TECHNICAL MEMORANDUM

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Date	**March 25, 2011**		

Subject **Windsor Essex Parkway Project**
Bridge B-12: Preliminary 60% Geotechnical Design – Rev.0

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LIST OF DRAWINGS

(To be included in final report)

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1.0 INTRODUCTION

This memo provides preliminary 60% geotechnical recommendations for the 2-span Bridge B-12 structure (Howard Avenue underpass) located near Sta. 10+000T.

The WEMG proposal design for Bridge B-12 comprised integral abutments and centre pier founded on deep end bearing piles as shown in Figure 1[1]. Close false abutments using RSS wall system were also included. The WEMG proposal design has been accepted as 30% preliminary design. The pile foundation and abutment solutions adopted in the 30% design were based on geotechnical data and interpretation reports provided with the background geotechnical information[2] available at time of design development (March 2010).

The present geotechnical assessment represents a more in depth review of the 30% design solution for the available soil condition information. The 60% designs for the individual bridge, tunnel and other structures were to be developed after completion of the proposed additional geotechnical investigation at structure specific locations. However, due to delays in the start up of the additional investigation fieldwork and the need to advance the design work for a select group of structures, the so called "60% geotechnical design" for these structures will be based on limited and insufficient soil data obtained prior to the 30% design work. Bridge B-12 is one of these priority structures to be designed prior to completion of the additional investigation. In this regard, the soil data interpretations, design assessments and design recommendations given hereafter for the Bridge B-12 location are considered preliminary and subject to revision at a later stage when the soil and groundwater data are updated following completion of the proposed additional investigation.

The locations of the previously executed and the proposed additional investigation test holes are shown in Figure 2. Instrumentation for monitoring pore water pressures and excavation base heave during excavation is planned at strategic locations.

Bridge B-12 construction is expected to involve the following sequence of earthwork, design elements and loading stages:

- Temporary excavations to about 8 m (south abutment) and 11.5 m (north abutment) depth below grade.
- Installation of a 1.5 m thick Reinforced Granular Mat (RGM) foundation at the north abutment.
- Installation of piles (HP310x110) for all bridge supports driven to mobilize a ULS factored capacity of 2000 kN.
- Installation of 600 mm CSP around the pile stickup

[1] Figures are included at the end of the memo text.
[2] Subsurface Conditions Interpretation Report, Golder Associates, Revised December 2009: Soil properties were assessed over large (1000 to 1200 m long) segments of the parkway with little soil data available at Bridge B-12 location.

- Construction of the RSS structures and associated drainage works, and granular backfill behind the RSS structure.
- Filling of the CSP casing with loose dry sand followed by construction of the structural abutment (pile cap) and bridge deck
- Completion of final stage of backfill behind the integral abutments (including EPS as required).
- Completion of the pavements over the Highway 401 and over Howard Avenue.

2.0 SIMPLIFIED SOIL CONDITIONS AND DESIGN SOIL PARAMETERS

1. The test holes located at Bridge B-12 site and included in the current assessment are Boreholes BH-104, BH-105 and BH-301, cone penetration profiles CPT-2 and CPT-302 and Nilcon profile at BH-105. It should be noted that the data from BH-301 and CPT-302 was not available at time of the preliminary design work for the WEMG proposal. The borehole logs are included in Appendix A.
2. An approximate excavation profile for this structure is shown in Figure 3 which was developed on the basis of the roadway cross section at Tecumseh Sta. 10+000.
3. The design soil parameters were interpreted from the CPT and Nilcon vane profiles and the available laboratory test results. The approximate natural moisture content (wN), plasticity index (PI) and liquidity index (LI) for the silty clay crust layer (elevation 182 to 178 m) are 15%, 12% and 0.2, respectively (see Figure 4a). The approximate wN, PI and LI value variations with depth for the grey silty clay layer (elevation 178 to 165 m) are 18 to 28%, 15 to 18% and 0.3 to 0.8, respectively.
4. The Nilcon vane undrained shear strength (Su) profile was corrected for plasticity index (Bjerrum, 1972) and the Su-profiles from the CPTs were estimated using cone resistance (qt-σvo) and an empirical factor (Nkt, dependent on the soil type) (Ladd and DeGroot, 2007). As shown on Figure 4b, the Su variation with depth for the grey silty clay stratum was from about 80 to 55-60 kPa according to CPT-2 and about 60 to 50 kPa according to CPT-302. In the absence of other test data, the Su profile from CPT-302 was considered applicable.
5. Other relevant soil properties required for the analysis of stress and deformation response of the soils and foundations are provided in the calculation sections (Figures 5, 6, 7, 8 and 9).

3.0 DESIGN OF EXCAVATION AND TEMPORARY CUT SLOPES

- Excavations are expected to encounter surficial granular soils and some deleterious materials, and will be extended into the stiff clayey silt to silty clay. 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 soils may be classified as Type 3 soils if appropriate dewatering has been carried out. The excavations 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.

- While the complete excavation for Highway 401 does not need to be advanced to the roadway subgrade within the same excavation operation as for the abutments/pier, the stress and deformation assessment in this memo assumes that the bulk of the general excavation is conducted close to the slope profile shown on Figure 3. If other staging of the excavation is intended, a revision of the stress and deformation analyses will be required.
- Groundwater control will be required based on timing of construction and prevailing weather conditions.
- The slope stability analyses for temporary open cut slopes were carried out using Slope/W Version 2007, the Morgenstern-Price method of analysis and circular failure surfaces.
- The maximum depth of temporary excavation required at the north abutment is expected to be 11.5 m (including the sub-excavation required to accommodate the RGM foundation for the proposed false RSS abutment).
- A factor of safety (FS) of 1.26 was calculated for the temporary deepest excavation of 11.5 m for the slope profile (average 2H:1V) and assumed soil properties shown in Figure 5. Load restrictions at the top of slope are required for the limited period (estimated to 4 to 7 days) of sub-excavation of 1.5 m and construction of the RGM required at this location. The subexcavation for the RGM and the immediate completion of the RGM structure should be staged in 4 subsections (short length segments) along the toe of the excavation. Formworks may be incorporated within the mass of the RGM to create the holes for the future piles. Once the RGM is completed, FS increases to 1.39, including the effect of a potential construction surcharge of 10kPa at the top of the slope.
- A FS of 1.59 was calculated for the temporary slope at the south abutment where the total height considered was 8 m (Figure 6). No RGM is deemed necessary for the south abutment. The average temporary slope considered was at 1:1. However, considering the length of time of slope exposure, an average slope inclination of 1.5H:1V should be considered.
- The recommendations provided herein are based on the assumptions that (a) the temporary slopes are properly protected at all times against surface erosion due to runoff, desiccation, freeze-thaw effects, etc., and (b) the duration of the slope exposure is in general limited to 4 to 5 months. To protect the subgrade integrity, 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.
- Based on the analysis, basal heave at completion of the excavation for construction was estimated to be about 40 mm. This heave should have no impact on the performance of

the road base; however this data provides an indication of the anticipated geotechnical response and is expected to be monitored during construction.

- The calculated FS against basal uplift instability at the excavation bottom (due to hydrostatic pressure in the lower granular deposits) was greater than 2.0, which is considered acceptable.

4.0 DEEP FOUNDATIONS

- It is understood that HP 310x110 steel H piles driven to competent foundation material to mobilize a target ULS capacity of 2000 kN are being considered. Preliminarily, the tips of piles are anticipated to be set at about elevation 155.5 m.
- The actual pile capacity should be confirmed by static load tests at strategic locations in conjunction with testing using Pile Driving Analyzer (PDA). The static load tests will facilitate proper calibration of the PDA, pile driving equipment performances and determine the appropriate driving criteria (set).
- The steel H piles should be installed and monitored in accordance with OPSD 3000.150 and OPSS 903 standards. The piles should be reinforced with Type I shoe flanges as shown in OPSD 3000.100. Provision should be made to re-tap the piles to confirm the set after adjacent piles have been driven.
- Due to the potential artesian conditions in bedrock, it is recommended that the pile splicing be completed by butt-welding to minimize the pathways for upward flow of artesian water along the piles to the surface. Indications of gas, water, and fines washout should be monitored. Provision to mitigate such occurrences (heavy mud, grouting of the cavities, etc.) should be considered.
- Consideration should be given to potential driving difficulties due to the presence of 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) will be considered.
- Backfill surcharge behind the abutments may cause some downdrag loads and bending of the piles. This bending moment is in addition to structural bending moment assessed in pile due to imposed loads by the bridge structure. The estimated potential negative skin friction and bending moment are as follows:

Maximum unfactored negative skin friction	= 140 kN per pile
Maximum unfactored bending moment along strong axis of pile	= 150 kN-m per pile

- In the case of piles installed before the construction of the RSS walls, it is estimated that the free pile heads may deflect from the initial position (after completion of driving) by up to 10 mm at the top of the RSS structure after the completion of the false abutment.

- The preliminary horizontal subgrade reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficients:

k_x = $n_h(z/d)$ - for cohesionless soils,
= 67 Su/d - for cohesive soils.

Where:

k_x (MPa/m)	= soil modulus of horizontal subgrade reaction
n_h (MPa/m)	= soil coefficient
Su (MPa)	= Undrained shear strength
z (m)	= Depth of calculation section below finished grade
d (m)	= pile diameter/width

- The recommended ranges of soil parameters are tabulated as follows:

Anticipated Soils surrounding the piles	Elevation Range (m)	nh (MPa/m)	Su
Compacted Granular Fill within RSS (*)	Above El.177 at North abutment Above El.179 at south abutment	10 to 15	-
Loose Sand (within CSP) (*)	Above El.177 at north abutment Above El.179 at south abutment	2 to 5	-
Native Stiff Silty Clay	El.180 to El.177	-	Decreases linearly with depth from 0.075 MPa to 0.05 MPa
Native Firm Silty Clay	Below El. 177	-	0.05 MPa

(*) Due to the close proximity of the piles to the face of the false abutments, the pile design to lateral loads acting towards the face of the RSS walls should consider also an additional assumption that n_h=0. The RSS suppliers should be informed and consulted on the impacts on the RSS structures of the deflecting piles towards the face of the RSS walls.

