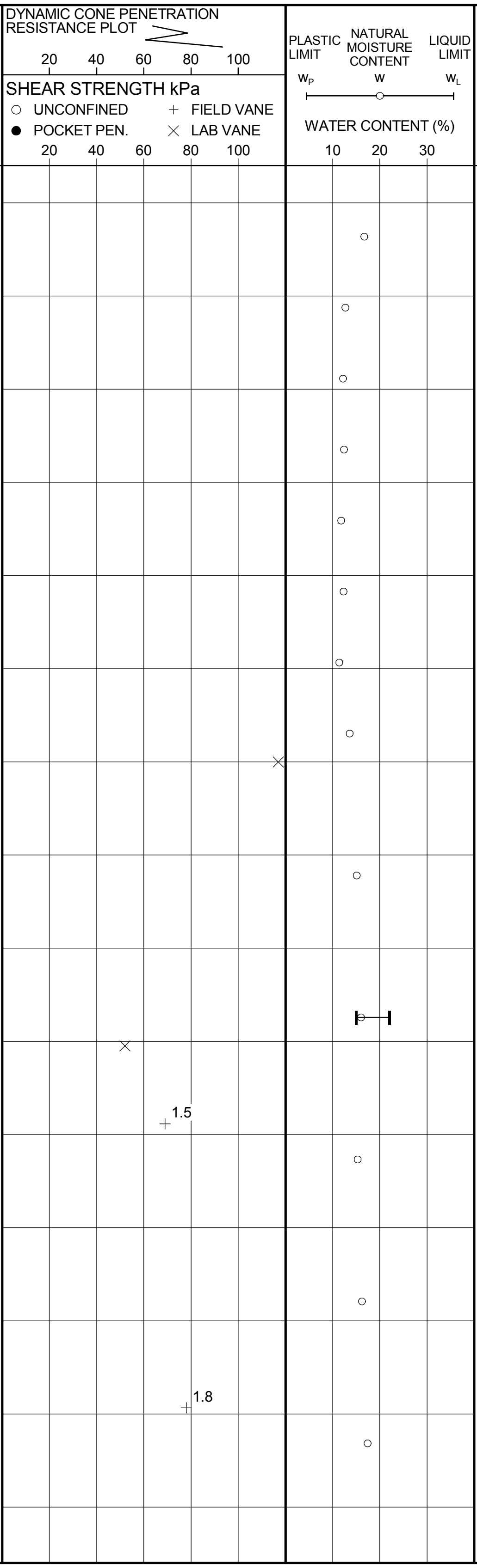
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B11-7

1 OF 3 **METRIC**

| | | | | | |
|---|---|---|---|---|---|
| W.P. | RFP No. 09-54-1007 | LOCATION | 4678125.8N, 334810.2E | ORIGINATED BY | NB |
| DIST | | HWY WEP | BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY | SS |
| DATUM | Geodetic | DATE | May 9, 11 - May 11, 11 | CHECKED BY | MSO |

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 185.4 | Ground Surface | | | | | | |
| 0.0<br>185.1<br>0.3 | 300mm Black **TOPSOIL** Organic Clay | | | | 185 | | -vibrating wire piezometers (VWP) installed in adjacent boring N4678126.1N, E334812.3 |
| | Mottled Brown-Grey **CLAYEY SILT** Some sand, trace gravel Firm | 1 | SS | 6 | | | |
| | Brown Stiff | 2 | SS | 13 | 184 | | -Inclinometer casing installed in sampled borehole |
| | Hard | 3 | SS | 31 | 183 | | |
| | | 4 | SS | 33 | 182 | | |
| | Grey | 5 | SS | 18 | | | |
| | Very stiff | 6 | SS | 10 | 181 | | |
| | Stiff | 7 | SS | 10 | 180 | | |
| | | 8 | TW | PH | 179 | 22.3 | |
| | | 9 | TW | PH | 178<br>177 | | |
| | | 10 | TW | PH | 176 | 21.6 | 3 31 40 26 |
| | | | VT | | 175 | | -VWP #P10 installed at 9.91m below ground surface |
| | | 11 | TW | PH | 174 | | |
| | | 12 | TW | PH | 173 | | |
| | | | VT | | 172 | | |
| | | 13 | TW | PH | 171 | | |



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

# RECORD OF BOREHOLE No B11-7

2 OF 3

**METRIC**

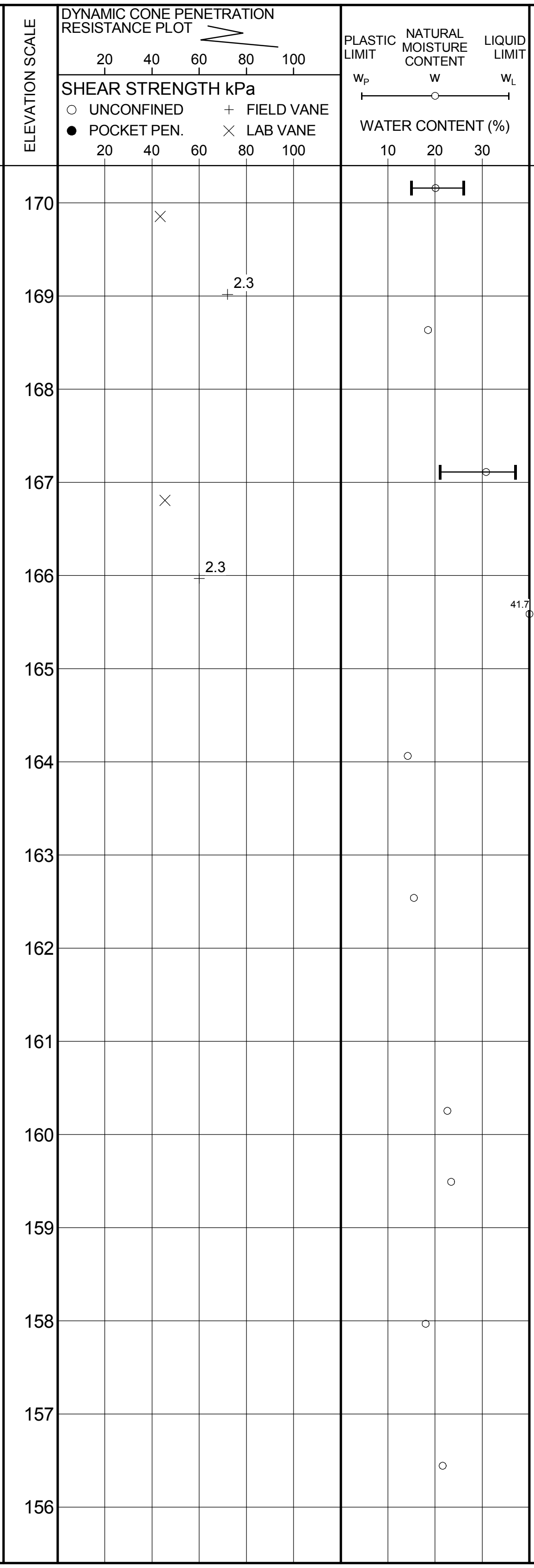
| | | | | | |
|---|---|---|---|---|---|
| W.P. | RFP No. 09-54-1007 | LOCATION | 4678125.8N, 334810.2E | ORIGINATED BY | NB |
| DIST | HWY WEP | BOREHOLE TYPE | CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY | SS |
| DATUM | Geodetic | DATE | May 9, 11 - May 11, 11 | CHECKED BY | MSO |

## SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| | Grey<br>**CLAYEY SILT** |
| 167.7 / 17.7 | Grey<br>**SILTY CLAY**<br>Trace pink clay nodules |
| 164.7 / 20.7 | Grey<br>**CLAYEY SILT**<br>Some sand, trace gravel<br>Very stiff |
| 161.6 / 23.8 | Grey<br>**CLAYEY SILT**<br>and<br>**FINE SILTY SAND**<br>to<br>**SANDY SILT**<br>In alternating layers |
| 155.5 | |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL | REMARKS |
|---|---|---|---|---|---|---|---|---|
| 14 | TW | PH | 20.8 | 2 | 28 | 40 | 30 | -continue by wash boring with casing |
| | VT | | | | | | | |
| 15 | SS | PH | | | | | | -no recovery with shelby tube; retrieve sample by pushing split spoon |
| 16 | TW | PH | 19.5 | 1 | 9 | 38 | 52 | -VWP #P19 installed at 19.05m below ground surface |
| | VT | | | | | | | |
| 17 | TW | PH | | | | | | |
| 18 | SS | PH | | | | | | -no recovery with shelby tube (damaged) retrieved sample by pushing split spoon |
| | VT | | | | | | | |
| 19 | TW | PH | | | | | | |
| 20 | SS | 33 | | | | | | -no recovery with split spoon |
| 21A,B | SS | 64 | | | | | | |
| 22 | SS | 18 | | | | | | |
| 23 | SS | 42 | | | | | | |
| 24 | SS | 13 | | | | | | |



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+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B11-7

3 OF 3 **METRIC**

| | | | | | |
|---|---|---|---|---|---|
| W.P. | RFP No. 09-54-1007 | LOCATION | 4678125.8N, 334810.2E | ORIGINATED BY | NB |
| DIST | | HWY WEP | | | |
| | | BOREHOLE TYPE | CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY | SS |
| DATUM | Geodetic | DATE | May 9, 11 - May 11, 11 | CHECKED BY | MSO |

| SOIL PROFILE | | | SAMPLES | | | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● POCKET PEN. × LAB VANE<br>20 40 60 80 100 | PLASTIC LIMIT $w_P$ — NATURAL MOISTURE CONTENT $w$ — LIQUID LIMIT $w_L$<br>WATER CONTENT (%)<br>10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | | | | |
| 29.9 | Grey **SILTY CLAY** to **SANDY CLAY,** Trace gravel *(continued)* | | 25 | SS | 5 | | 155 | | ○ | | -split spoon sank 300mm under weight of hammer and rods |
| 153.9<br>31.5 | Grey **SAND** Trace Gravel Very dense | | 26 | SS | 59 | | 154<br>153<br>152<br>151 | | | | -weathered limestone (possible boulder)<br>-casing advanced past 33.5m without sampling |
| | | | 27 | SS | | | | | | | |
| 150.0<br>35.4 | Light Grey to white **LIMESTONE** Fine grained, semi-hard, faintly porous, with stylolites Light blue-grey nodules, fractured | | 28 | RC | | | 150<br>149<br>148 | | | | RQD = 55%<br>TCR = 100%<br>SCR = 55% |
| 147.7<br>37.7 | Light brown to tan grey **LIMESTONE** Fine grained, semi-hard, porous with stylolites | | 29 | RC | | | | | | | RQD = 82%<br>TCR = 100%<br>SCR = 90% |
| 146.9<br>38.5 | **END OF BOREHOLE** | | | | | | 147<br>146<br>145<br>144<br>143<br>142<br>141 | | | | |

Piezometric levels in VWP #P10:
May 24, 2011: EL. 184.5m
June 25, 2011: EL. 184.4m
July 24, 2011: EL. 183.9m

Plezometric levels in VWP #P19:
May 24, 2011: EL. 184.2m
June 25,2011: EL. 184.1m
July 24, 2011: EL. 183.6m

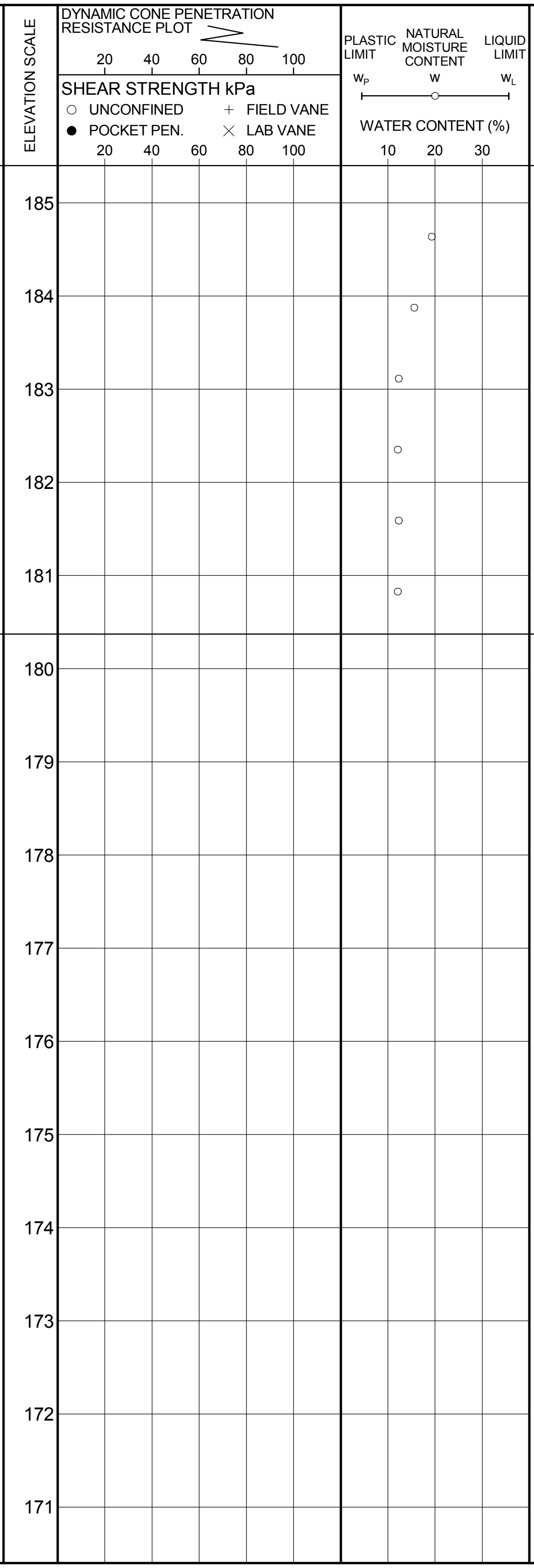
$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

# RECORD OF BOREHOLE No CPT B11-1

1 OF 1 **METRIC**

W.P. RFP No. 09-54-1007
LOCATION N4678195.2, E334595.9
ORIGINATED BY TA
DIST
HWY WEP
BOREHOLE TYPE CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE May 4, 11 - May 4, 11
CHECKED BY MSO

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES |
|---|---|---|---|---|
| 185.4 | Pavement Surface | | | |
| 185.2<br>0.0 | 150mm **ASPHALT** | | | |
| 0.2 | Grey **FILL** Crushed Limestone sand and gravel | | | |
| 184.7<br>0.7 | Mottled brown **CLAYEY SILT** Some sand, trace gravel -Horizontal and vertical fractures/fissures from 2m to 4m Stiff | 1 | SS | 11 |
| | | 2 | SS | 7 |
| | Brown | | | |
| | Hard | 3 | SS | 37 |
| | | 4 | SS | 39 |
| | Very stiff<br>Grey | 5 | SS | 22 |
| 180.4 | | 6 | SS | 16 |
| 5.0 | **END OF SAMPLED BOREHOLE** (Continue with CPT to refusal) | | | |



SOIL PROFILE | SAMPLES | STRAT PLOT | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

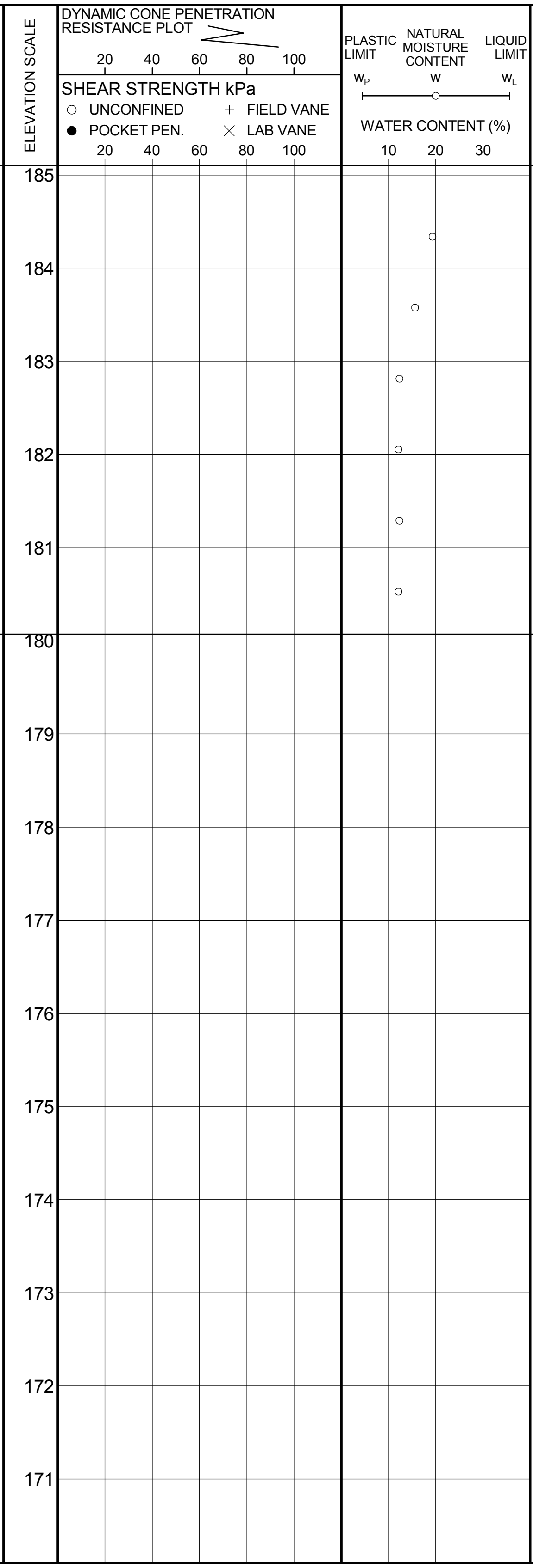
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No DMT B11-1

1 OF 1 **METRIC**

|  |  |  |  |  |  |
|---|---|---|---|---|---|
| W.P. | RFP No. 09-54-1007 | LOCATION | N4678223.6, E334579.2 | ORIGINATED BY | LC |
| DIST | HWY WEP | BOREHOLE TYPE | CME 55 - 200mm Dia. Continuous Flight Hollow Stem Augers | COMPILED BY | SS |
| DATUM | Geodetic | DATE | Apr 29, 11 - Apr 29, 11 | CHECKED BY | MSO |

| SOIL PROFILE |  |  | SAMPLES |  |  |  |  | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE |  |  |
| 185.1 | Ground Surface |  |  |  |  |  |  |  |  |
| 0.0<br>184.9<br>0.2 | 175mm Black organic Clay **TOPSOIL** |  |  |  |  |  | 185 |  |  |
|  | Mottled Brown-Grey **CLAYEY SILT** Some sand, trace gravel Stiff |  | 1 | SS | 11 |  | 184 |  |  |
|  | -Thin sand partings to 2.0m Very stiff |  | 2 | SS | 21 |  | 183 |  |  |
|  | Hard |  | 3 | SS | 51 |  |  |  |  |
|  | Brown<br>Grey |  | 4 | SS | 35 |  | 182 |  |  |
|  | Very stiff |  | 5 | SS | 17 |  | 181 |  |  |
|  | Stiff |  | 6 | SS | 13 |  |  |  |  |
| 180.1<br>5.0 | **END OF SAMPLED BOREHOLE** (continue with DMT to refusal) |  |  |  |  |  | 180 |  |  |



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

# RECORD OF CONE PENETRATION TEST CPT B11-1

**METRIC**

PROJECT Windsor-Essex Parkway

TEST DATE 5/4/2011 - 5/4/2011

SHEET 1 OF 2

LOCATION N4678195.2; 334595.9

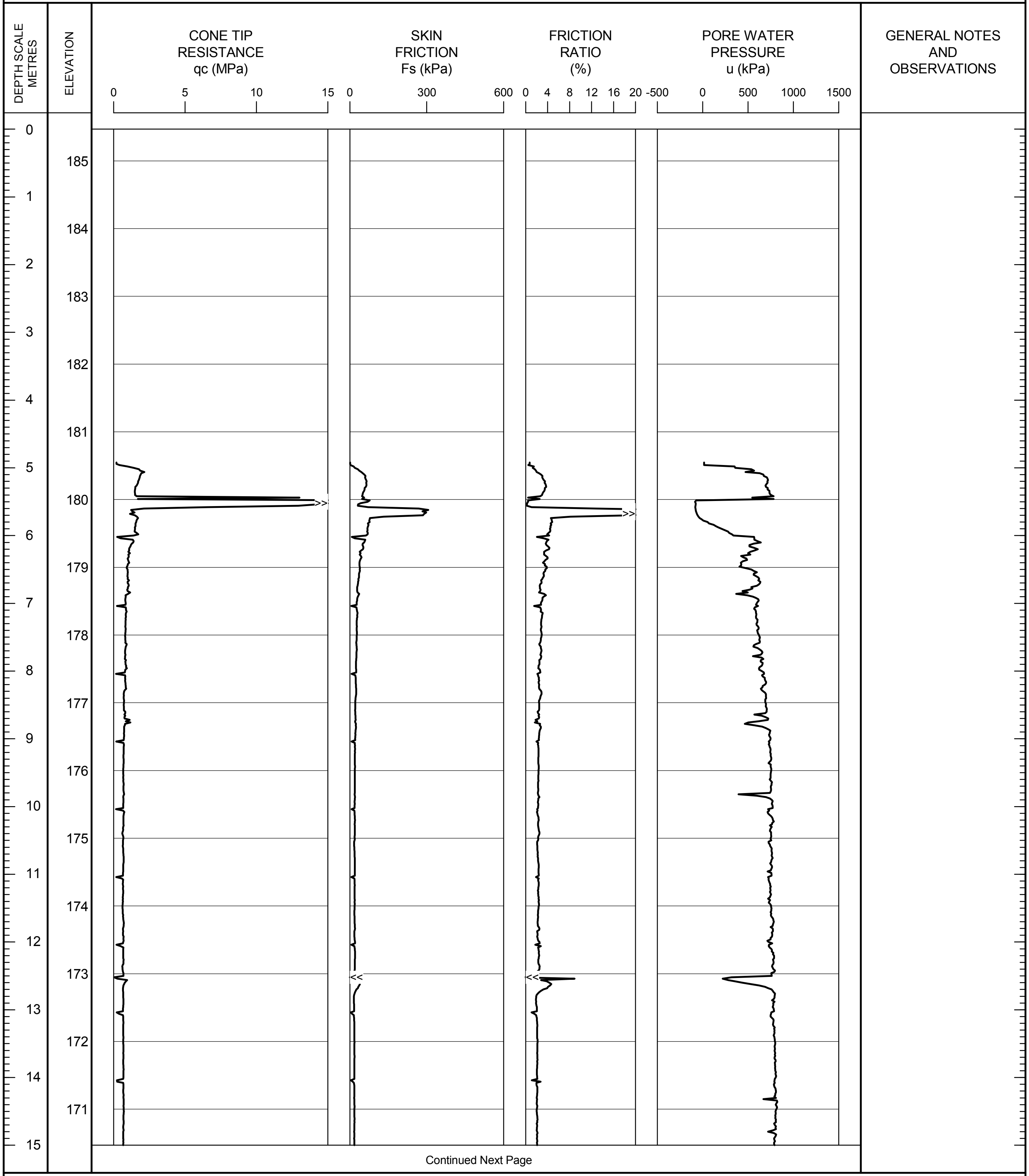
DATUM Geodetic

GROUND SURFACE ELEVATION: 185.5

PREDRILL DEPTH: 4.9

CORRECTION FACTOR A: 0.8

CORRECTION FACTOR B: 0



Continued Next Page

OPERATOR: TA

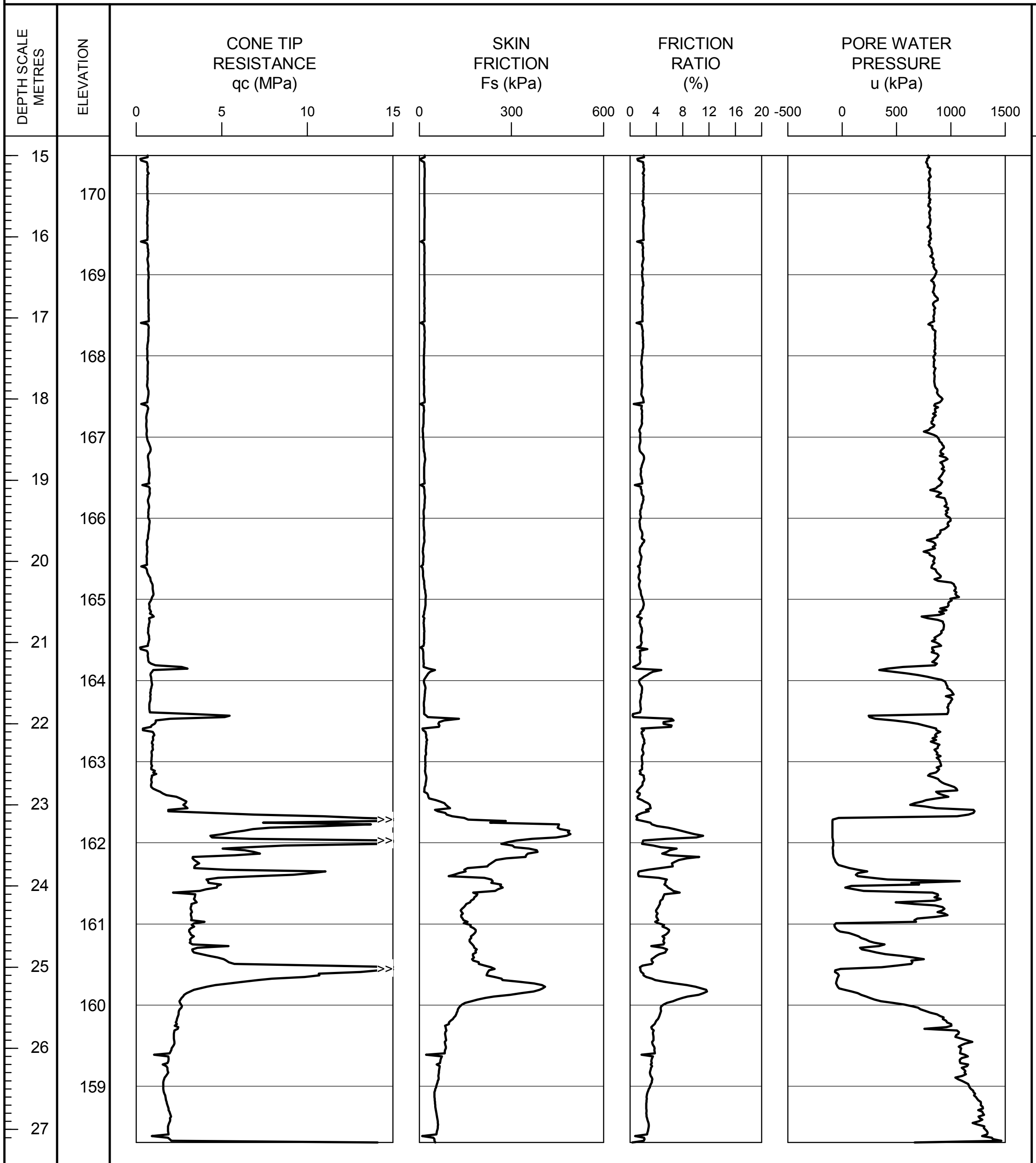
CHECKED: DD

## RECORD OF CONE PENETRATION TEST CPT B11-1

**METRIC**

| | | | | | |
|---|---|---|---|---|---|
| PROJECT | Windsor-Essex Parkway | TEST DATE | 5/4/2011 - 5/4/2011 | SHEET | 2 OF 2 |
| LOCATION | N4678195.2; 334595.9 | | | DATUM | Geodetic |

GROUND SURFACE ELEVATION: 185.5 PREDRILL DEPTH: 4.9 CORRECTION FACTOR A: 0.8 CORRECTION FACTOR B: 0



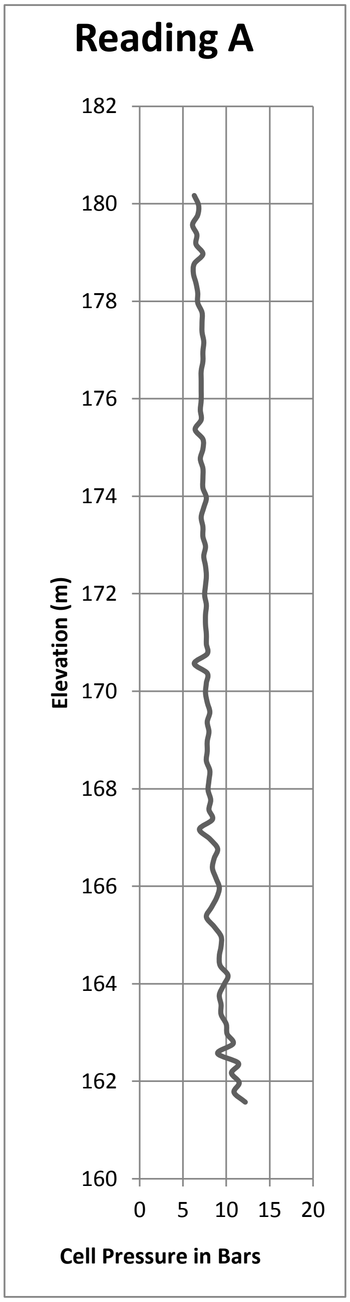
GENERAL NOTES AND OBSERVATIONS

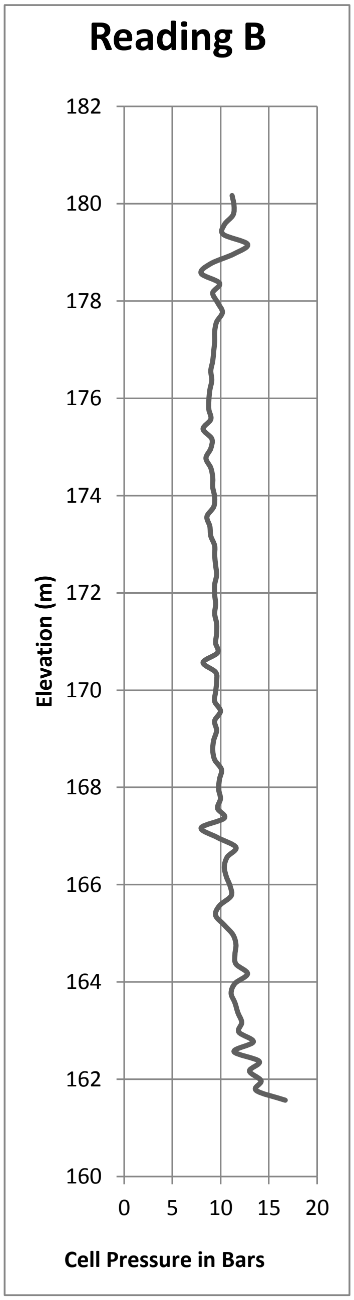
OPERATOR: TA

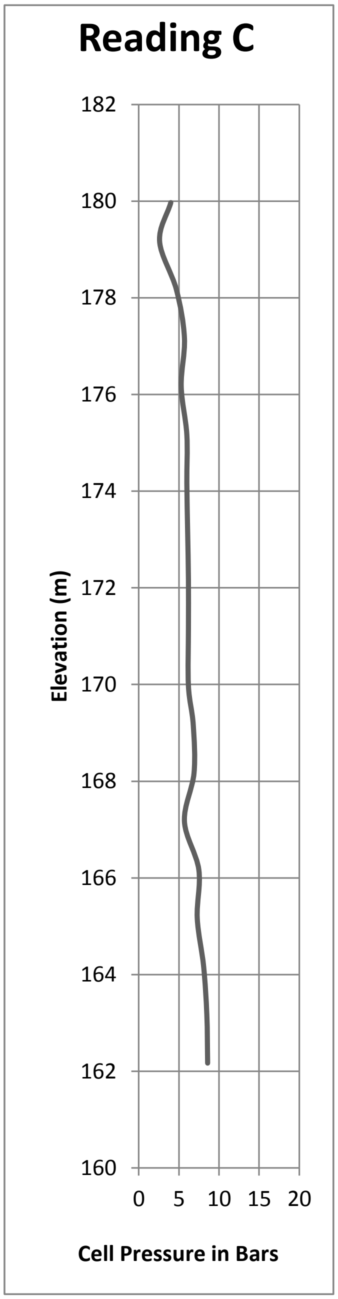
CHECKED: DD

# RECORD OF DILATOMETER TEST DMT B11-1

| | | |
|---|---|---|
| Project : Windsor-Essex Parkway | Test Date: 4/29/2011 | Sheet 1 of 1 |
| Location: N 4678223.6; E 334579.2 | Predrill Depth : 5 m | Datum Geodetic |
| Ground Surface Elevation : 185.2 | Delta A: 0.18 Bar | Delta B: 0.33 Bar |







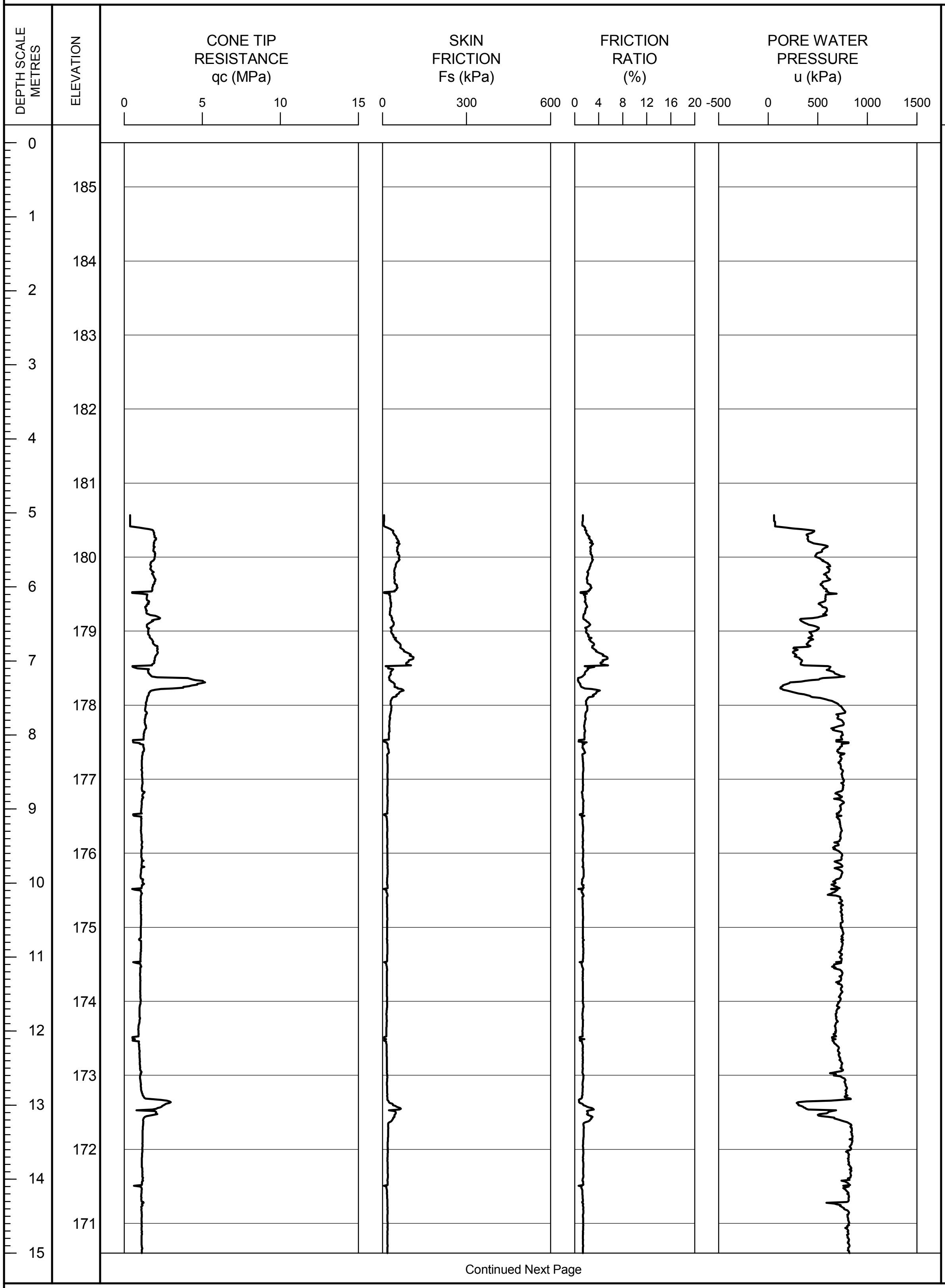
Operator: LC

Checked: DD

# RECORD OF CONE PENETRATION TEST CPT 49-RW

**METRIC**

PROJECT Windsor-Essex Parkway
TEST DATE 5/30/2011 - 5/30/2011
SHEET 1 OF 2
LOCATION N4678107.8; E334725.3
DATUM Geodetic
GROUND SURFACE ELEVATION: 185.6
PREDRILL DEPTH: 5
CORRECTION FACTOR A: 0.8
CORRECTION FACTOR B: 0