5.0 RSS ABUTMENT WALLS

5.1 Global Stability

- Figures 7 and 8 illustrate the slope stability models for short-term and long-term loading conditions for the north abutment which poses more challenges due to the greater height (10 m from the top of the slope to the top of the RGM) and lower foundation grade (El.175.5 m at the base of RGM) compared to the south abutment (8 m high, founded at El. 179 m). The RSS structure parameters were assumed. The actual design of the RSS is to be provided by the RSS supplier, and is beyond the scope of this design memo.
- The calculated FS values are in excess of 1.3 against global instability and satisfy the PA criteria. Incorporation of the RGM beneath the RSS wall will have no effect on this FS.

- The stability conditions of the south abutment, whose height is 2 m less than the north abutment, was not analyzed at this time but it is expected that they are similar, or greater, than those for the north wall.

5.2 ULS Bearing Capacity

- The following gross factored geotechnical resistance values (q_u) were determined for the native subgrade soils at the two abutments:

Abutment	Assumed Subgrade Elevation	Condition	q_u (kPa)
North	175.5	Short-Term (Undrained)	160
		Long-Term (Drained)	465
South	179	Short-Term (Undrained)	155
		Long-Term (Drained)	335

The above resistances are applicable in conjunction with the specific RSS wall and RGM configurations and sizes described below.

The overall dimensions and makeup of the false abutments at this site have been checked for the following Loading Combinations:

- SLS (1D+1E+0.9LL)
- ULS Combination 1a – (1.25D + 1.25E +1.7LL)
- ULS Combination 1b – (0.8D +1.25E)
- ULS Combination 9 – (1.35D + 1.25E)

Where: D – dead loads (based on an average characteristic unit weight of the backfills of 21 kN/m3
E – Earth pressures
LL – Live Loads on top of the wall (assumed uniform distributed with the characteristic value of 12 kPa)

The following total abutment (RSS wall and associated top fill) dimensions were determined to meet the most severe of the above conditions:

Abutment Location	Assumed Total Height(1), m	RGM Size (thickness x length)	EPS Size, m (thickness x length)(2)	RSS Structure Size, m (width x height)(3)
North	10	1.5 x 10	3x14	6.5x5
South	8	Not Required	2x13	6.5x4

(1) Measured from top of finished pavement to the base of the RSS structure

(2) Assumes EPS is placed at/near the top of the RSS structure with the balance of soil backfill placed above the EPS. The use of EPS (or equivalent light-weight fill) was required at both abutments to meet the ULS design for the undrained (short-term) bearing conditions.

(3) The RSS supplier may require wider structures to meet the internal design requirement

5.3 SLS Performance

- A preliminary stress and deformation analysis was conducted on a structure – subgrade soils model illustrated in Figure 9.

The estimated horizontal deflections of the RSS wall face are as follows:

Loading Stage	Horizontal Deflection of RSS Wall Face at Top (mm)	Horizontal Deflection of RSS Wall Face at Bottom (mm)	Estimated Wall Rotation	PA Allowable Rotation based on 1H:24V Batter
End of RSS Wall Construction	<(-)10	<10	0.004	0.021
End of Construction	10 to 15	<30	0.004	0.021
Long-term Post-construction	10 to 15	<35	0.005	0.021

Note: (-) indicates lateral movement toward the back of the wall

- The RSS wall is to be designed and constructed in accordance with MTO's RSS Design Guidelines and Special Provisions SP599S22 and SP599S23.
- The post-construction (long-term steady state loading condition) settlements at the face of the RSS structure and on top of the approach way were estimated as follow:

Loading Stage	Settlement at Top of RSS Wall (mm)	Settlement at Top of Pavement at Edge of Approach Slab (mm)
End of RSS Wall Construction	~10	N/A
End of Construction	< 25	<40 (*)
Long-term Post-construction	< 30 (**)	<10 (**)

(*) The pavement surface settlement indicated above for the end of construction will be compensated by additional fill during preparation of the subgrade surface.

(**) Expected to occur within a few months to one or two years following the completion of the fill if the soil stresses within the zone of influence remain below the pre-consolidation pressure.

- The deformations and 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 that may occur further to inadequate compaction.
- It should be noted that the above RSS wall face deflections depend on the deformability of the foundation soils as well as of the RSS wall itself. The deformability characteristics of the latter have been assumed as for a homogeneous material characterised by a deformation modulus of 60 MPa and a unit weight of 21 kN/m^3. This assumption has to be confirmed by the RSS supplier.

6.0 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.
- The fill should be compacted in maximum 200 mm thick loose lifts in accordance with SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 and 3190.100.
- Behind the RSS structure a particular attention should be given to the critical subdrainage system along the face of the temporary slope (see Figure 8). The drainage of backfill behind the RSS wall is critical and is required to ensure the long-term global stability of the abutment. Subdrainage should be provided if clay backfill is used between the back of the RSS wall and the excavation slope face. Alternatively, free draining sand and gravel fill (Granular B Type I, or approved equivalent) may be used for backfill behind the RSS wall, which will ensure good long-term drainage and keep the phreatic surface low.
- 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 Canadian Highway Bridge Design Code CAN/CSA-S6-06.
- Earth pressures on abutment and wing walls may be calculated on the basis of the following parameters:

Parameter	Group I Soils (*)	Group II Soils(*)	Group III Soils (*)
Fill unit weight:	22 kN/m^3	21 kN/m^3	20.5 kN/m^3
Coefficients of static lateral earth pressure:			
'active' or unrestrained, K_a	0.27-0.30	0.30-0.35	0.35-0.45
'at rest' or restrained, K_o	0.45-0.50	0.50-0.55	0.60-0.70
“passive’	3.3 – 3.7	2.8 – 3.3	2.2 – 2.8

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

Due to the weight of the approachway surcharge above the top of the RSS structure, the total thrust exercised by the abutment wall on the bridge structure may be larger than the total force calculated from the conventional earth pressures. The actual thrust will depend also on the level of restraint to lateral displacement of the pile cap caused by the girders and bridge deck.

7.0 RGM FOUNDATION

- A 1.5 m thick, 10 m wide, RGM foundation, or equivalent, is be required under the taller north false abutment wall to meet the ULS bearing capacity requirements for undrained conditions (See Figure 10 for the assumed geometry of RGM). The following loads where estimated to act on top of the RGM (i.e., the underside of the RSS wall) on the basis of conventional calculation of the bearing pressures under gravity retaining walls.

Loading Stage	SLS Stresses (kPa) (1)		Max. ULS Stresses (kPa) (2)
End of Construction	157	148	193
Long-Term	186	122	211

SLS load combination (1xD+1E+0.9LL) as per CHBDC

ULS - 1 load combination (1.25xD+1.25E+1.7LL) was determined to be the most critical.

- The properties used for the backfill materials were those defined for the Global Stability analyses, and are given as follows:

  - Unit weight for Clay Fill — 21 kN/m^3
  - Unit weight EPS — 0.5 kN/m^3 (ignored in calculations)
  - Undrained Strength of Clay Fill, Su — 50 kPa
  - Drained Angle of Internal Friction of Clay Fill, φ´ — 30º

8.0 DEWATERING

- Further details of temporary and permanent dewatering needs will be determined when additional soil information becomes available for this particular bridge site.
- The design of the dewatering system should comply with the OPSS 517 and 518 provisions.

9.0 TAF INSERTS

9.1 Design/Assessment Criteria

- The designs are as per Project Agreement – Schedule 15-2, Part 2 – Design and Construction Requirements, Article 5
- The foundations' designs are as per the principles of Limit States Design (LS Method) based on Load and Resistance Factors (CHBDC and Canadian Foundation Engineering Manual)
- Working Stress Design (WS Method) is employed for global stability of the false abutment foundations and/or earthworks
- Deep foundations are designed to meet or exceed the applicable requirements of MTO Structural Manual and OPSS 903 of 2009.
- All piles at this project are designed as end-bearing piles generally on bedrock.

- The design pile capacities (axial and lateral loads) will be assured by suitable driving equipment and procedures.
- Negative skin friction and shaft bending due to soil deformation have been considered.
- The geotechnical design of the RSS foundations was conducted on the basis of LS method.
- Proprietary retaining systems will be designed and constructed in accordance with MTO's RSS Design Guidelines and Special Provisions of SP599S22 and SP599S23. RSS walls will not be used as or for True Abutments.
- The internal design of the RSS structures will be based on the LS method.
- The stability of the soil mass containing the retaining wall was checked for all potential surfaces of sliding and have a minimum factor of safety exceeding 1.3.
- The face batter of the permanent retaining walls will not be steeper than 1H:24V. At no time during the project term, The differential rotational displacements of the wall face was checked to ensure that they did not exceed 50% of the as-built wall batter.
- Long-term creep is not a factor since the soil stress increases at this structure are maintained below the pre-consolidation stresses.

9.2 GROUND CONDITIONS

- The soil and groundwater condition data provided in the Baseline Report (from previous geotechnical investigations) are considered valid and applicable. The soil stratigraphic conditions and soil properties will be interpreted and updated from the results of the geotechnical investigations carried out previously by others and the additional investigation to be carried out by AMEC.
- The soil conditions and design parameters will be based on investigation data at the structure location with due consideration for the data in the vicinity.
- As noted in Section 1.0, the geotechnical analyses and design recommendations provided in this memo are preliminary and are subject to change based on interpretation of the updated soil data (combined results of the previous and proposed additional geotechnical investigations).
- Details of geotechnical investigation proposed to validate basis of design/assessment.

	Borehole #s	***CPT #s***	***Nilcon***	***DMT***	***Consolidation & Triaxial Tests***	***Instrumentation***
Background Investigations (Golder, 2009 & 2010)	*BH 104* *BH 105* *BH 301*	*CPT 2* *CPT 302*	*BH 105*	*na*	*4 one point CIUC*	*1 br OW+1 sh OW* *1 br OW* *1 br OW*
Proposed Additional Investigation	*B12-1* *B12-2* *B12-3*	*CPT 12-1*	*B12-1*	*DMT 6-RW*	*1-set CIUC* *1 CT*	*1 set of 3 VWP* *1 set of 2 MHSR*

(sh) – Shallow ; (br) – Bedrock; MHSR – Magnetic Heave/Settlement Rings; VWP – Vibrating Wire Piezometer; OW – Observation Well

9.3 Description of Foundations

PILES

Structural Foundation is designed on end bearing HP 310x110 piles driven to adequate bearing strata using an ULS capacity of 2000 kN. The design capacity and associated driving criteria will be confirmed by load tests and PDA. Driving Refusal (blows/25 mm) and Hiley charts will be developed and calibrated with the static load tests and PDA.