GENERAL NOTES AND OBSERVATIONS

Continued Next Page

OPERATOR: TA
CHECKED: DD

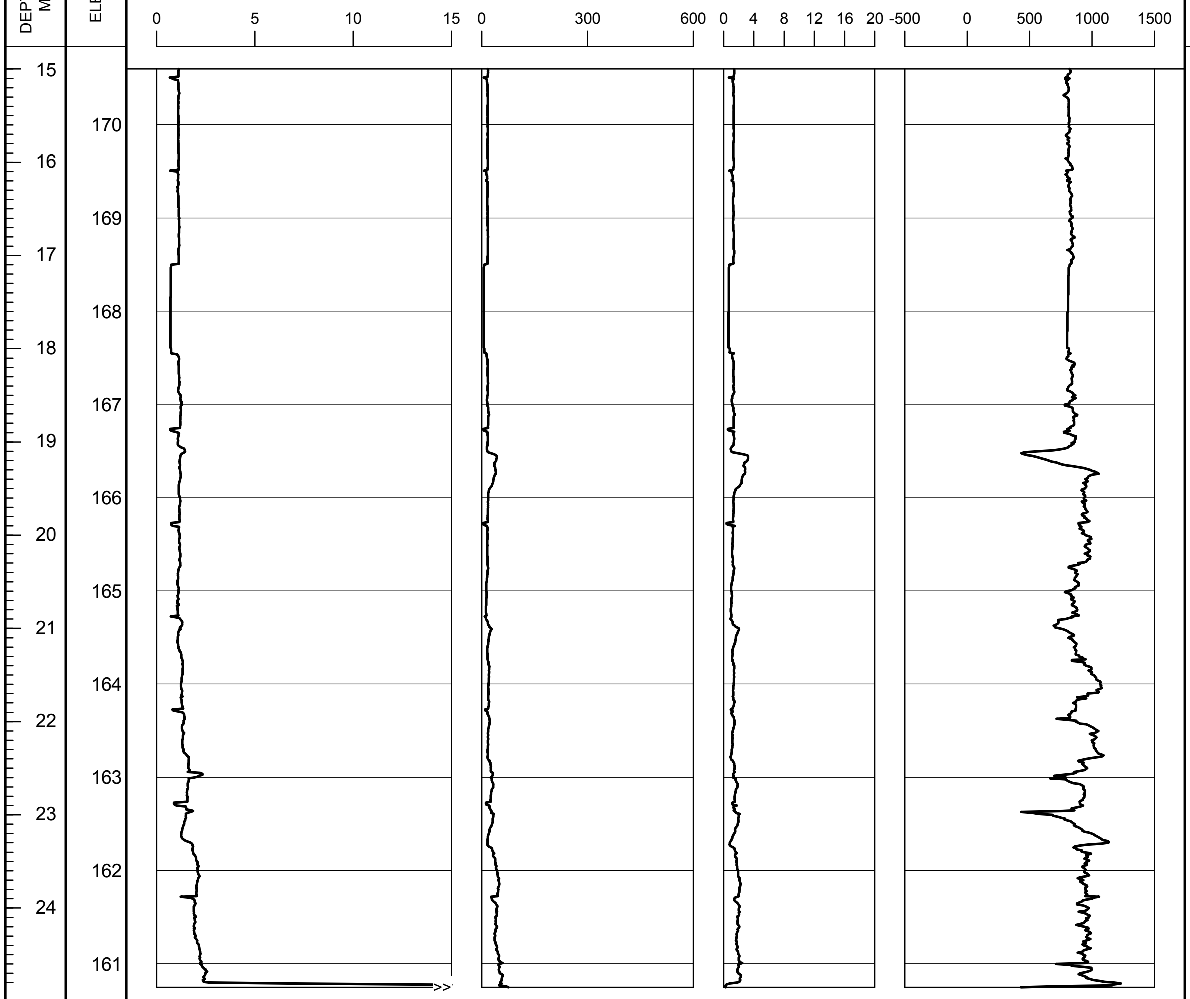
# RECORD OF CONE PENETRATION TEST CPT 49-RW

**METRIC**

PROJECT Windsor-Essex Parkway
TEST DATE 5/30/2011 - 5/30/2011
SHEET 2 OF 2
LOCATION N4678107.8; E334725.3
DATUM Geodetic

GROUND SURFACE ELEVATION: 185.6 PREDRILL DEPTH: 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

# Appendix B: Test Hole Logs - Previous Investigations (2007-2009)

Golder Associates
London, Ontario

# RECORD OF BOREHOLE No 109

1 OF 4 **METRIC**

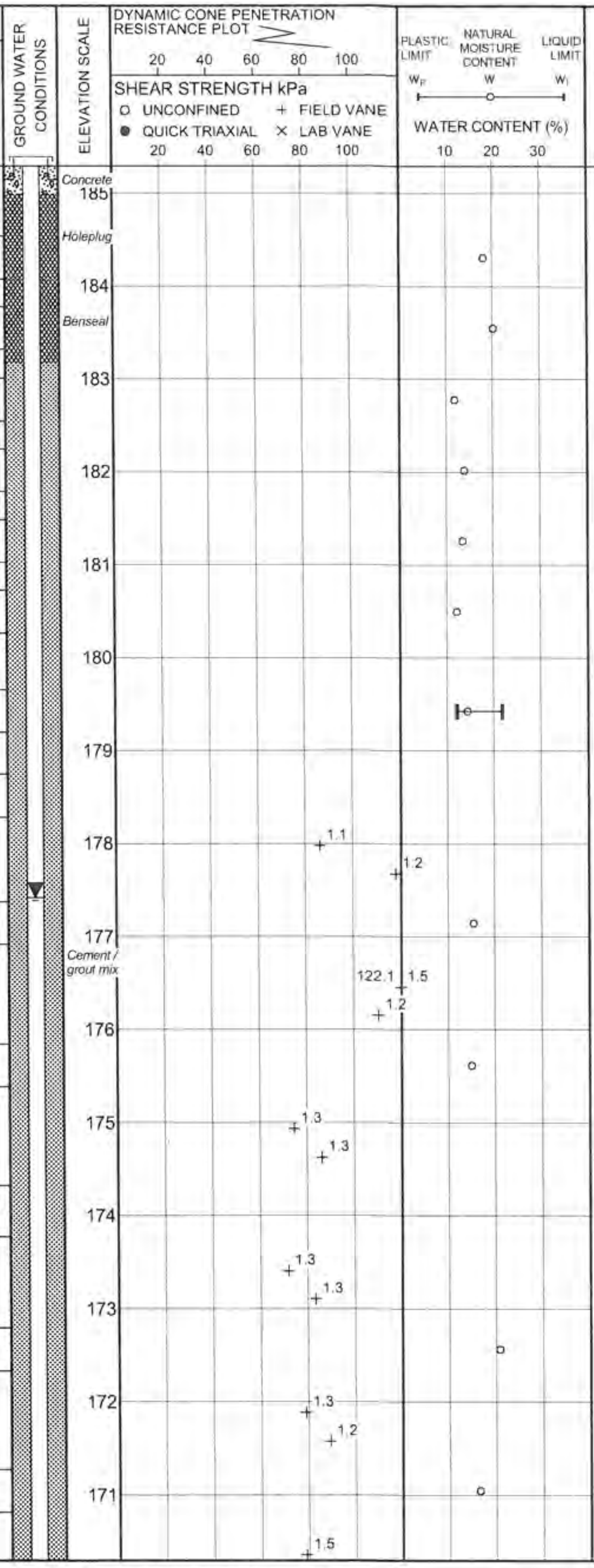
|  |  |  |
|---|---|---|
| PROJECT 07-1130-207-0 | | |
| W.P. | LOCATION N 4678155.0 ;E 334716.3 | ORIGINATED BY MA |
| DIST WEST HWY 401/3 | BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC | COMPILED BY LMK |
| DATUM GEODETIC | DATE January 17, 2008 - January 29, 2008 | CHECKED BY SJB |

## SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 185.30 | GROUND SURFACE |
| 0.00 | TOPSOIL, clayey<br>Brown |
| 0.15 | FILL, clayey silt, trace sand, trace gravel, trace topsoil<br>Brown |
| 184.64<br>0.66 | CLAYEY SILT, trace sand, trace gravel<br>Firm to stiff<br>Mottled brown and grey |
| 183.17<br>2.13 | CLAYEY SILT, some sand, trace gravel<br>Firm to Hard<br>Brown becoming grey at about elev 181.6m |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|
| 1 | SS | 6 | |
| 2 | SS | 13 | |
| 3 | SS | 28 | |
| 4 | SS | 22 | |
| 5 | SS | 20 | |
| 6 | SS | 14 | |
| 7 | SS | 9 | 10 30 41 19 |
| 8 | TO | PH | |
| 9 | SS | 5 | |
| 10 | SS | 6 | |
| 11 | TO | PH | |
| 12 | SS | 6 | |
| 13 | SS | 7 | |



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

$○^{3\%}$ STRAIN AT FAILURE

## RECORD OF BOREHOLE No 109

2 OF 4 **METRIC**

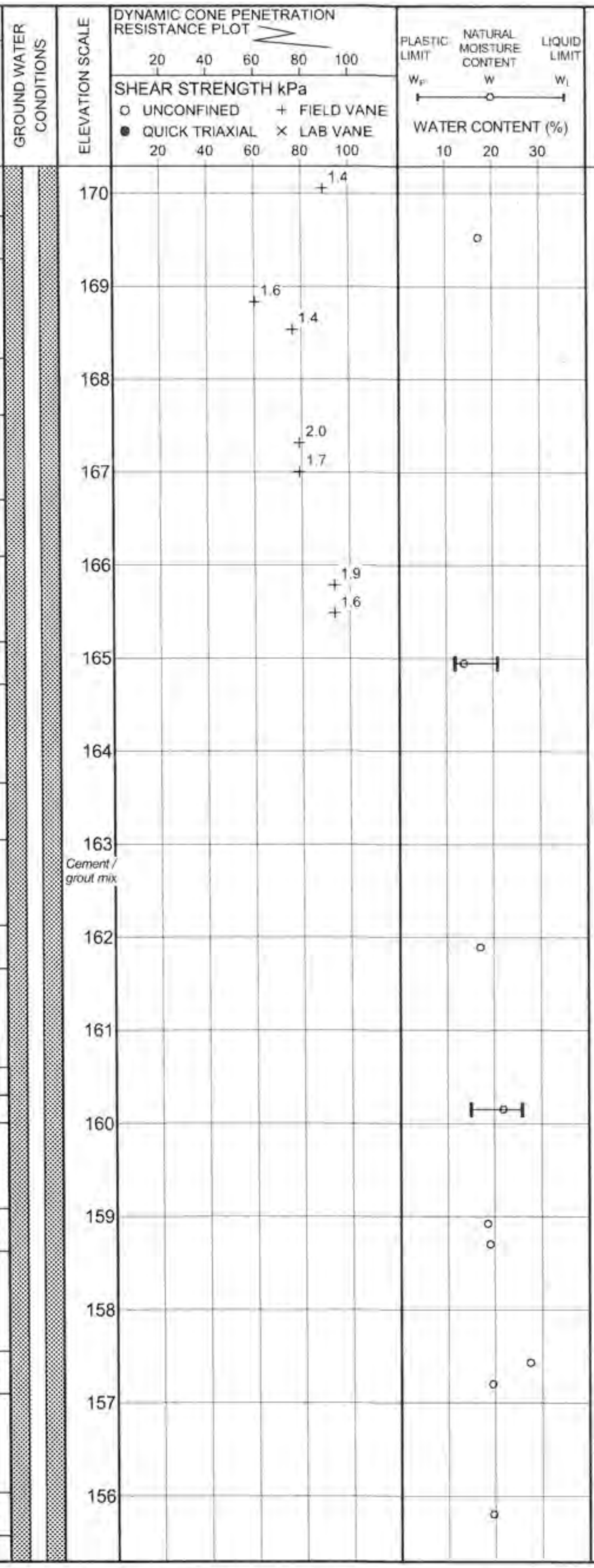
| | | | |
|---|---|---|---|
| PROJECT 07-1130-207-0 | | | |
| W.P. | LOCATION N 4678155.0 ;E 334716.3 | | ORIGINATED BY MA |
| DIST WEST HWY 401/3 | BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC | | COMPILED BY LMK |
| DATUM GEODETIC | DATE January 17, 2008 - January 29, 2008 | | CHECKED BY SJB |

### SOIL PROFILE

| ELEV DEPTH | DESCRIPTION |
|---|---|
| | CLAYEY SILT, some sand, trace gravel<br>Firm to Hard<br>Brown becoming grey at about elev. 181.6m |
| 159.39<br>25.91 | SILT, some sand, trace clay<br>Dense<br>Grey |
| 158.78<br>26.52 | CLAYEY SILT, trace sand, trace gravel<br>Dense to very dense<br>Grey |
| 157.32<br>27.98 | SANDY SILT,<br>Very dense<br>Grey |

### SAMPLES

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|
| 14 | SS | 6 | |
| 15 | TO | PH | |
| 16 | TO | PH | |
| 17 | SS | 9 | 6 26 47 21 |
| 18 | TO | PH | |
| 19 | SS | 19 | |
| 20 | TO | PH | |
| 21 | SS | 31 | 7 12 45 36 |
| 22 | SS | 37 | (83) |
| 23 | SS | 67 | (73) |
| 24 | SS | 66 | |



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

# RECORD OF BOREHOLE No 109

3 OF 4 **METRIC**

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4678155.0 ;E 334716.3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE January 17, 2008 - January 29, 2008
ORIGINATED BY MA
COMPILED BY LMK
CHECKED BY SJB

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | T.C.R. (%) | S.C.R. (%) | R.Q.D. (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 155.12<br>30.18 | SANDY SILT, some clay, trace gravel<br>Compact<br>Grey | | | | Cement / grout mix | 155 | | | | |
| | | 25 | SS | 18 | | | | | | (49) |
| | | | | | Cave in | 154 | | | | |
| 153.30<br>32.00 | SAND AND GRAVEL, trace silt<br>Very dense<br>Grey | | | | | 153 | | | | |
| | | 26 | SS | 54 | | | | | | 15 71 10 4 |
| | | | | | Bentonite | 152 | | | | |
| | | 27 | SS | 72/<br>127mm | Sand | 151 | | | | |
| | | | | | Screen | 150 | | | | |
| | | 28 | SS | 67 | | | | | | |
| 149.18<br>36.12 | LIMESTONE, fresh, medium strong, fine grained, moderately porous<br>Light grey<br><br>(FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE) | | | | Bentonite | 149 | | | | |
| | | 29 | SS | 100/<br>13mm | | | | | | |
| | | 30 | NQ RC | | | 148 | 90 | 69 | 10 | |
| | | 31 | NQ RC | | Sand | 147<br>146 | 100 | 98 | 73 | |
| | | 32 | NQ RC | | | 145 | 98 | 89 | 75 | UC |
| 143.79<br>41.51 | | 33 | NQ RC | | | 144 | 100 | 92 | 77 | |

END OF BOREHOLE

Water levels in borehole at about elev. 159.39m, 157.32m and 153.30m during drilling between January 18 and 28, 2008.

Water level measured in deep piezometer at elev. 178.11m on March 20, 2008.
Water level measured in deep piezometer at elev. 177.75m on July 24, 2008.
Water level measured in deep piezometer at elev. 177.20m on November 14, 2008.
Water level measured in deep piezometer at elev. 177.42m on January 28, 2009.

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: 07-1130-207-0

# RECORD OF DRILLHOLE: 109

SHEET 4 OF 4

LOCATION: N 4678155.0 ;E 334716.3

INCLINATION: -90° AZIMUTH: --

DRILLING DATE: January 17, 2008 - January 29, 2008

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

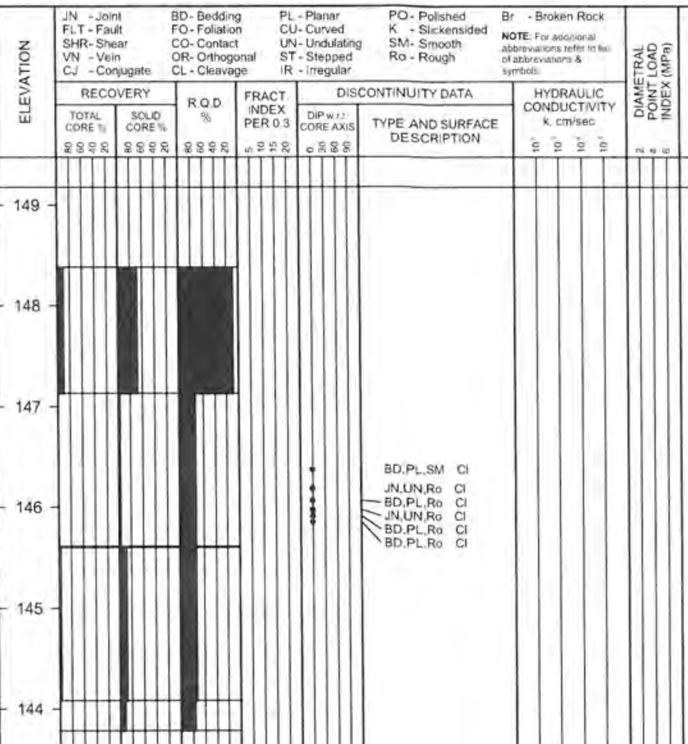
DRILLING CONTRACTOR: AARDVARK DRILLING INC

DATUM: GEODETIC

| | | | | |
|---|---|---|---|---|
| JN - Joint | BD - Bedding | PL - Planar | PO - Polished | Br - Broken Rock |
| FLT - Fault | FO - Foliation | CU - Curved | K - Slickensided | NOTE: For additional abbreviations refer to list of abbreviations & symbols. |
| SHR - Shear | CO - Contact | UN - Undulating | SM - Smooth | |
| VN - Vein | OR - Orthogonal | ST - Stepped | Ro - Rough | |
| CJ - Conjugate | CL - Cleavage | IR - Irregular | | |

| DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. / DEPTH (m) | RUN No | PENETRATION RATE (m/min) | FLUSH COLOUR % RETURN |
|---|---|---|---|---|---|---|
| | ROCK SURFACE | | 149.16 / 36.12 | | | |
| MUD ROTARY<br>NO ROCK CORE | LIMESTONE, fresh, medium strong, laminated, very fine grained, faintly porous,<br>light grey/tan to grey<br>37.16m to 37.4m - zone of broken core<br>Numerous fractures, unable to ID origin due to wave in barrel | | 147.41 / 37.89 | 1 | | |
| | LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, vuggy to faintly porous with depth, white to light grey | | 146.71 / 38.59 | 2 | | |
| | LIMESTONE, fresh, medium strong, laminated, fine grained moderately porous, grey to broken | | 145.71 / 39.59 | | | |
| | LIMESTONE, fresh, medium strong, thinly laminated to laminated, fine to medium grained, moderately porous, grey to greyish white<br>zone of broken core from 39.81m to 39.84 and<br>from 40.29m to 40.33m | | 144.76 / 40.54 | 3 | | |
| | LIMESTONE, fresh, medium strong, laminated, medium grained, faintly to moderately porous, brownish grey | | 143.79 / 41.51 | 4 | | |
| | END OF DRILLHOLE | | | | | |

DEPTH SCALE METRES: 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51



TYPE AND SURFACE DESCRIPTION:
BD,PL,SM Cl
JN,UN,Ro Cl
BD,PL,Ro Cl
JN,UN,Ro Cl
BD,PL,Ro Cl
BD,PL,Ro Cl

NOTES
WATER LEVELS
INSTRUMENTATION

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: SG
CHECKED: SJB

Golder Associates
London, Ontario

PROJECT 07-1130-207-0

# RECORD OF BOREHOLE No 109A

1 OF 1

**METRIC**

W.P.
LOCATION N 4678155.0 ;E 334716.3
ORIGINATED BY MA
DIST WEST HWY 401/3
BOREHOLE TYPE POWER AUGER, SOLID STEM
COMPILED BY LMK
DATUM GEODETIC
DATE January 29, 2008 - January 29, 2008
CHECKED BY SJB

SOIL PROFILE | 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
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

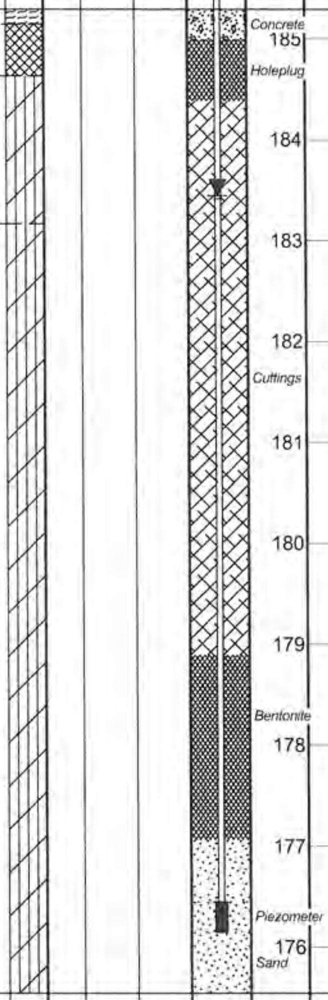
PLASTIC LIMIT $W_P$ — NATURAL MOISTURE CONTENT $W$ — LIQUID LIMIT $W_L$

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

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

| ELEV<br>DEPTH | DESCRIPTION<br>SOIL CONDITIONS INFERRED FROM BOREHOLE No. 109 |
|---|---|
| 185.30 | GROUND SURFACE |
| 0.00 | TOPSOIL, clayey<br>Brown |
| 0.15 | FILL, clayey silt, trace sand, trace gravel, trace topsoil<br>Brown |
| 184.64<br>0.66 | CLAYEY SILT, trace sand, trace gravel<br>Firm to Stiff<br>Mottled brown and grey |
| 183.17<br>2.13 | CLAYEY SILT, some sand, trace gravel<br>Firm to Hard<br>Brown becoming grey at about elev 181.6m |
| 175.55<br>9.75 | END OF BOREHOLE |



Water level measured in shallow piezometer at elev. 183.39m on March 20, 2008.

Water level measured in shallow piezometer at elev. 183.37m on July 24, 2008.

Water level measured in shallow piezometer at elev. 183.45m on January 28, 2009.

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

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

Golder Associates
London, Ontario

PROJECT 07-1130-207-0

# RECORD OF BOREHOLE No CPT-108

1 OF 1

**METRIC**

W.P.

LOCATION N 4678051.6 ;E 334826.8

ORIGINATED BY MA

DIST WEST HWY 401/3

BOREHOLE TYPE POWER AUGER, SOLID STEM

COMPILED BY BRS

DATUM GEODETIC

DATE March 31, 2008

CHECKED BY SJB

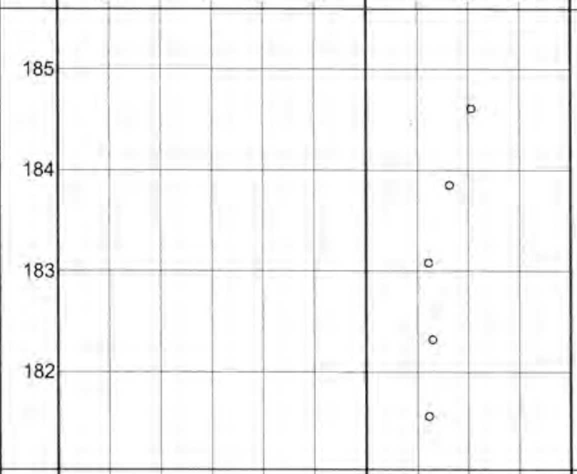
| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | | | |
| 185.60 | GROUND SURFACE | | | | | | | |
| 0.00 | TOPSOIL, silty | | | | | | | |
| 0.15 | Brown<br>CLAYEY SILT, some sand, trace gravel, with silty sand partings<br>Soft to hard<br>Mottled brown and grey, becoming grey at about elev. 182.7m | | | | | 185 | | |
| | | 1 | SS | 4 | | | | |
| | | 2 | SS | 8 | | 184 | | |
| | | 3 | SS | 27 | | 183 | | |
| | | 4 | SS | 31 | | 182 | | |
| | | 5 | SS | 16 | | | | |
| 181.03<br>4.57 | END OF BOREHOLE<br><br>Borehole dry during drilling on March 31, 2008. | | | | | | | |

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



+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No CPT-307

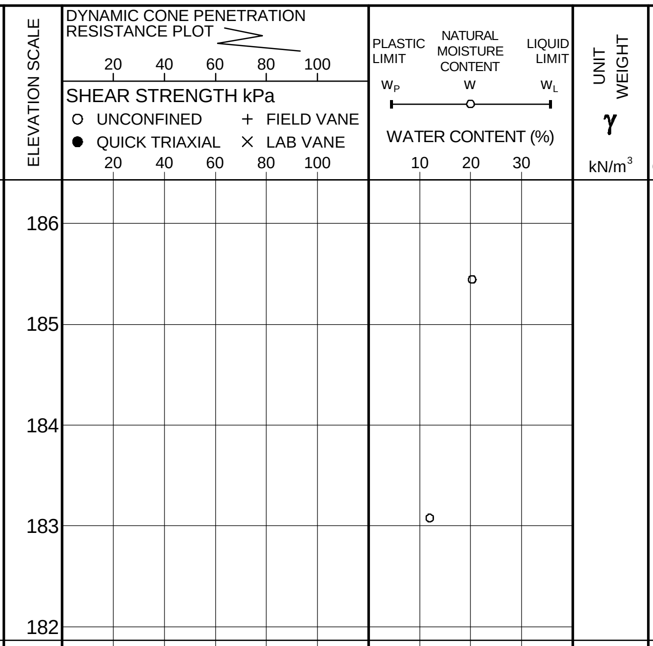
1 OF 1 **METRIC**

PROJECT 09-1132-0080
W.P.
DIST WEST HWY 401 / 3
DATUM GEODETIC

LOCATION N 4678157.2 ;E 334805.1
BOREHOLE TYPE POWER AUGER, SOLID STEM
DATE January 13, 2010

ORIGINATED BY TA
COMPILED BY DMB
CHECKED BY

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | |
| 186.43 | GROUND SURFACE | | | | | |
| 0.00 | TOPSOIL, clayey<br>Black | | | | | |
| 0.30 | FILL, clayey silt, some sand, trace gravel, trace topsoil<br>Brown | | | | | |
| 185.67<br>0.76 | CLAYEY SILT, some sand, trace gravel, with occasional silt partings<br>Stiff to hard<br>Brown | 1 | SS | 13 | | |
| | | 2 | SS | 15 | | |
| | | 3 | SS | 36 | | |
| | | 4 | SS | 69 | | |
| 181.86<br>4.57 | END OF BOREHOLE<br><br>Borehole dry during drilling on January 13, 2010. | | | | | |



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

Golder Associates
London, Ontario

# RECORD OF BOREHOLE No CPT-309

1 OF 1 **METRIC**

PROJECT 09-1132-0080

W.P.

LOCATION N 4678204.8 ;E 334657.1

ORIGINATED BY TA

DIST WEST HWY 401 / 3

BOREHOLE TYPE POWER AUGER, SOLID STEM

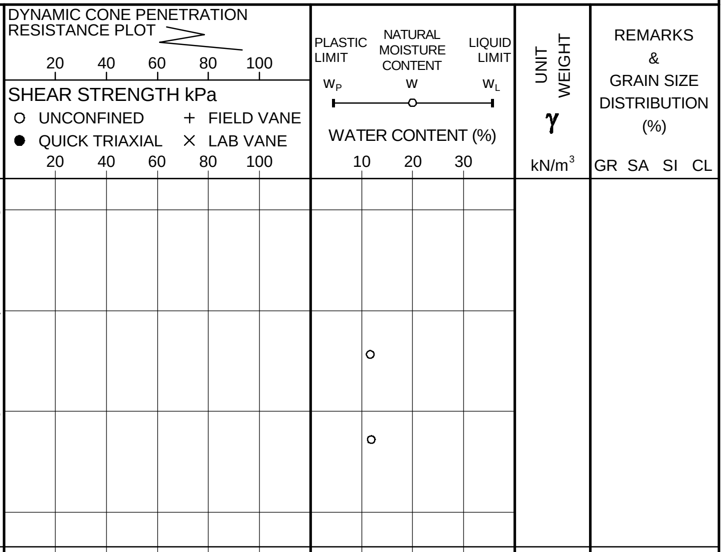
COMPILED BY DMB

DATUM GEODETIC

DATE December 22, 2009

CHECKED BY

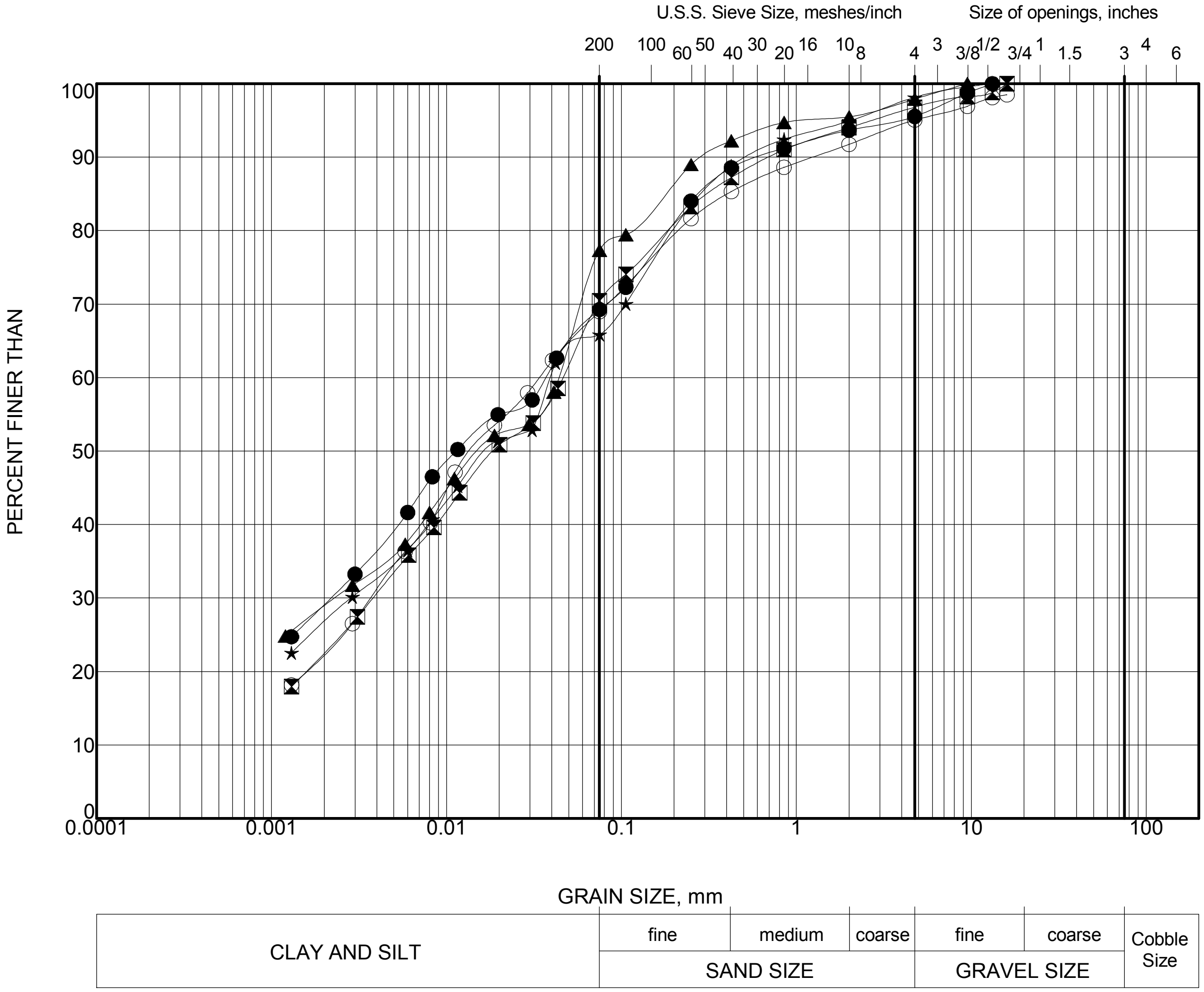
| SOIL PROFILE | | | SAMPLES | | | | |
|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE |
| 185.31 | GROUND SURFACE | | | | | | |
| 0.00 | FILL, limestone gravel, crushed<br>Grey | | | | | | 185 |
| 0.13 | FILL, sand and gravel<br>Brown | | | | | | |
| 0.30 | CLAYEY SILT, some sand, trace gravel, with occasional fissures, silt partings and seams<br>Stiff to hard<br>Brown | | 1 | SS | 14 | | 184 |
| | | | 2 | SS | 20 | | 183 |
| | | | 3 | SS | 55 | | |
| | | | 4 | SS | 70 | | 182 |
| 181.65<br>3.66 | END OF BOREHOLE<br><br>Borehole dry during drilling on December 22, 2009. | | | | | | |



+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

# Appendix C: Geotechnical Laboratory Test Results



**LEGEND:**

| SYMBOL | BOREHOLE | SAMPLE | DEPTH (m) |
|---|---|---|---|
| ● | B11-1 | 13 | 13.7 |
| ☒ | B11-1 | 18 | 21.3 |
| ▲ | B11-2 | 3 | 2.3 |
| ★ | B11-2 | 9 | 7.6 |
| ○ | B11-2 | 19 | 22.9 |

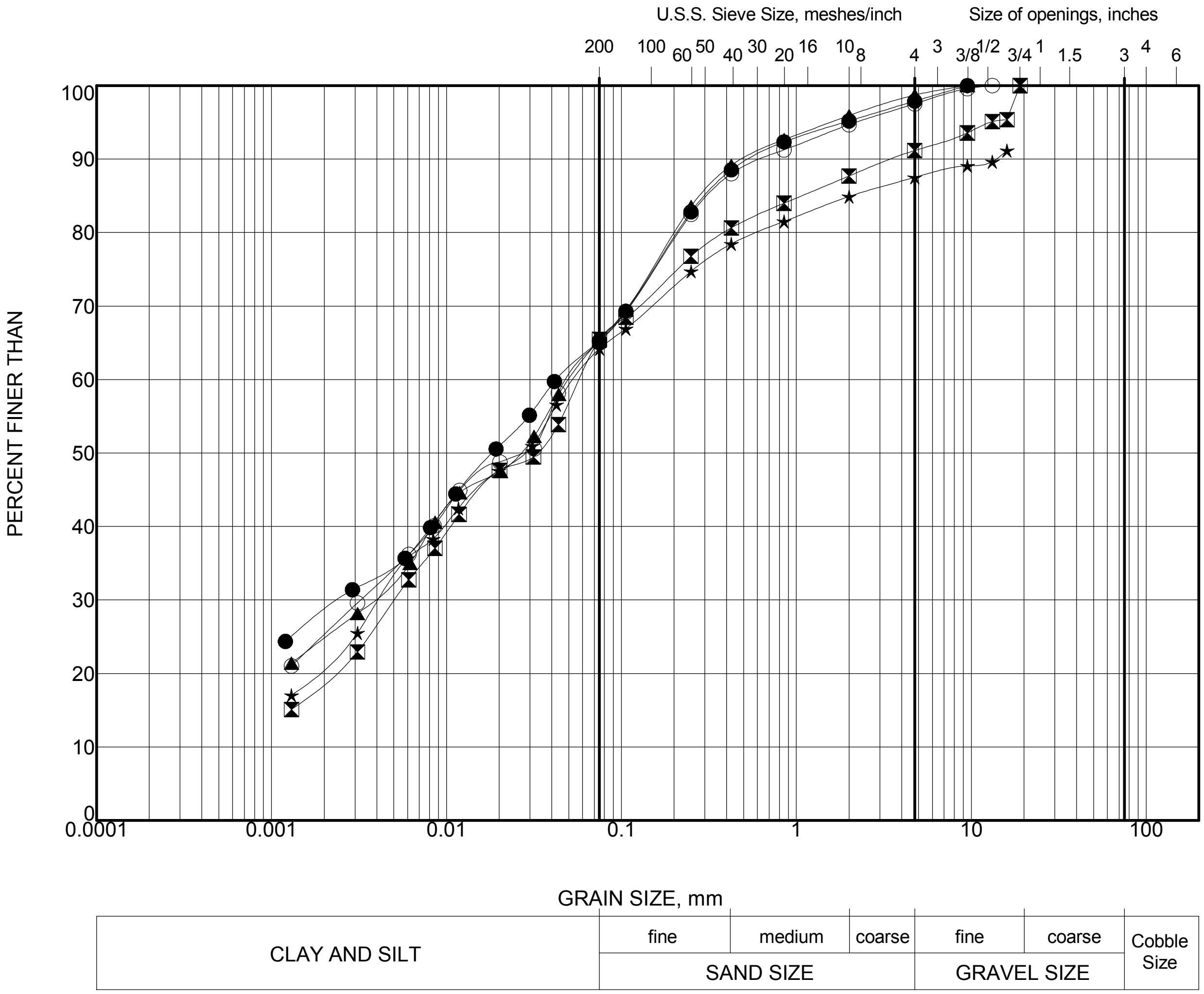
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
**GRAIN SIZE DISTRIBUTION**
**Clayey Silt to Silty Clay**

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

| | | | |
|---|---|---|---|
| PROJECT No. | SW8801.1004.101 | FILE No. | SW8801.1004.101 |
| | | SCALE | REV. |
| DRAWN | SS | Nov 2, 2011 | |
| CHECK | MSO | | FIGURE **C1** |

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 02/11/11



**LEGEND:**

| SYMBOL | BOREHOLE | SAMPLE | DEPTH (m) |
|---|---|---|---|
| ● | B11-3 | 4 | 3 |
| ⧗ | B11-3 | 19 | 22.9 |
| ▲ | B11-4 | 8 | 6.1 |
| ★ | B11-4 | 20 | 24.4 |
| ○ | B11-5 | 8 | 6.1 |

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 02/11/11

PROJECT

Windsor Essex Parkway (WEP)
Windsor, Ontario

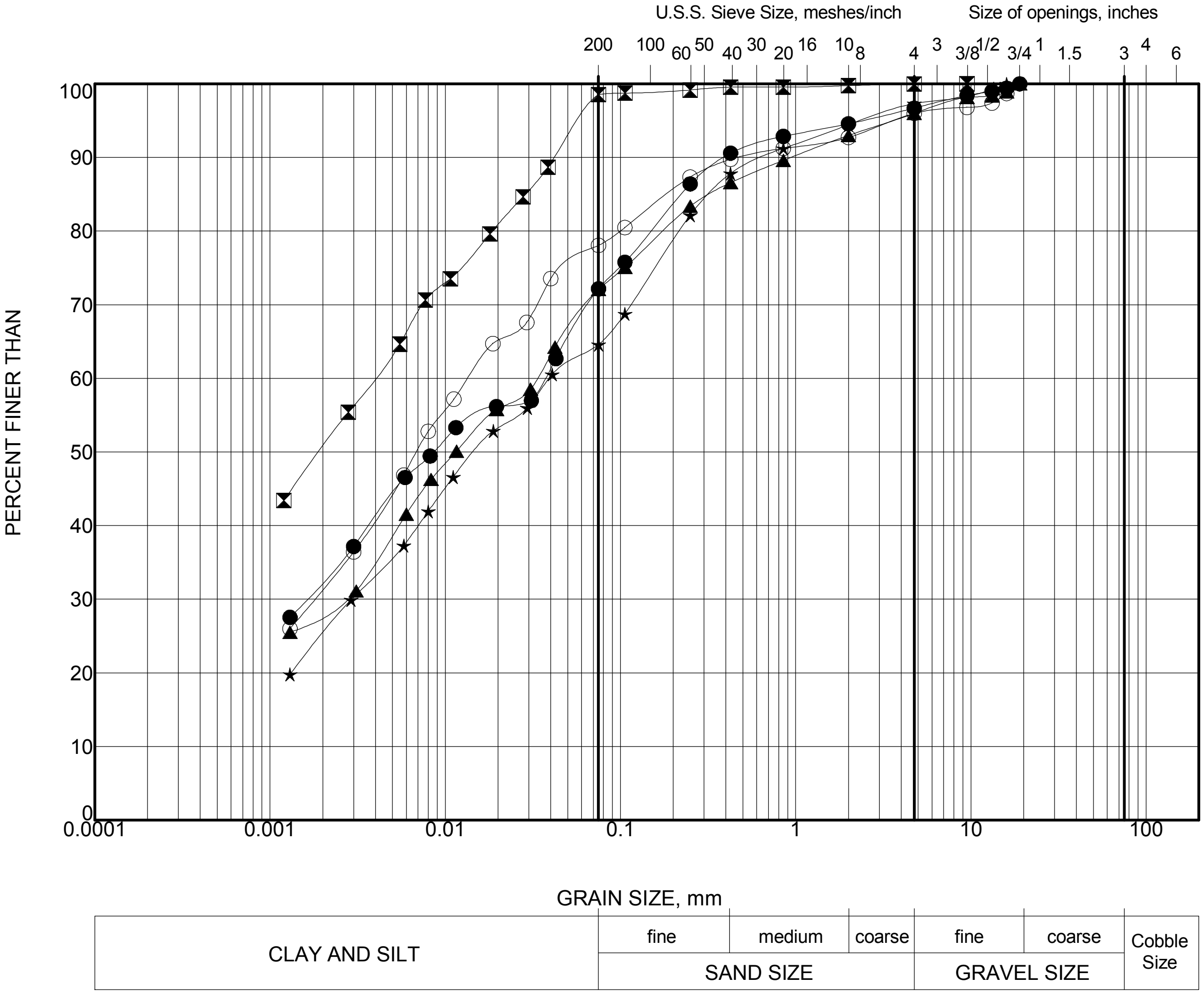
TITLE

# GRAIN SIZE DISTRIBUTION

## Clayey Silt to Silty Clay

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

| | | | |
|---|---|---|---|
| PROJECT No. SW8801.1004.101 | | | FILE No. SW8801.1004.101 |
| | | | SCALE / REV. |
| DRAWN | SS | Nov 2, 2011 | FIGURE **C2** |
| CHECK | MSO | | |



## LEGEND:

| SYMBOL | BOREHOLE | SAMPLE | DEPTH (m) |
|---|---|---|---|
| ● | B11-5 | 14 | 15.2 |
| ⧗ | B11-5 | 16 | 18.3 |
| ▲ | B11-5 | 19 | 22.9 |
| ★ | B11-6/HGMW-05 | 9 | 6.1 |
| ○ | B11-6/HGMW-05 | 18 | 18.3 |

PROJECT

Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE

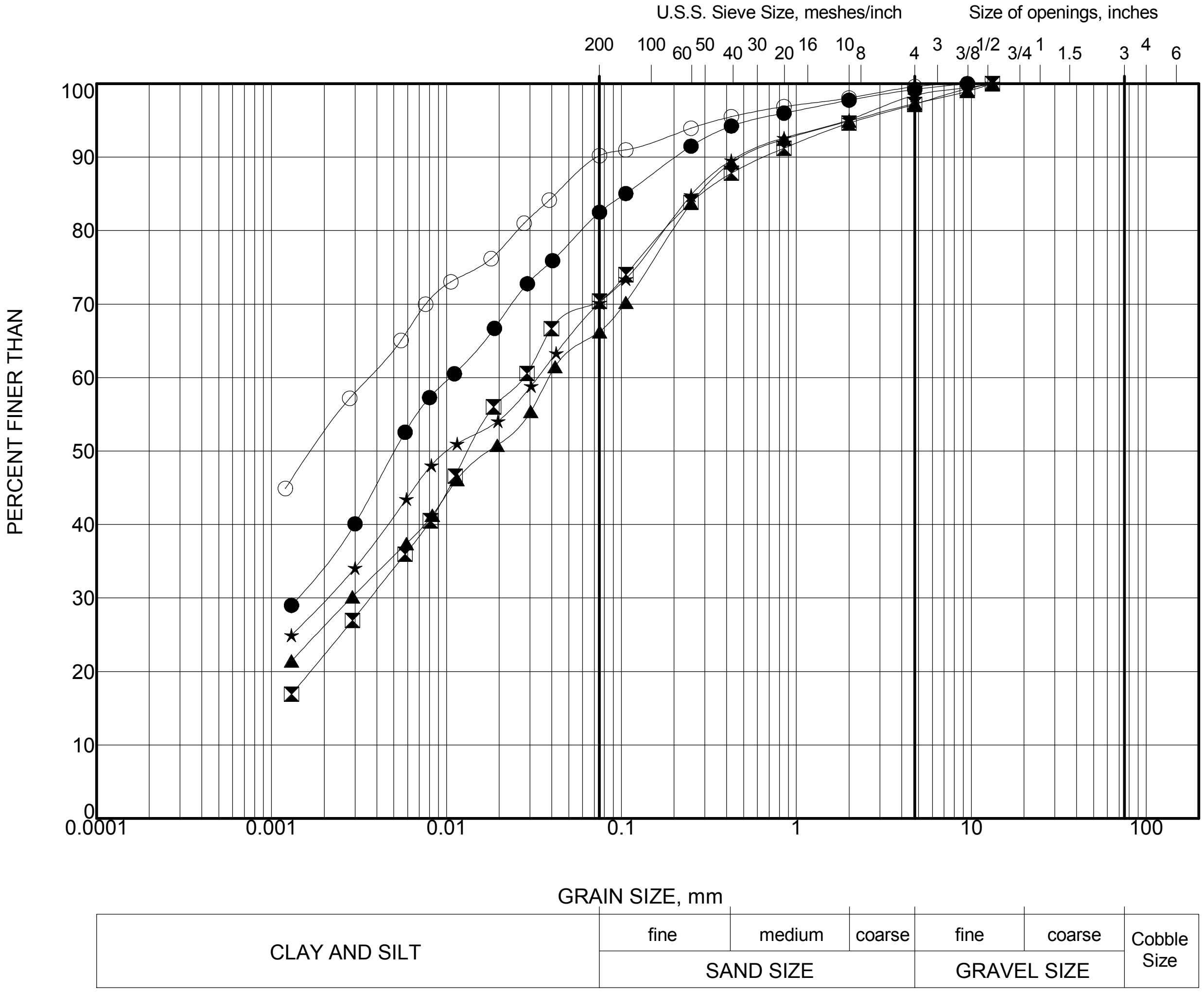
# GRAIN SIZE DISTRIBUTION

## Clayey Silt to Silty Clay

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

| | | | |
|---|---|---|---|
| PROJECT No. SW8801.1004.101 | | | FILE No. SW8801.1004.101 |
| | | | SCALE / REV. |
| DRAWN | SS | Nov 2, 2011 | FIGURE **C3** |
| CHECK | MSO | | |

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 02/11/11



**LEGEND:**

| SYMBOL | BOREHOLE | SAMPLE | DEPTH (m) |
|---|---|---|---|
| ● | B11-6/HGMW-05 | 20 | 19.8 |
| ⧗ | B11-6/HGMW-05 | 24 | 22.9 |
| ▲ | B11-7 | 10 | 9.1 |
| ★ | B11-7 | 14 | 15.2 |
| ○ | B11-7 | 16 | 18.3 |

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE

## GRAIN SIZE DISTRIBUTION

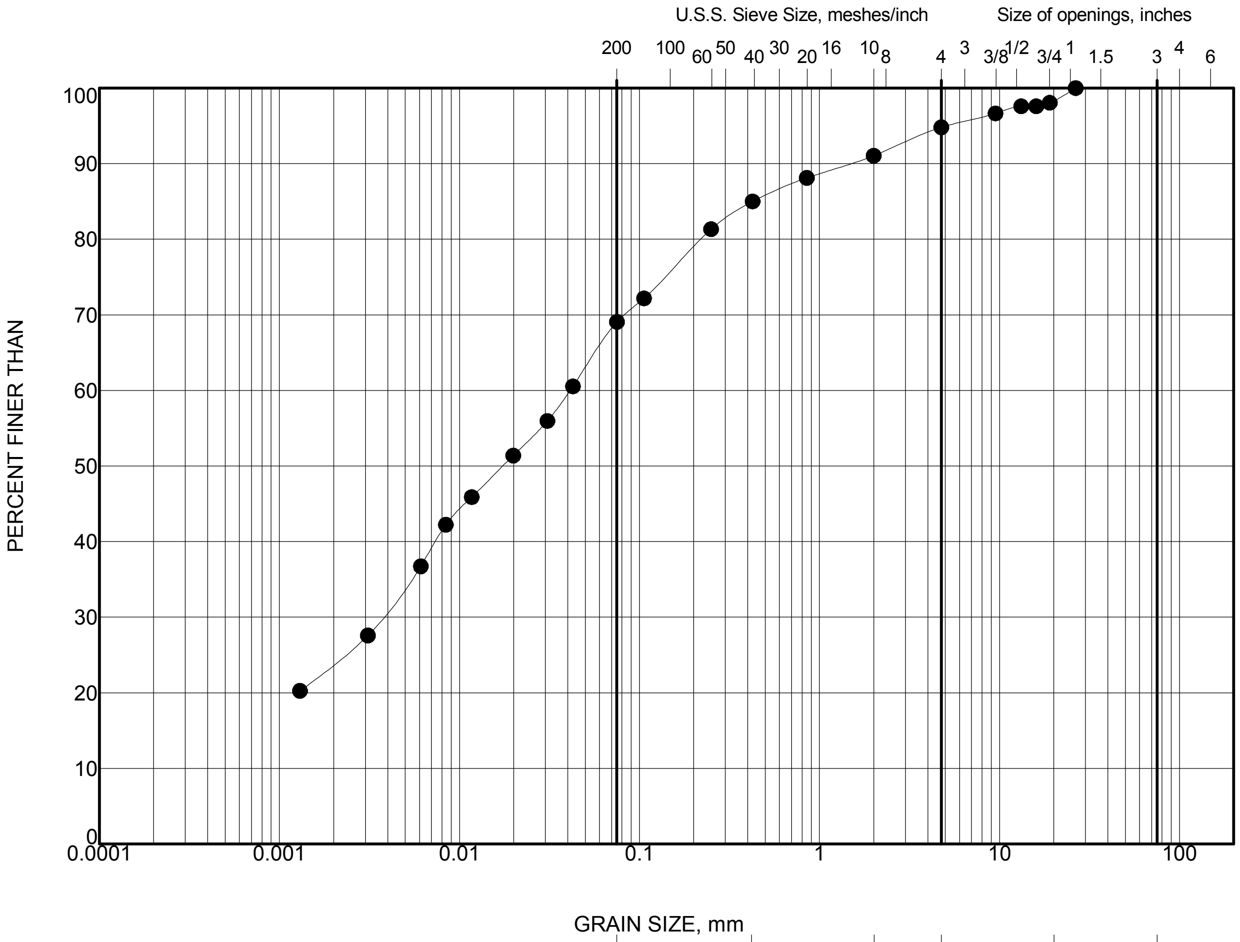
### Clayey Silt to Silty Clay

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

|  |  |  |  |
|---|---|---|---|
| PROJECT No. | SW8801.1004.101 | FILE No. | SW8801.1004.101 |
| DRAWN | SS | Nov 2, 2011 | SCALE |
| CHECK | MSO |  | REV. |

FIGURE **C4**

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 02/11/11



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

**LEGEND:**

| SYMBOL | BOREHOLE | SAMPLE | DEPTH (m) |
|---|---|---|---|
| ● | B11-7 | 19 | 22.9 |

PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

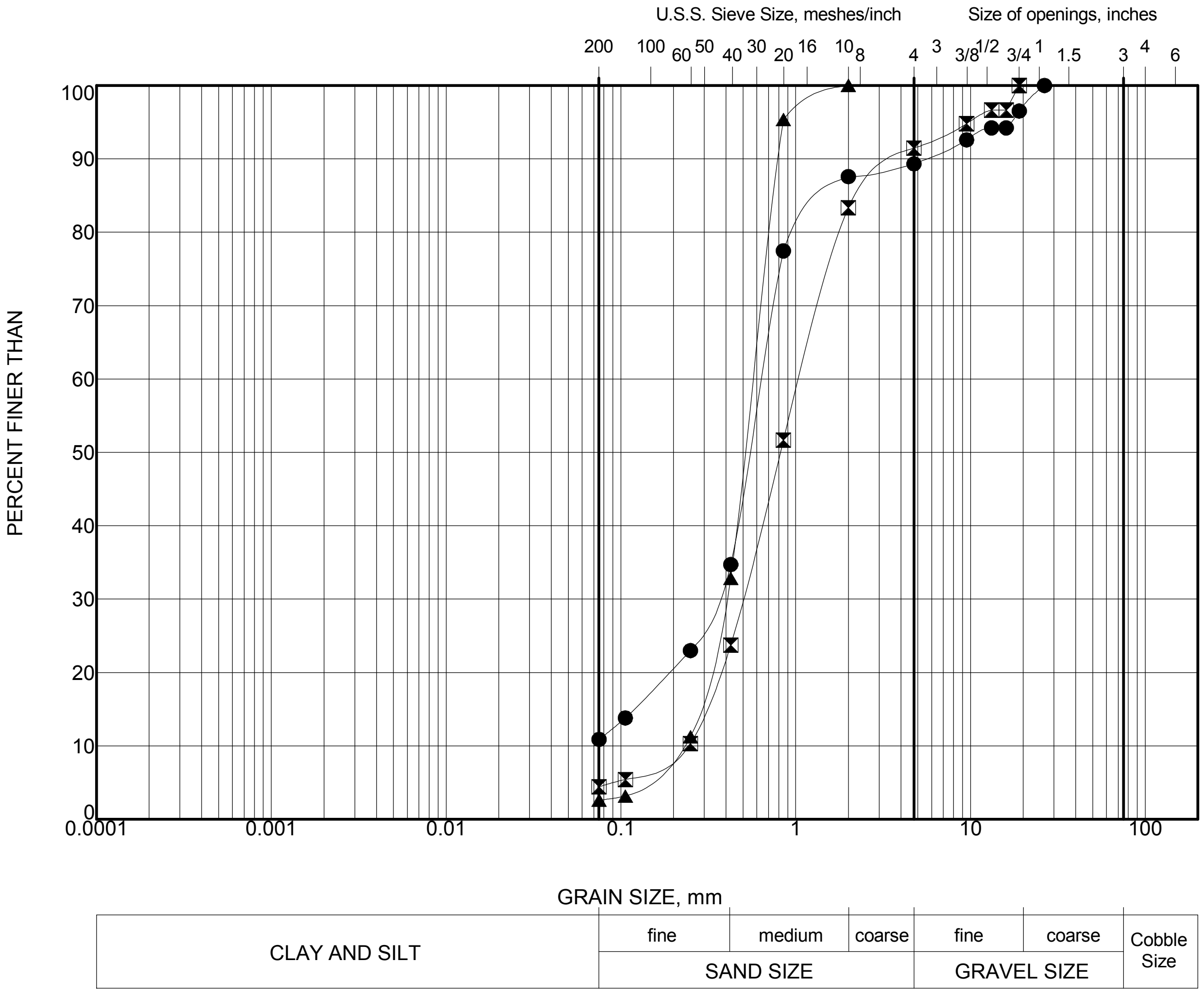
TITLE

# GRAIN SIZE DISTRIBUTION

## Clayey Silt to Silty Clay

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

| | | | | |
|---|---|---|---|---|
| PROJECT No. SW8801.1004.101 | | | FILE No. SW8801.1004.101 | |
| | | | SCALE | REV. |
| DRAWN | SS | Nov 2, 2011 | FIGURE **C5** | |
| CHECK | MSO | | | |



**LEGEND:**

| SYMBOL | BOREHOLE | SAMPLE | DEPTH (m) |
|---|---|---|---|
| ● | B11-1 | 25 | 32 |
| ⧗ | B11-3 | 27 | 35.1 |
| ▲ | B11-4 | 28 | 37.8 |

PROJECT

Windsor Essex Parkway (WEP)
Windsor, Ontario

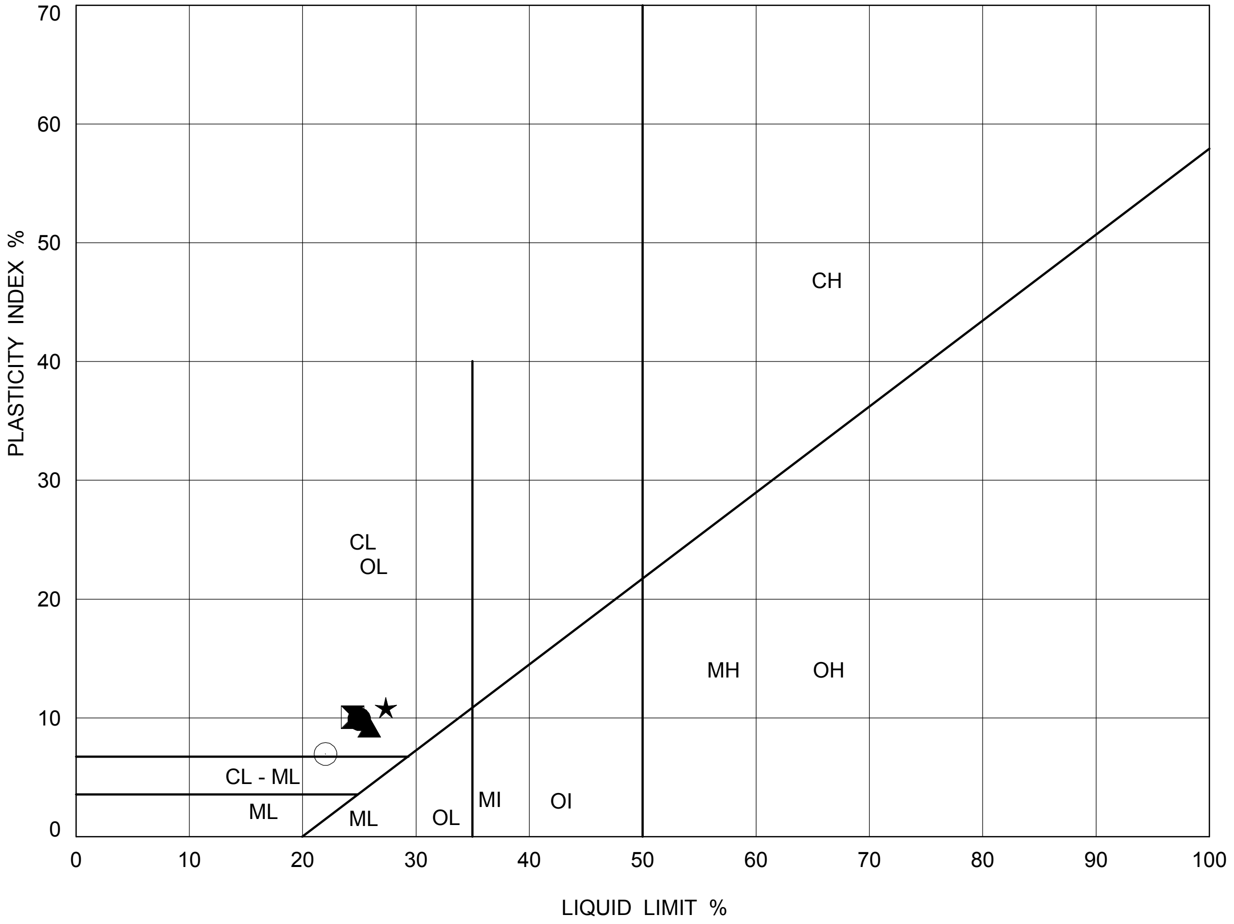
TITLE

# GRAIN SIZE DISTRIBUTION
## Lower Till/Granular

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

| | | | |
|---|---|---|---|
| PROJECT No. SW8801.1004.101 | | | FILE No. SW8801.1004.101 |
| | | | SCALE / REV. |
| DRAWN | SS | Nov 2, 2011 | FIGURE **C6** |
| CHECK | MSO | | |

WEP GRAIN SIZE SW8801.1004.101.GPJ ONTARIO MOT.GDT 02/11/11



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

**LEGEND:**

| SYMBOL | BOREHOLE | SAMPLE | DEPTH (m) | LL(%) | PL(%) | PI |
|---|---|---|---|---|---|---|
| ● | B11-5 | 19 | 22.9 | **25** | **15** | **10** |
| ⧗ | B11-6/HGMW-05 | 9 | 6.1 | **24** | **14** | **10** |
| ▲ | B11-6/HGMW-05 | 18 | 18.3 | **26** | **17** | **9** |
| ★ | B11-6/HGMW-05 | 20 | 19.8 | **27** | **16** | **11** |
| ○ | B11-6/HGMW-05 | 24 | 22.9 | **22** | **15** | **7** |

PROJECT

Windsor Essex Parkway (WEP)
Windsor, Ontario

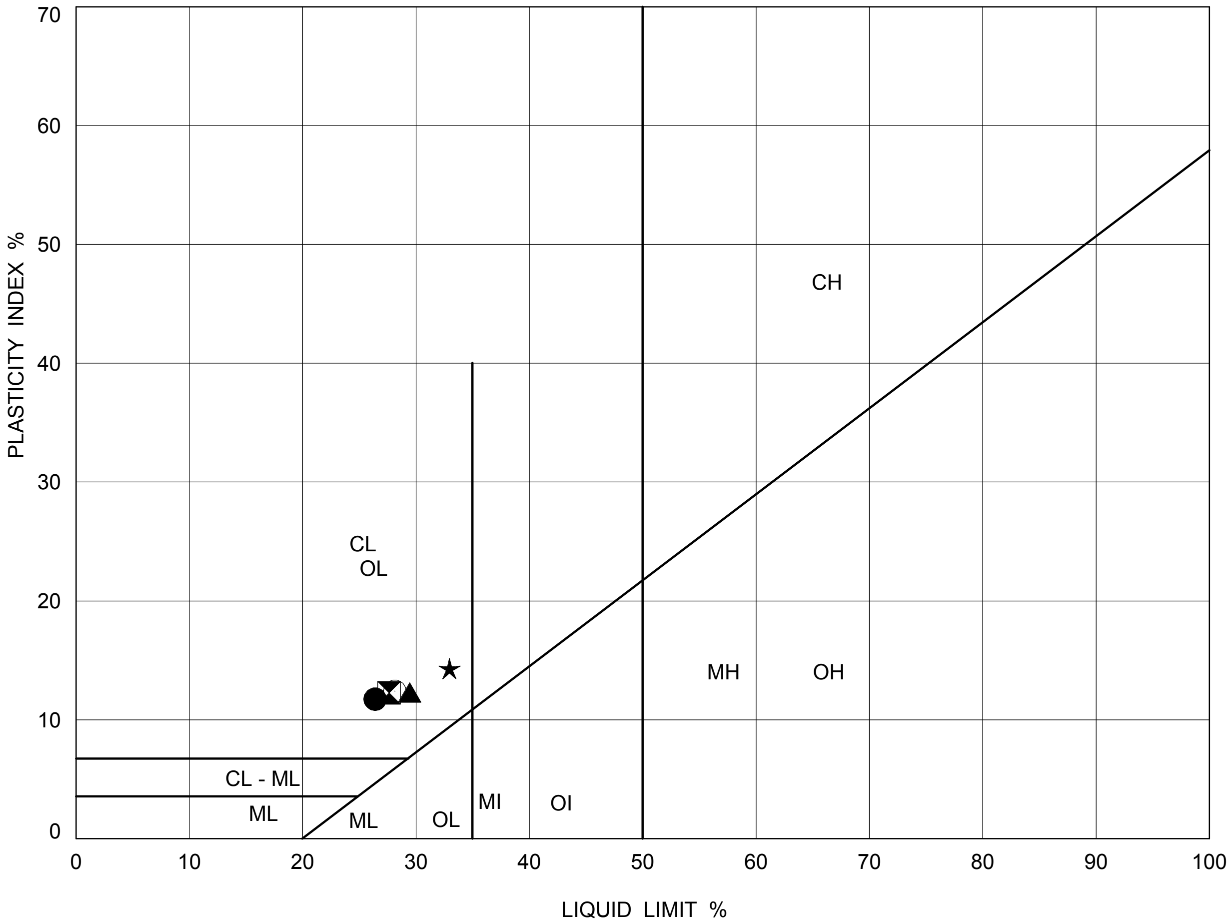
TITLE

PLASTICITY CHART

**Silty Clay to Clayey Silt**

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

| | | | | |
|---|---|---|---|---|
| PROJECT No. | SW8801.1004.101 | | FILE No. | SW8801.1004.101 |
| | | | SCALE | REV. |
| DRAWN | SS | Nov 2, 2011 | FIGURE C7 | |
| 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 |
|---|---|---|---|---|---|---|
| ● | B11-1 | 13 | 13.7 | **26** | **15** | **11** |
| ⧗ | B11-1 | 15 | 16.8 | **28** | **15** | **13** |
| ▲ | B11-1 | 16 | 18.3 | **29** | **17** | **12** |
| ★ | B11-1 | 17 | 19.8 | **33** | **19** | **14** |
| ○ | B11-2 | 3 | 2.3 | **28** | **16** | **12** |

PROJECT

Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE

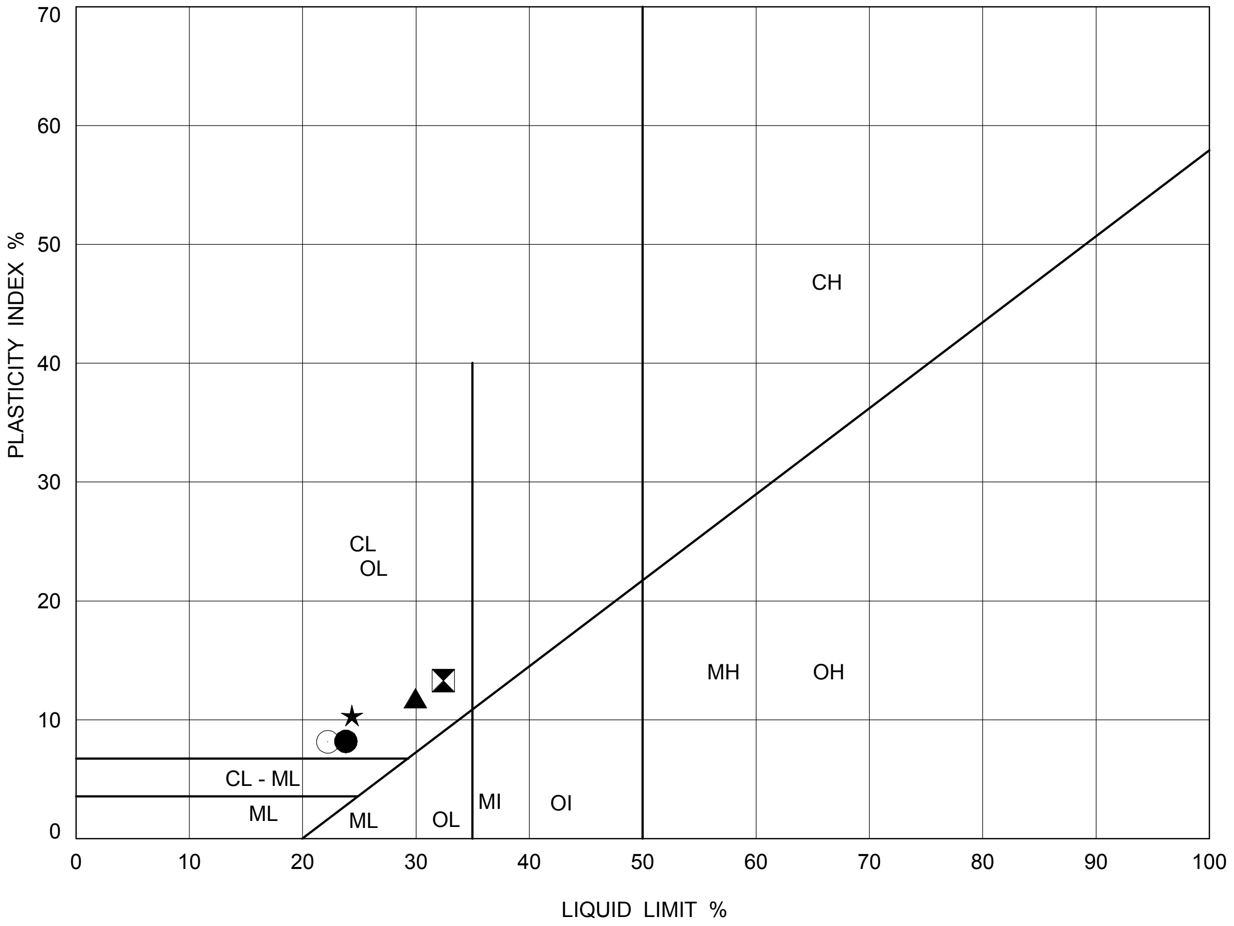
# PLASTICITY CHART

## Silty Clay to Clayey Silt

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

| | | | |
|---|---|---|---|
| PROJECT No. SW8801.1004.101 | | | FILE No. SW8801.1004.101 |
| | | | SCALE / REV. |
| DRAWN | SS | Nov 2, 2011 | FIGURE **C8** |
| CHECK | MSO | | |

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 02/11/11



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

**LEGEND:**

| SYMBOL | BOREHOLE | SAMPLE | DEPTH (m) | LL(%) | PL(%) | PI |
|---|---|---|---|---|---|---|
| ● | B11-3 | 3 | 2.3 | **24** | **16** | **8** |
| ⧗ | B11-3 | 16 | 18.3 | **32** | **19** | **13** |
| ▲ | B11-3 | 17 | 19.8 | **30** | **18** | **12** |
| ★ | B11-3 | 18 | 21.3 | **24** | **14** | **10** |
| ○ | B11-3 | 19 | 22.9 | **22** | **14** | **8** |

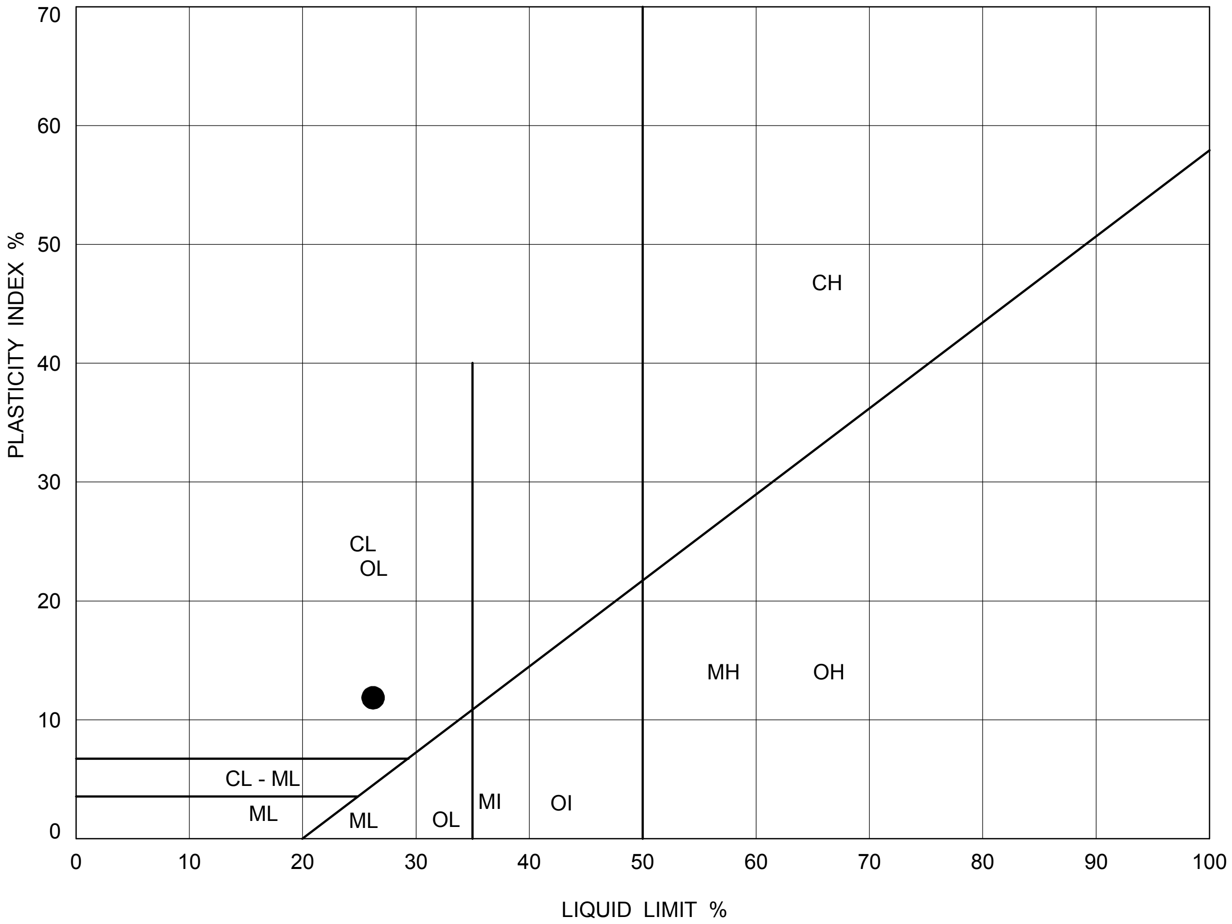
PROJECT
Windsor Essex Parkway (WEP)
Windsor, Ontario

TITLE
PLASTICITY CHART
**Silty Clay to Clayey Silt**

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

| | | | | |
|---|---|---|---|---|
| PROJECT No. | SW8801.1004.101 | | FILE No. | SW8801.1004.101 |
| | | | SCALE | REV. |
| DRAWN | SS | Nov 2, 2011 | FIGURE **C9** | |
| CHECK | MSO | | | |

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 02/11/11



SOIL TYPE

C = Clay
M = Silt
O = Organic

PLASTICITY

L = Low
I = Intermediate
H = High

**LEGEND:**

| SYMBOL | BOREHOLE | SAMPLE | DEPTH (m) | LL(%) | PL(%) | PI |
|---|---|---|---|---|---|---|
| ● | B11-7 | 19 | 22.9 | **26** | **14** | **12** |

| | |
|---|---|
| PROJECT | Windsor Essex Parkway (WEP)<br>Windsor, Ontario |
| TITLE | PLASTICITY CHART<br>**Silty Clay to Clayey Silt** |

Parkway Infrastructure Engineers amec Hatch Mott MacDonald

| | | | | |
|---|---|---|---|---|
| PROJECT No. | SW8801.1004.101 | | FILE No. | SW8801.1004.101 |
| | | | SCALE | REV. |
| DRAWN | | SS Nov 2, 2011 | | |
| CHECK | MSO | | FIGURE | **C10** |

WEP PLASTICITY CHART SW8801.1004.101.GPJ ONTARIO MOT.GDT 02/11/11

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

| | | | | | |
|---|---|---|---|---|---|
| **Project:** | **WEP** | | | **Project No.:** | **SW8801.1004.101** |
| **Client:** | **Hatch Mott MacDonald Limited** | | | **Date:** | **24-Aug-11** |
| **Location:** | **Windsor, ON.** | **Sample ID:** | **B11-6_TW14** | **Depth(m):** | **13.7** |