SLS resistance to vertical loads is not an issue since the bedrock is anticipated to not yield under the ultimate loads. Hence the pile axial deformations should be comparable with the elastic compression of the pile shaft (less than 18 mm for a 30 m long shaft loaded to an estimated SLS = 1400 kN).

Lateral pile response and axial stress increase due to soil stress increase from approachway fill was assessed on the basis of the acceptable methods of analyses (MSR and 'p-y' concept) using commercial software (L-Pile, Sigma/W), and will be confirmed & calibrated by field load tests and laboratory tests.

FALSE ABUTMENTS

The use of RSS solution was adopted as a preferred option due to the weak and compressible foundation soils and economical considerations.

The internal design of the RSS will be provided by the specialty supplier and verified by us to meet the specifications in the PA.

The external global stability was designed for a minimum factor of safety in excess of 1.3 for both the short-term and long-term conditions.

The bearing conditions are verified at ULS and SLS using the methods applicable to gravity type of retaining walls as per CHBDC.

To assess the required Site Performance Rating (SPR) of "HIGH", modeling of the wall expected deformations was carried out using SIGMA-W along with soil and material deformation properties determined by tests on the retained soils and strips.

9.4 Results of Test of Ground Water (E.G. Ph Value, Chloride Or Sulphate Content) and Any Counteracting Measures Proposed

The corrosion potential will be tested and, if required, appropriate mitigation measures will be considered (cathodic protection, sacrificial steel thickness, etc). Elevated content of H2S in the groundwater is anticipated.

9.5 Differential Settlement to be Allowed for in Design of Structure

5 mm to 100 mm measured at distances from the back of the abutment stub from 0 m to 100 m at the Expiry Date.

9.6 Anticipated Ground Movements or Settlement Due to Embankment Loading, Flowing Water

Total post-construction settlement of about 10 mm is anticipated at the top of the approachway due to the weight of the RSS, additional surcharge, and drawdown of the groundwater table. This long-term ground settlements are expected to occur substantially within 2 years following completion of construction.

9.7 List of Drawings

- 285380-04-090-SEG0-0015 Location Plan and Profile Sta13+400L to Sta 10+100T
- 285380-04-091-SEG1-0122 Location Plan and Sections at Bridge B-12
- 285380-04-091-SEG1-0123 Stratigraphic Sections at Bridge B-12

NR/dd/nsv

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Attachments:

Figures 1 to 10
Appendix A - Earlier Borehole Logs

FIGURES

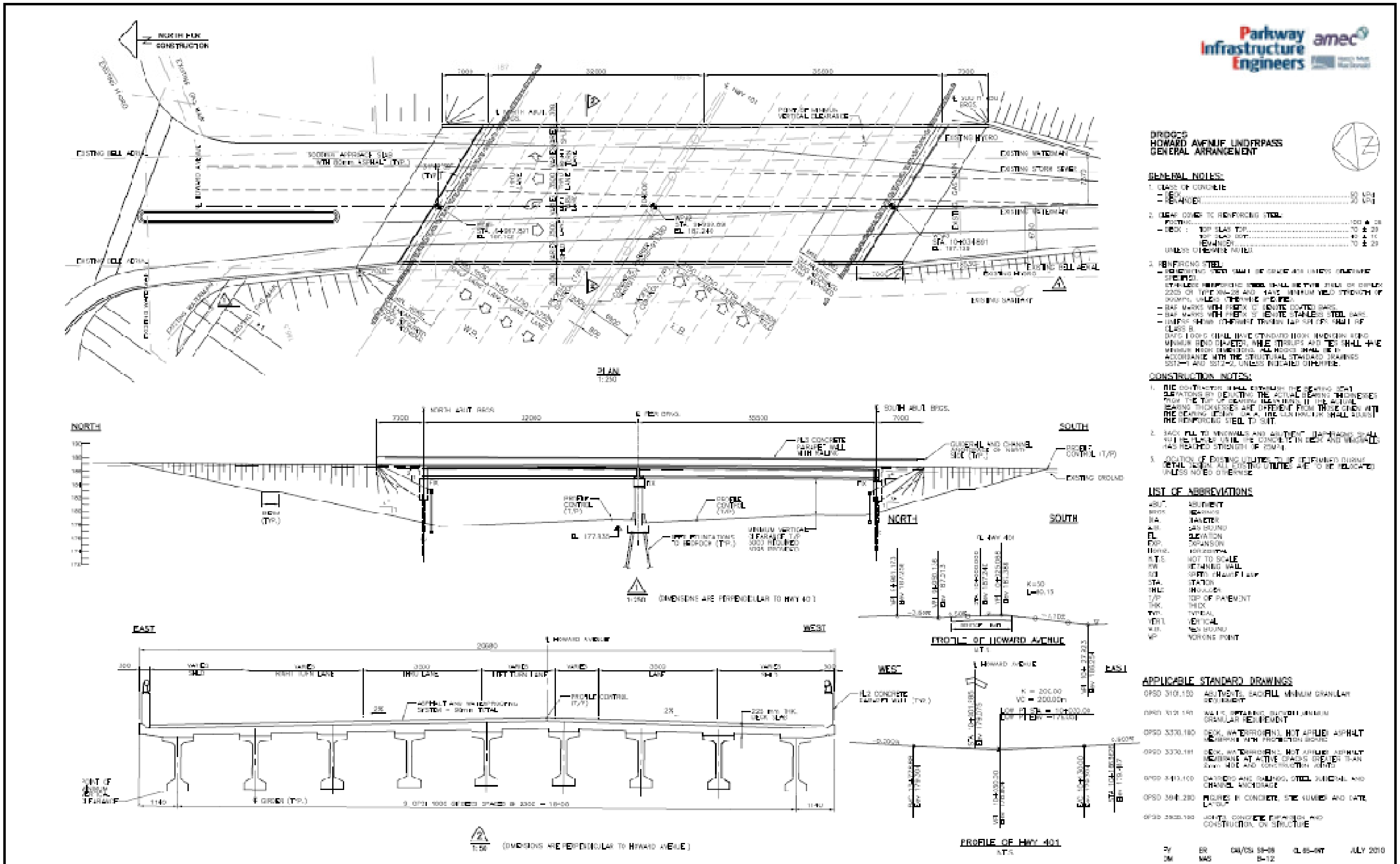


Figure 1: Structural Layout of Proposed Bridge B-12

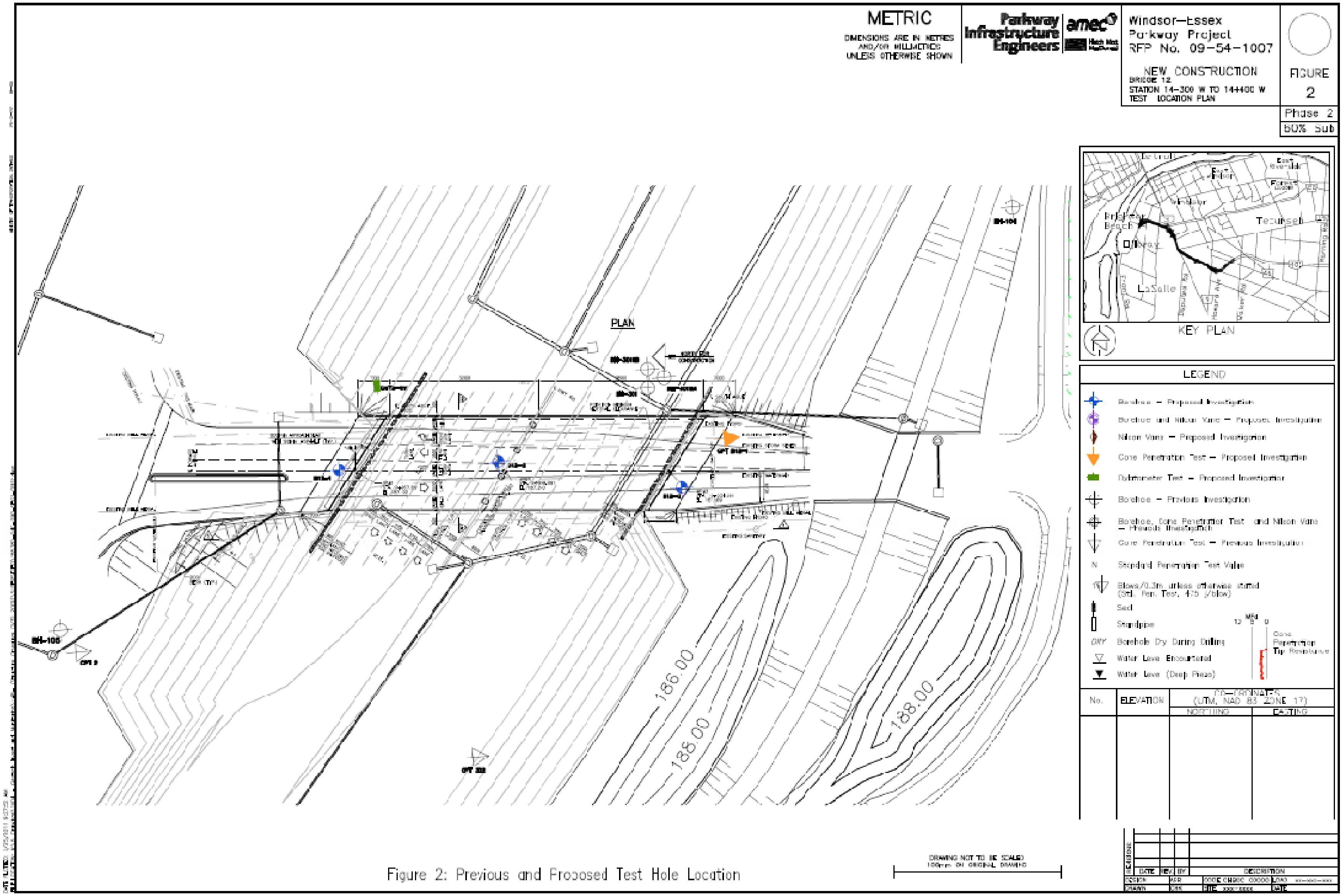


Figure 2: Previous and Proposed Test Hole Location

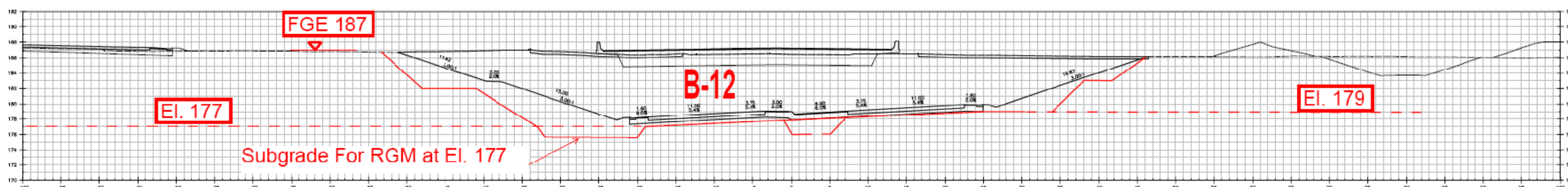


Figure 3: Temporary Excavation Profile for Bridge B-12 at Station 10+000T

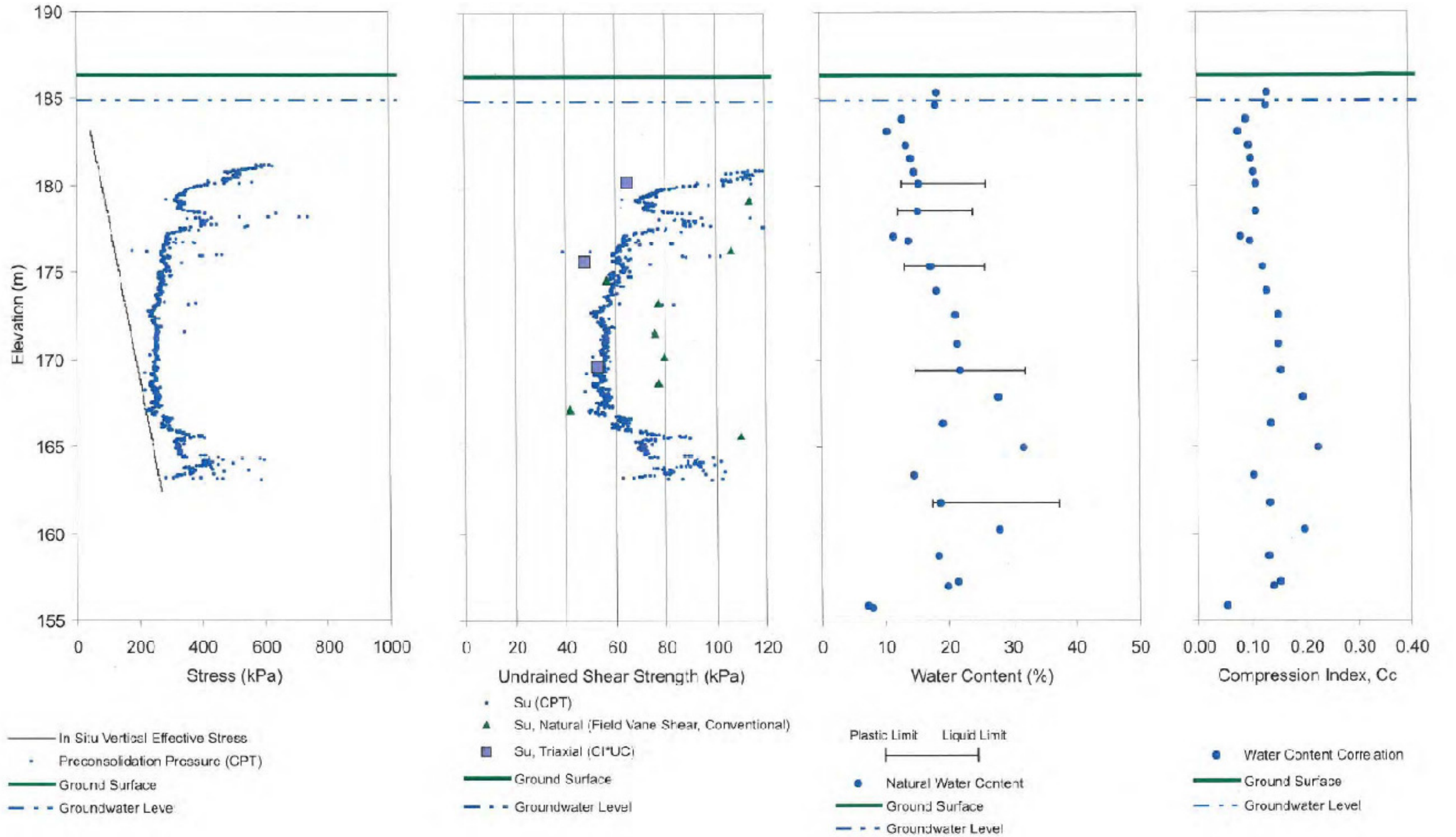


Figure 4a: Soil Properties from Previous Investigations (BH-105 / CPT-2)

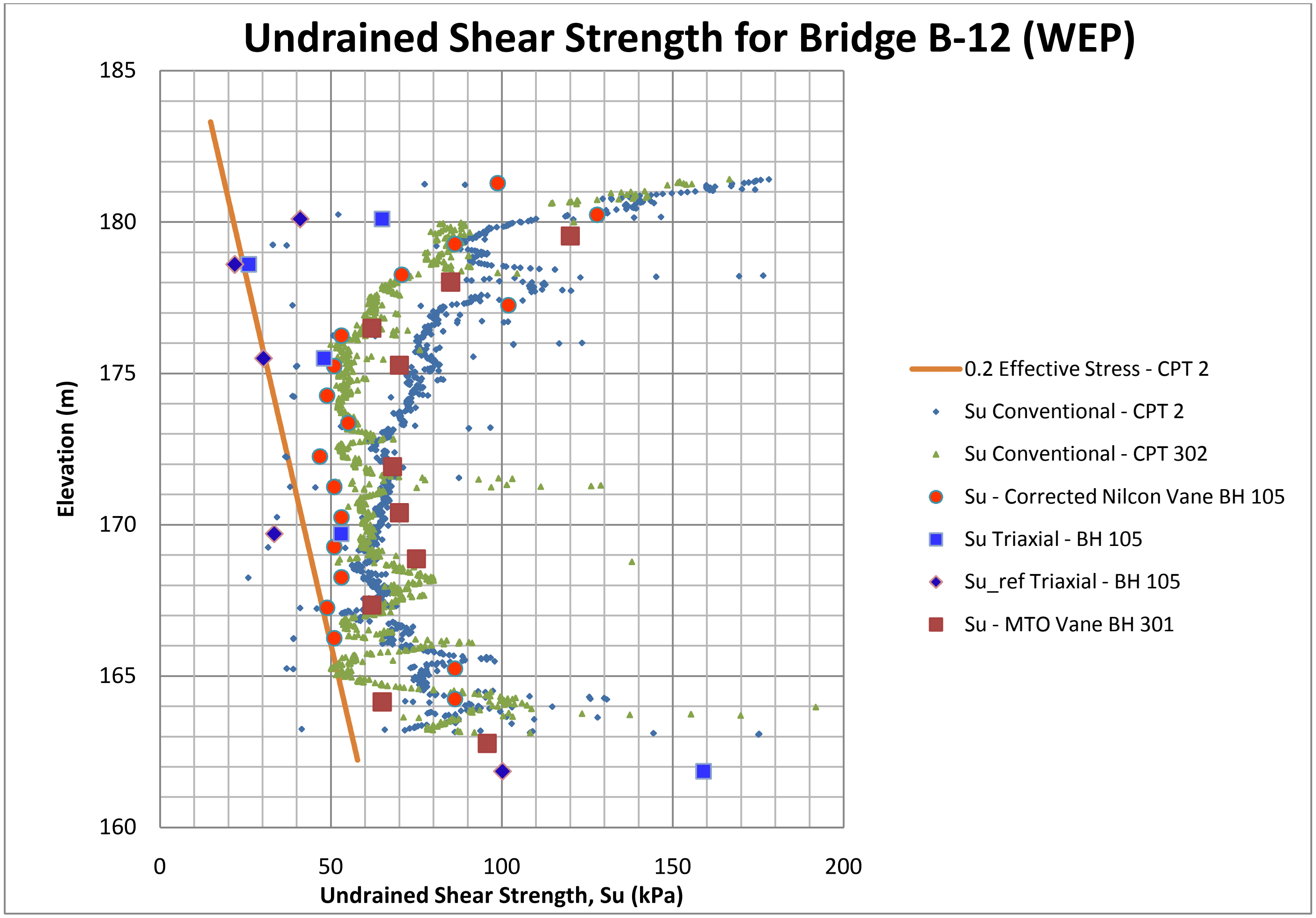


Figure 4b: Undrained Shear Strength Profile at Bridge B-12 Site (Re-Interpreted)