**Sample Description:** **Sandy Silty Clay trace gravel**

## Sample Parameters

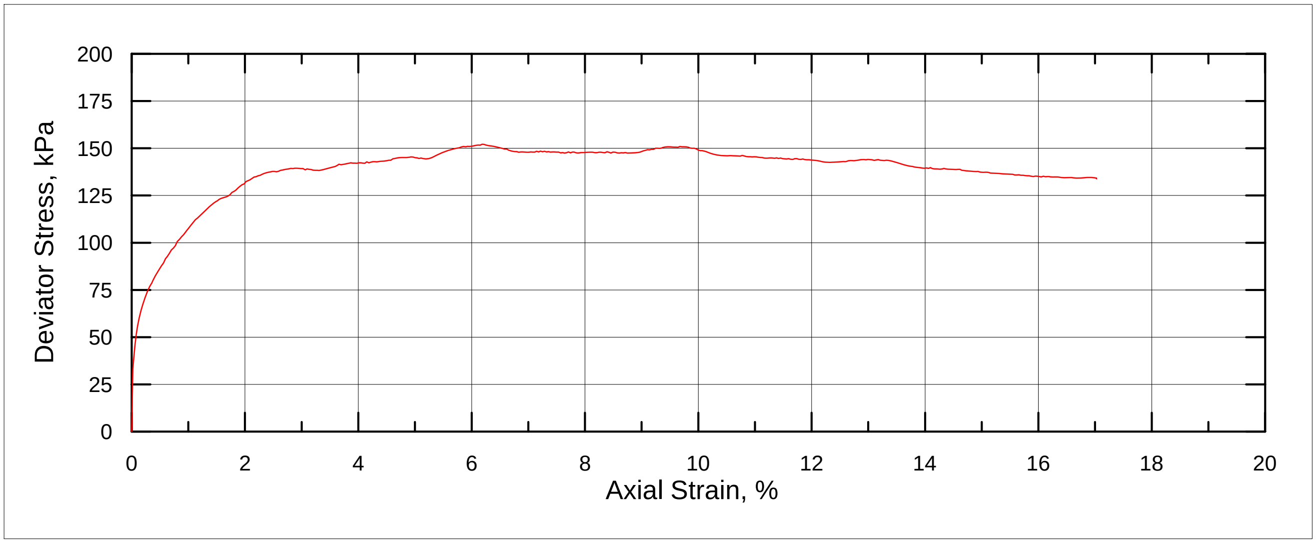
| Initial | | Specimen 1 | Specimen 2 | Specimen 3 |
|---|---|---|---|---|
| Diameter | cm | 6.940 | | |
| Height | cm | 14.088 | | |
| Volume | $cm^3$ | 532.915 | | |
| Wet Mass | g | 1177.70 | | |
| Dry Density | $kg/m^3$ | 1927 | | |
| Water Content | % | 14.7 | | |
| Specific Gravity | Actual | 2.700 | | |
| Void Ratio | | 0.40 | | |
| Degree of Saturation | | 98.8 | | |
| **Before Shear** (after consolidation) | | | | |
| Volume | $cm^3$ | 518.115 | | |
| B - Value | | 1.00 | | |
| **After Shear** | | | | |
| Wet Mass | g | 1167.83 | | |
| Dry Density | $kg/m^3$ | 1985 | | |
| Water Content | % | 13.5 | | |
| Void Ratio | | 0.36 | | |
| Degree of Saturation | | 100.0 | | |

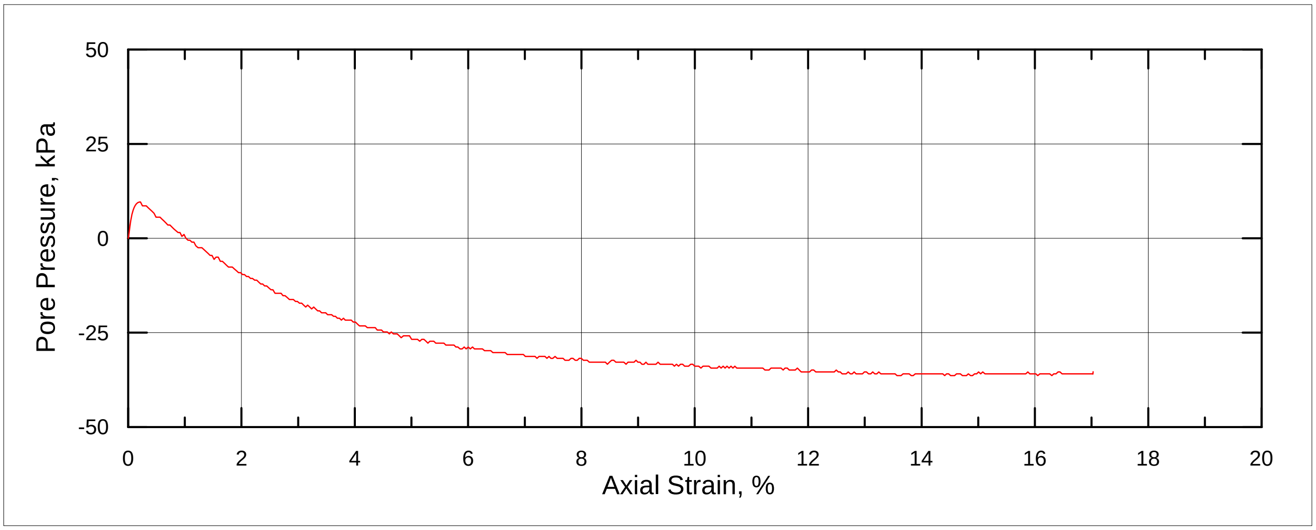
| Stress - Strain | | | | |
|---|---|---|---|---|
| Cell Pressure | kPa | 235.00 | | |
| Back Pressure | kPa | 190.00 | | |
| Consolidation Stress | kPa | 45.00 | | |
| Rate of Strain | mm/min | 0.0500 | | |
| Vertical Strain at Failure | % | 6.18 | | |
| Deviator Stress at Failure | kPa | 152.09 | | |
| Pore Pressure at Failure | kPa | -29.30 | | |

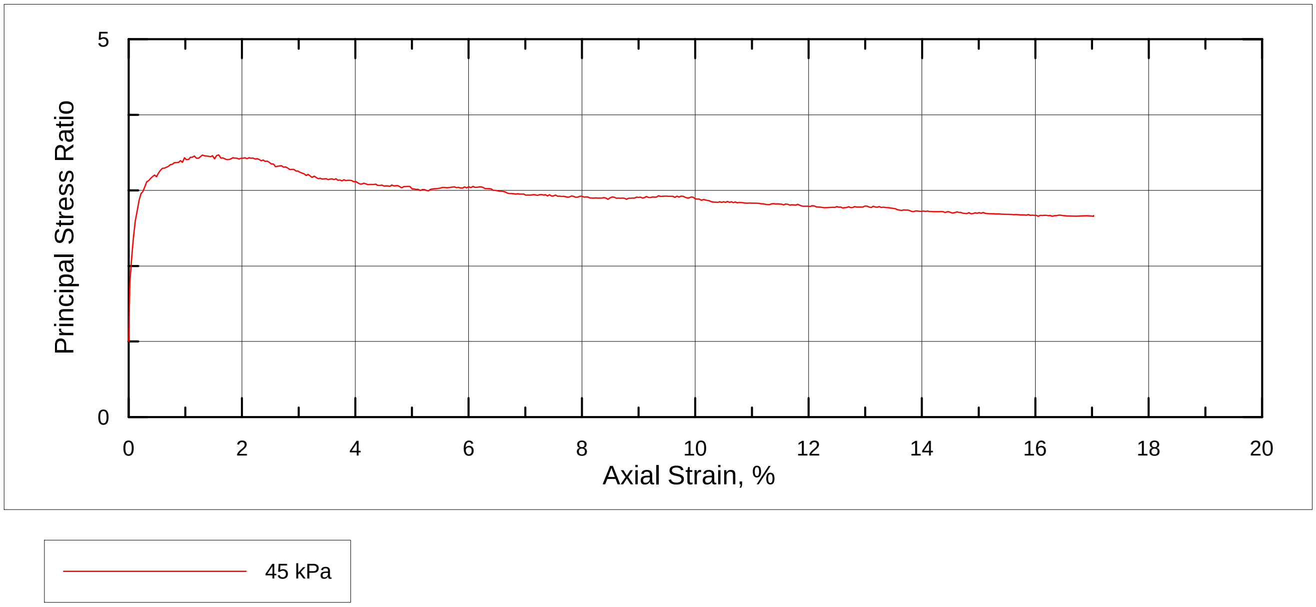
| Total Stress | | | | |
|---|---|---|---|---|
| Minor Principal Stress, $\sigma_3$ | kPa | 45.00 | | |
| Major Principal Stress, $\sigma_1$ | kPa | 197.09 | | |
| Radius, $(\sigma_1-\sigma_3)/2$ | kPa | 76.04 | | |
| Intersection Point, $(\sigma_1+\sigma_3)/2$ | kPa | 121.04 | | |

| Effective Stress | | | | |
|---|---|---|---|---|
| Minor Principal Stress, $\sigma_3'$ | kPa | 74.30 | | |
| Major Principal Stress, $\sigma_1'$ | kPa | 226.39 | | |
| Radius, $(\sigma_1'-\sigma_3')/2$ | kPa | 76.04 | | |
| Intersection Point, $(\sigma_1'+\sigma_3')/2$ | kPa | 150.34 | | |

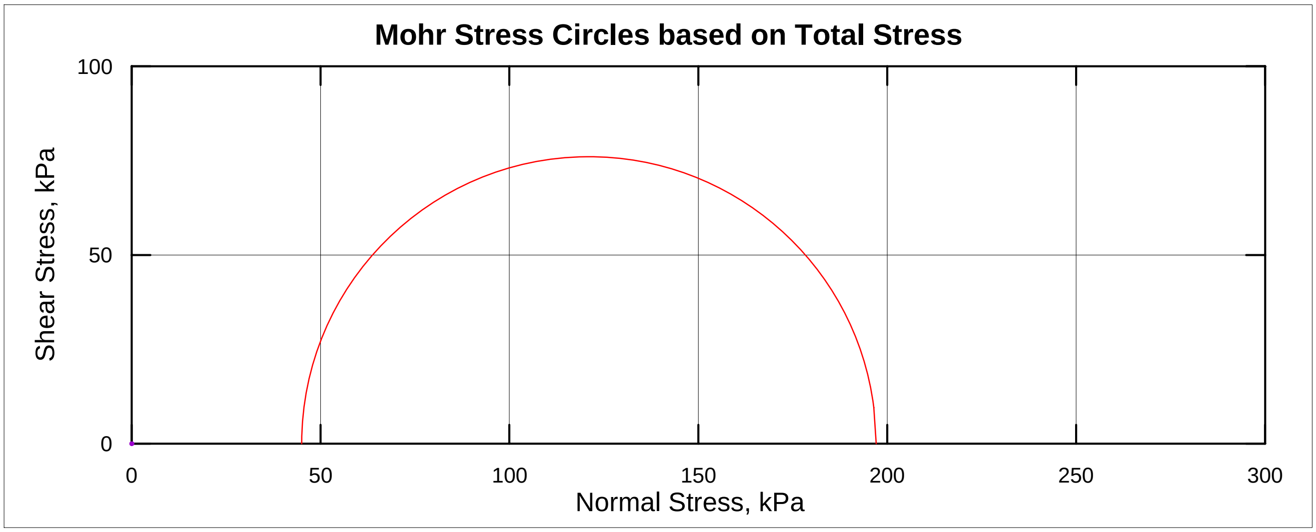
# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST
# FOR COHESIVE SOILS (ASTM D-4767)
# (Multi specimen - Single stage)

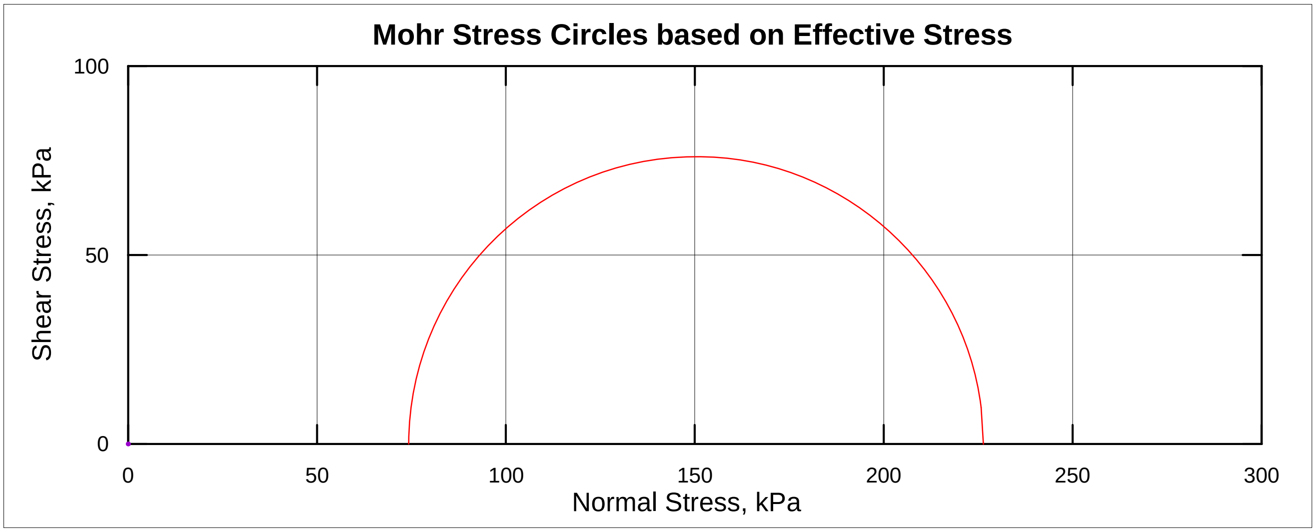


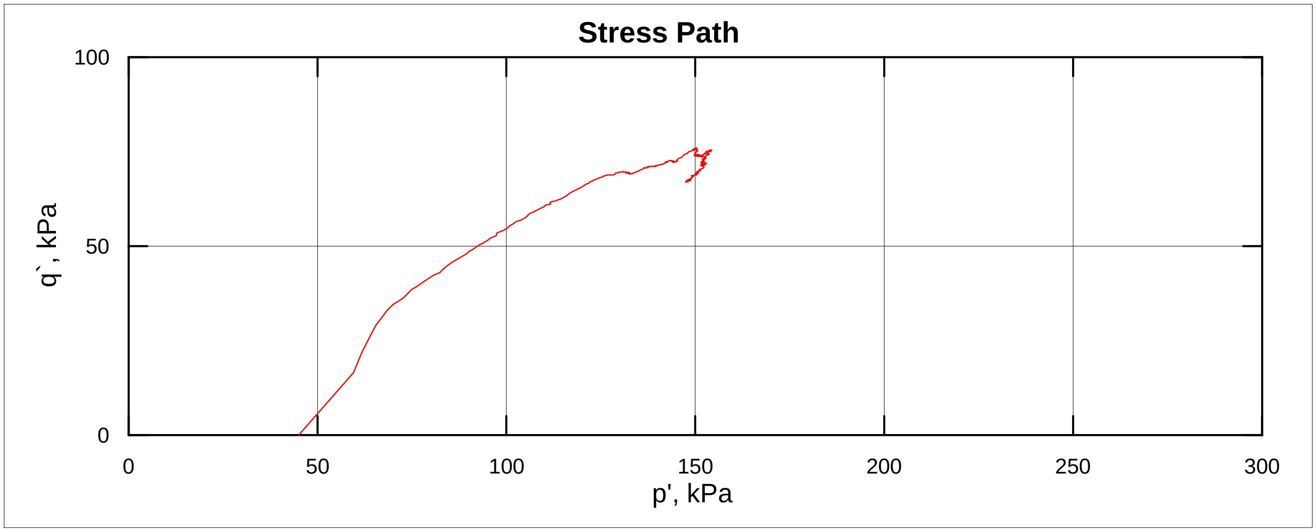




# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST FOR COHESIVE SOILS (ASTM D- 4767)
## (Multi specimen - single stage)
## (Failure based on maximum deviator stress)


Mohr Stress Circles based on Total Stress


Mohr Stress Circles based on Effective Stress


Stress Path

— 45 kPa

# Appendix D: Analytical Laboratory Test Results

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

Date Received: 25-JUL-11
Report Date: 29-JUL-11 20:50 (MT)
Version: FINAL

Client Phone: 519-735-2499

# Certificate of Analysis

**Lab Work Order #: L1035542**
Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 112634

Gayle Braun

Gayle Braun
Senior Account Manager

# ALS ENVIRONMENTAL ANALYTICAL REPORT

| | **Sample ID**<br>**Description**<br>**Sampled Date**<br>**Sampled Time**<br>**Client ID** | L1035542-1<br>SOIL<br>22-JUL-11<br><br>B11-1-SI,SA#9@25'SILTY CLAY, GREY | L1035542-2<br>SOIL<br>22-JUL-11<br><br>B11-6,TW,10@25'SILTY CLAY, GREY | L1035542-3<br>SOIL<br>22-JUL-11<br><br>B11-2,SA#9@100'SILTY CLAY, GREY | | |
|---|---|---|---|---|---|---|
| **Grouping** | **Analyte** | | | | | |
| **SOIL** | | | | | | |
| **Physical Tests** | % Moisture (%) | 13.3 | 12.4 | 9.15 | | |
| | pH (pH units) | 7.86 | 7.94 | 8.05 | | |
| | Redox Potential (mV) | 121 | 122 | 115 | | |
| | Resistivity (ohm cm) | 4170 | 2780 | 1710 | | |
| **Leachable Anions & Nutrients** | Sulphide (mg/kg) | <0.20 | <0.20 | <0.20 | | |
| **Anions and Nutrients** | Sulphate (mg/kg) | 218 | 549 | 745 | | |

# 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:**

112634

**GLOSSARY OF REPORT TERMS**

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

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

# Quality Control Report

Workorder: L1035542 Report Date: 29-JUL-11

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 R2224277** | | | | | | | | |
| **WG1318502-2 LCS** | | | | | | | | |
| % Moisture | | | 92 | | % | | 70-130 | 25-JUL-11 |
| **WG1318502-1 MB** | | | | | | | | |
| % Moisture | | | <0.10 | | % | | 0.1 | 25-JUL-11 |
| **PH-WT** | **Soil** | | | | | | | |
| **Batch R2226613** | | | | | | | | |
| **WG1321682-1 CVS** | | | | | | | | |
| pH | | | 100 | | % | | 80-120 | 27-JUL-11 |
| **RESISTIVITY-WT** | **Soil** | | | | | | | |
| **Batch R2226581** | | | | | | | | |
| **WG1319414-2 CVS** | | | | | | | | |
| Resistivity | | | 99 | | % | | 70-130 | 27-JUL-11 |
| **SO4-WT** | **Soil** | | | | | | | |
| **Batch R2225769** | | | | | | | | |
| **WG1319770-3 LCS** | | | | | | | | |
| Sulphate | | | 101 | | % | | 60-140 | 27-JUL-11 |
| **WG1319770-1 MB** | | | | | | | | |
| Sulphate | | | <20 | | mg/kg | | 20 | 27-JUL-11 |
| **SULPHIDE-WT** | **Soil** | | | | | | | |
| **Batch R2224730** | | | | | | | | |
| **WG1319337-1 CVS** | | | | | | | | |
| Sulphide | | | 96 | | % | | 50-120 | 26-JUL-11 |
| **WG1319332-1 MB** | | | | | | | | |
| Sulphide | | | <0.20 | | mg/kg | | 0.2 | 26-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. |

**Hold Time Exceedances:**

| ALS Product Description | Sample ID | Sampling Date | Date Processed | Rec. HT | Actual HT | Units | Qualifier |
|---|---|---|---|---|---|---|---|
| **Physical Tests** | | | | | | | |
| Redox Potential | | | | | | | |
| | 1 | 22-JUL-11 | 27-JUL-11 14:06 | 24 | 122 | hours | EHTR |
| | 2 | 22-JUL-11 | 27-JUL-11 14:07 | 24 | 122 | hours | EHTR |
| | 3 | 22-JUL-11 | 27-JUL-11 14:08 | 24 | 122 | hours | EHTR |

**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 L1035542 were received on 25-JUL-11 10:00.

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.

112634

C of C # 00000

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

ALS Environmental

# CHAIN OF CUSTODY / ANALYTICAL SERVICES REQUEST FORM

Page ___ of ___

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

Specify date required

| Service requested | | | |
|---|---|---|---|
| 5 day (regular) | ✓ | 2 day TAT (50%) | |
| 3-4 day (25%) | | Next day TAT (100%) | |
| | | Same day TAT (200%) | |

COMPANY NAME: Amec E+I
OFFICE: Windsor
PROJECT MANAGER: Shane Macleod
PROJECT #: SWE601.1004.101
PHONE: 519-735-2499
FAX: 519-735-8668
ACCOUNT #:
QUOTATION #: Q28643
PO #:

CRITERIA — Criteria on report YES___ NO___
Reg 153/04 ☐ Reg 511/09 ☐
Table 1 2 3 4 5 6 7 8 9
TCLP ___ MISA ___ PWQO ___
ODWS ___ OTHER ___

REPORT FORMAT/DISTRIBUTION
EMAIL ✓ FAX ___ BOTH ___
SELECT: PDF ___ DIGITAL ___ BOTH ___
EMAIL 1: Shane.Macleod@Amec.com
EMAIL 2:

ANALYSIS REQUEST: Corrosion Package

PLEASE INDICATE FILTERED, PRESERVED OR BOTH <---- (F, P, F/P)

SUBMISSION #: L1035542
ENTERED BY: PSta Stny
DATE/TIME ENTERED: 25-July-11
BIN #:

SAMPLING INFORMATION

| Date (dd-mm-yy) | Time (24hr) (hh:mm) | COMP | GRAB | WATER | SOIL | OTHER | SAMPLE DESCRIPTION TO APPEAR ON REPORT | NUMBER OF CONTAINERS | Corrosion Package | COMMENTS | LAB ID |
|---|---|---|---|---|---|---|---|---|---|---|---|
| July 22, 2011 | | | | | X | | BH-1-SI, Sa#9@25' silty clay grey | 1 | X | | -1 |
| July 22, 2011 | | | | | X | | BH-G, TW 10@25' silty clay, grey | 1 | X | | -2 |
| July 22, 2011 | | | | | X | | BH-2, Sa#25@105' sandy, grey | 1 | X | | -3 |

SPECIAL INSTRUCTIONS/COMMENTS

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

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

SAMPLED BY:
DATE & TIME:
RECEIVED BY:
DATE & TIME:

RELINQUISHED BY:
DATE & TIME:
RECEIVED AT LAB BY:
DATE & TIME: 25-July-11 10:00

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

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

Date Received: 30-JUN-11
Report Date: 08-JUL-11 07:09 (MT)
Version: FINAL

Client Phone: 519-735-2499

# Certificate of Analysis

**Lab Work Order #: L1025351**
Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 092732-1

Gayle Braun

Gayle Braun
Senior Account Manager

# ALS ENVIRONMENTAL ANALYTICAL REPORT

| | **Sample ID**<br>**Description**<br>**Sampled Date**<br>**Sampled Time**<br>**Client ID** | L1025351-1<br>SOIL<br>29-JUN-11<br><br>B11-3 SA#10 30’ | L1025351-2<br>SOIL<br>29-JUN-11<br><br>B11-4 SA#27 115’ | L1025351-3<br>SOIL<br>29-JUN-11<br><br>B11-7 SA#10 30’ | | |
|---|---|---|---|---|---|---|
| **Grouping** | **Analyte** | | | | | |
| **SOIL** | | | | | | |
| **Physical Tests** | % Moisture (%) | 14.8 | 13.1 | 13.6 | | |
| | pH (pH units) | 7.83 | 7.86 | 7.80 | | |
| | Redox Potential (mV) | 136 | 154 | 98.0 | | |
| | Resistivity (ohm cm) | 3680 | 1790 | 4080 | | |
| **Leachable Anions & Nutrients** | Sulphide (mg/kg) | <0.20 | <0.20 | <0.20 | | |
| **Anions and Nutrients** | Sulphate (mg/kg) | 106 | 608 | 120 | | |

# 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:**

092732-1

**GLOSSARY OF REPORT TERMS**

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

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

# Quality Control Report

Workorder: L1025351 Report Date: 08-JUL-11

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 R2212761** | | | | | | | | |
| **WG1305298-3 DUP** | | **L1025351-1** | | | | | | |
| % Moisture | | 14.8 | 14.3 | | % | 3.3 | 30 | 30-JUN-11 |
| **WG1305298-2 LCS** | | | | | | | | |
| % Moisture | | | 98 | | % | | 70-130 | 30-JUN-11 |
| **WG1305298-1 MB** | | | | | | | | |
| % Moisture | | | <0.10 | | % | | 0.1 | 30-JUN-11 |
| **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 |
| **WG1307906-2 DUP** | | **L1025351-1** | | | | | | |
| pH | | 7.83 | 7.91 | | pH units | 1.0 | 20 | 06-JUL-11 |
| **REDOX-POTENTIAL-WT** | **Soil** | | | | | | | |
| **Batch R2215161** | | | | | | | | |
| **WG1308648-1 DUP** | | **L1025351-2** | | | | | | |
| Redox Potential | | 154 | 156 | | mV | 1.3 | 25 | 07-JUL-11 |
| **RESISTIVITY-WT** | **Soil** | | | | | | | |
| **Batch R2215155** | | | | | | | | |
| **WG1308646-1 CVS** | | | | | | | | |
| Resistivity | | | 100 | | % | | 70-130 | 07-JUL-11 |
| **WG1308646-2 DUP** | | **L1025351-2** | | | | | | |
| Resistivity | | 1790 | 1760 | | ohm cm | 1.4 | 25 | 07-JUL-11 |
| **SO4-WT** | **Soil** | | | | | | | |
| **Batch R2213607** | | | | | | | | |
| **WG1306314-2 DUP** | | **L1025351-1** | | | | | | |
| Sulphate | | 106 | 106 | | mg/kg | 0.49 | 30 | 04-JUL-11 |
| **WG1306314-3 LCS** | | | | | | | | |
| Sulphate | | | 101 | | % | | 60-140 | 04-JUL-11 |
| **WG1306314-1 MB** | | | | | | | | |
| Sulphate | | | <20 | | mg/kg | | 20 | 04-JUL-11 |
| **SULPHIDE-WT** | **Soil** | | | | | | | |

# Quality Control Report

| Test | Matrix | Reference | Result | Qualifier | Units | RPD | Limit | Analyzed |
|---|---|---|---|---|---|---|---|---|
| **SULPHIDE-WT** | **Soil** | | | | | | | |
| **Batch R2213798** | | | | | | | | |
| **WG1307079-1 CVS** | | | | | | | | |
| Sulphide | | | 79 | | % | | 50-120 | 05-JUL-11 |
| **WG1307075-2 DUP** | | **L1025351-1** | | | | | | |
| Sulphide | | <0.20 | <0.20 | RPD-NA | mg/kg | N/A | 20 | 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 17:00 | 24 | 197 | hours | EHTL |
| | 2 | 29-JUN-11 | 07-JUL-11 17:01 | 24 | 197 | hours | EHTL |
| | 3 | 29-JUN-11 | 07-JUL-11 17:03 | 24 | 197 | hours | EHTL |
| Resistivity | | | | | | | |
| | 1 | 29-JUN-11 | 07-JUL-11 16:57 | 7 | 8 | days | EHT |
| | 2 | 29-JUN-11 | 07-JUL-11 16:58 | 7 | 8 | days | EHT |
| | 3 | 29-JUN-11 | 07-JUL-11 17:00 | 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 L1025351 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.

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

ALS
ALS Environmental

# CHAIN OF CUSTODY / ANALYTICAL SERVICES REQUEST FORM

C of C # 092732-1
PAGE 1 OF 1

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

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

COMPANY NAME AMEC E+I
OFFICE SW Windsor / Tecumseh
PROJECT MANAGER Brian Lapos
PROJECT # SW8801.1004.101
PHONE 519-735-3459 FAX 519-735-9669
ACCOUNT # 15555
QUOTATION # Q28643 PO#

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

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

PLEASE INDICATE FILTERED, PRESERVED OR BOTH ⇦ (F, P, F/P)
SUBMISSION # L1025351
ENTERED BY: PStastny
DATE/TIME ENTERED: 30-June-11
BIN #

SAMPLING INFORMATION

ANALYSIS REQUEST

| Date (dd-mm-yy) | Time (24 hr) (hh:mm) | COMP | GRAB | WATER | SOIL | OTHER | SAMPLE DESCRIPTION TO APPEAR ON REPORT | NUMBER OF CONTAINERS | Corrosion package | | | | | | | | | | COMMENTS | LAB ID |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| June 29/11 | | | | | X | | B11-3 Sa# 10 30' | 1 | ✓ | | | | | | | | | | | -1 |
| June 29/11 | | | | | X | | B11-4 Sa# 27 115' | 1 | ✓ | | | | | | | | | | | -2 |
| June 29/11 | | | | | X | | B11-7 Sa# 10 30' | 1 | ✓ | | | | | | | | | | | -3 |
| | | | | | | | | | | | | | | | | | | | | |
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SPECIAL INSTRUCTIONS/COMMENTS

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

SAMPLE CONDITION
FROZEN ☐
COLD ☑
COOLING INITIATED ☑
AMBIENT ☐
MEAN TEMP 11.9
OBSERVATIONS Yes ☐ No ☐ If yes add SIF
INIT

SAMPLED BY: M.K.
DATE & TIME June 29, 2011
RECEIVED BY:
DATE & TIME
RELINQUISHED BY:
DATE & TIME
RECEIVED AT LAB BY:
DATE & TIME 6/30/11 1100

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

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

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

Date Received: 03-JUN-11
Report Date: 09-JUN-11 12:42 (MT)
Version: FINAL

Client Phone: 519-735-2499

# Certificate of Analysis

**Lab Work Order #: L1012447**
Project P.O. #: NOT SUBMITTED
Job Reference: SW8801.1004.101
Legal Site Desc:
C of C Numbers: 092961

Gayle Braun

Gayle Braun
Senior Account Manager

# ALS ENVIRONMENTAL ANALYTICAL REPORT

| | | | | | | |
|---|---|---|---|---|---|---|
| | **Sample ID** | L1012447-1 | | | | |
| | **Description** | SOIL | | | | |
| | **Sampled Date** | 29-MAY-11 | | | | |
| | **Sampled Time** | | | | | |
| | **Client ID** | B11-5<br>SA#29@120-121’ | | | | |
| **Grouping** | **Analyte** | | | | | |
| **SOIL** | | | | | | |
| **Physical Tests** | % Moisture (%) | 10.3 | | | | |
| | pH (pH units) | 7.83 | | | | |
| | Redox Potential (mV) | 119 | | | | |
| | Resistivity (ohm cm) | 5680 | | | | |
| **Leachable Anions & Nutrients** | Sulphide (mg/kg) | <0.20 | | | | |
| **Anions and Nutrients** | Sulphate (mg/kg) | 52 | | | | |

# 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:**

092961

**GLOSSARY OF REPORT TERMS**

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

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

# Quality Control Report

Workorder: L1012447 Report Date: 09-JUN-11

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 R2199427** | | | | | | | | |
| **WG1290566-2 LCS** | | | | | | | | |
| % Moisture | | | 93 | | % | | 70-130 | 06-JUN-11 |
| **WG1290566-1 MB** | | | | | | | | |
| % Moisture | | | <0.10 | | % | | 0.1 | 06-JUN-11 |
| **PH-WT** | **Soil** | | | | | | | |
| **Batch R2198896** | | | | | | | | |
| **WG1290364-1 CVS** | | | | | | | | |
| pH | | | 100 | | % | | 80-120 | 04-JUN-11 |
| **RESISTIVITY-WT** | **Soil** | | | | | | | |
| **Batch R2198903** | | | | | | | | |
| **WG1290368-1 CVS** | | | | | | | | |
| Resistivity | | | 98 | | % | | 70-130 | 04-JUN-11 |
| **SO4-WT** | **Soil** | | | | | | | |
| **Batch R2200711** | | | | | | | | |
| **WG1291932-3 LCS** | | | | | | | | |
| Sulphate | | | 94 | | % | | 60-140 | 08-JUN-11 |
| **WG1291932-1 MB** | | | | | | | | |
| Sulphate | | | <20 | | mg/kg | | 20 | 08-JUN-11 |
| **SULPHIDE-WT** | **Soil** | | | | | | | |
| **Batch R2200565** | | | | | | | | |
| **WG1292239-1 CVS** | | | | | | | | |
| Sulphide | | | 84 | | % | | 50-120 | 08-JUN-11 |
| **WG1292235-1 MB** | | | | | | | | |
| Sulphide | | | <0.20 | | mg/kg | | 0.2 | 08-JUN-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-MAY-11 | 04-JUN-11 19:49 | 24 | 152 | hours | EHTR |
| **Leachable Anions & Nutrients** | | | | | | | |
| Sulphide | | | | | | | |
| | 1 | 29-MAY-11 | 08-JUN-11 14:49 | 7 | 10 | 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 L1012447 were received on 03-JUN-11 10:42.

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

---

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

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

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

ALS
ALS Environmental

# CHAIN OF CUSTODY / ANALYTICAL SERVICES REQUEST FORM

C of C # 092961
PAGE ____ OF ____

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

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

| | |
|---|---|
| COMPANY NAME | AMEC E + I |
| OFFICE | SW \| Windsor/Tecumseh |
| PROJECT MANAGER | Brian Lapos |
| PROJECT # | SW8801.1004.101 |
| PHONE | 519-735-2499 |
| FAX | 519-735-7669 |
| ACCOUNT # | |
| QUOTATION# | Q28645 |
| PO# | |

**CRITERIA** Criteria on report Yes ☐ No ☐

Reg 153/04
Table 1 2 3
TCLP ______ MISA ____ PWQO ______
ODWS ______ OTHER ______________

**REPORT FORMAT / DISTRIBUTION**

EMAIL ✓ FAX ______ BOTH ______
SELECT: PDF ____ DIGITAL ____ BOTH ______
EMAIL1 Shane.macleod@amec.com
EMAIL2 ______________

PLEASE INDICATE FILTERED, PRESERVED OR BOTH ⇦ (F, P, F/P)

SUBMISSION # L1012447
ENTERED BY:
DATE/TIME ENTERED: 3June11
BIN #

**SAMPLING INFORMATION**

| Sample Date/Time | | TYPE | | MATRIX | | | | | ANALYSIS REQUEST | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Date (dd-mm-yy) | Time (24 hr) (hh:mm) | COMP | GRAB | WATER | SOIL | OTHER | SAMPLE DESCRIPTION TO APPEAR ON REPORT | NUMBER OF CONTAINERS | Corrosion Package | | | | | | | | | | COMMENTS | LAB ID |
| 27 05 11 | | | | | ✓ | | B11-S Sa# 29@120-121' | 1 | ✓ | | | | | | | | | | | -1 |

**SPECIAL INSTRUCTIONS/COMMENTS**

**THE QUESTIONS BELOW MUST BE ANSWERED FOR WATER SAMPLES (CHECK Yes OR No)**

Are any samples taken from a regulated DW System? Yes ☐ No ☐
If yes, an authorized drinking water COC MUST be used for this submission.
Is the water sampled intended to be potable for human consumption? Yes ☐ No ☐

**SAMPLE CONDITION**

| | | |
|---|---|---|
| FROZEN | ☐ | MEAN TEMP |
| COLD | ☑ | 7.5 |
| COOLING INITIATED | ☐ | |
| AMBIENT | ☐ | |

| | | | |
|---|---|---|---|
| SAMPLED BY: T.A. | DATE & TIME | RECEIVED BY: | DATE & TIME |
| RELINQUISHED BY: | DATE & TIME | RECEIVED AT LAB BY: | DATE & TIME 3June11 1000 |

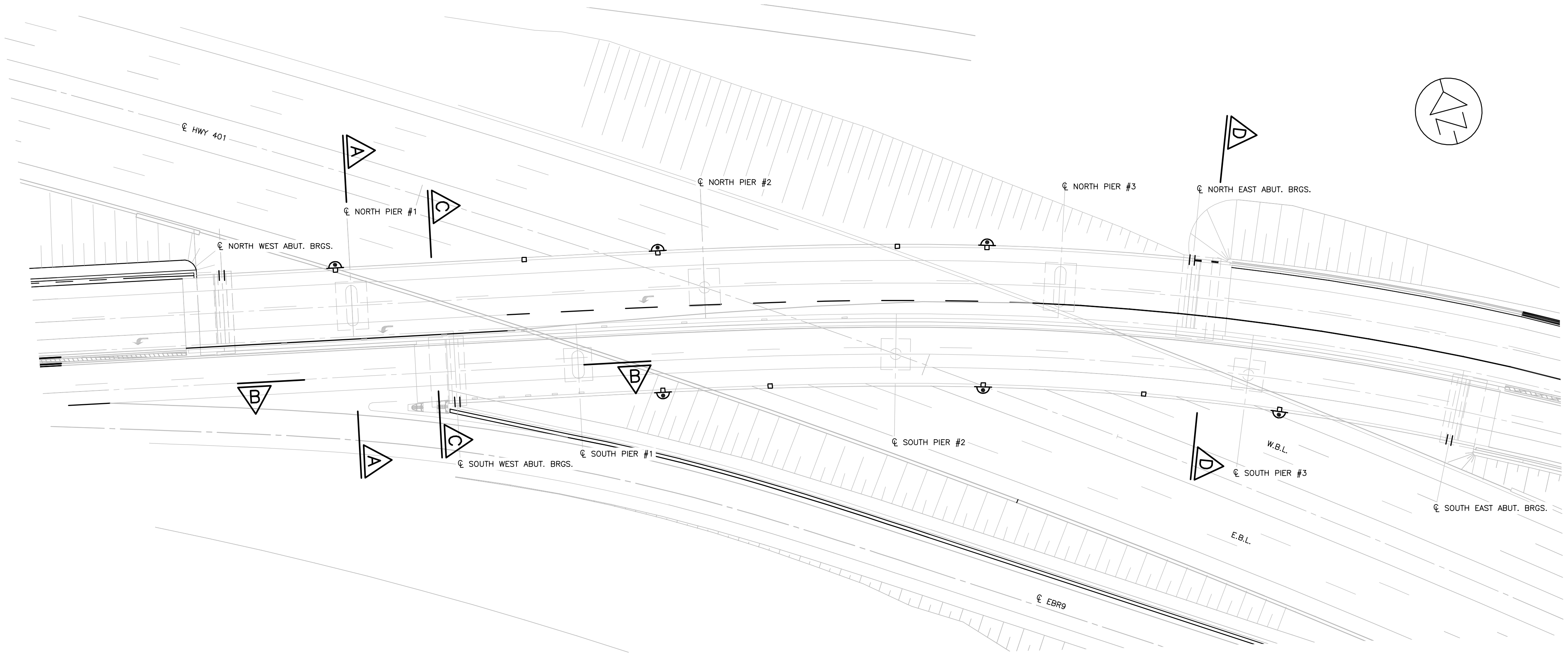
OBSERVATIONS Yes ☐ No ☐
If yes add SIF
INIT

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

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

Reg COC Rev#4 09

# Appendix E: Slope Stability Analyses



PLAN

SCALE 1:500

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR
CONSTRUCTION

**Parkway Infrastructure Engineers**

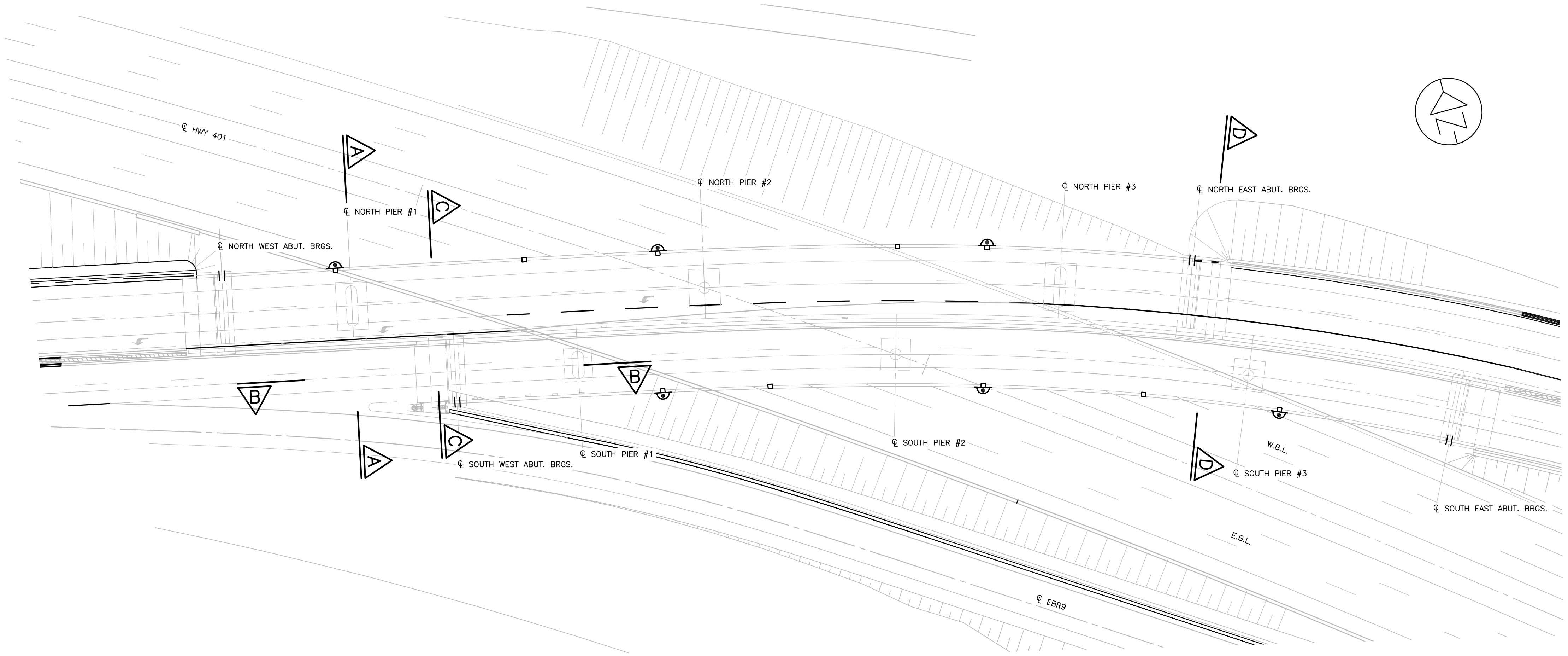
amec

Hatch Mott MacDonald

BRIDGE B–11
GLOBAL STABILITY
ANALYZED SECTIONS

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

DOC: GLOBAL STABILITY FOR ANALYZED SECTIONS_FIG E.1



PLAN
SCALE 1:500

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR
CONSTRUCTION

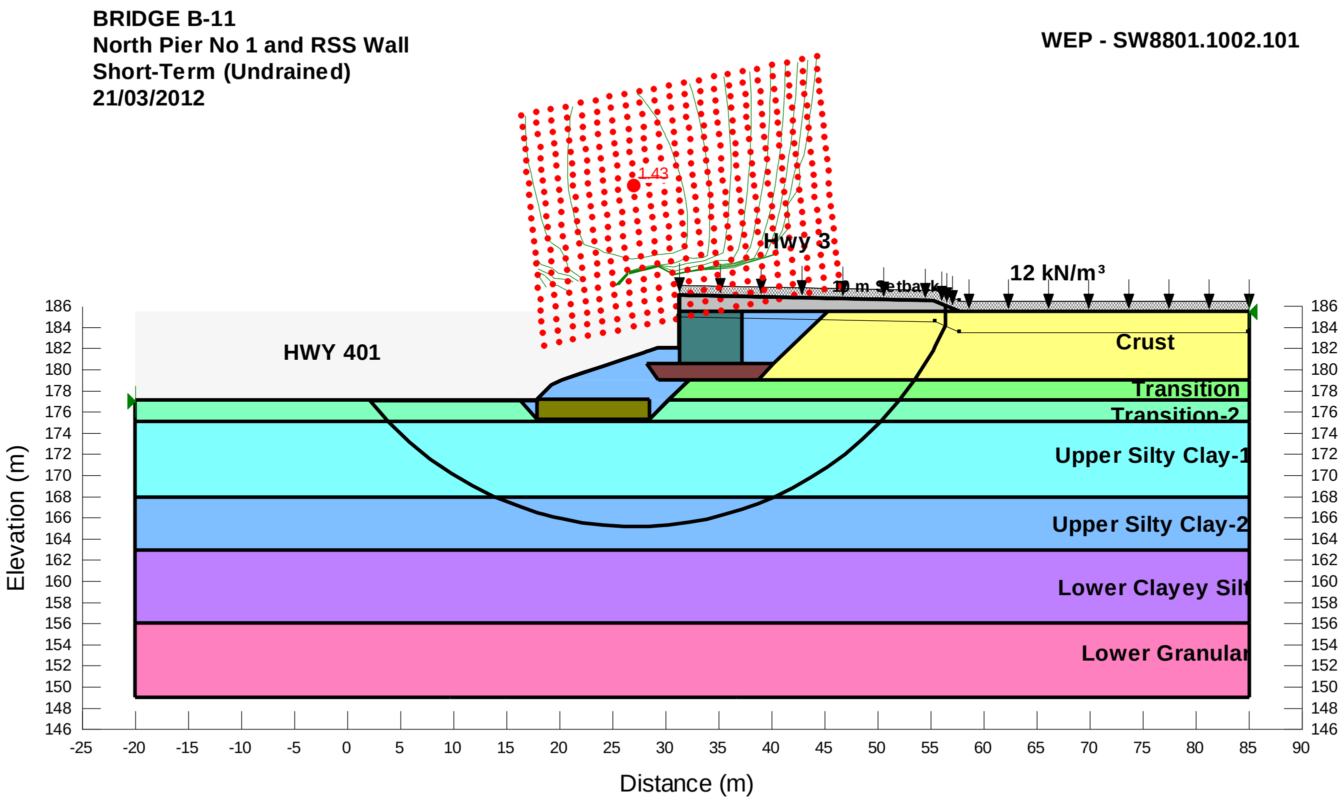
**Parkway Infrastructure Engineers**
amec
Hatch Mott MacDonald

BRIDGE B–11
GLOBAL STABILITY
ANALYZED SECTIONS

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

DOC: GLOBAL STABILITY FOR ANALYZED SECTIONS_FIG E.1

## Figure E-2: Global Stability Result – North Pier No. 1 and RSS Wall – Short-Term (Undrained properties)

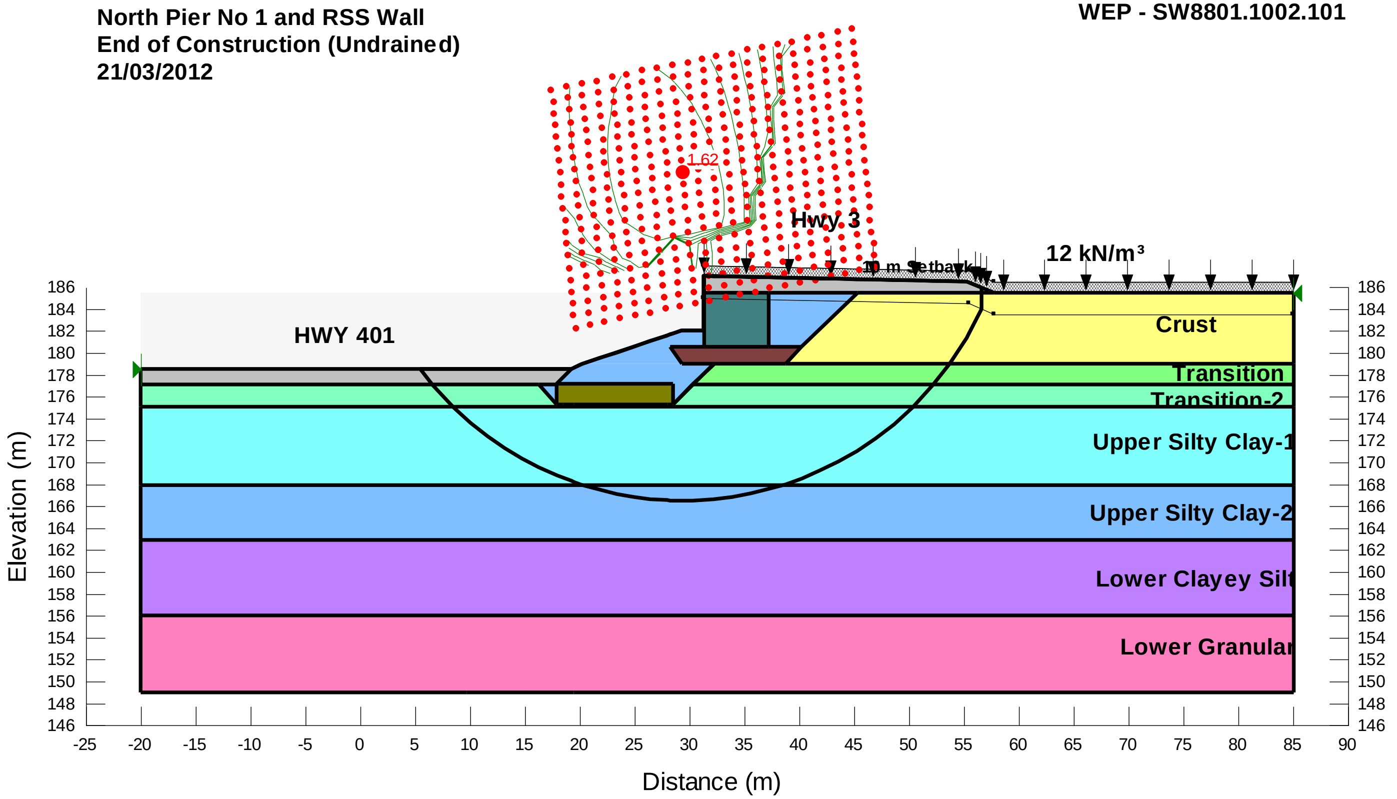


Name: Crust Unit Weight: 22 kN/m³ Cohesion: 75 kPa
Name: Transition-1 Unit Weight: 21.5 kN/m³ Cohesion: 75 kPa
Name: Transition-2 Unit Weight: 21.5 kN/m³ C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m
Name: Upper Silty Clay-1 Unit Weight: 21 kN/m³ C-Datum: 55 kPa C-Rate of Change: -0.5556 kPa/m Limiting C: 51 kPa Elevation: 175 m
Name: Upper Silty Clay-2 Unit Weight: 20.5 kN/m³ C-Datum: 51.1 kPa C-Rate of Change: 1.3888 kPa/m Limiting C: 58 kPa Elevation: 168 m
Name: Lower Clayey Silt Unit Weight: 22 kN/m³ C-Datum: 58 kPa C-Rate of Change: 21 kPa/m Limiting C: 100 kPa Elevation: 163 m
Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Clay Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa
Name: RGM Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 35 °
Name: Concrete Unit Weight: 22 kN/m³ Cohesion: 2000 kPa Phi: 35 °

## Figure E-3: Global Stability Result – North Pier No. 1 and RSS Wall – End of Construction Loading (Undrained properties

**BRIDGE B-11**
**North Pier No 1 and RSS Wall**
**End of Construction (Undrained)**
**21/03/2012**

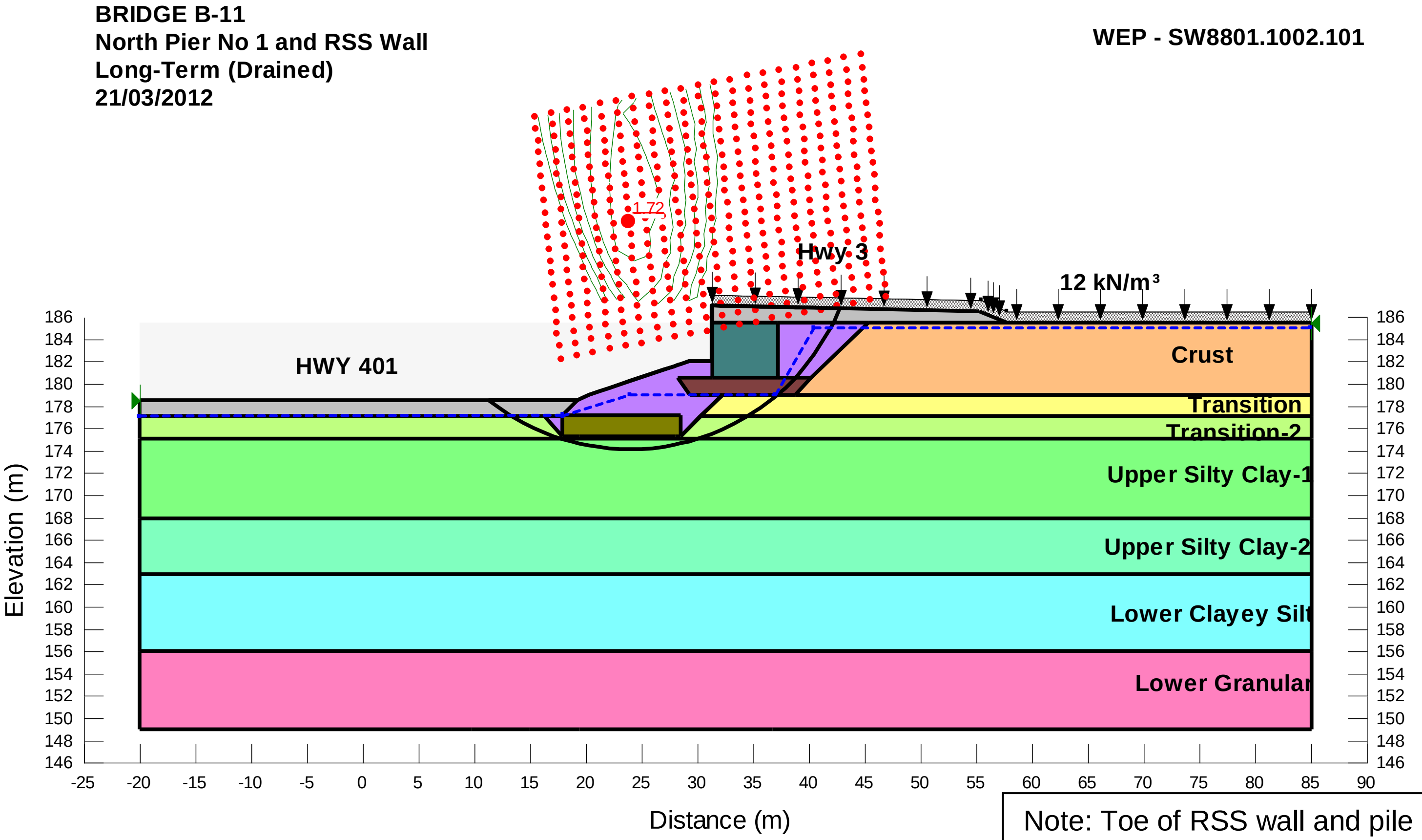
**WEP - SW8801.1002.101**



Name: Crust Unit Weight: 22 kN/m³ Cohesion: 75 kPa
Name: Transition-1 Unit Weight: 21.5 kN/m³ Cohesion: 75 kPa
Name: Transition-2 Unit Weight: 21.5 kN/m³ C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m
Name: Upper Silty Clay-1 Unit Weight: 21 kN/m³ C-Datum: 55 kPa C-Rate of Change: -0.5556 kPa/m Limiting C: 51 kPa Elevation: 175 m
Name: Upper Silty Clay-2 Unit Weight: 20.5 kN/m³ C-Datum: 51.1 kPa C-Rate of Change: 1.3888 kPa/m Limiting C: 58 kPa Elevation: 168 m
Name: Lower Clayey Silt Unit Weight: 22 kN/m³ C-Datum: 58 kPa C-Rate of Change: 21 kPa/m Limiting C: 100 kPa Elevation: 163 m
Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Clay Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa
Name: RGM Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 35 °
Name: Concrete Unit Weight: 22 kN/m³ Cohesion: 2000 kPa Phi: 35 °

Note: Toe of RSS wall and pile cap excavation should be backfilled prior to construction of RSS wall.

# Figure E-4: Global Stability Result – North Pier No. 1 and RSS Wall – Long-term Loading (Drained properties)



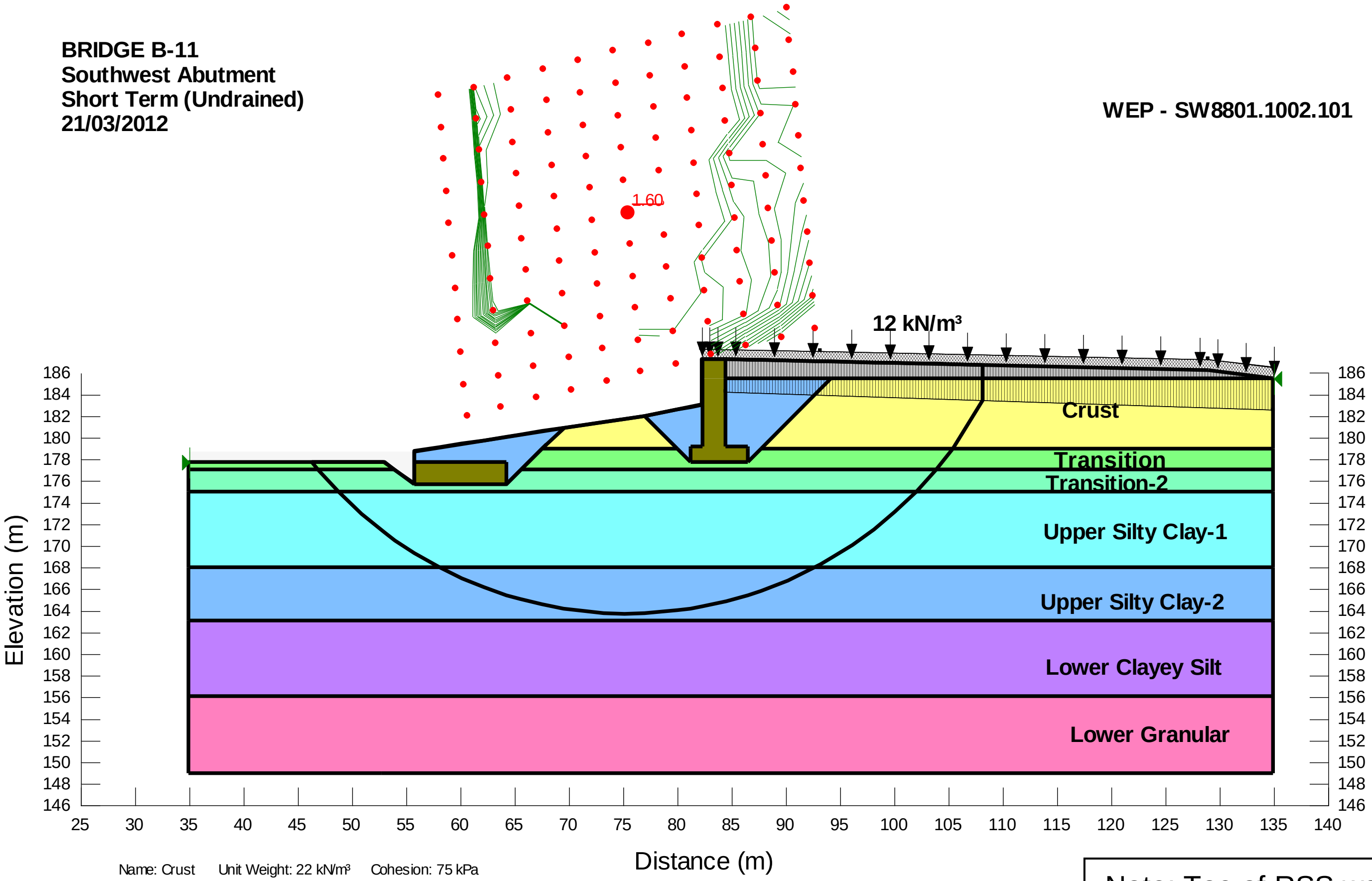
**BRIDGE B-11**
**North Pier No 1 and RSS Wall**
**Long-Term (Drained)**
**21/03/2012**

**WEP - SW8801.1002.101**

Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Crust (Drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Transition-1 (Drained) Unit Weight: 21.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Transition-2 (Drained) Unit Weight: 21.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-1 (Drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-2 (Drained) Unit Weight: 20.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Lower Clayey Silt (Drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clay Backfill (Drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: RGM Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 35 °
Name: Concrete Unit Weight: 22 kN/m³ Cohesion: 2000 kPa Phi: 35 °

Note: Toe of RSS wall and pile cap excavation should be backfilled prior to construction of RSS wall.

## Figure E-5: Global Stability Result – Southwest Abutment – Short-Term Loading (Undrained properties)



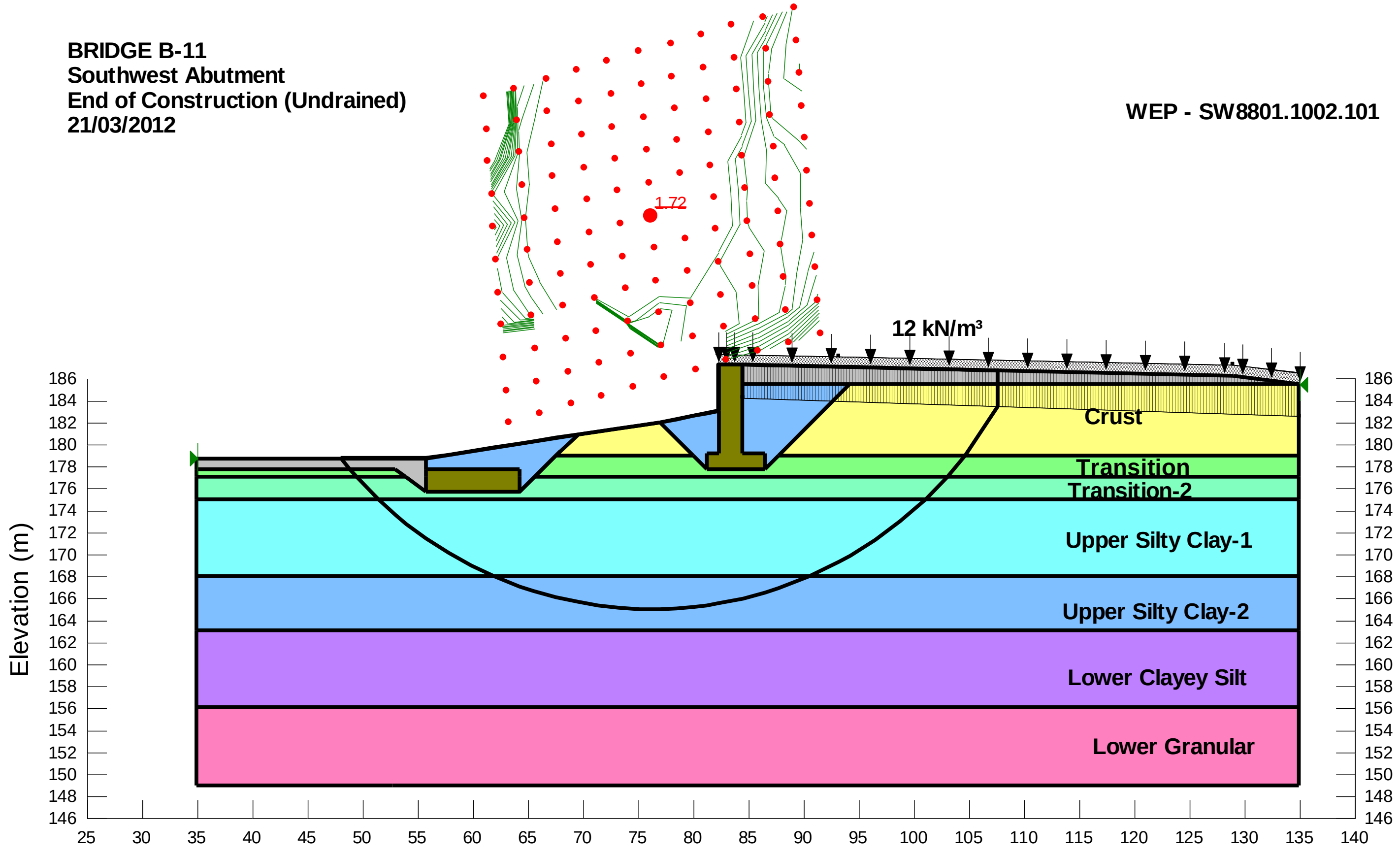
Name: Crust Unit Weight: 22 kN/m³ Cohesion: 75 kPa
Name: Transition-1 Unit Weight: 21.5 kN/m³ Cohesion: 75 kPa
Name: Transition-2 Unit Weight: 21.5 kN/m³ C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m
Name: Upper Silty Clay-1 Unit Weight: 21 kN/m³ C-Datum: 55 kPa C-Rate of Change: -0.5556 kPa/m Limiting C: 51 kPa Elevation: 175 m
Name: Upper Silty Clay-2 Unit Weight: 20.5 kN/m³ C-Datum: 51.1 kPa C-Rate of Change: 1.3888 kPa/m Limiting C: 58 kPa Elevation: 168 m
Name: Lower Clayey Silt Unit Weight: 22 kN/m³ C-Datum: 58 kPa C-Rate of Change: 21 kPa/m Limiting C: 100 kPa Elevation: 163 m
Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Clay Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Concrete Unit Weight: 22 kN/m³ Cohesion: 2000 kPa Phi: 35 °

Note: Toe of RSS wall and pile cap excavation should be backfilled prior to construction of RSS wall.

## Figure E-6: Global Stability Result – Southwest Abutment – End of Construction Loading (Undrained properties)

**BRIDGE B-11**
**Southwest Abutment**
**End of Construction (Undrained)**
**21/03/2012**

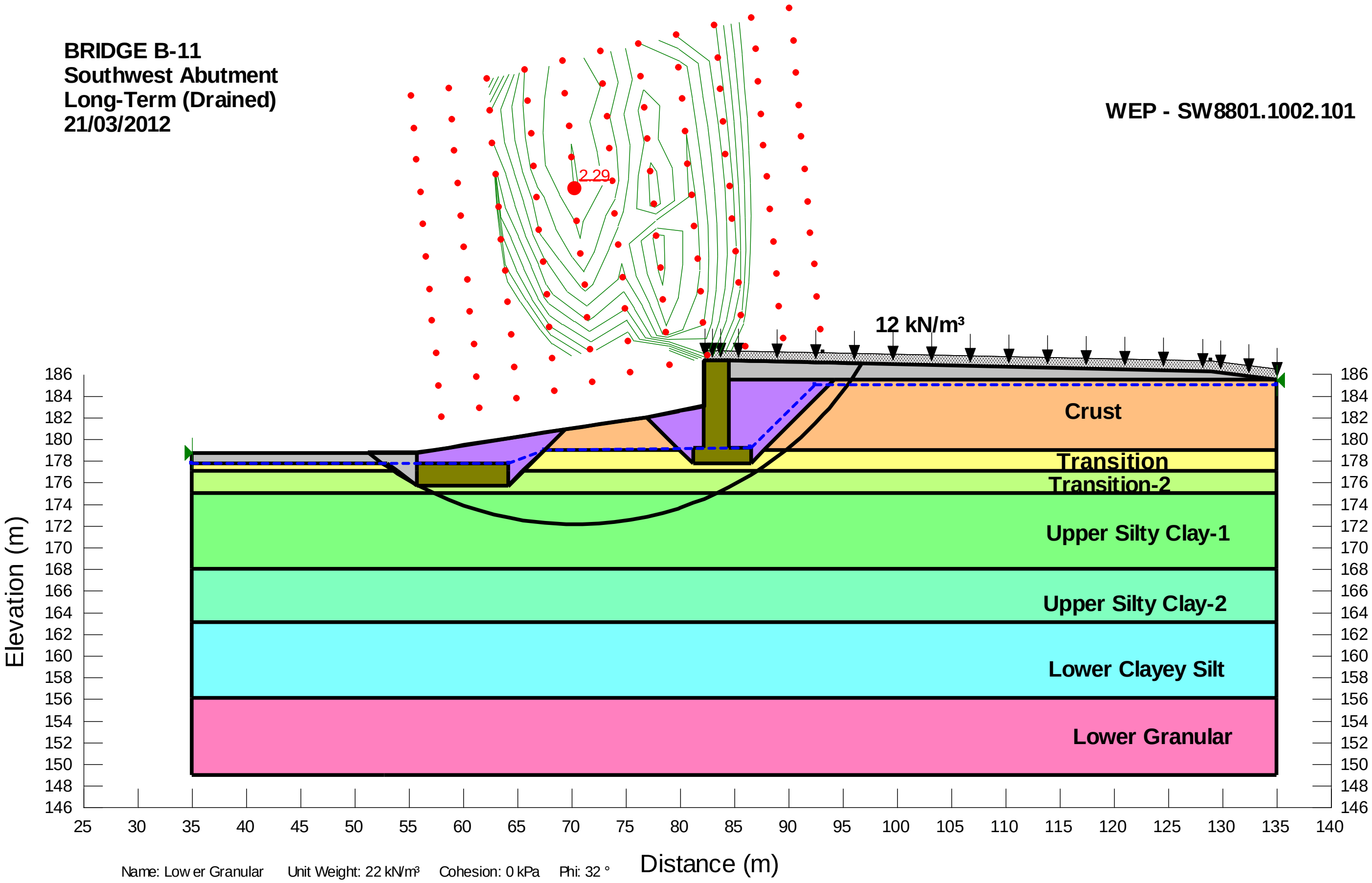
**WEP - SW8801.1002.101**



Name: Crust Unit Weight: 22 kN/m³ Cohesion: 75 kPa
Name: Transition-1 Unit Weight: 21.5 kN/m³ Cohesion: 75 kPa
Name: Transition-2 Unit Weight: 21.5 kN/m³ C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m
Name: Upper Silty Clay-1 Unit Weight: 21 kN/m³ C-Datum: 55 kPa C-Rate of Change: -0.5556 kPa/m Limiting C: 51 kPa Elevation: 175 m
Name: Upper Silty Clay-2 Unit Weight: 20.5 kN/m³ C-Datum: 51.1 kPa C-Rate of Change: 1.3888 kPa/m Limiting C: 58 kPa Elevation: 168 m
Name: Lower Clayey Silt Unit Weight: 22 kN/m³ C-Datum: 58 kPa C-Rate of Change: 21 kPa/m Limiting C: 100 kPa Elevation: 163 m
Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Clay Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Concrete Unit Weight: 22 kN/m³ Cohesion: 2000 kPa Phi: 35 °

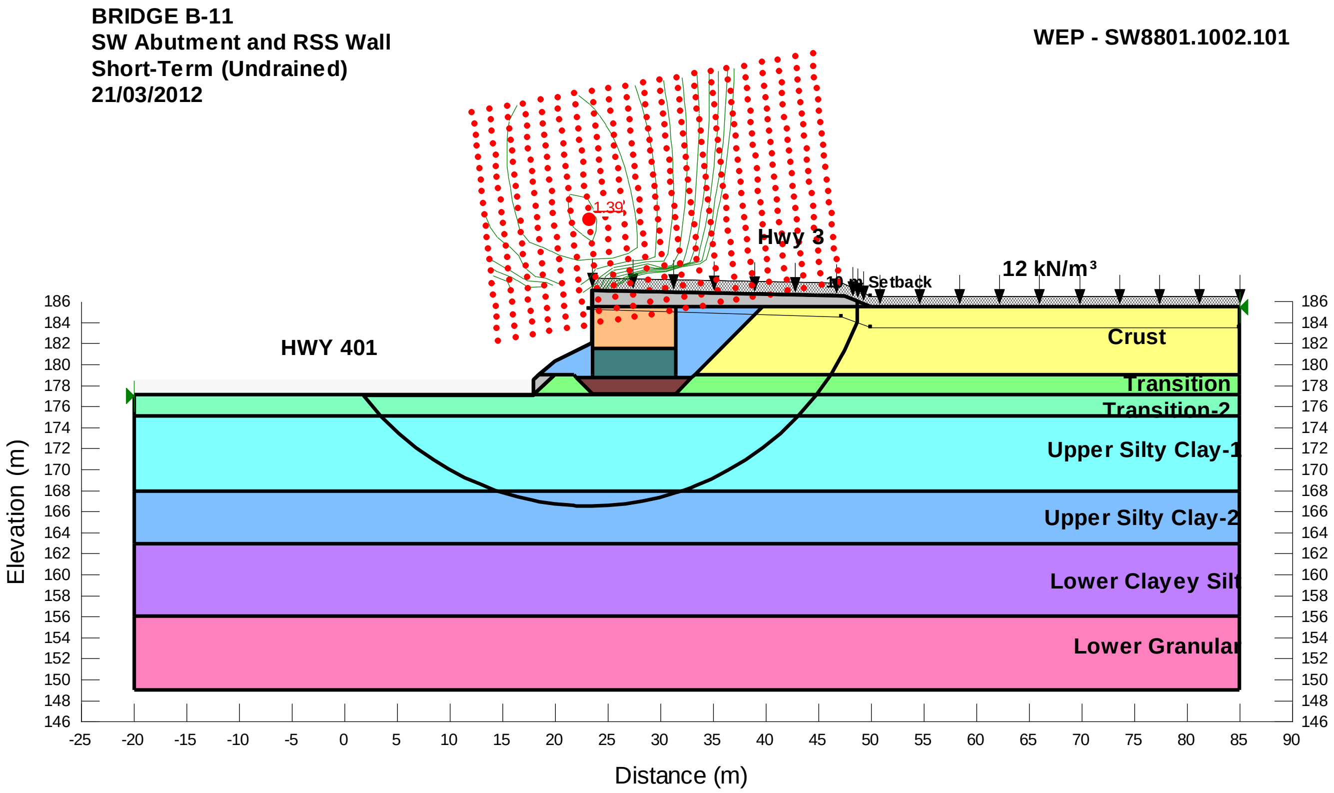
Note: Assumes that pile cap excavation should be backfilled prior to backfilling abutment.