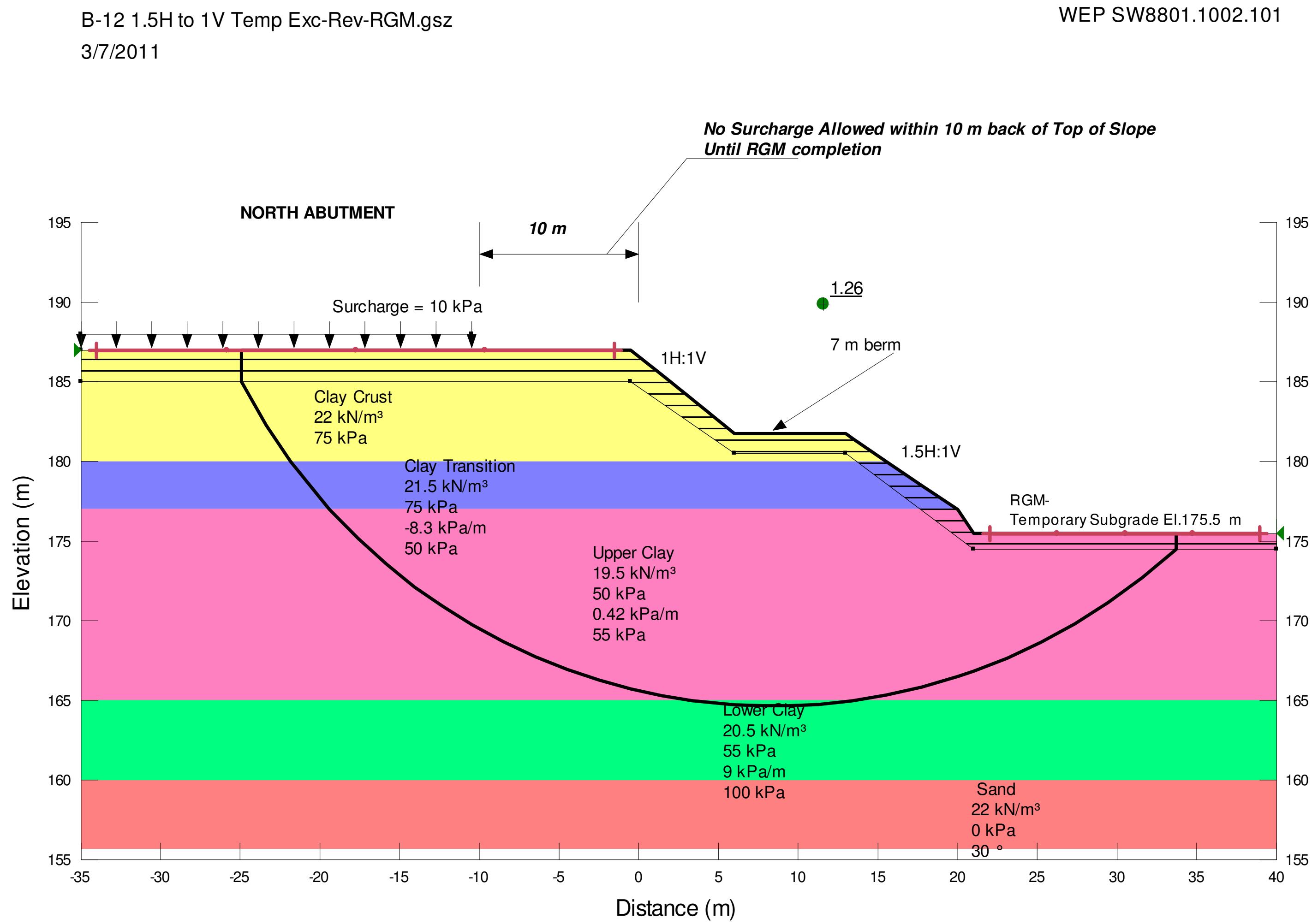


Figure 5: Stability Analysis of Temporary Excavation Slope at North Abutment

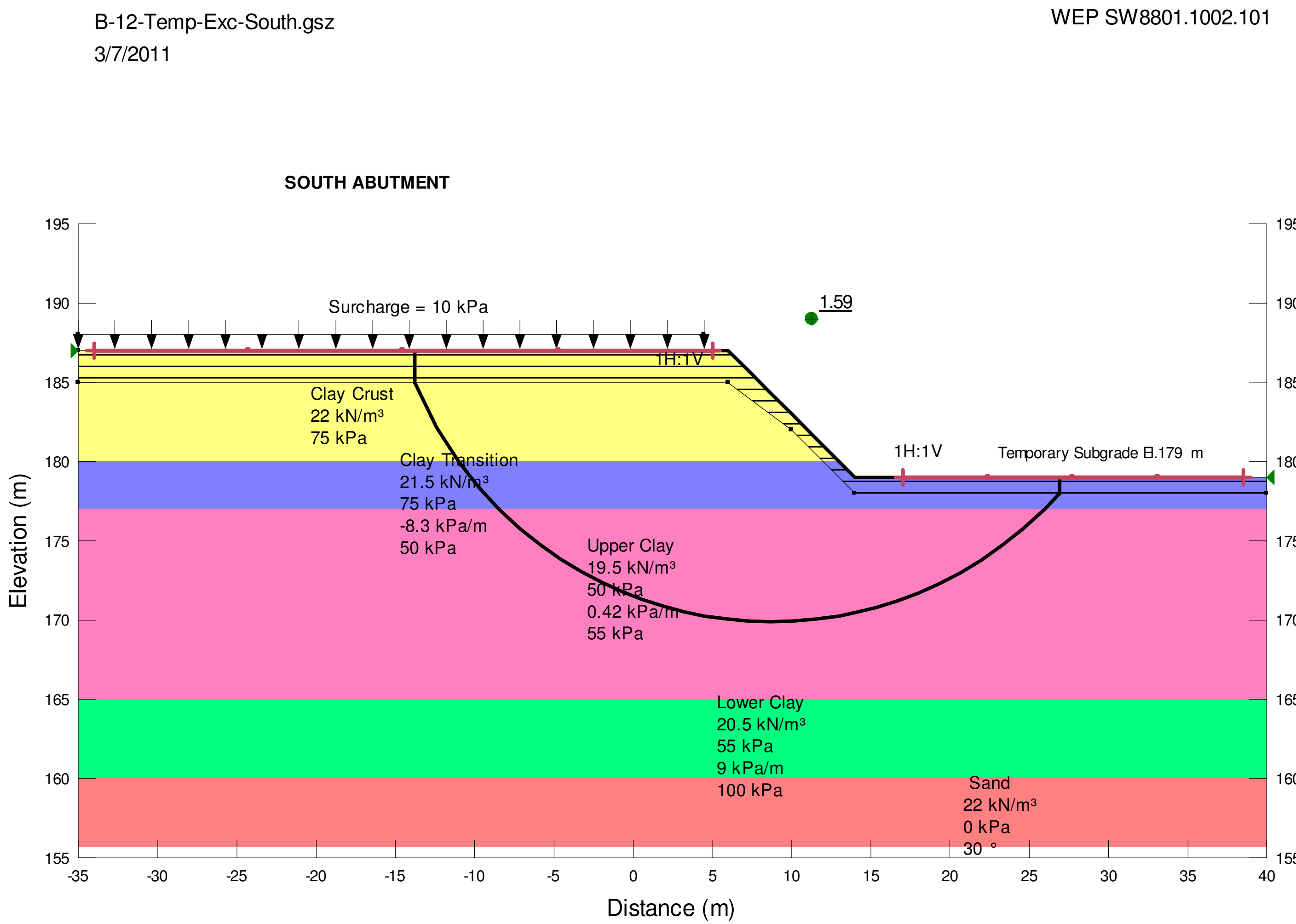


Figure 6: Stability Analysis of Temporary Excavation Slope at South Abutment

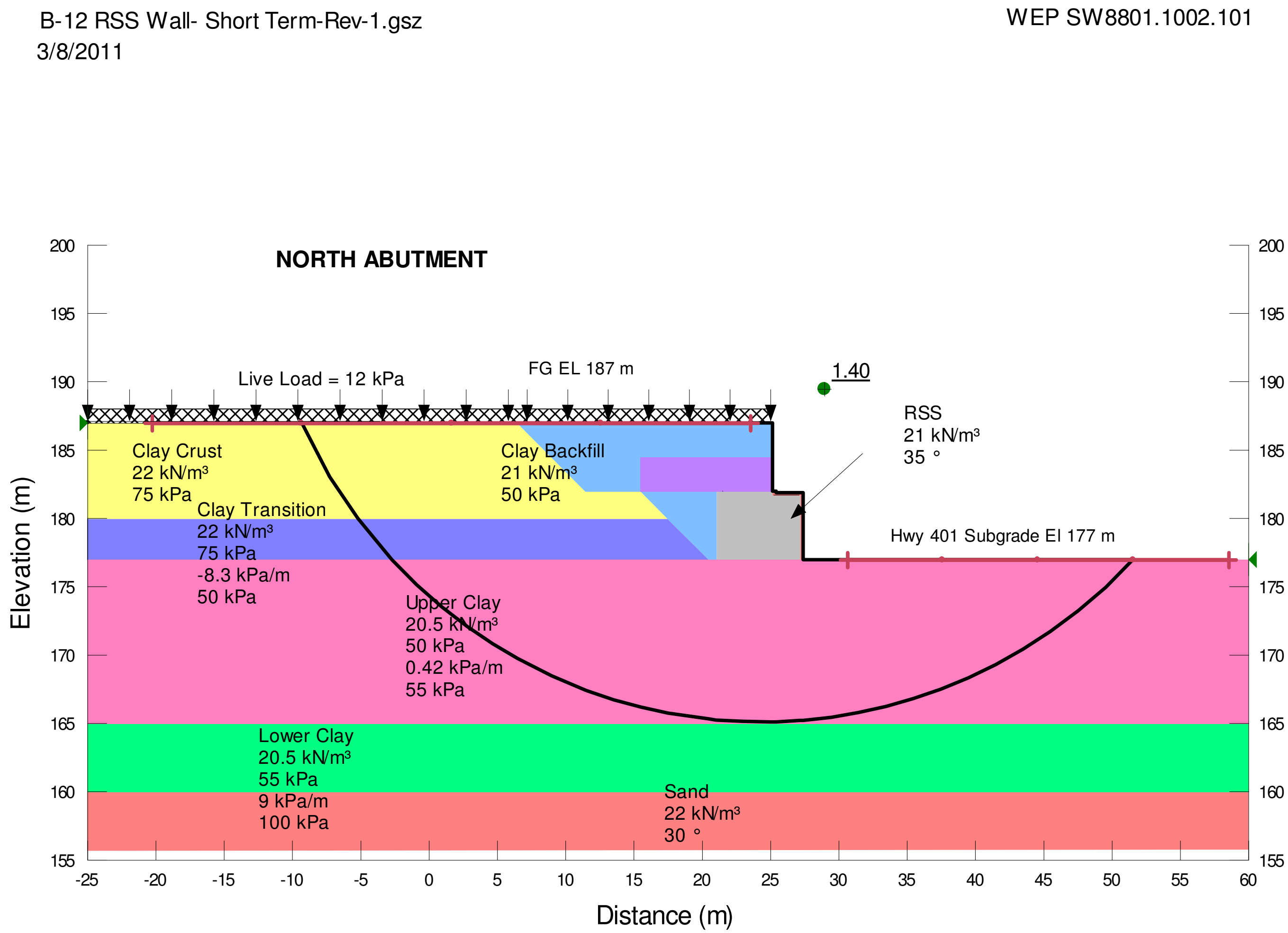


Figure 7: Global Stability (Short-term Loading) of North Abutment RSS Wall

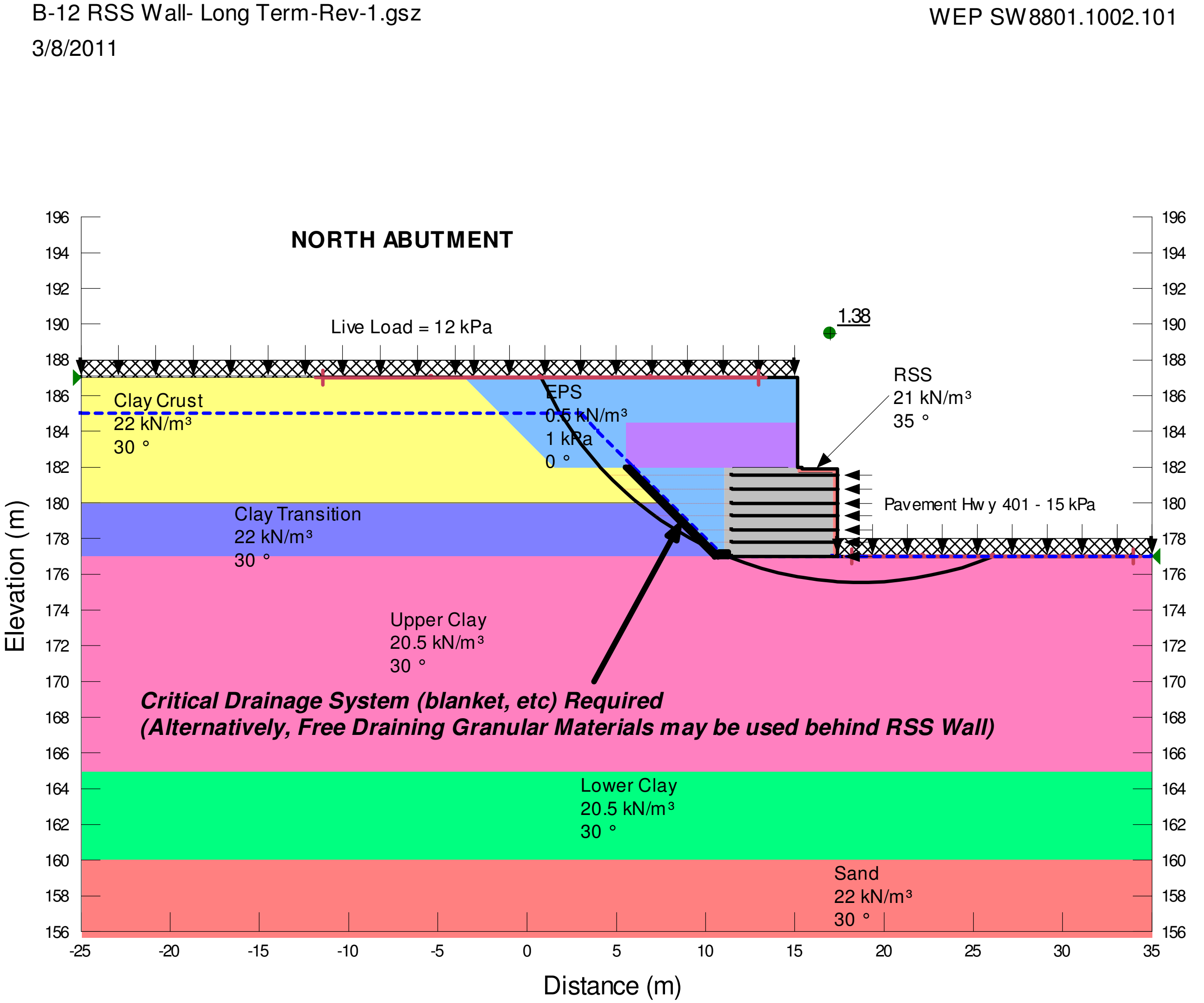


Figure 8: Global Stability (Long-term Loading) of North Abutment RSS Wall

WEP
Bridge B-12 Stress-Deformation Analysis
Date of Analysis: March 12, 2011

Name: Clay Crust Young's Modulus (E): 36000 kPa Poisson's Ratio: 0.49 Cohesion: 75 kPa Phi: 0 ° Unit Weight: 22 kN/m³
Name: Clay Transition Young's Modulus (E): 24000 kPa Poisson's Ratio: 0.49 Cohesion: 60 kPa Phi: 0 ° Unit Weight: 22 kN/m³
Name: Lower Clay Young's Modulus (E): 24000 kPa Poisson's Ratio: 0.49 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 20.5 kN/m³
Name: Deep Clay Young's Modulus (E): 26500 kPa Poisson's Ratio: 0.49 Cohesion: 70 kPa Phi: 0 ° Unit Weight: 20.5 kN/m³