## Figure E-7: Global Stability Result – Southwest Abutment – Long-Term (Drained properties)



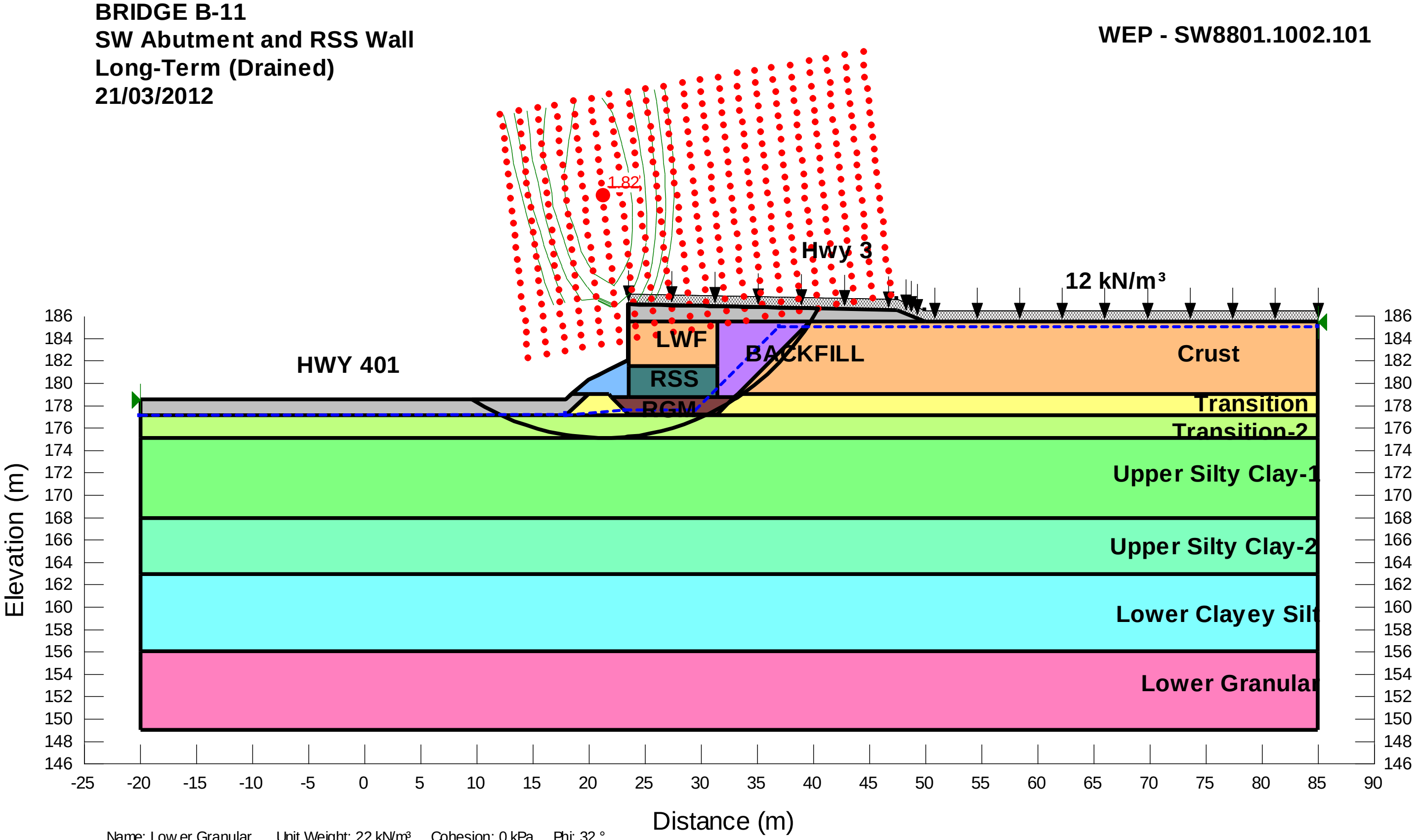
Name: Lower Granular Unit Weight: 22 $kN/m^3$ Cohesion: 0 kPa Phi: 32 °
Name: Crust (Drained) Unit Weight: 22 $kN/m^3$ Cohesion: 0 kPa Phi: 30 °
Name: Transition-1 (Drained) Unit Weight: 21.5 $kN/m^3$ Cohesion: 0 kPa Phi: 30 °
Name: Transition-2 (Drained) Unit Weight: 21.5 $kN/m^3$ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-1 (Drained) Unit Weight: 21 $kN/m^3$ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-2 (Drained) Unit Weight: 20.5 $kN/m^3$ Cohesion: 0 kPa Phi: 30 °
Name: Lower Clayey Silt (Drained) Unit Weight: 22 $kN/m^3$ Cohesion: 0 kPa Phi: 30 °
Name: Clay Backfill (Drained) Unit Weight: 21 $kN/m^3$ Cohesion: 0 kPa Phi: 30 °
Name: Roadway Granular Unit Weight: 22 $kN/m^3$ Cohesion: 0 kPa Phi: 35 °
Name: Concrete Unit Weight: 22 $kN/m^3$ Cohesion: 2000 kPa Phi: 35 °

## Figure E-8: Global Stability Result – Southwest Abutment Transverse – Short-Term Loading (Undrained properties)



Name: Crust Unit Weight: 22 kN/m³ Cohesion: 75 kPa
Name: Transition-1 Unit Weight: 21.5 kN/m³ Cohesion: 75 kPa
Name: Transition-2 Unit Weight: 21.5 kN/m³ C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m
Name: Upper Silty Clay-1 Unit Weight: 21 kN/m³ C-Datum: 55 kPa C-Rate of Change: -0.5556 kPa/m Limiting C: 51 kPa Elevation: 175 m
Name: Upper Silty Clay-2 Unit Weight: 20.5 kN/m³ C-Datum: 51.1 kPa C-Rate of Change: 1.3888 kPa/m Limiting C: 58 kPa Elevation: 168 m
Name: Lower Clayey Silt Unit Weight: 22 kN/m³ C-Datum: 58 kPa C-Rate of Change: 21 kPa/m Limiting C: 100 kPa Elevation: 163 m
Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Clay Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa
Name: RGM Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: LWF Unit Weight: 12 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 35 °

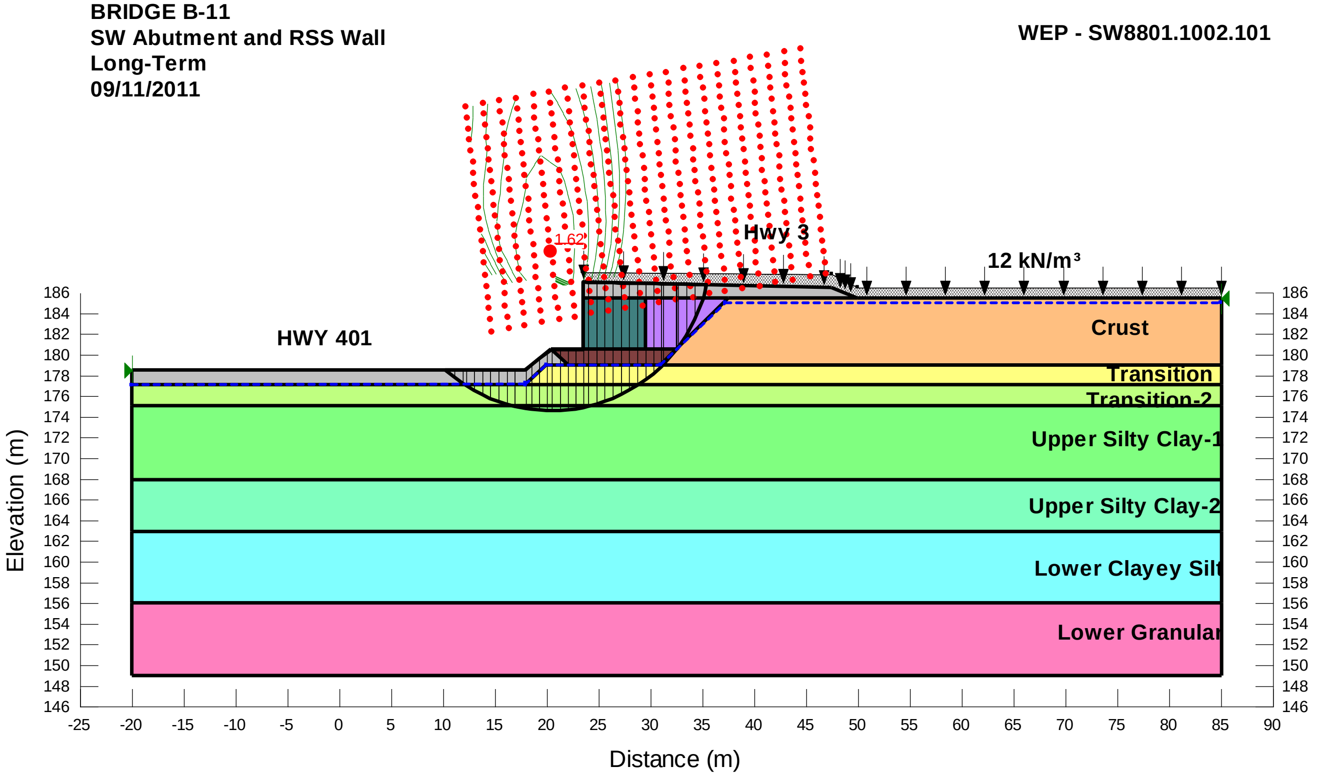
## Figure E-9: Global Stability Result – Southwest Abutment Transverse – End of Construction Loading (Undrained properties)



Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Crust (Drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Transition-1 (Drained) Unit Weight: 21.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Transition-2 (Drained) Unit Weight: 21.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-1 (Drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-2 (Drained) Unit Weight: 20.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Lower Clayey Silt (Drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clay Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa
Name: Clay Backfill (Drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: RGM Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: LWF Unit Weight: 12 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 35 °

Note: Assumes that pile cap excavation should be backfilled prior to backfilling abutment.

# Figure E-10: Global Stability Result – Southwest Abutment Transverse – Long-Term (Drained properties)



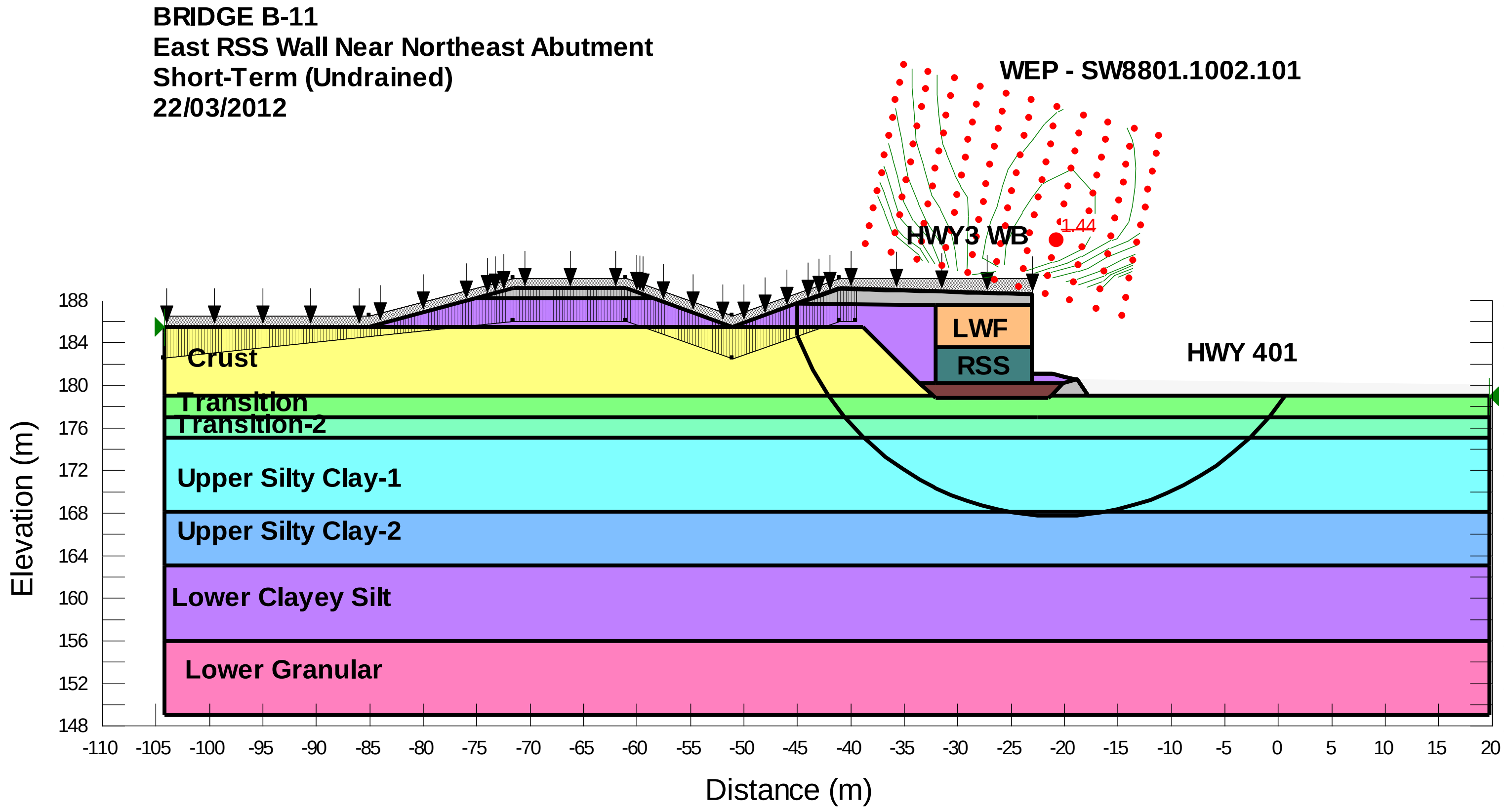
BRIDGE B-11
SW Abutment and RSS Wall
Long-Term
09/11/2011

WEP - SW8801.1002.101

Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Crust (Drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Transition-1 (Drained) Unit Weight: 21.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Transition-2 (Drained) Unit Weight: 21.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-1 (Drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-2 (Drained) Unit Weight: 20.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Lower Clayey Silt (Drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clay Backfill (Drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: RGM Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa Phi: 35 °

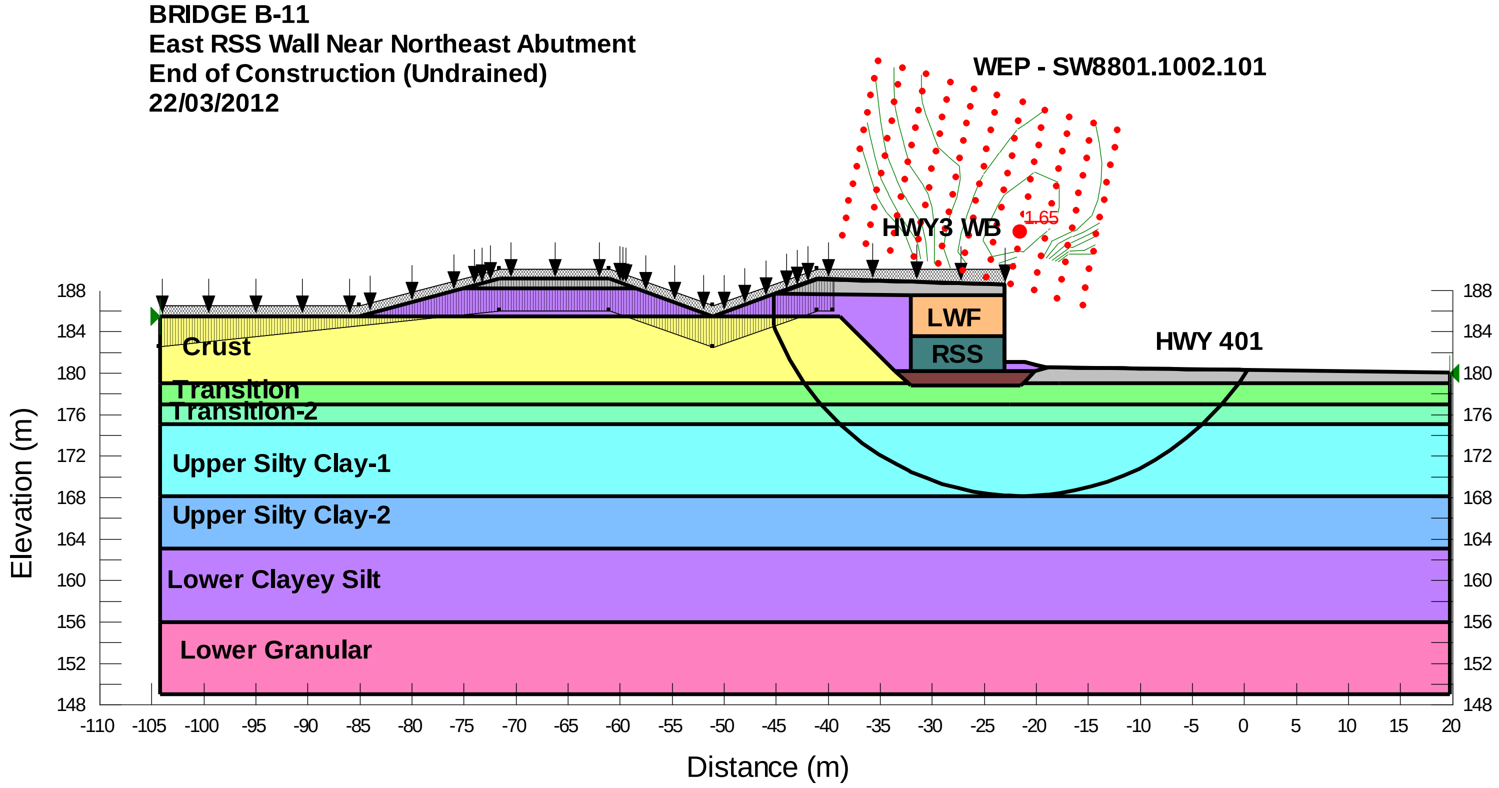
Note: Assumes that pile cap excavation should be backfilled prior to backfilling abutment.

Figure E-11: Global Stability Result – Northeast Abutment Transverse – Short-Term Loading (Undrained properties)



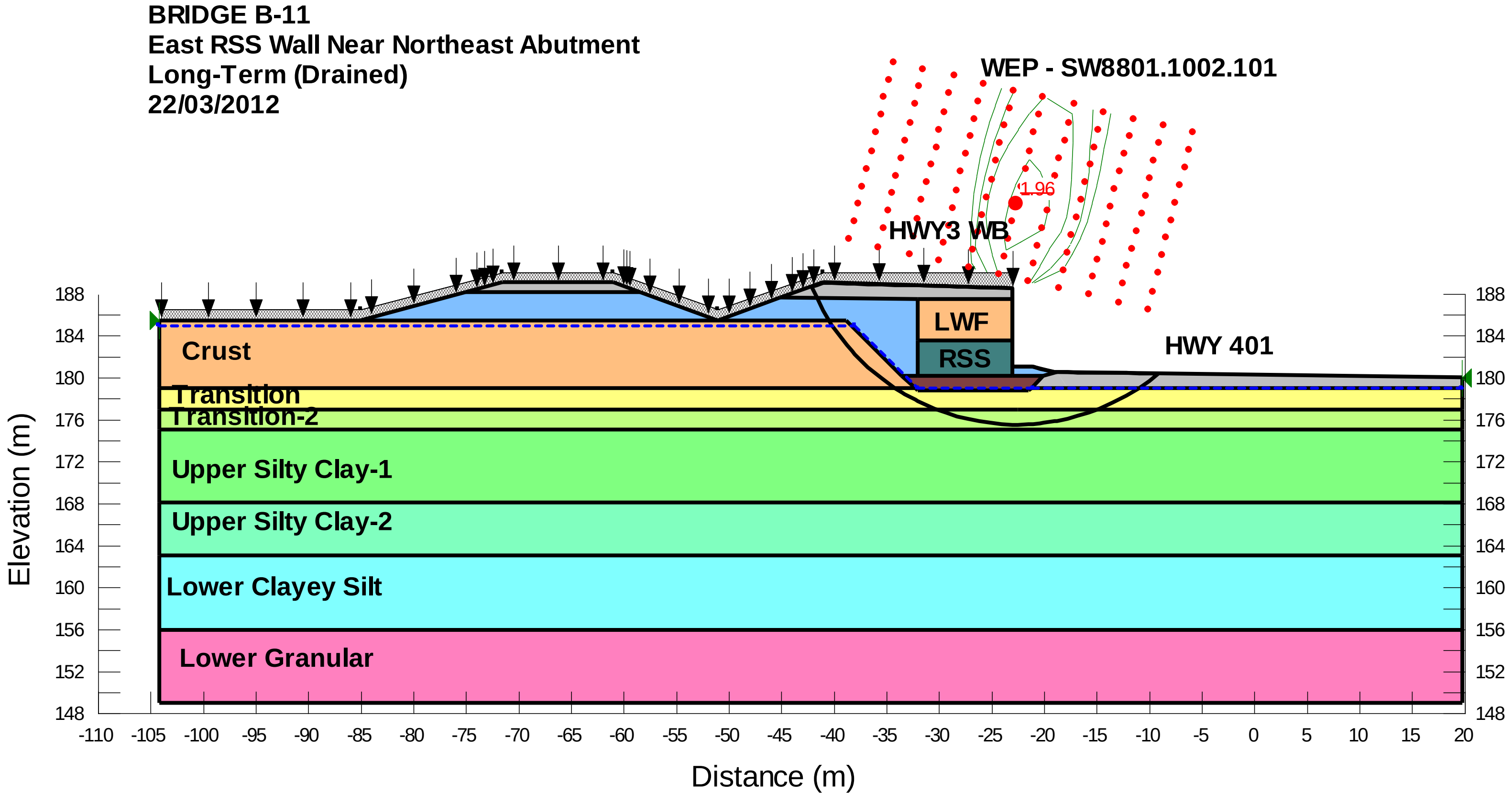
Name: Crust Unit Weight: 22 kN/m$^3$ Cohesion: 75 kPa
Name: Transition-1 Unit Weight: 21.5 kN/m$^3$ Cohesion: 75 kPa
Name: Transition-2 Unit Weight: 21.5 kN/m$^3$ C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m
Name: Upper Silty Clay-1 Unit Weight: 21 kN/m$^3$ C-Datum: 55 kPa C-Rate of Change: -0.5556 kPa/m Limiting C: 51 kPa Elevation: 175 m
Name: Upper Silty Clay-2 Unit Weight: 20.5 kN/m$^3$ C-Datum: 51.1 kPa C-Rate of Change: 1.3888 kPa/m Limiting C: 58 kPa Elevation: 168 m
Name: Lower Clayey Silt Unit Weight: 22 kN/m$^3$ C-Datum: 58 kPa C-Rate of Change: 21 kPa/m Limiting C: 100 kPa Elevation: 163 m
Name: Lower Granular Unit Weight: 22 kN/m$^3$ Cohesion: 0 kPa Phi: 32 °
Name: Clay Backfill Unit Weight: 21 kN/m$^3$ Cohesion: 50 kPa
Name: RGM Backfill Unit Weight: 21 kN/m$^3$ Cohesion: 0 kPa Phi: 35 °
Name: Roadway Granular Unit Weight: 22 kN/m$^3$ Cohesion: 0 kPa Phi: 35 °
Name: LWF Unit Weight: 12 kN/m$^3$ Cohesion: 0 kPa Phi: 35 °
Name: RSS Backfill Unit Weight: 21 kN/m$^3$ Cohesion: 75 kPa Phi: 35 °

## Figure E-12: Global Stability Result – Northeast Abutment Transverse – End of Construction Loading (Undrained properties)



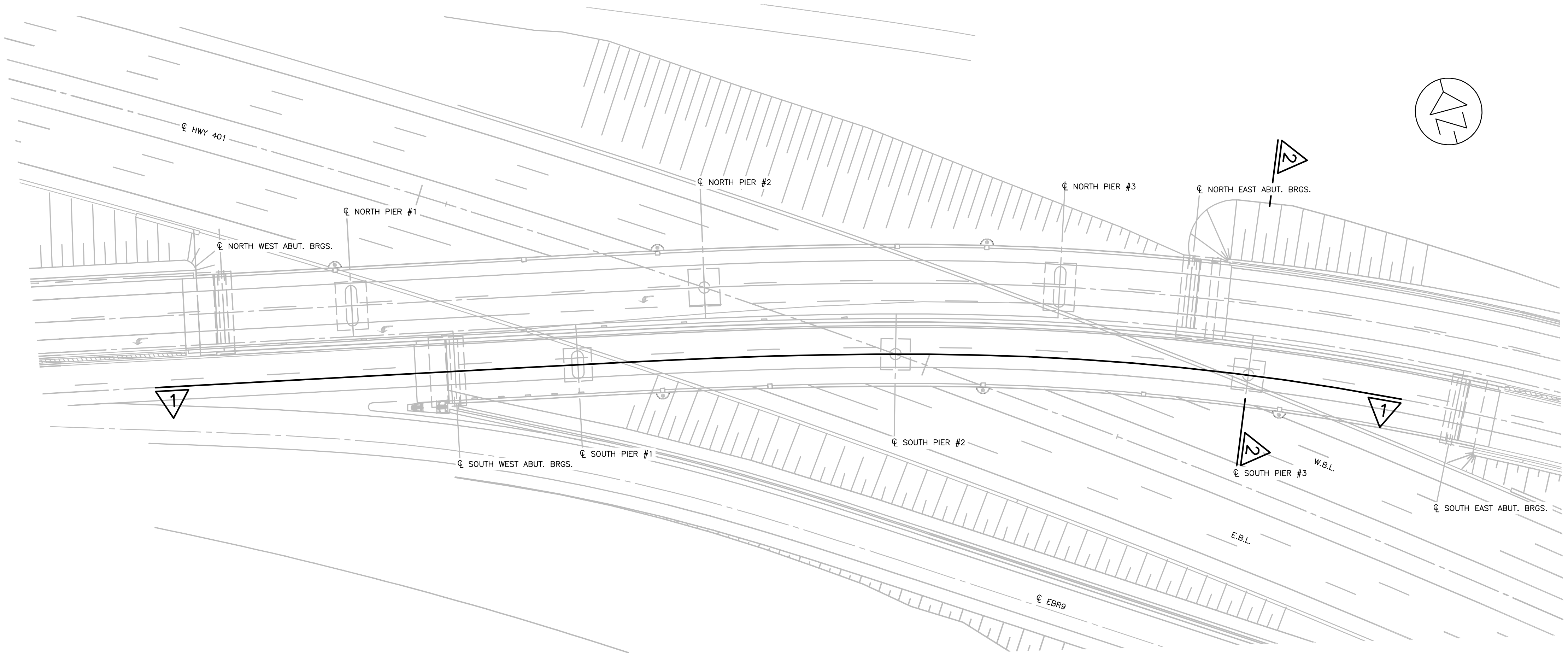
Name: Crust Unit Weight: 22 kN/m³ Cohesion: 75 kPa
Name: Transition-1 Unit Weight: 21.5 kN/m³ Cohesion: 75 kPa
Name: Transition-2 Unit Weight: 21.5 kN/m³ C-Datum: 75 kPa C-Rate of Change: -10 kPa/m Limiting C: 55 kPa Elevation: 177 m
Name: Upper Silty Clay-1 Unit Weight: 21 kN/m³ C-Datum: 55 kPa C-Rate of Change: -0.5556 kPa/m Limiting C: 51 kPa Elevation: 175 m
Name: Upper Silty Clay-2 Unit Weight: 20.5 kN/m³ C-Datum: 51.1 kPa C-Rate of Change: 1.3888 kPa/m Limiting C: 58 kPa Elevation: 168 m
Name: Lower Clayey Silt Unit Weight: 22 kN/m³ C-Datum: 58 kPa C-Rate of Change: 21 kPa/m Limiting C: 100 kPa Elevation: 163 m
Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Clay Backfill Unit Weight: 21 kN/m³ Cohesion: 50 kPa
Name: RGM Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: LWF Unit Weight: 12 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS Backfill Unit Weight: 21 kN/m³ Cohesion: 75 kPa Phi: 35 °

## Figure E-13: Global Stability Result - Northeast Abutment Transverse - Long-Term (Drained properties)



Name: Lower Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Crust (Drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Transition-1 (Drained) Unit Weight: 21.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Transition-2 (Drained) Unit Weight: 21.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-1 (Drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Upper Silty Clay-2 (Drained) Unit Weight: 20.5 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Lower Clayey Silt (Drained) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Clay Backfill (Drained) Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: RGM Backfill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: Roadway Granular Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: LWF Unit Weight: 12 kN/m³ Cohesion: 0 kPa Phi: 35 °
Name: RSS Backfill Unit Weight: 21 kN/m³ Cohesion: 75 kPa Phi: 35 °

# Appendix F: Stress-Deformation Analyses



PLAN
SCALE 1:500

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR
CONSTRUCTION

DOC: STRESS DEFORMATION FOR ANALYZED SECTIONS_FIG F.1

Parkway
Infrastructure
Engineers

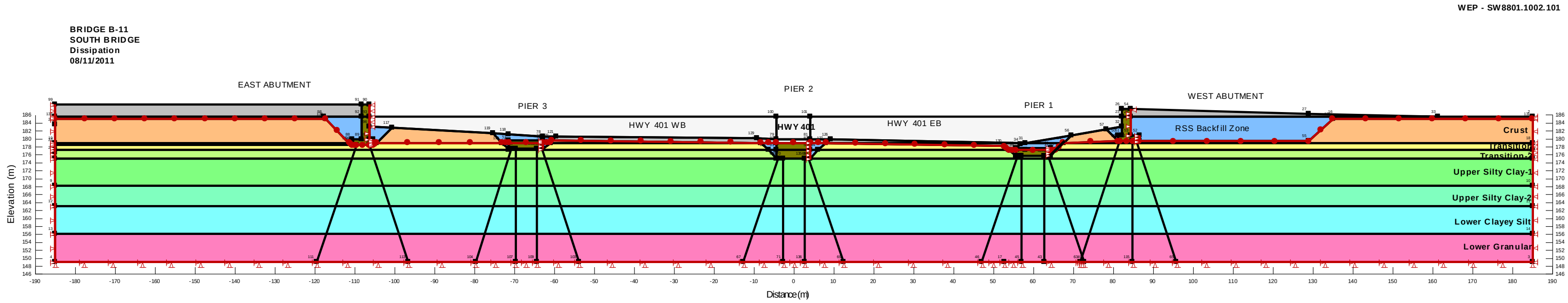
amec

Hatch Mott
MacDonald

BRIDGE B–11
STRESS DEFORMATION
ANALYZED SECTIONS

DWG. BY: SJL
CHK. BY: TL
FIGURE NO.: F.1
DATE: Mar–12
SHEET: 1 OF 1

# Figure F-2:  SDA Model South Bridge Longitudinal Section



Name: Insitu  Effective Young's Modulus (E'): 10000000 kPa  Poisson's Ratio: 0.334  Unit Weight: 21 kN/m³

Name: Lower Granular  Effective Young's Modulus (E'): 40000 kPa  Poisson's Ratio: 0.334  Cohesion': 0 kPa  Phi': 32 °  Unit Weight: 22 kN/m³

Name: Crust (Drained)  Effective Young's Modulus (E'): 31500 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 22 kN/m³

Name: Transition-1 (Drained)  Effective Young's Modulus (E'): 20250 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 21.5 kN/m³

Name: Transition-2 (Drained)  Effective Young's Modulus (E'): 17550 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 21.5 kN/m³

Name: Upper Silty Clay-1 (Drained)  O.C. Ratio: 1.8  Poisson's Ratio: 0.35  Lambda: 0.063  Kappa: 0.009  Initial Void Ratio: 0.48  Unit Weight: 21 kN/m³  Phi': 26 °

Name: Upper Silty Clay-2 (Drained)  O.C. Ratio: 1.4  Poisson's Ratio: 0.35  Lambda: 0.084  Kappa: 0.0126  Initial Void Ratio: 0.63  Unit Weight: 20.5 kN/m³  Phi': 26 °

Name: Lower Clayey Silt (Drained)  Effective Young's Modulus (E'): 25500 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 22 kN/m³

Name: Clay Backfill (drained)  Effective Young's Modulus (E'): 20000 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 21 kN/m³

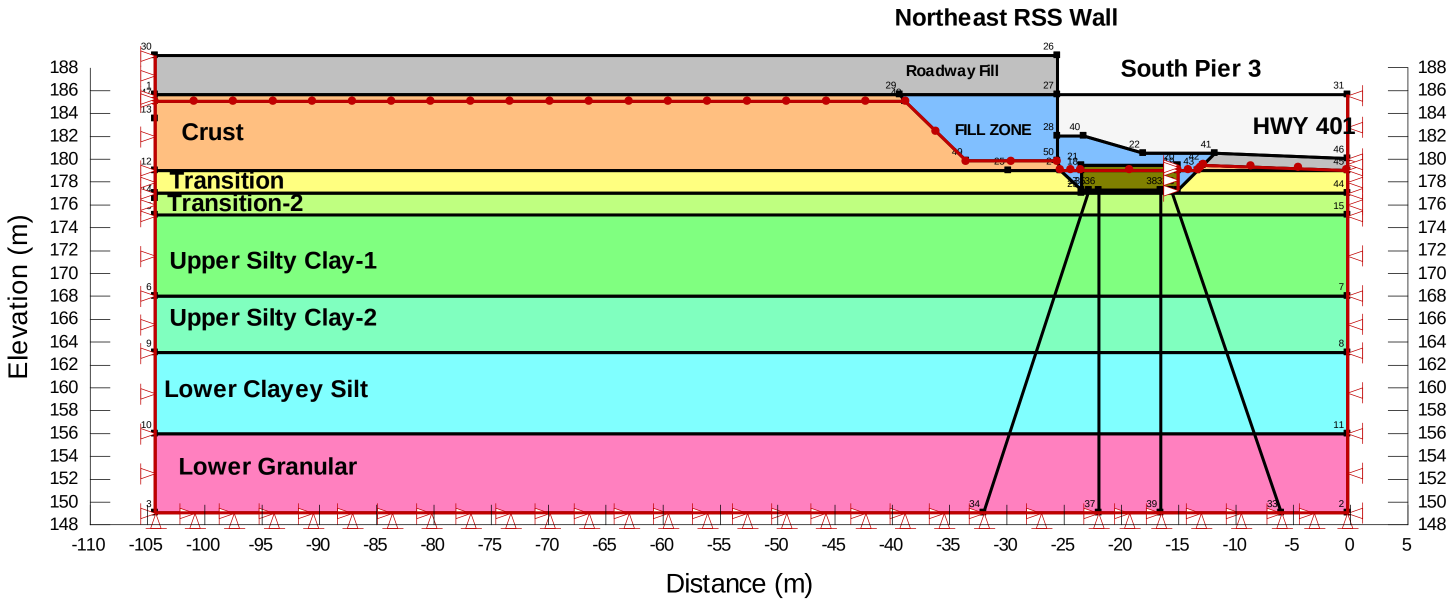
Name: Roadway Granular  Effective Young's Modulus (E'): 50000 kPa  Poisson's Ratio: 0.35  Unit Weight: 22 kN/m³

Name: Concrete  Young's Modulus (E): 200000 kPa  Unit Weight: 0.1 kN/m³  Poisson's Ratio: 0.35

## Figure F-3: SDA Model Northeast Abutment

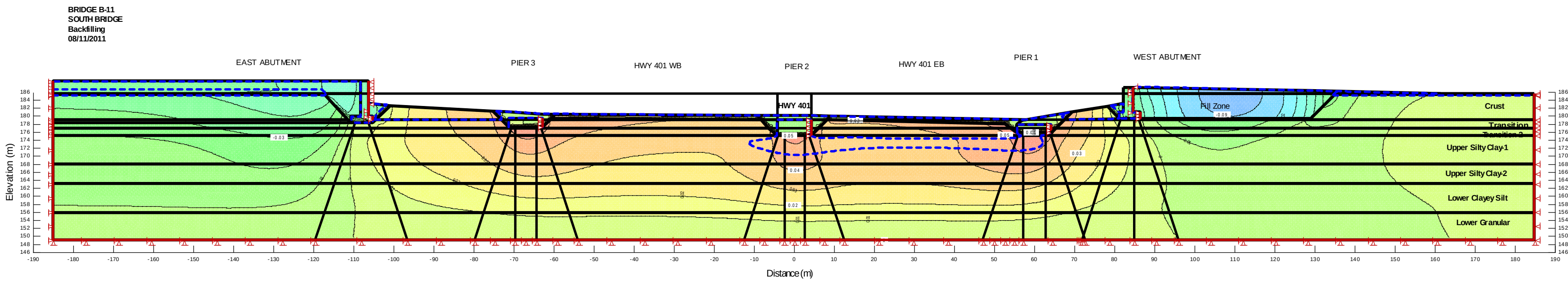
**BRIDGE B-11**
**East Abutment 13+250 Through South Pier No. 3**
**Dissipation**
**08/11/2011**

**WEP - SW8801.1002.101**



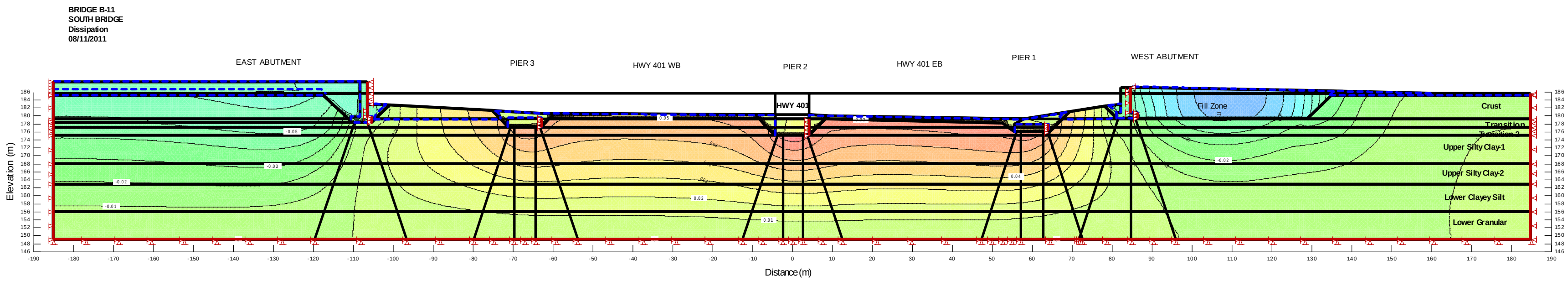
Name: Lower Granular  Effective Young's Modulus (E'): 40000 kPa  Poisson's Ratio: 0.334  Cohesion': 0 kPa  Phi': 32 °  Unit Weight: 22 kN/m³
Name: Crust (Drained)  Effective Young's Modulus (E'): 31500 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 22 kN/m³
Name: Transition-1 (Drained)  Effective Young's Modulus (E'): 20250 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 21.5 kN/m³
Name: Transition-2 (Drained)  Effective Young's Modulus (E'): 17550 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 21.5 kN/m³
Name: Upper Silty Clay-1 (Drained)  O.C. Ratio: 1.8  Poisson's Ratio: 0.35  Lambda: 0.063  Kappa: 0.009  Initial Void Ratio: 0.48  Unit Weight: 21 kN/m³  Phi': 26 °
Name: Upper Silty Clay-2 (Drained)  O.C. Ratio: 1.4  Poisson's Ratio: 0.35  Lambda: 0.084  Kappa: 0.0126  Initial Void Ratio: 0.63  Unit Weight: 20.5 kN/m³  Phi': 26 °
Name: Lower Clayey Silt (Drained)  Effective Young's Modulus (E'): 25500 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 22 kN/m³
Name: Clay Backfill (Drained)  Effective Young's Modulus (E'): 20000 kPa  Poisson's Ratio: 0.35  Cohesion': 0 kPa  Phi': 30 °  Unit Weight: 21 kN/m³
Name: Roadway Granular  Effective Young's Modulus (E'): 50000 kPa  Poisson's Ratio: 0.35  Unit Weight: 22 kN/m³
Name: Concrete  Young's Modulus (E): 200000 kPa  Unit Weight: 0.1 kN/m³  Poisson's Ratio: 0.35

# Figure F-4: Longitudinal Section - Cumulative Heave/Settlement - End of Construction Condition



Name: Insitu Effective Young's Modulus (E'): 10000000 kPa Poisson's Ratio: 0.334 Unit Weight: 21 kN/m³
Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.334 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Crust (Drained) Effective Young's Modulus (E'): 31500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Transition-1 (Drained) Effective Young's Modulus (E'): 20250 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Transition-2 (Drained) Effective Young's Modulus (E'): 17550 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Upper Silty Clay-1 (Drained) O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.009 Initial Void Ratio: 0.48 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Upper Silty Clay-2 (Drained) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.084 Kappa: 0.0126 Initial Void Ratio: 0.63 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Clayey Silt (Drained) Effective Young's Modulus (E'): 25500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Backfill (drained) Effective Young's Modulus (E'): 200000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³
Name: Roadway Granular Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Unit Weight: 22 kN/m³
Name: Concrete Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m³ Poisson's Ratio: 0.35

## Figure F-5: Longitudinal Section – Cumulative Heave/Settlement – Long-term Condition

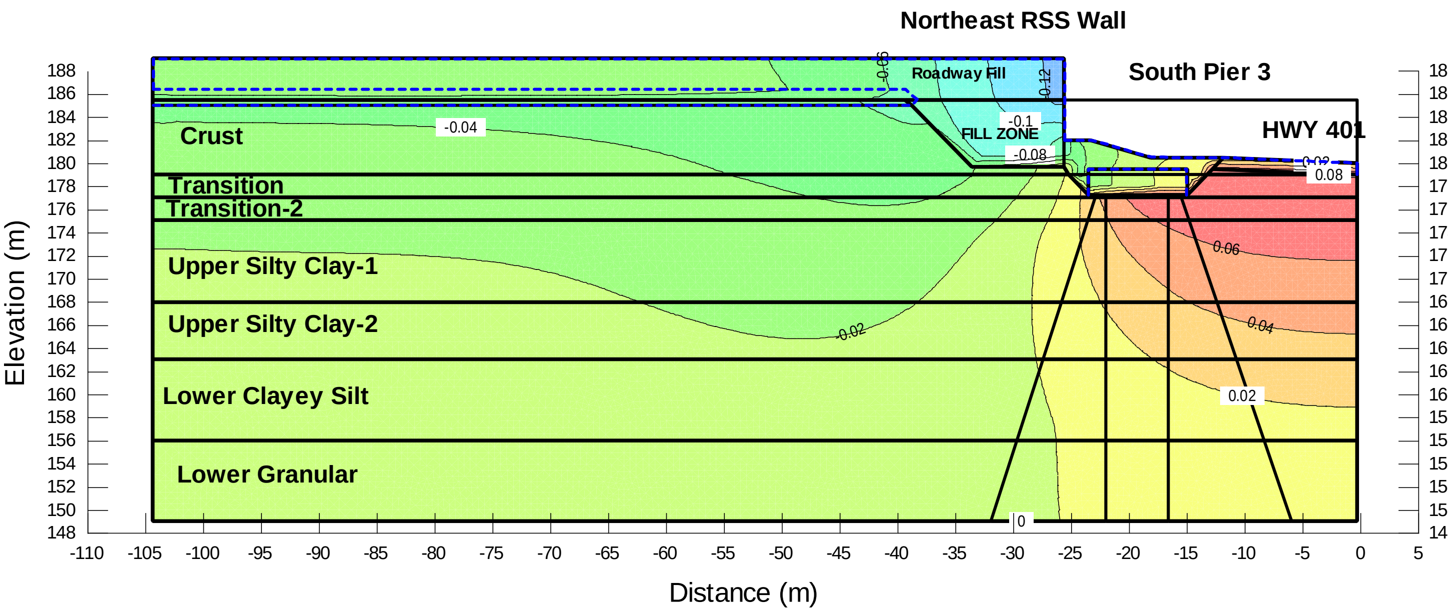


Name: Insitu Effective Young's Modulus (E'): 10000000 kPa Poisson's Ratio: 0.334 Unit Weight 21 kN/m³
Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.334 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Crust (Drained) Effective Young's Modulus (E'): 31500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Transition-1 (Drained) Effective Young's Modulus (E'): 20250 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Transition-2 (Drained) Effective Young's Modulus (E'): 17550 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Upper Silty Clay-1 (Drained) O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.009 Initial Void Ratio: 0.48 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Upper Silty Clay-2 (Drained) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.084 Kappa: 0.0126 Initial Void Ratio: 0.63 Unit Weight 20.5 kN/m³ Phi': 26 °
Name: Lower Clayey Silt (Drained) Effective Young's Modulus (E'): 25500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Backfill (drained) Effective Young's Modulus (E'): 200000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight 21 kN/m³
Name: Roadway Granular Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Unit Weight 22 kN/m³
Name: Concrete Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m³ Poisson's Ratio: 0.35

## Figure F-6: Cumulative Heave/Settlement – End of Construction Condition

**BRIDGE B-11**
**East Abutment 13+250 Through South Pier No. 3**
**Backfilling**
**08/11/2011**

**WEP - SW8801.1002.101**

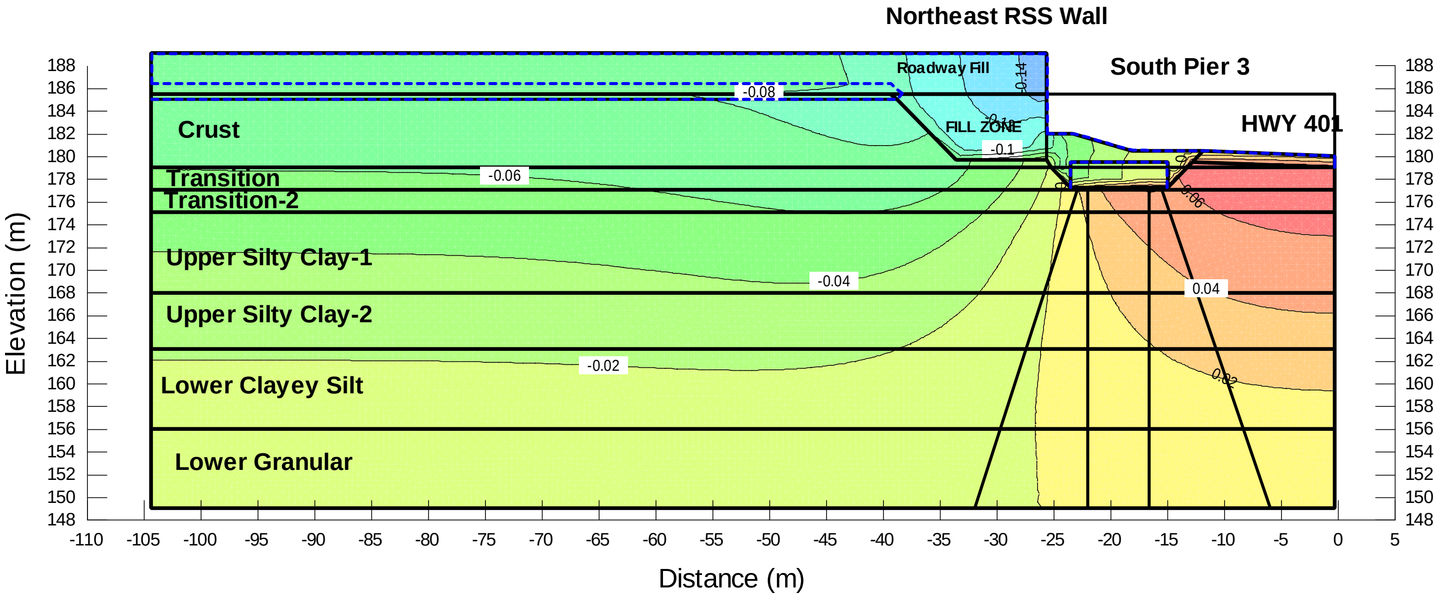


Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.334 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Crust (Drained) Effective Young's Modulus (E'): 31500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Transition-1 (Drained) Effective Young's Modulus (E'): 20250 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Transition-2 (Drained) Effective Young's Modulus (E'): 17550 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Upper Silty Clay-1 (Drained) O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.009 Initial Void Ratio: 0.48 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Upper Silty Clay-2 (Drained) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.084 Kappa: 0.0126 Initial Void Ratio: 0.63 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Clayey Silt (Drained) Effective Young's Modulus (E'): 25500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Backfill (Drained) Effective Young's Modulus (E'): 20000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³
Name: Roadway Granular Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Unit Weight: 22 kN/m³
Name: Concrete Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m³ Poisson's Ratio: 0.35

Figure F-7: Cumulative Heave/Settlement – Long-Term Condition

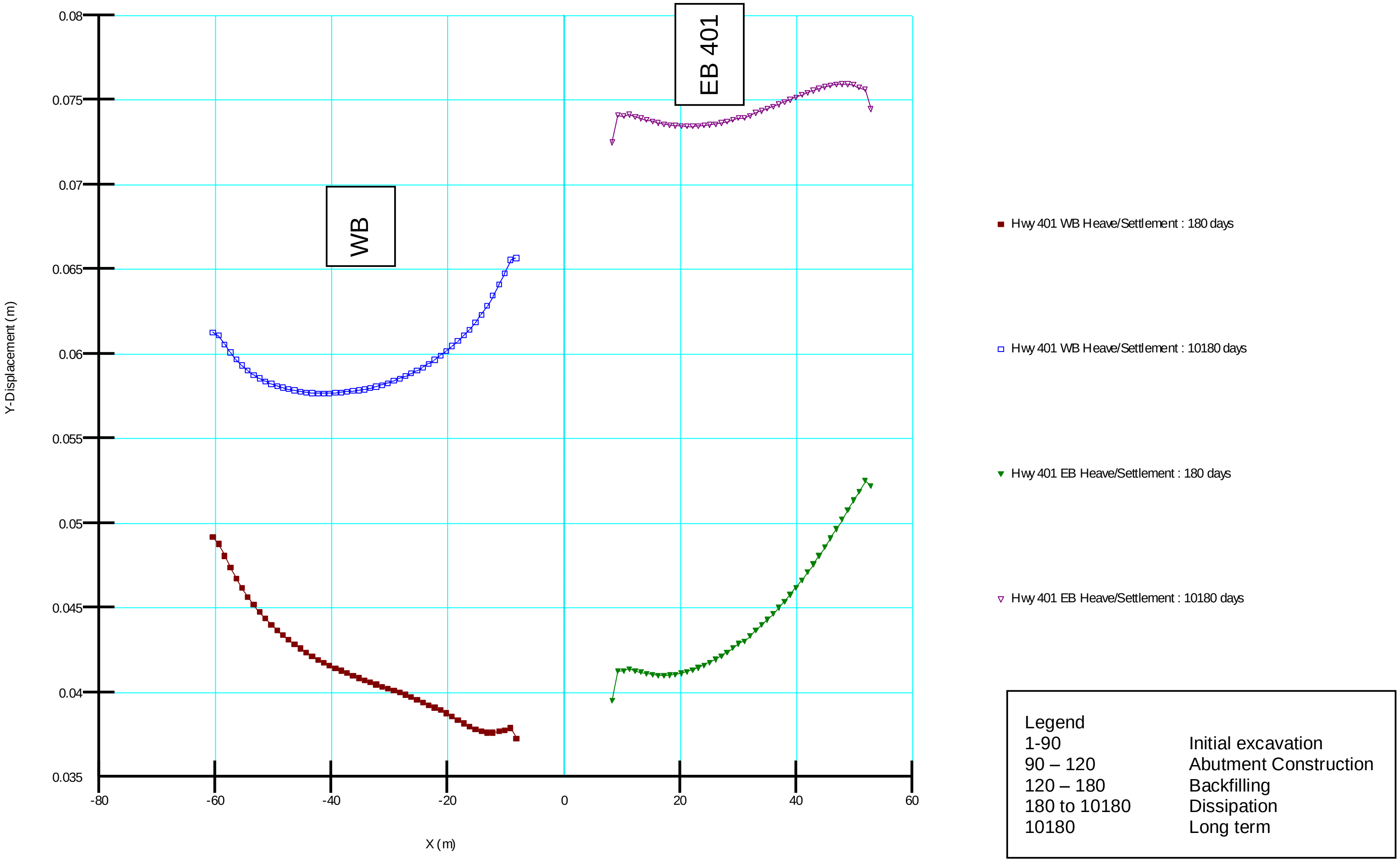
**BRIDGE B-11**
**East Abutment 13+250 Through South Pier No. 3**
**Dissipation**
**08/11/2011**

**WEP - SW8801.1002.101**



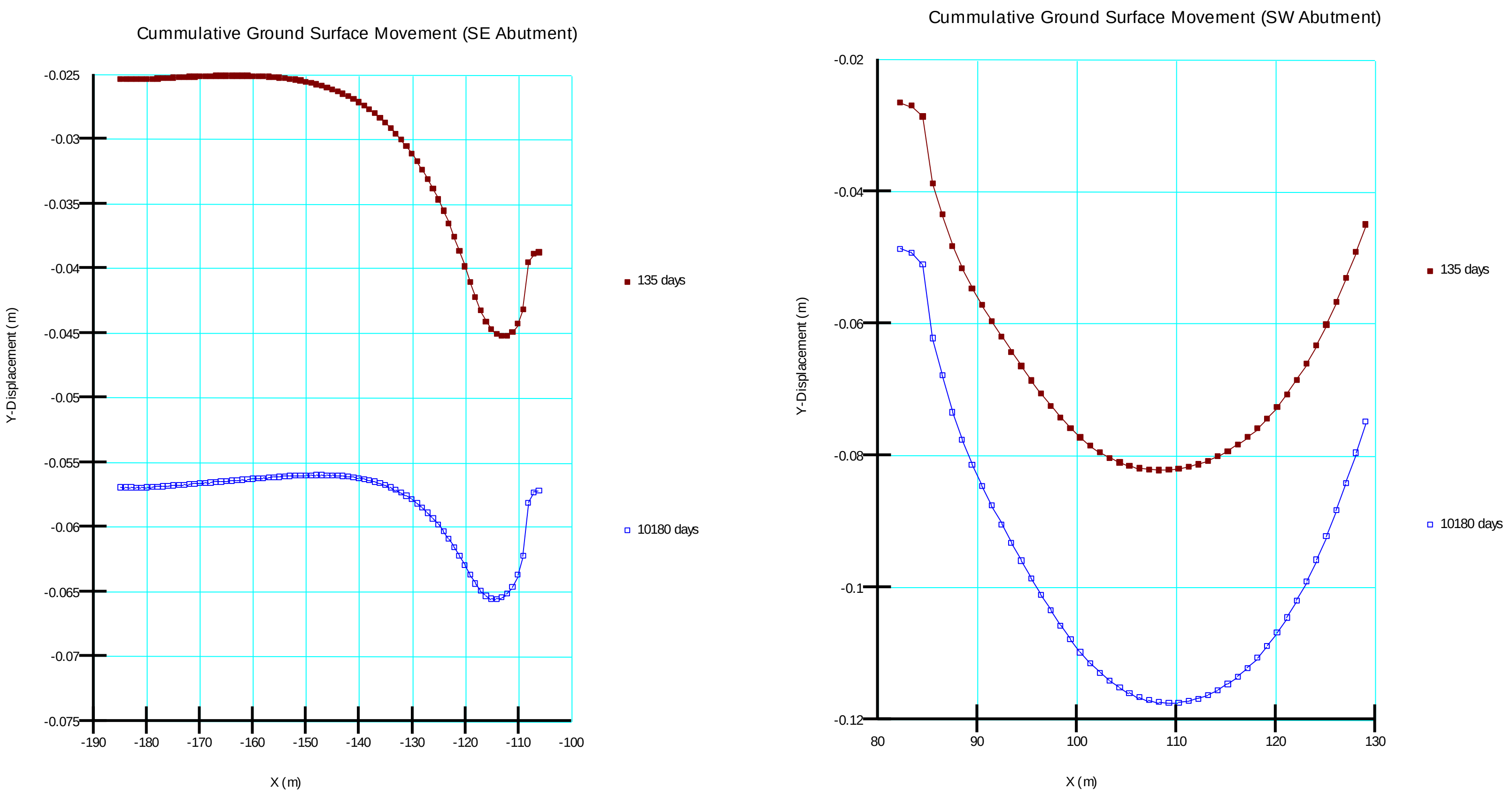
Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.334 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Crust (Drained) Effective Young's Modulus (E'): 31500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Transition-1 (Drained) Effective Young's Modulus (E'): 20250 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Transition-2 (Drained) Effective Young's Modulus (E'): 17550 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Upper Silty Clay-1 (Drained) O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.009 Initial Void Ratio: 0.48 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Upper Silty Clay-2 (Drained) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.084 Kappa: 0.0126 Initial Void Ratio: 0.63 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Clayey Silt (Drained) Effective Young's Modulus (E'): 25500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Backfill (Drained) Effective Young's Modulus (E'): 20000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³
Name: Roadway Granular Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Unit Weight: 22 kN/m³
Name: Concrete Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m³ Poisson's Ratio: 0.35

Figure F-8: Cumulative Ground Settlements Highway 401 Subbase Along Longitudinal Profile of B11 South Bridge



| | |
|---|---|
| 1-90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 | Dissipation |
| 10180 | Long term |

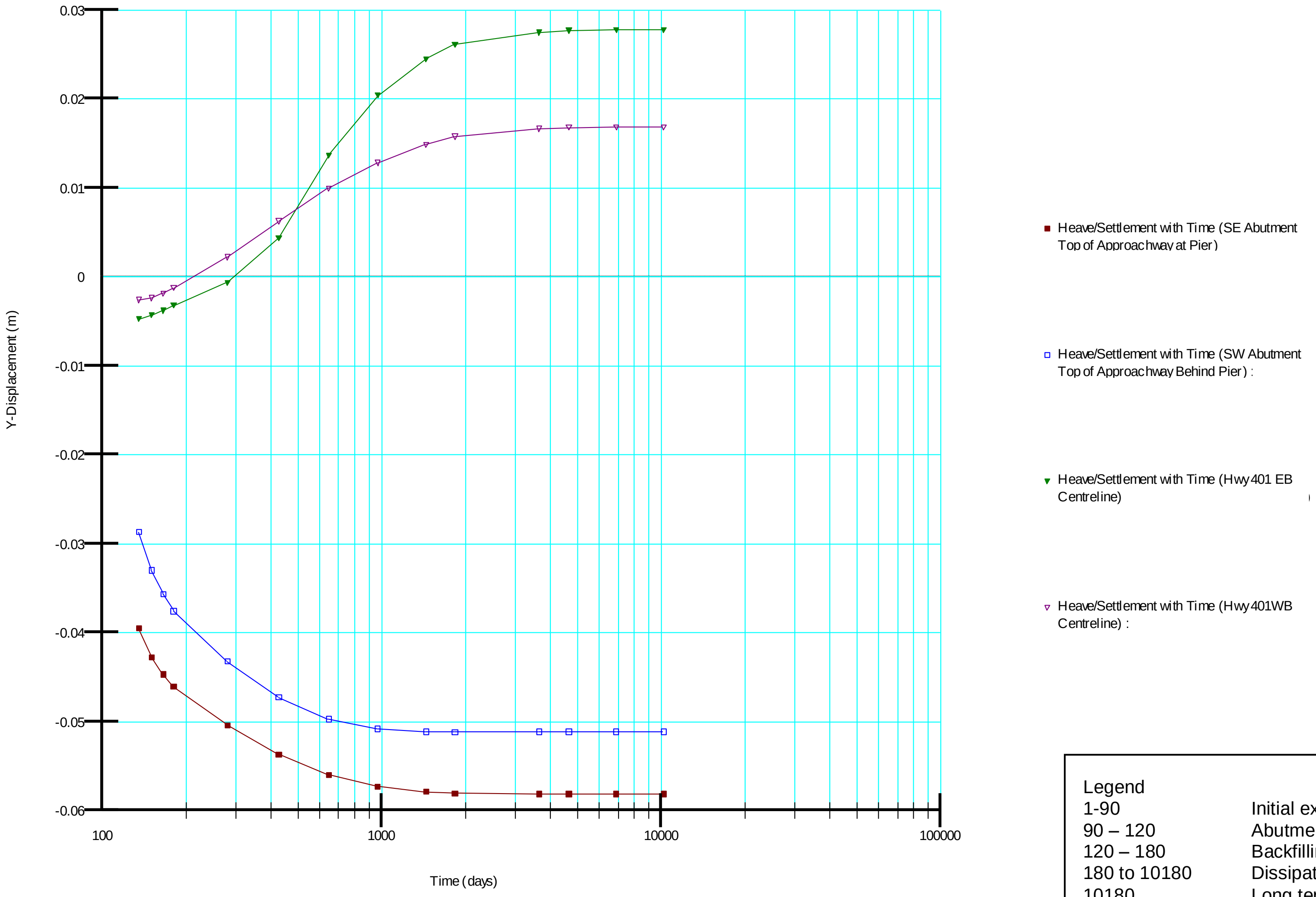
# Figure F-9: Abutment Section – Ground Settlements



| Legend | |
|---|---|
| 1-90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 | Dissipation |
| 10180 | Long term |

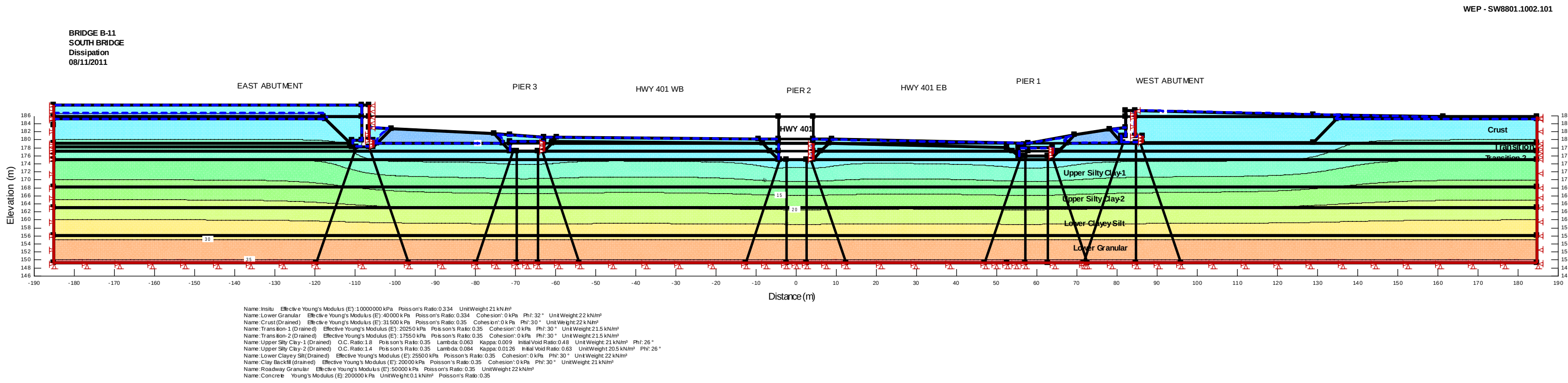
Note: For SE Abutment Distance -106 is front of abutment.
For SW Abutment Distance – 82 is front of abutment.

## Figure F-10: Heave/Settlement Rate Various Locations



| Legend | |
|---|---|
| 1-90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 | Dissipation |
| 10180 | Long term |

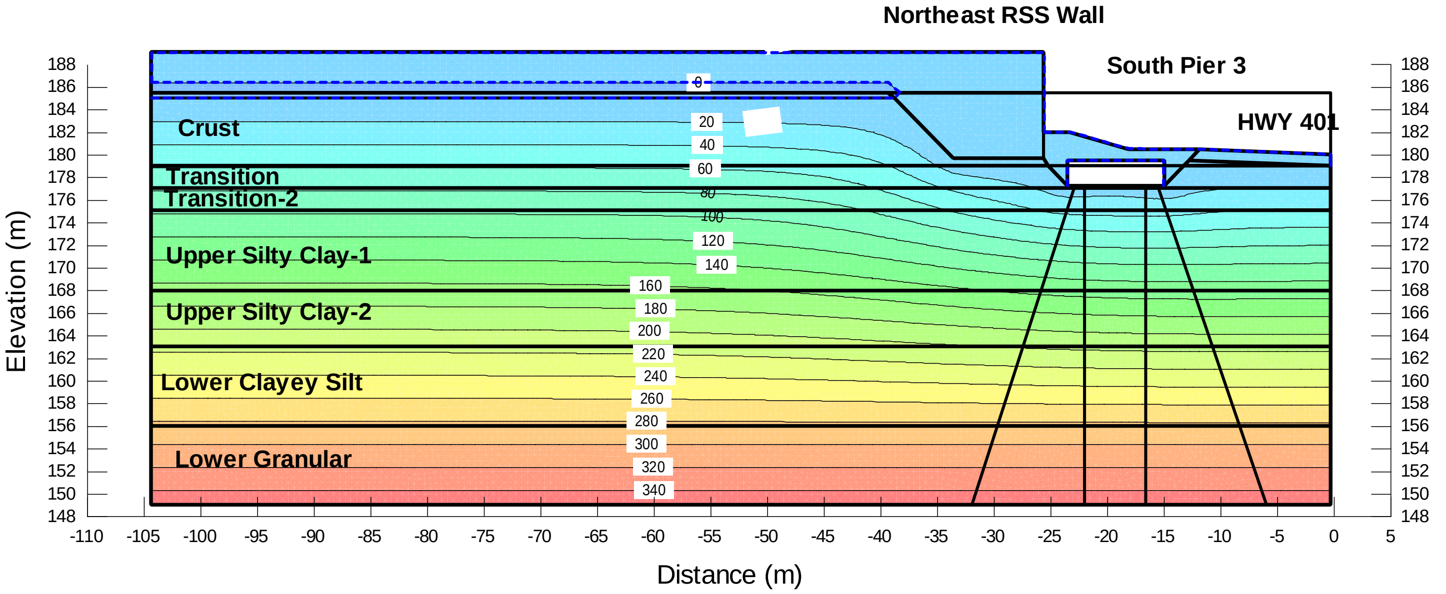
## Figure F-11: Longitudinal Section B-11 South Bridge Pore-Water Pressures Long-Term



## Figure F-12: Transverse Section B-11 Pier No 3 Pore-Water Pressures Long-Term

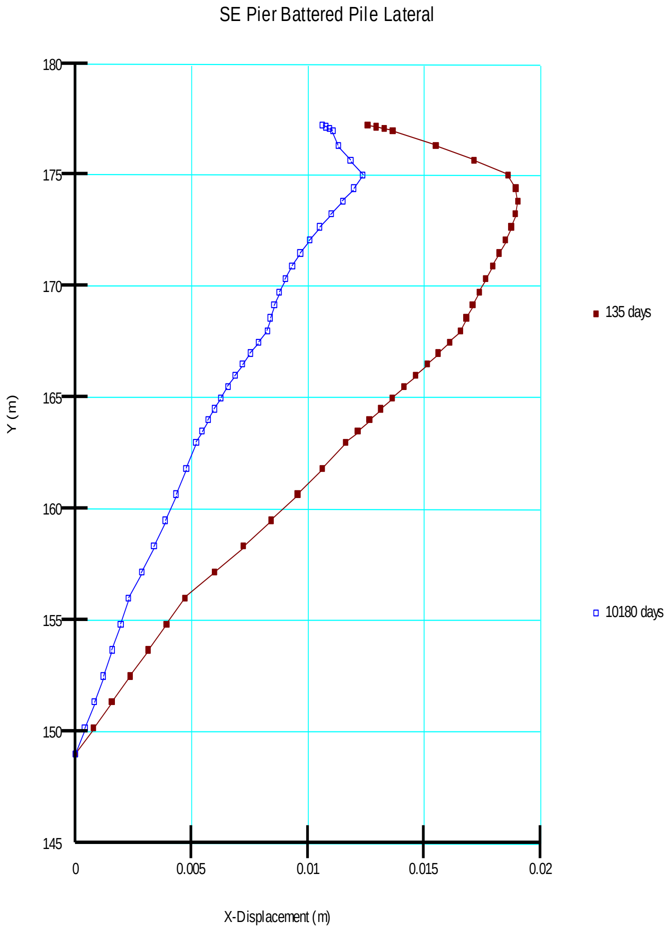
**BRIDGE B-11**
**East Abutment 13+250 Through South Pier No. 3**
**Dissipation**
**08/11/2011**

**WEP - SW8801.1002.101**



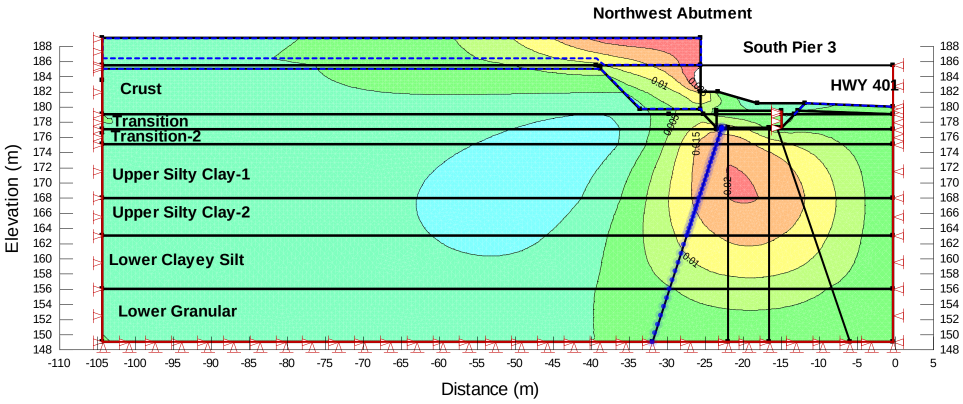
Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.334 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m³
Name: Crust (Drained) Effective Young's Modulus (E'): 31500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Transition-1 (Drained) Effective Young's Modulus (E'): 20250 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Transition-2 (Drained) Effective Young's Modulus (E'): 17550 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m³
Name: Upper Silty Clay-1 (Drained) O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.009 Initial Void Ratio: 0.48 Unit Weight: 21 kN/m³ Phi': 26 °
Name: Upper Silty Clay-2 (Drained) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.084 Kappa: 0.0126 Initial Void Ratio: 0.63 Unit Weight: 20.5 kN/m³ Phi': 26 °
Name: Lower Clayey Silt (Drained) Effective Young's Modulus (E'): 25500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m³
Name: Clay Backfill (Drained) Effective Young's Modulus (E'): 20000 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21 kN/m³
Name: Roadway Granular Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Unit Weight: 22 kN/m³
Name: Concrete Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m³ Poisson's Ratio: 0.35

# Figure F-13: Net Lateral Soil Movement at South Pier 3 – Transverse Section



**BRIDGE B-11**
**East Abutment 13+250 Through South Pier No. 3**
**Backfilling**
**02/11/2011**

**WEP - SW8801.1002.101**

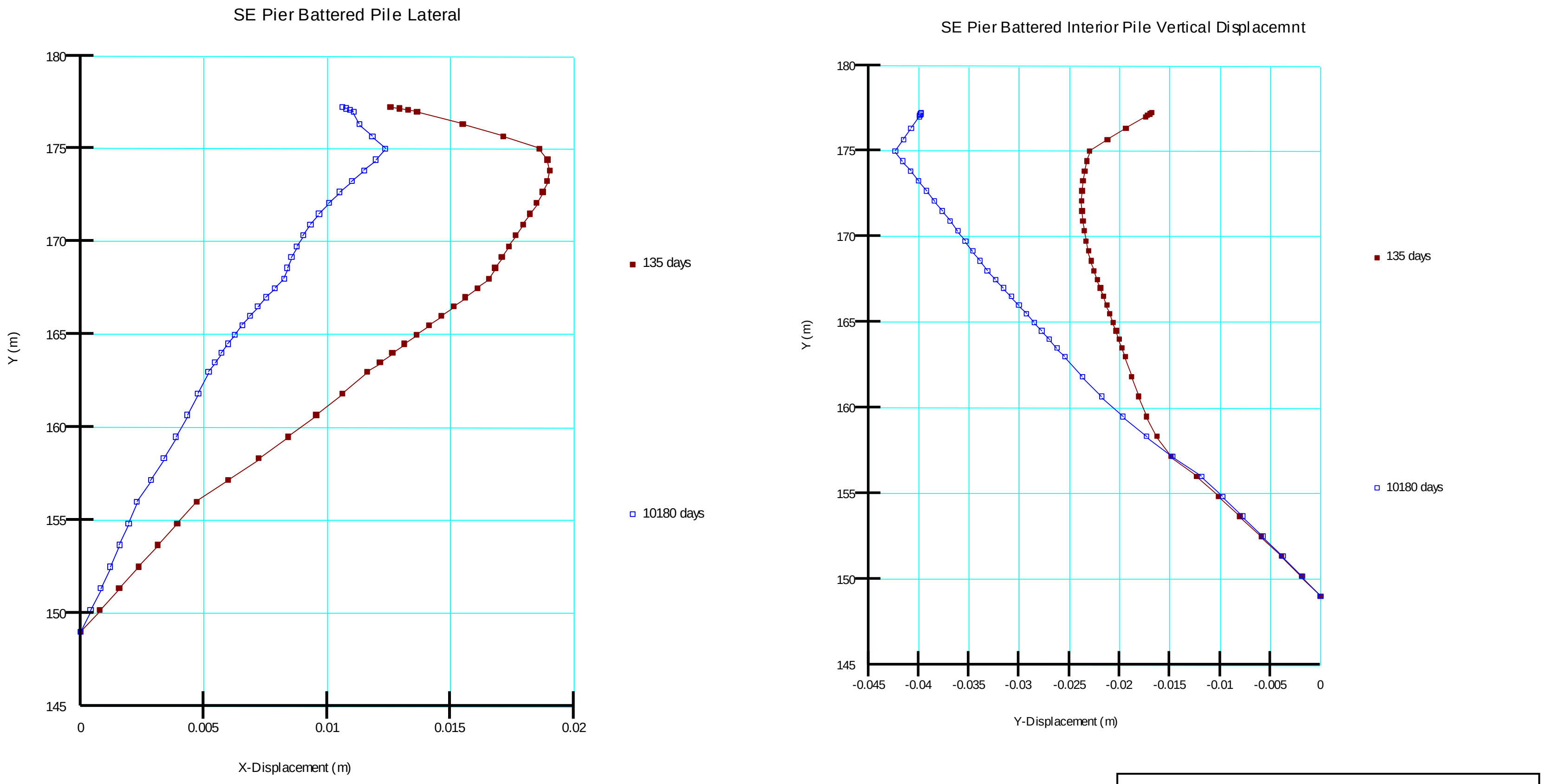


Name: Lower Granular Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.334 Cohesion': 0 kPa Phi': 32 ° Unit Weight: 22 kN/m$^3$
Name: Crust (Drained) Effective Young's Modulus (E'): 31500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m$^3$
Name: Transition-1 (Drained) Effective Young's Modulus (E'): 20250 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m$^3$
Name: Transition-2 (Drained) Effective Young's Modulus (E'): 17550 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 21.5 kN/m$^3$
Name: Upper Silty Clay-1 (Drained) O.C. Ratio: 1.8 Poisson's Ratio: 0.35 Lambda: 0.063 Kappa: 0.009 Initial Void Ratio: 0.48 Unit Weight: 21 kN/m$^3$ Phi': 26 °
Name: Upper Silty Clay-2 (Drained) O.C. Ratio: 1.4 Poisson's Ratio: 0.35 Lambda: 0.084 Kappa: 0.0126 Initial Void Ratio: 0.63 Unit Weight: 20.5 kN/m$^3$ Phi': 26 °
Name: Lower Clayey Silt (Drained) Effective Young's Modulus (E'): 25500 kPa Poisson's Ratio: 0.35 Cohesion': 0 kPa Phi': 30 ° Unit Weight: 22 kN/m$^3$
Name: Clay Backfill Young's Modulus (E): 20000 kPa Poisson's Ratio: 0.49 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m$^3$
Name: Roadway Granular Effective Young's Modulus (E'): 50000 kPa Poisson's Ratio: 0.35 Unit Weight: 22 kN/m$^3$
Name: Concrete Young's Modulus (E): 200000 kPa Unit Weight: 0.1 kN/m$^3$ Poisson's Ratio: 0.35

Legend

| | |
|---|---|
| 1-90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 | Dissipation |
| 10180 | Long term |

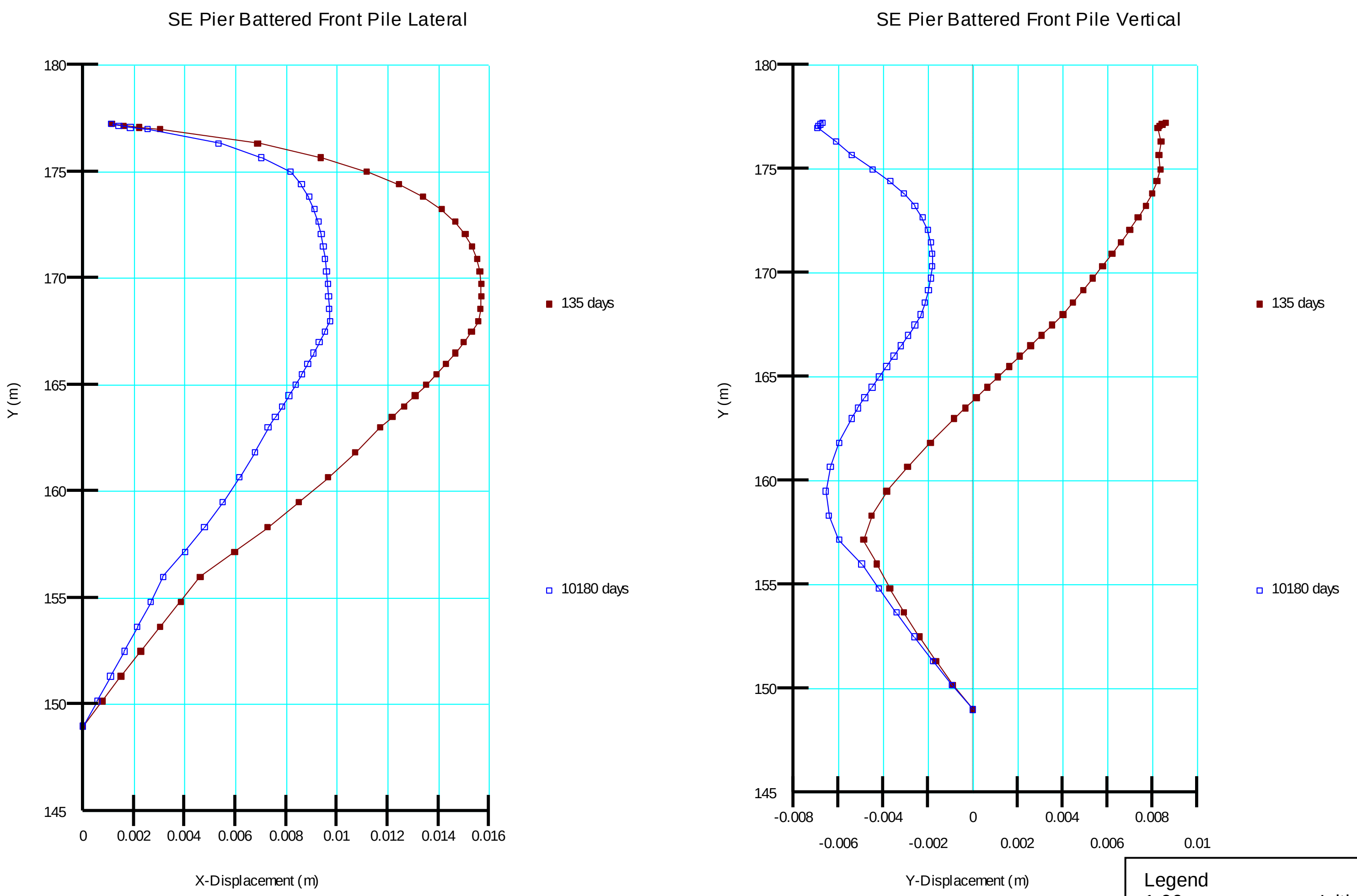
# Figure F-14: Net Soil Movement at South Pier 3 Interior Pile– Transverse Section