Name: Sand Young's Modulus (E): 60000 kPa Poisson's Ratio: 0.35 Cohesion: 0 kPa Phi: 30 ° Unit Weight: 22 kN/m³
Name: Concrete Young's Modulus (E): 23000000 kPa Unit Weight: 24 kN/m³ Poisson's Ratio: 0.2
Name: RSS Backfill Young's Modulus (E): 60000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.35
Name: Clay Backfill Young's Modulus (E): 25000 kPa Poisson's Ratio: 0.49 Cohesion: 50 kPa Phi: 0 ° Unit Weight: 21 kN/m³
Name: Pavement Young's Modulus (E): 20000000 kPa Unit Weight: 23 kN/m³ Poisson's Ratio: 0.2
Name: Infinite Material (2) Young's Modulus (E): 5000 kPa Unit Weight: 21 kN/m³ Poisson's Ratio: 0.49
Name: Interface-Backfill Unit Weight: 20 kN/m³ Poisson's Ratio: 0.49

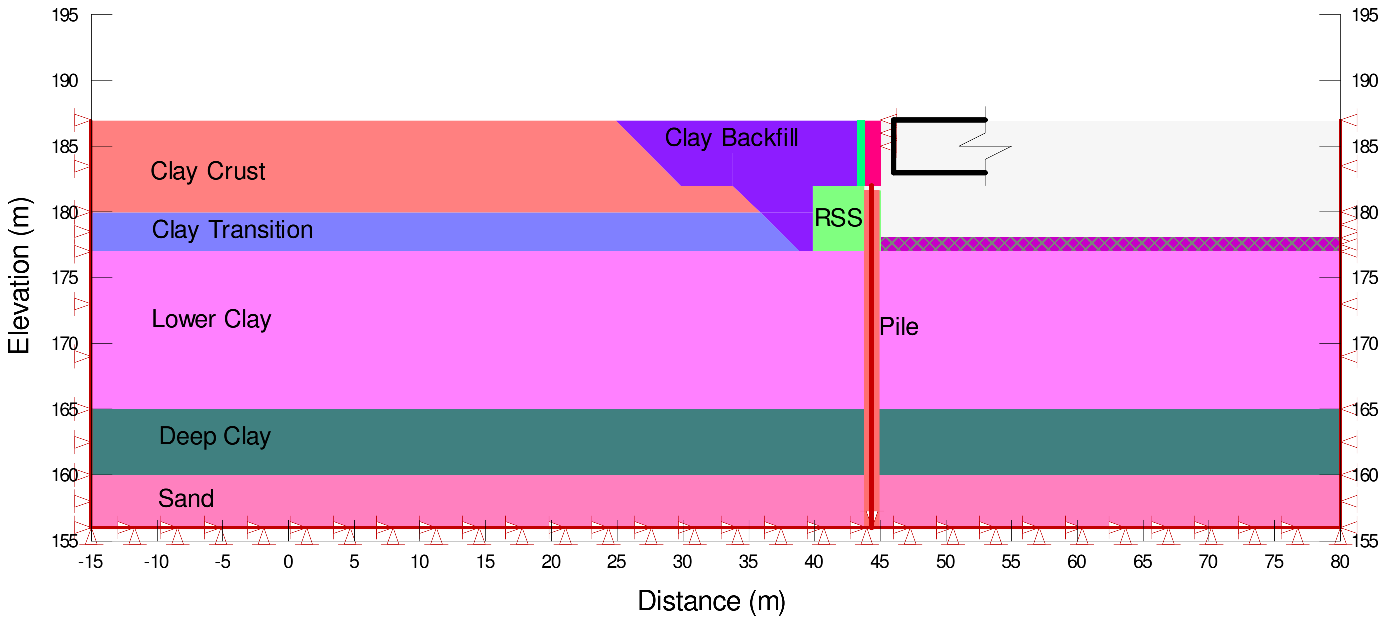
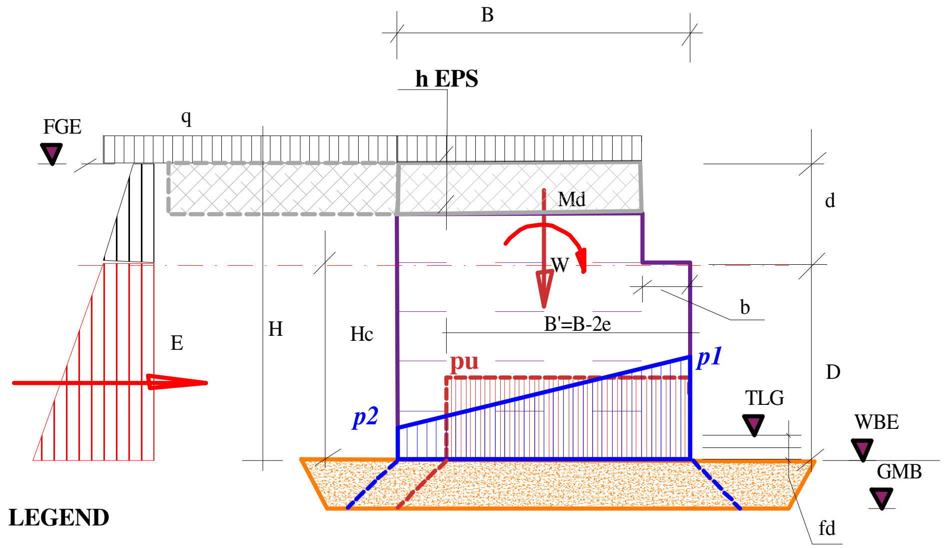


Figure 9: Stress-Deformation Analysis Model of Structure–Soil Configuration at Bridge B-12



LEGEND

TLG = Top Lower Grade
FGE = Finished Grade Elevation
WBE = Wall Base Elevation
GMB = Granular Mat Base
h EPS = EPS thickness

E = Unfactored max. Latedral Wall Load
Hc = Actual RSS Wall Height
H = Total wall height
W = Wall Weight (kN)

p1, p2 = Bearing Pressures at SLS
pu = Bearing pressures at ULS
e = Load eccentricity
Md = Driving Moment
fd = Foundation Depth

Figure 10: Schematic Arrangement of RGM with RSS Gravity Wall

APPENDIX A – EARLIER BOREHOLE LOGS IN VICINITY OF BRIDGE B-12

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Golder Associates
London, Ontario

RECORD OF BOREHOLE No 104

 METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4677630.3 ;E 335263.1		ORIGINATED BY MA
DIST WEST HWY 401/3	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY BRS
DATUM GEODETIC	DATE April 1, 2008 - April 2, 2008		CHECKED BY SJB

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
186.15	GROUND SURFACE
0.00	SILTY SAND, some clay, trace gravel, Loose Mottled brown and grey
184.78 / 1.37	CLAYEY SILT, some sand, trace gravel Very stiff Brown becoming grey at about elev. 183.1m
182.49 / 3.66	CLAYEY SILT, some sand, trace gravel, with sandy silt layers Very stiff Grey
181.73 / 4.42	CLAYEY SILT, some sand, trace gravel Very stiff Grey
178.85 / 7.35	SAND AND GRAVEL, some silt, some clay Grey
178.17 / 7.98	CLAYEY SILT, some sand, trace gravel Grey
8.23	SANDY SILT, trace gravel Grey
177.16 / 8.99	SILTY SAND, trace gravel, trace clay, with clayey silt layers Loose to compact Grey
	CLAYEY SILT, some sand, trace gravel, with silt and sand partings Firm Grey
176.09 / 10.06	CLAYEY SILT, some sand, trace gravel Stiff Grey
171.21	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	
2	SS	23	
3	SS	28	
4	SS	22	
5	SS	16	
6	SS	14	
7	SS	12	
8	SS	7	
9	TO	PH	
10	SS	10	4 46 37 13
11	SS	8	
12	SS	6	2 27 43 28
13	TO	PH	
14	SS	7	

GROUND WATER CONDITIONS: Concrete, Bentonite, Grout

ELEVATION SCALE: 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172

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

Field vane sensitivity values plotted: 1.5, 1.6, 1.3, 1.8, 1.6

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

UNIT WEIGHT γ kN/m³

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

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

Parkway Infrastructure Engineers | amec | Hatch Mott MacDonald

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 104

METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4677630.3 ;E 335263.1		ORIGINATED BY MA
DIST WEST HWY 401/3	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY BRS
DATUM GEODETIC	DATE April 1, 2008 - April 2, 2008		CHECKED BY SJB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE, sensitivity)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14.94	CLAYEY SILT, some sand, trace gravel, with sand partings Stiff Grey	15	TO	PH	171			
					170	+ 1.7		
		16	SS	6	169		w_p–w–w_L	1 23 41 35
168.62 17.53	SILTY CLAY, trace sand, trace gravel Stiff Grey					+ 1.5		
		17	SS	4	168		○	
166.95 19.20	CLAYEY SILT, trace sand, trace gravel, with silt partings Stiff Grey				167	+ 2.3		
		18	TO	PH	166			
165.42 20.73	CLAYEY SILT, trace sand, trace gravel Stiff to hard Grey					+ 1.8		
		19	SS	5	165		○	
					164	+ 1.2		
		20	SS	31	163		○	
162.53 23.62	SAND AND GRAVEL, trace silt Very dense Grey				162			
		21	SS	68			○	8 74 (18)
160.85 25.30	SANDY SILT Very dense Grey				161			
		22	SS	71	160		○ ○	(66)
159.85 26.30	CLAYEY SILT, trace sand, trace gravel Hard Grey				159			
		23	SS	39			○ ○	(92)
158.35 27.80	SILT, trace sand Dense Grey				158			
157.65 28.50	CLAYEY SILT, trace sand, trace gravel, with sandy silt partings Stiff to very stiff Grey	24	SS	15	157		○	

Grout

Continued Next Page

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

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

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RECORD OF BOREHOLE No 104

 METRIC

PROJECT 07-1130-207-0
W.P.
LOCATION N 4677630.3 ;E 335263.1
ORIGINATED BY MA
DIST WEST HWY 401/3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY BRS
DATUM GEODETIC
DATE April 1, 2008 - April 2, 2008
CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
					Grout	156					
155.70 30.45	LIMESTONE, fresh, medium strong, thinly laminated, fine grained, faintly porous Light grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	25	SS	100/ 76mm	Bentonite					o	
		26	HQ RC			155	88	82	82		
		27	HQ RC		Sand	154	95	81	70		UC
						153					
		28	HQ RC		Screen	152	100	100	100		
151.45 34.70	END OF BOREHOLE										

Water level in borehole at about elev. 162.4m during drilling on April 1 and 2, 2008.

Water level measured in deep piezometer at elev. 177.92m on April 4, 2008.

Water level measured in deep piezometer at elev. 176.09m on September 19, 2008.

Water level measured in deep piezometer at elev. 177.25 on November 14, 2008.

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

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

RECORD OF DRILLHOLE: 104

PROJECT: 07-1130-207-0
LOCATION: N 4677630.3 ;E 335263.1
INCLINATION: -90°   AZIMUTH: —

DRILLING DATE: April 1, 2008 - April 2, 2008
DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC
DRILLING CONTRACTOR: AARDVARK DRILLING INC

DATUM: GEODETIC

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	ELEVATION	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE	155.70 / 30.45				
31	MUD ROTARY / NO ROCK CORE	LIMESTONE, fresh, medium strong, thinly laminated to laminated, fine grained, faintly porous, light brown to tan		1	155		
32				2	154		
		LIMESTONE, fresh, medium strong, laminated to bedded, very fine grained to fine grained, moderately porous with occasional pits, light grey and grey	153.51 / 32.64				
33		LIMESTONE, fresh, medium strong, thinly laminated, fine to medium grained, moderately porous, grey	153.11 / 33.04		153	JN,IR,Ro CI	
34		LIMESTONE, fresh, medium strong, thinly laminated, stylolitic, fine grained, faintly porous, grey	152.32 / 33.83	3	152	JN,UN,Ro CI	
		END OF DRILLHOLE	151.45 / 34.70				

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

Depth scale: 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: SG
CHECKED: SXB

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 104A

 METRIC

PROJECT 07-1130-207-0
W.P.
DIST WEST HWY 401/3
DATUM GEODETIC
LOCATION N 4677630.3 ;E 335263.1
BOREHOLE TYPE POWER AUGER, SOLID STEM
DATE April 1, 2008
ORIGINATED BY MA
COMPILED BY BRS
CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE: DESCRIPTION (SOIL CONDITIONS INFERRED FROM BOREHOLE No. 104)	GROUND WATER CONDITIONS
186.15	GROUND SURFACE	
0.00	SILTY SAND, some clay, trace gravel, Loose Mottled brown and grey	Concrete
184.78 / 1.37	CLAYEY SILT, some sand, trace gravel Very stiff Brown becoming grey at about elev. 183.1m	
182.49 / 3.66	CLAYEY SILT, some sand, trace gravel, with sandy silt layers Very stiff Grey	Bentonite
181.73 / 4.42	CLAYEY SILT, some sand, trace gravel Very stiff Grey	
178.85 / 7.35	SAND AND GRAVEL, some silt, some clay Grey	
178.17 / 7.98	CLAYEY SILT, some sand, trace gravel Grey	Sand
8.23	SANDY SILT, trace gravel Grey	
177.16 / 8.99	SILTY SAND, trace gravel, trace clay, with clayey silt layers Loose to compact Grey	Piezometer
176.09 / 10.06	CLAYEY SILT, some sand, trace gravel, with silt and sand partings Firm Grey	
	END OF BOREHOLE	

Water level measured in shallow piezometer at elev. 183.01m on April 4, 2008.

Water level measured in shallow piezometer at elev. 183.76m on September 19, 2008.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (185, 184, 183, 182, 181, 180, 179, 178, 177); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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London, Ontario

RECORD OF BOREHOLE No 105

METRIC

PROJECT 07-1130-207-0
W.P.
LOCATION N 4677843.2 ;E 335190.1
ORIGINATED BY SM
DIST WEST HWY 401/3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY BRS
DATUM GEODETIC
DATE February 26, 2008 - February 28, 2008
CHECKED BY SJB

ELEV DEPTH	SOIL PROFILE – DESCRIPTION
186.16	GROUND SURFACE
0.00	TOPSOIL, clayey Brown
0.30	SAND AND GRAVEL Brown
0.46	CLAYEY SILT Brown
0.76	CLAYEY SILT, with black silty topsoil Firm to very stiff Mottled brown and grey
184.03 2.13	CLAYEY SILT, some sand, trace gravel, with sand partings Stiff to hard Mottled brown and grey becoming grey at about elev. 182.5m
177.78 8.38	SANDY SILT, some clay, trace gravel Loose Grey
176.71 9.45	CLAYEY SILT, some sand, trace gravel Firm to stiff Grey

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
1	SS	15	
2	SS	7	
3	SS	24	
4	SS	34	
5	SS	17	
6	SS	13	
7	SS	13	
8	TO	PH	CIUC
9	SS	7	CIUE
10	SS	9	
11	TO	PH	CIUC
12	SS	8	
13	TO	PH	

GROUND WATER CONDITIONS: Bentonite; Grout

ELEVATION SCALE: 186, 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172

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

Field vane sensitivity values: 1.7, 1.8, 1.5, 1.5, 2.0, 1.5

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

UNIT WEIGHT γ kN/m³

GR SA SI CL

Continued Next Page

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

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RECORD OF BOREHOLE No 105

METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4677843.2 ;E 335190.1		ORIGINATED BY SM
DIST WEST HWY 401/3	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY BRS
DATUM GEODETIC	DATE February 26, 2008 - February 28, 2008		CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, some sand, trace gravel Firm to stiff Grey	14	SS	6	171	
					170	
		15	TO	PH	169	CIUC
					168	
		16	TO	PH	167	
		17	TO	PH	166	
165.59 20.57	SILTY CLAY, some sand, trace gravel Very stiff Grey	18	TO	PH	165	
					164	
		19	TO	PH	163	
					162	
		20	TO	PH		CIUC
161.01 25.15	SILTY FINE SAND, trace clay Grey				161	
		21	TO	PH	160	
159.49 26.67	CLAYEY SILT, some sand Very stiff Grey				159	
		22	SS	25		(69)
157.97 28.19	SILTY CLAY, some sand, trace gravel Hard Grey				158	
156.90 29.26	SILTY SAND AND GRAVEL, trace clay Dense Grey	23	SS	37	157	