Legend

| | |
|---|---|
| 1-90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 | Dissipation |
| 10180 | Long term |

# Figure F-15: Net Soil Movement at South Pier 3 Exterior Pile– Transverse Section



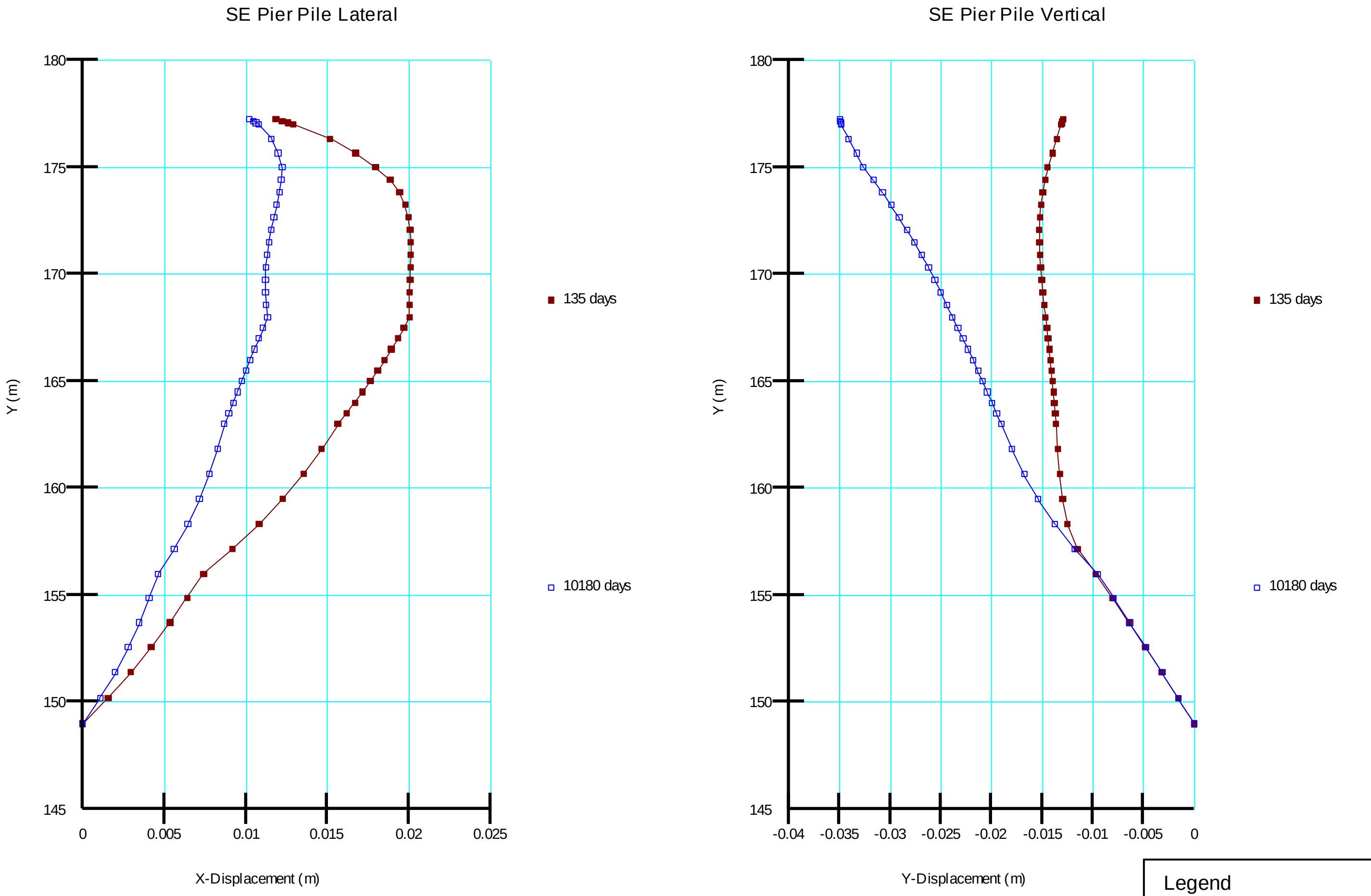
SE Pier Battered Front Pile Lateral

SE Pier Battered Front Pile Vertical

Legend

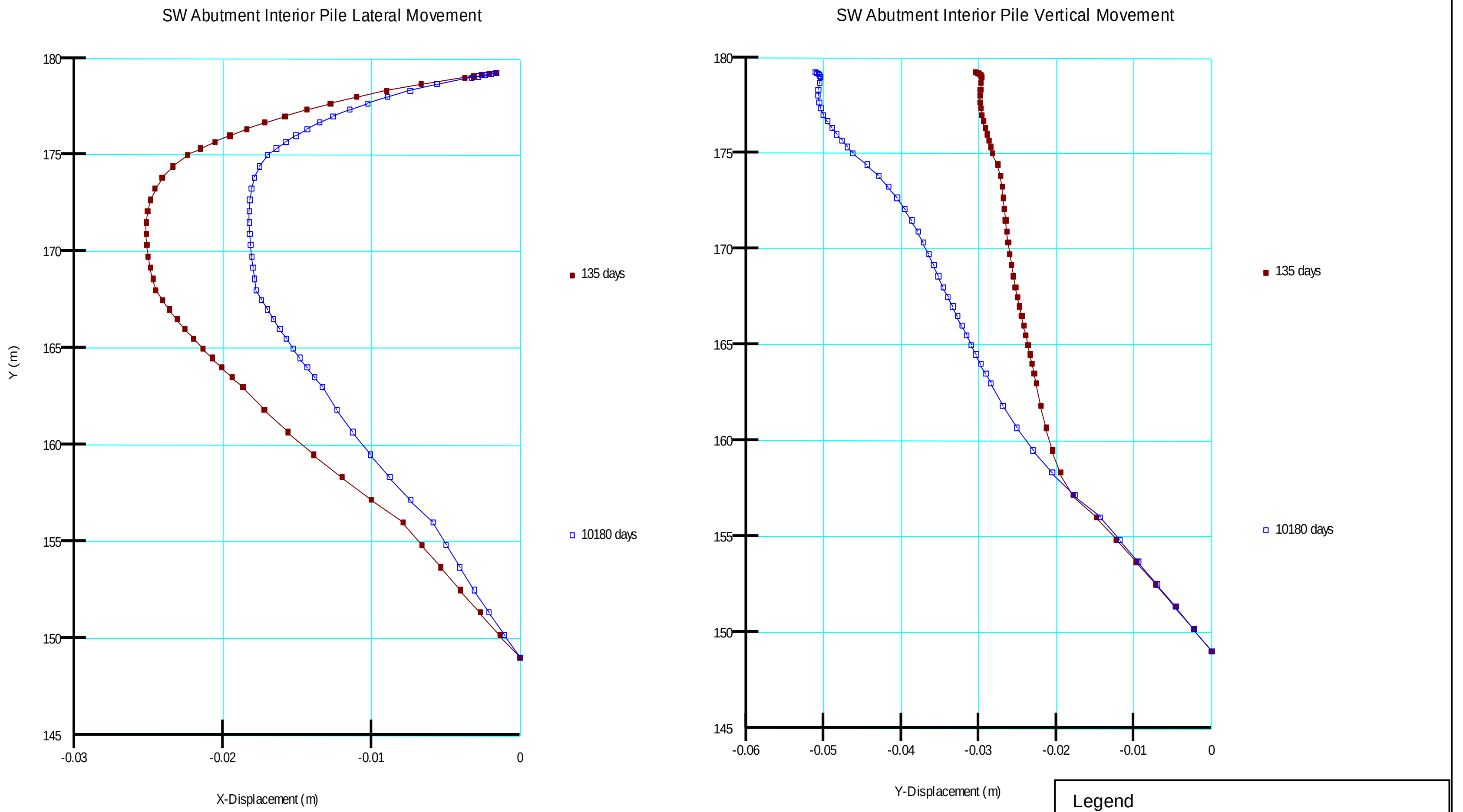
| | |
|---|---|
| 1-90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 | Dissipation |
| 10180 | Long term |

## Figure F-16: Net Soil Movement at South Pier 3 Vertical Pile– Transverse Section



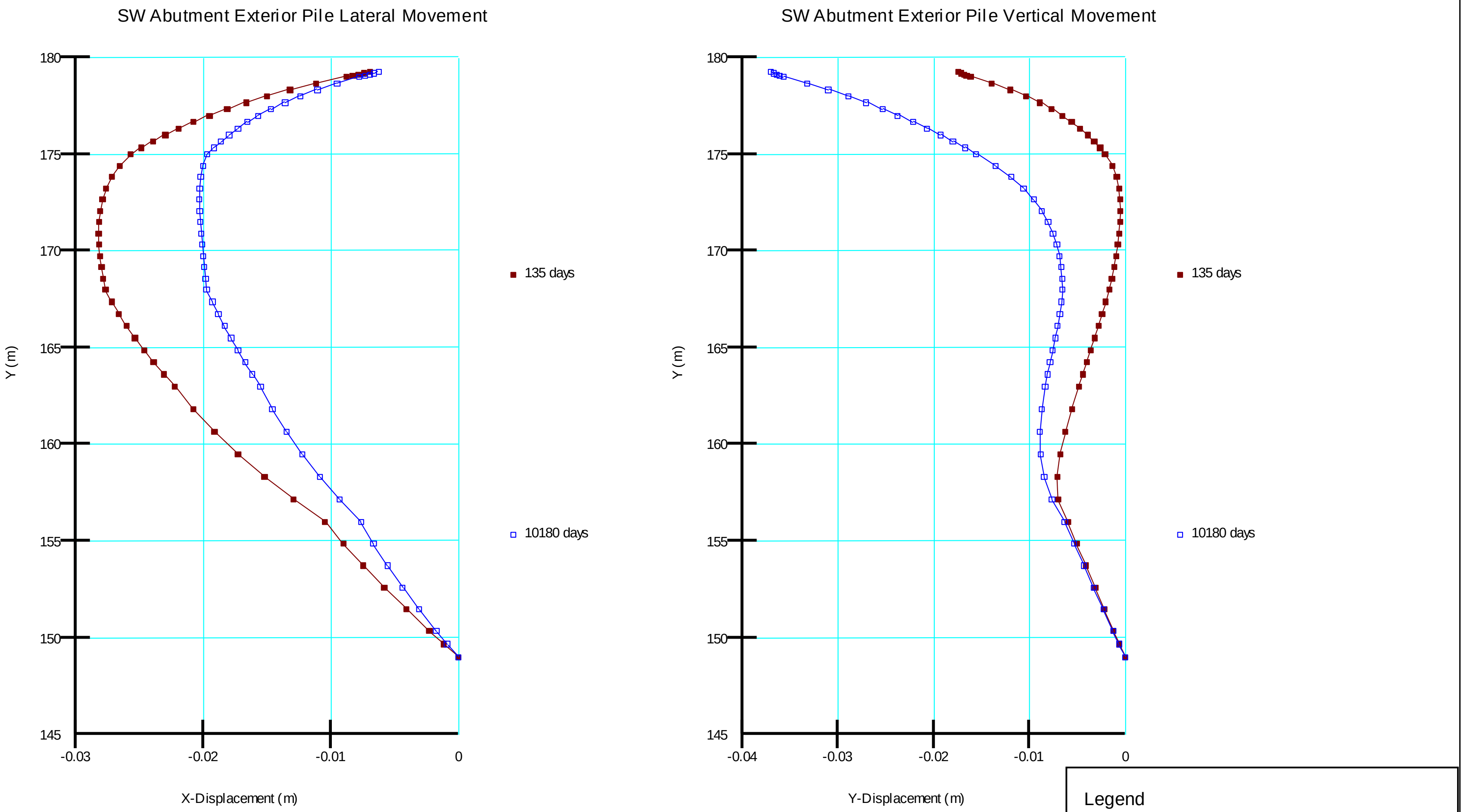
| Legend | |
|---|---|
| 1-90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 | Dissipation |
| 10180 | Long term |

## Figure F-17: SW Abutment Section – Net Soil Movement Interior Pile



| Legend | |
|---|---|
| 1-90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 | Dissipation |
| 10180 | Long term |

# Figure F-18: SW Abutment Section – Net Soil Movement Exterior Pile



| Legend | |
|---|---|
| 1-90 | Initial excavation |
| 90 – 120 | Abutment Construction |
| 120 – 180 | Backfilling |
| 180 to 10180 | Dissipation |
| 10180 | Long term |

# Appendix G: Selected Site Photographs

Centreline of Talbot Rd. looking East

East Side of Talbot Rd. looking South towards Surrey Rd. and Montgomery Dr.

NBL of Talbot Rd. at Montgomery Dr. looking South

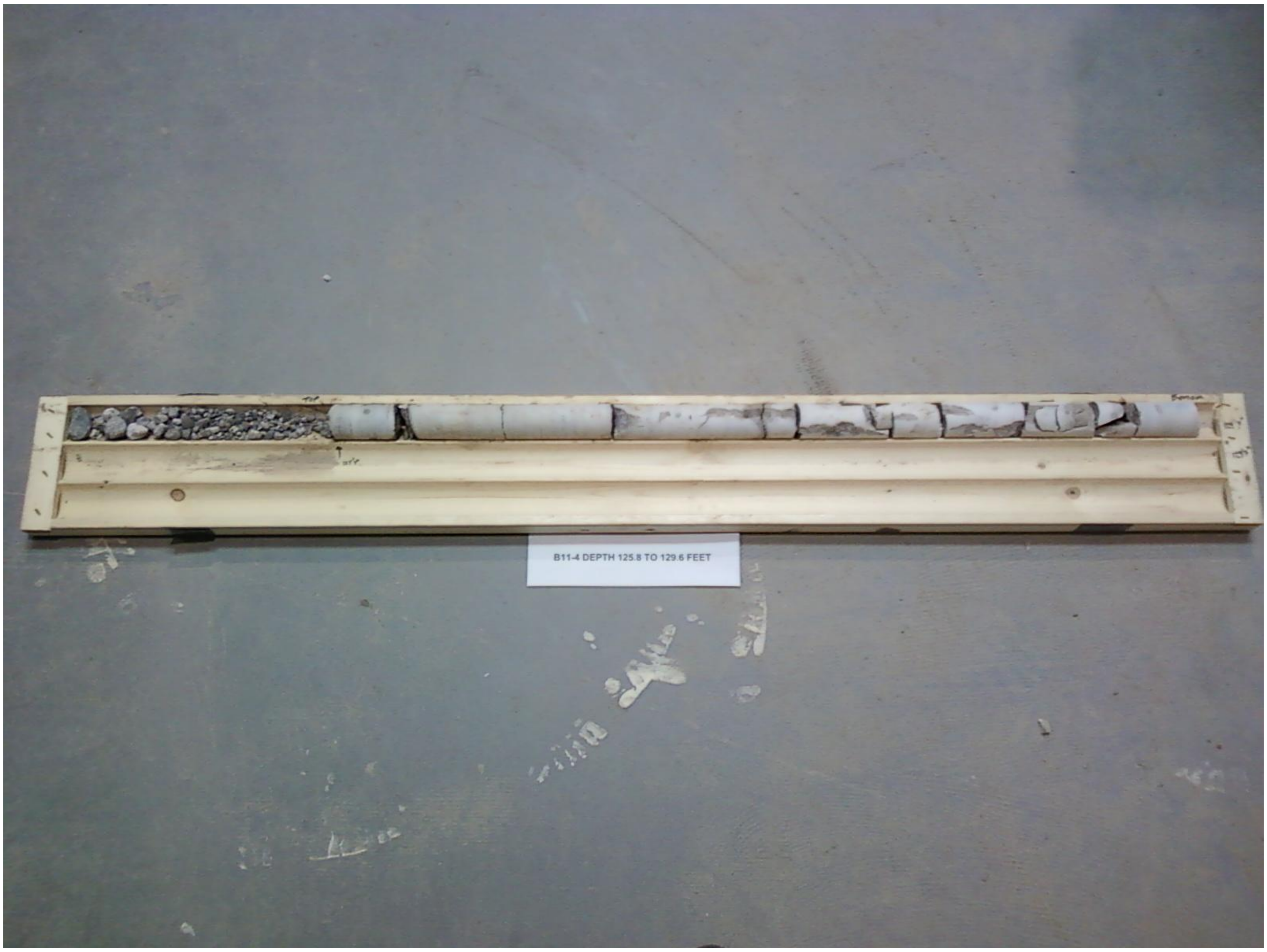
SBL of Talbot Rd. at Montgomery Dr. looking South

# Appendix H: Selected Rock Core Photographs

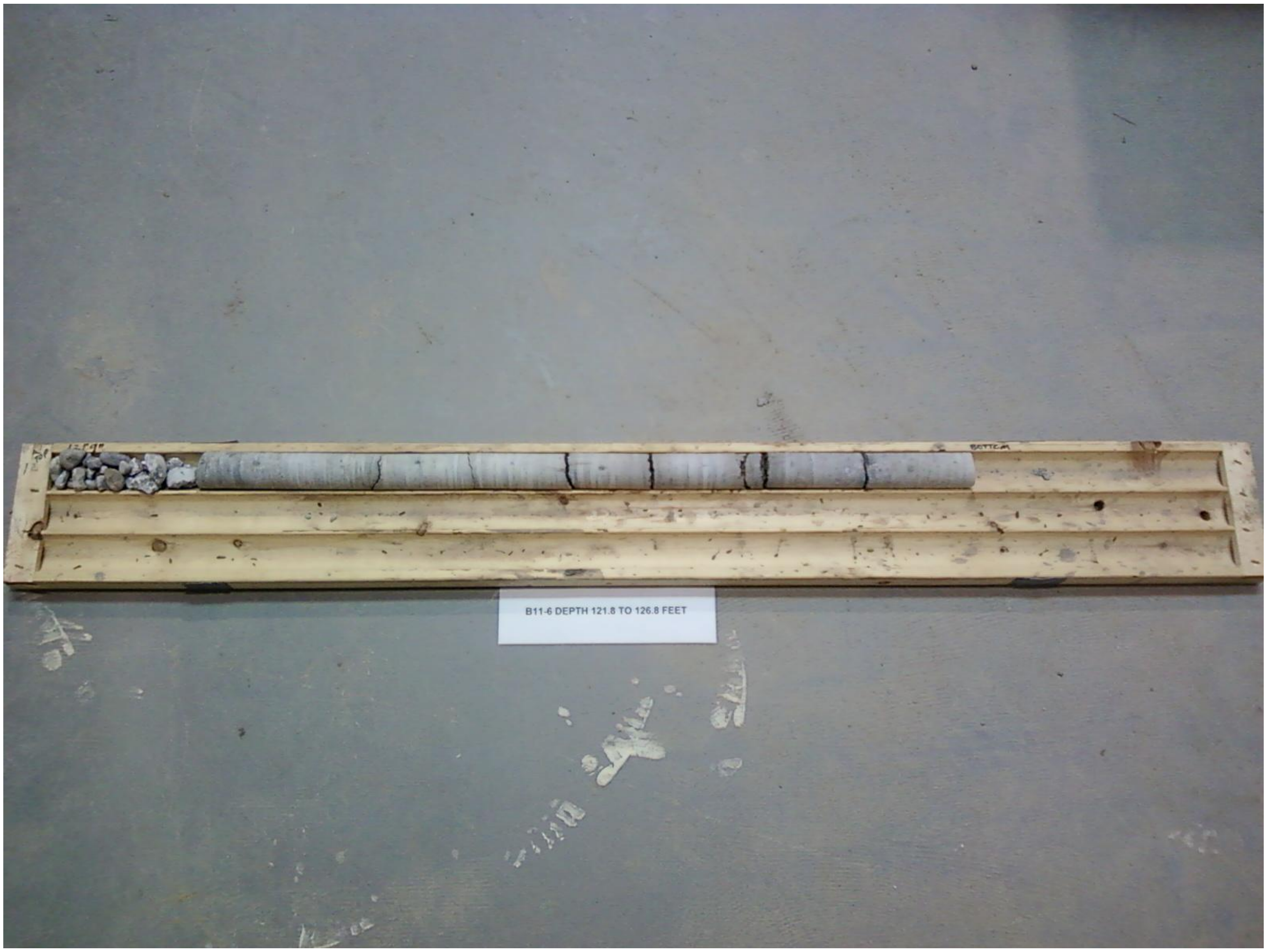

TOP
B11-1 DEPTH 121.5 TO 131.7 FEET
BOTTOM


Top 120'4"
124'2"
126'4"
Bottom 126'4"
B11-2 DEPTH 120.3 TO 126.3 FEET


B11-3 DEPTH 121.0 TO 132.0 FEET


B11-4 DEPTH 125.8 TO 129.6 FEET


B11-5 DEPTH 122.7 TO 130.1 FEET


BOTTOM
B11-6 DEPTH 121.8 TO 126.8 FEET


top
116'3"
126'3"
BOTTOM
B11-7 DEPTH 116.3 TO 126.3 FEET

## ROCK CORE PHOTOGRAPHS

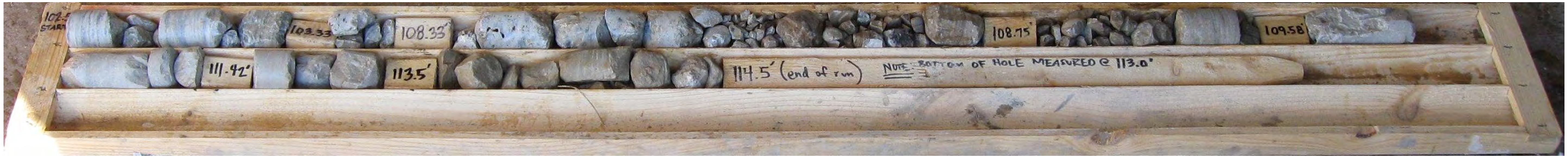


Photo 5: Borehole No. 107 – Rock Core. Elevation 155.17 metres to 150.95 metres.

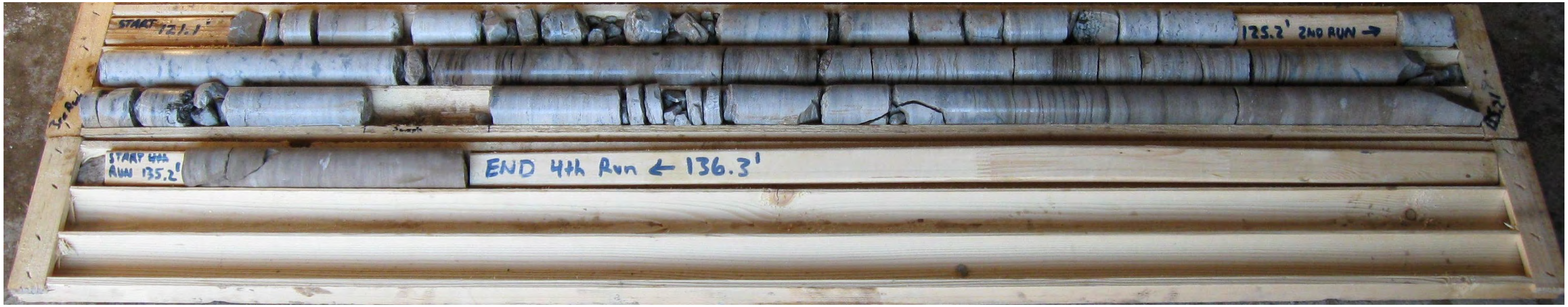
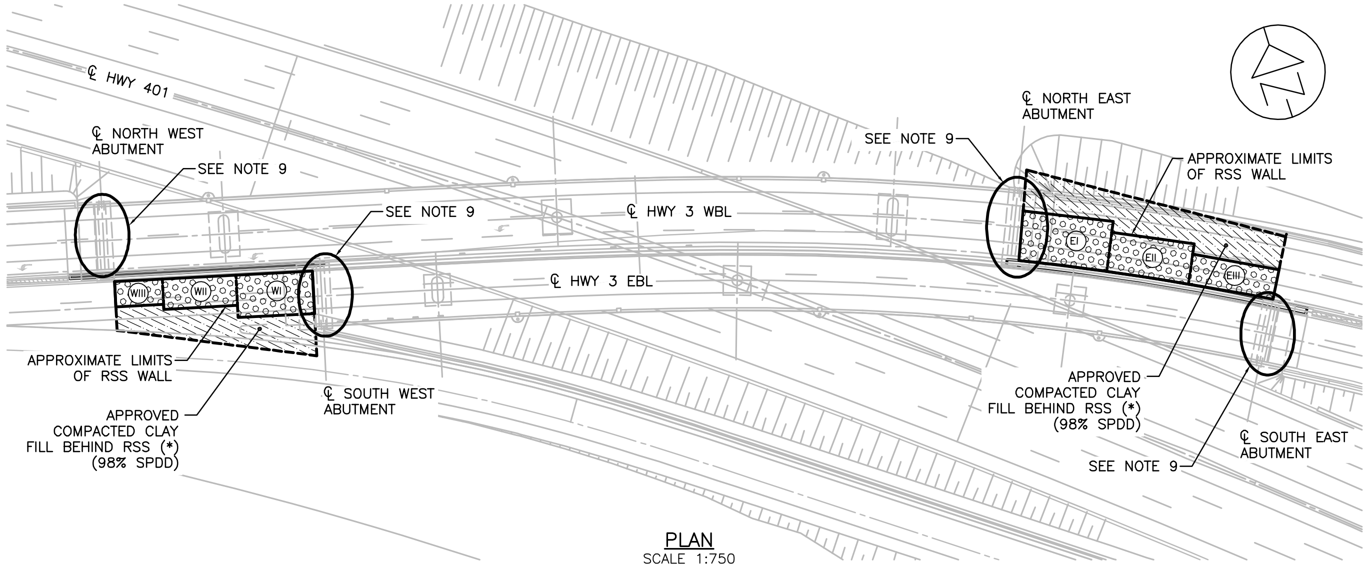


Photo 6: Borehole No. 109 – Rock Core. Elevation 149.18 metres to 143.79 metres.

# Appendix I: Conceptual Design

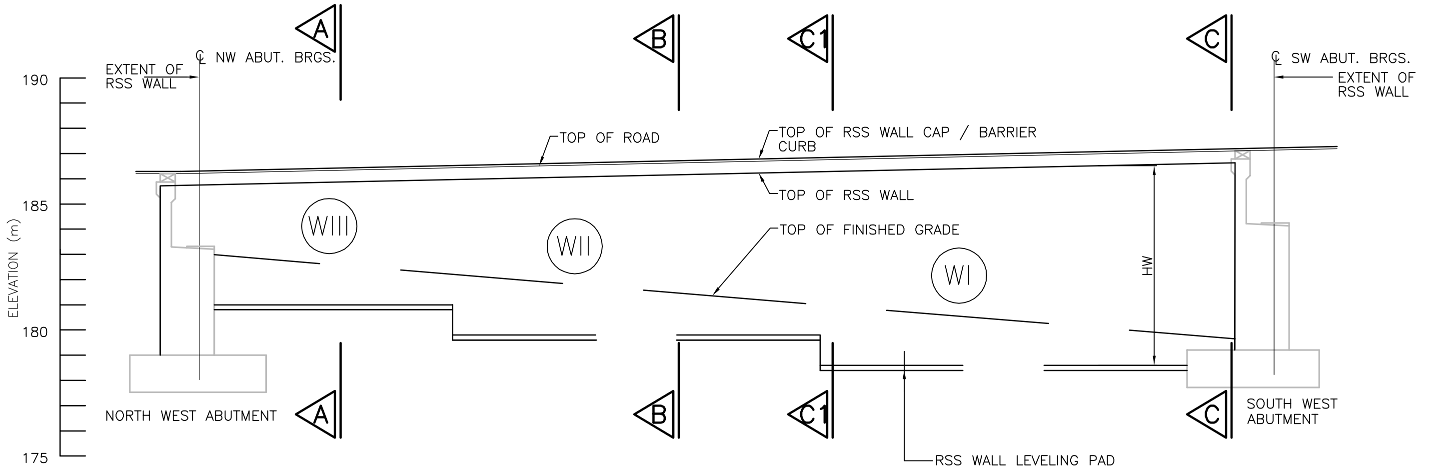


PLAN
SCALE 1:750

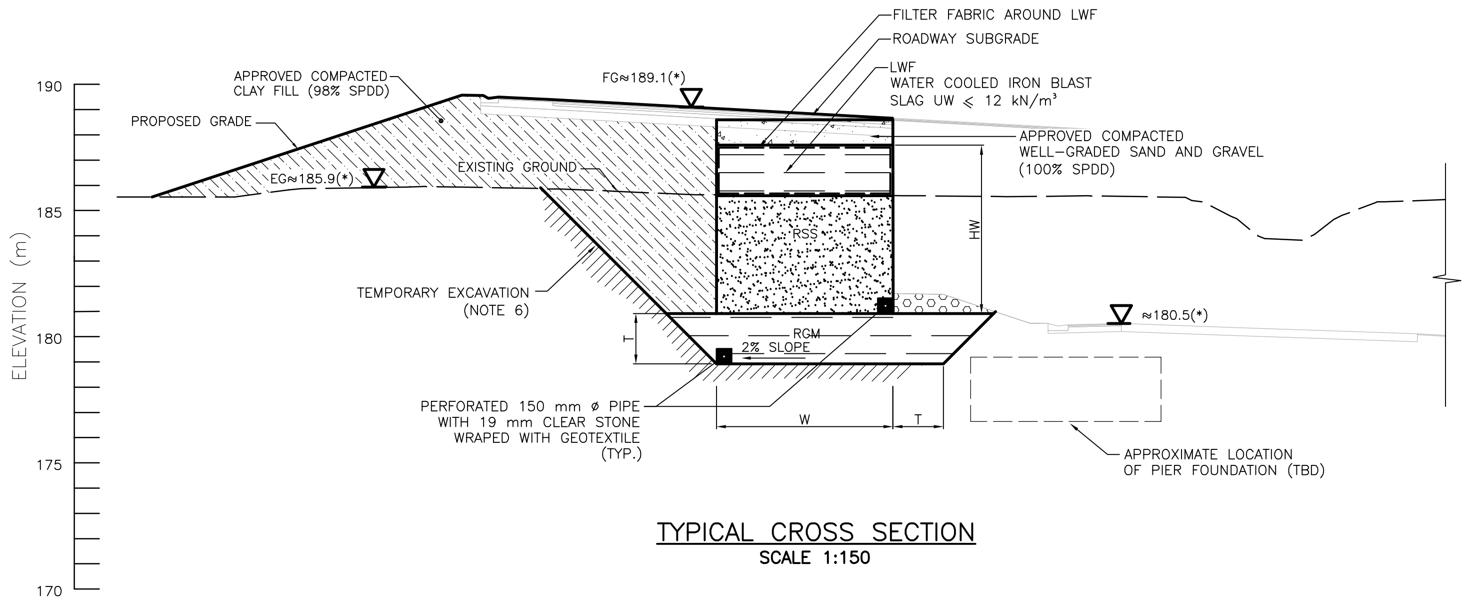
## NOTES:

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GEOTECHNICAL DESIGN REPORT.
2. THE ILLUSTRATED RSS DIMENSIONS REPRESENT THE MINIMUM DIMENSIONS FOR EXTERNAL AND GLOBAL STABILITY REQUIREMENTS. THE FINAL DESIGN OF RSS, RGM AND STRUCTURAL ELEMENTS ARE TO BE DEVELOPED BY OTHERS.
3. ABUTMENT AND APPROACHWAY EMBANKMENT ELEVATIONS AND DIMENSIONS FOR THE GEOTECHNICAL DESIGN WERE OBTAINED FROM STRUCTURAL DRAWINGS AVAILABLE IN JANUARY 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 EXERCISED.
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 MUST BE APPROPRIATELY PROTECTED, REGULARLY INSPECTED AND TREATED AS REQUIRED.
6. BACKFILL OF THE PIER FOUNDATION NEAR RSS WALL MUST BE ENTIRELY COMPLETED BEFORE BEGINNING OF THE RSS ERECTION.
7. BACKFILL IN FRONT OF RSS WALL TO BE COMPLETED TO NOT LESS THAN 500 mm OVER THE TOP OF RGM BEFORE THE CONSTRUCTION OF THE RSS WALL EXCEEDS 4 m IN HEIGHT.
8. SEE ACCOMPANYING DRAWINGS FOR APPLICABLE CONSTRUCTION NOTES.
9. GRANULAR BACKFILL BEHIND CONCRETE ABUTMENT AS PER OPSD 3101.150.

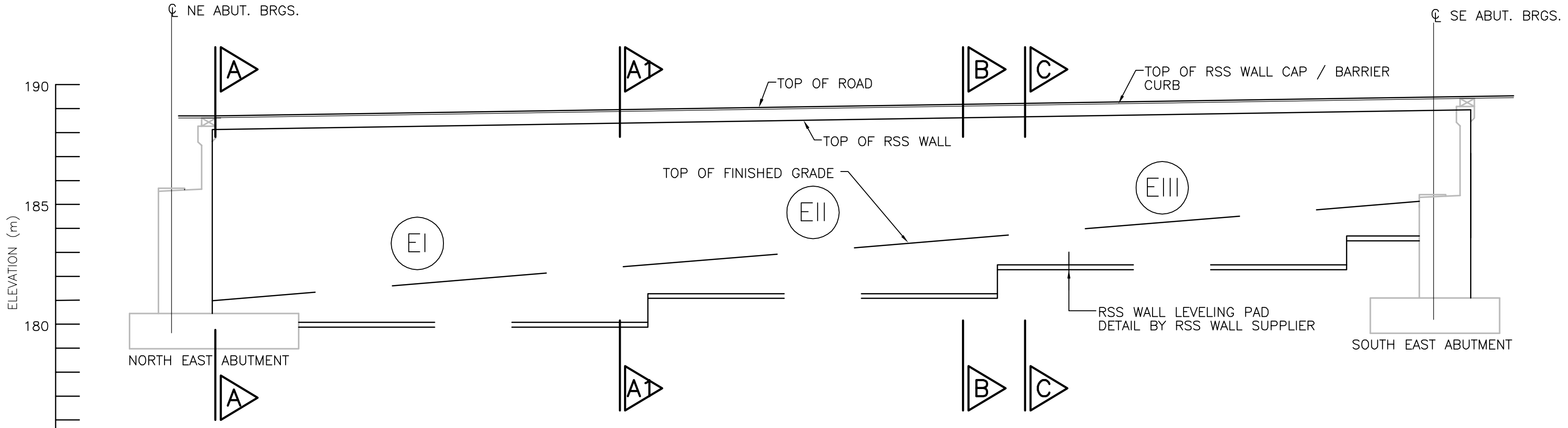
(*) VARIES



WEST RSS WALL
SCALE 1:150



TYPICAL CROSS SECTION
SCALE 1:150



EAST RSS WALL
SCALE 1:150

| RSS WALL | RSS STRUCTURE SIZE WIDTH, W x HEIGHT, HW (m) * | RGM THICKNESS, T (m) | LWF THICKNESS (m) |
|---|---|---|---|
| WEST RSS WALL | | | |
| SECTION A | 5.0 x 5.0 | 1.0 | NONE |
| SECTION B | 6.0 x 6.5 | 1.5 | NONE |
| SECTION C1 | 8.0 x 7.7 | 1.5 | 2 |
| SECTION C | 8.0 x 8.0 | 1.5 | 4 |
| EAST RSS WALL | | | |
| SECTION A | 8.0 x 8.1 | 1.5 | 4 |
| SECTION A1 | 8.0 x 8.3 | 1.5 | 2.5 |
| SECTION B | 6.5 x 7.4 | 1.5 | NONE |
| SECTION C | 6.0 x 6.2 | 1.5 | NONE |

* THE RSS WIDTHS ARE THE MINIMUM FROM GLOBAL STABILITY REQUIREMENT CONSIDERATIONS.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

NOT FOR CONSTRUCTION

Parkway Infrastructure Engineers
amec
Hatch Mott MacDonald

BRIDGE B–11
HWY 3 UNDERPASS NEAR MONTGOMERY
ABUTMENT EXCAVATION AND BACKFILL DETAILS

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

DOC: ABMT EXVTN & BACKFILL_FIG I.1