Ground water conditions (piezometer): Grout; Bentonite; Sand; Screen

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

Field vane results (sensitivity): + 1.6; + 1.9; + 1.2; + 1.4; 125.7 + 1.3; (>143.6) +

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

LDN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

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RECORD OF BOREHOLE No 105

METRIC

PROJECT 07-1130-207-0
W.P.
LOCATION N 4677843.2 ;E 335190.1
ORIGINATED BY SM
DIST WEST HWY 401/3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY BRS
DATUM GEODETIC
DATE February 26, 2008 - February 28, 2008
CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
						Screen, Bentonite	156						
155.68 / 30.48	LIMESTONE, fresh, medium strong, thinly laminated to laminated, very fine to fine grained, faintly to strongly porous. Light grey to tan. (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		24	SS	106						o		
			25	NQ RC			155	43	18	0			
			26	NQ RC		Sand	154	92	88	67			UC
			27	NQ RC			153, 152	98	88	56			
151.54 / 34.62	END OF BOREHOLE												

END OF BOREHOLE

Borehole dry during drilling on February 27, 2008

Water level measured in deep piezometer at elev. 178.26m on March 20, 2008

Water level measured in deep piezometer at elev. 177.93m on July 22, 2008

Water level measured in deep piezometer at elev. 175.77m on August 11, 2008

Water level measured in deep piezometer at elev. 176.84m on September 19, 2008

Water level measured in deep piezometer at elev. 177.35m on November 14, 2008

Water level measured in deep piezometer at elev. 177.94m 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; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

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

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PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 105

SHEET 4 OF 4

LOCATION: N 4677843.2 ;E 335190.1

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: February 26, 2008 - February 28, 2008

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DATUM: GEODETIC

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	ELEVATION	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.3 (5 10 15 20)	DISCONTINUITY DATA DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE		155.68 30.48													
31	MUD ROTARY NQ ROCK CORE	LIMESTONE, fresh, medium strong, thinly laminated, fine to very fine grained, faintly porous, light grey to tan		154.59 31.57	1			155									
32		LIMESTONE, fresh, medium strong, laminated, very fine grained, strongly porous to pitted, whitish grey		153.06 33.10	2			154						JN,CU,SM Cl JN,PL,SM Cl JN,PL,SM Cl JN,PL,SM Cl JU,UN,SM Cl			
33–34		LIMESTONE, fresh, medium strong, laminated, fine grained, porous with localized pitting, light grey, occasional fossils		151.54 34.62	3			153 152						JN,PL,Ro Cl			
35		END OF DRILLHOLE															

36 37 38 39 40 41 42 43 44 45

LDN_ROCK_03 07-1130-207-0-ROCK.GPJ GLDR_LDN.GDT 6/29/09 DATA INPUT: WDF

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: SG

CHECKED: SJB

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 105A

METRIC

PROJECT 07-1130-207-0			
W.P.	LOCATION N 4677843.2 ;E 335190.1	ORIGINATED BY SM
DIST WEST HWY 401/3	BOREHOLE TYPE POWER AUGER, HOLLOW STEM	COMPILED BY BRS
DATUM GEODETIC	DATE February 28, 2008	CHECKED BY SJB

SOIL CONDITIONS INFERRED FROM BOREHOLE No. 105

ELEV DEPTH	DESCRIPTION	GROUND WATER CONDITIONS
186.16	GROUND SURFACE	
0.00	TOPSOIL, clayey Brown	Bentonite
0.30	SAND AND GRAVEL Brown	
0.46	CLAYEY SILT Brown	
0.76	CLAYEY SILT, with black silty topsoil Firm to very stiff Mottled brown and grey	
184.03 2.13	CLAYEY SILT, some sand, trace gravel, with sand partings Stiff to hard Mottled brown and grey becoming grey at about elev. 182.5m	Sand Bentonite
177.78 8.38	SANDY SILT, some clay, trace gravel Loose Grey	Sand Piezometer
177.02 9.14	END OF BOREHOLE	

Elevation scale: 186, 185, 184, 183, 182, 181, 180, 179, 178

Water level measured in shallow piezometer at elev. 184.72m on March 20, 2008.

Water level measured in shallow piezometer at elev. 184.36m on July 22, 2008.

Water level measured in shallow piezometer at elev. 184.12m on August 11, 2008.

Water level measured in shallow piezometer at elev. 184.05m on September 19, 2008.

Water level measured in shallow piezometer at elev. 183.69m on January 28, 2009.

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

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

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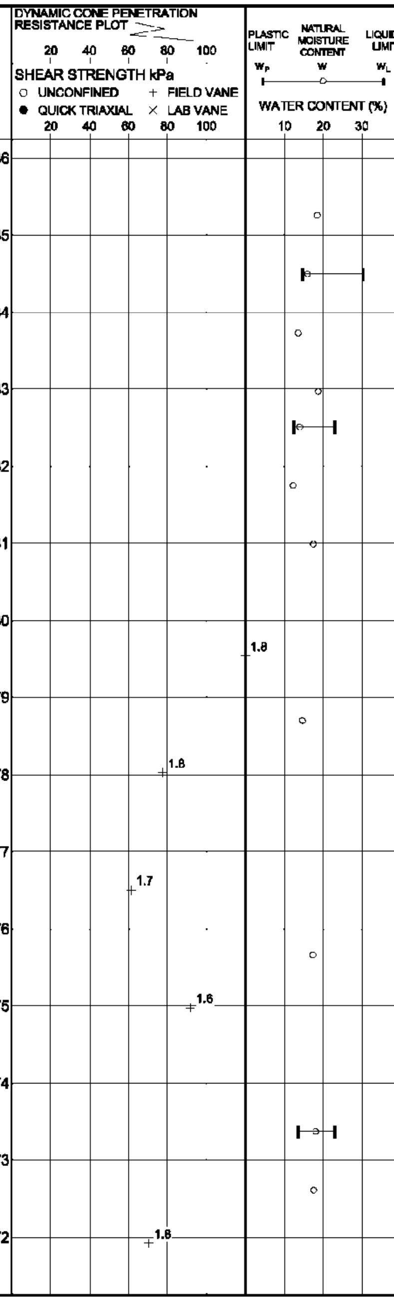
RECORD OF BOREHOLE No 301

METRIC

PROJECT 09-1132-0080
W.P.
LOCATION N 4677712.2 ;E 335231.1
ORIGINATED BY MR
DIST WEST HWY 401 / 3
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
COMPILED BY LMK/DMB
DATUM GEODETIC
DATE December 2, 2009 - December 3, 2009
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
186.25	GROUND SURFACE					
0.00	TOPSOIL, clayey Dark brown				186	
0.24	CLAYEY SILT, some sand, trace gravel Stiff to very stiff Brown becoming grey below about elev. 181.4m	1	SS	11		
		2	SS	9	185	
		3	SS	24	184	
		4	SS	27	183	
		5	SS	20		0 27 46 27
		6	SS	14	182	
		7	SS	13	181	
		8	TO	PH	180	
		9	SS	8	179	6 33 40 21
		10	TO	PH	178, 177	
		11	SS	5	176	
174.67 11.58	SILT, some clay, trace sand, trace gravel Grey	12	TO	PH	175	
173.75 12.50	CLAYEY SILT, some sand, trace gravel Stiff Grey	13	SS	6	174	2 31 43 24
		14	SS	8	173, 172	

Ground water conditions: Grout



UNIT WEIGHT γ kN/m³

Continued Next Page

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

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RECORD OF BOREHOLE No 301

METRIC

PROJECT 09-1132-0080			
W.P.	LOCATION N 4677712.2 ;E 335231.1		ORIGINATED BY MR
DIST WEST HWY 401 / 3	BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC		COMPILED BY LMK/DMB
DATUM GEODETIC	DATE December 2, 2009 - December 3, 2009		CHECKED BY

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (field vane, sensitivity)	REMARKS GR SA SI CL
	CLAYEY SILT, some sand, trace gravel Stiff Grey	15	TO	PH	171	+ 1.8	
		16	SS	7	170	+ 1.9	
		17	TO	PH	168	+ 1.9	
167.04 19.21	SILTY CLAY, trace sand Firm Grey	18	TO	PH	167		
		19	TO	PH	166		
165.37 20.88	CLAYEY SILT, some sand, trace gravel Stiff Grey	20	SS	8	165	+ 1.9	1 15 53 31
		21	TO	PH	163	(>95.7)+	
		22	SS	28	162		1 24 44 31
		23	SS	18	160		
159.73 26.52	SAND, fine, trace silt Very dense Grey	24	SS	72	159		
157.29 28.96	CLAYEY SILT, some sand, trace gravel Hard Grey	25	SS	66	157		

Ground water conditions: Grout; Bentonite

Elevation scale: 171 170 169 168 167 166 165 164 163 162 161 160 159 158 157

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 301

METRIC

PROJECT	09-1132-0080
W.P.	
DIST	WEST HWY 401 / 3
DATUM	GEODETIC
LOCATION	N 4677712.2 ;E 335231.1
BOREHOLE TYPE	POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC
DATE	December 2, 2009 - December 3, 2009
ORIGINATED BY	MR
COMPILED BY	LMK/DMB
CHECKED BY	

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)
		26	SS	100/125mm		156			
155.63 / 30.42	LIMESTONE, fresh, medium strong, laminated, very fine to fine grained, faintly porous. Light grey to brown. (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)	27	NQ RC	-			71	0	0
		26	NQ RC	-		155	98	48	45
		29	NQ RC	-	Sand	154	77	60	50
		30	NQ RC	-		153 / 152	100	82	65
		31	NQ RC	-	Piezometer	151	100	73	67
150.28 / 35.97	END OF BOREHOLE								

Groundwater encountered at about elev. 159.7m during drilling on December 2 and 3, 2009.

Water level measured at elev. 178.15m on February 24, 2010.

Water level measured at elev. 177.92m on January 6, 2010.

Borehole sealed with cement-bentonite grout.

Column headings: SOIL PROFILE; SAMPLES; 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

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