



October 5, 2011

**REVISED  
FOUNDATION INVESTIGATION AND DESIGN REPORT**

**WHITE CLAY RIVER BRIDGE REPLACEMENT  
HIGHWAY 11, SITE NO. 47-005  
TOWNSHIP OF MAISONVILLE, ONTARIO  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5239-06-00**

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REPORT

**GEOCRES No. 42A-77**

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## **Executive Summary**

The final Foundation Investigation and Design Report (FIDR) prepared by Golder Associates Ltd. (Golder) for the White Clay River Bridge replacement was issued on September 11, 2009. The review comments of the Contract Documents were documented in January 2010. Review comments of the final FIDR by MTO Foundations in October 2009 prompted an internal (Golder) review of the Approach Embankment Design and Construction (Section 6.7) of the FIDR which, in turn, resulted in updates to the Contract Documents (drawings and specifications) in May 2010. The design changes were summarized in Addendum No. 1 to the final FIDR, dated July 15, 2010.

Prior to the submission of the design package to Contracts and Tenders Section (CTS) of MTO in January 2011, an additional review of the Contract Documents was carried out and documented by Golder. Immediately prior to the contract being tendered in late March 2011, MTO Foundations noted that not all of their review comments on the Approach Embankment Design and Construction (Section 6.7) had been satisfactorily addressed, including ensuring that the foundation design was adequately reflected in the design drawings. As a result, a second internal review of Section 6.7 of the report was carried out by Golder and additional revisions were recommended and subsequently incorporated into the Contract Documents in late April 2011.

This Revised FIDR documents the revisions made to the Approach Embankment Design and Construction (Section 6.7) of the report which were ultimately reflected in the final Contract Documents. In addition, after issuance of the final FIDR, rock fill was identified as an available material for construction of the approach embankments, in lieu of earth fill, and Section 6.7 now includes an assessment of the estimate settlement of rock fill. Minor revisions were also made to Section 6.8 related to sub-excavation and embankment construction over swamps and compressible soils. There are no other revisions to the remaining sections of the original final FIDR.



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NSSP	Unwatering
OC	Obstructions - Timber Piles
OC	Obstructions - Boulders
NSSP	Vibration Monitoring



# PART A

FOUNDATION INVESTIGATION REPORT  
WHITE CLAY RIVER BRIDGE REPLACEMENT  
HIGHWAY 11, SITE NO. 47-005  
TOWNSHIP OF MAISONVILLE, ONTARIO  
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GWP 5239-06-00



## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the replacement of the structure carrying Highway 11 over White Clay River in the Township of Maisonville and Sesekinika (northwest of Kirkland Lake), Ontario.

The terms of reference for the scope of work for the foundation investigation are outlined in MTO's Request for Proposal dated March 12, 2007 for foundation engineering services associated with the White Clay River Crossing and as outlined in Section 6.8 of LEA's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for foundation engineering services for this project, dated September 18, 2007. The General Arrangement drawing for the bridge structure, dated May 2008, was provided to Golder by LEA in September 2008.

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement structure and the approach embankments by borehole drilling, rock coring, in situ testing and laboratory testing on selected samples. The location of the investigated area is shown in plan on Drawings 1 and 2.

## **2.0 SITE DESCRIPTION**

The site is situated in the Township of Maisonville on Highway 11 crossing the White Clay River, approximately 18 km north of the junction with Highway 66. The existing road grade is about 2 m above the river water level. The surrounding land is mainly used for recreational activities, with grass and tree cover extending beyond the limits of the site. The banks adjacent to the river are vegetated with grass and small shrubs. A low-lying swampy area is present on the northwest side of the bridge, extending to Swan Lake Park Road. The river is about 40 m across at its narrowest at the existing crossing location where the north and south causeways have been extended some 40 m and 140 m into the river, respectively. On either side of the crossing, the river widens to about 250 m and then narrows toward the west. Approximately 1 km upstream of the river, a small lake is present. The river is mainly used for recreation and is an unregulated watercourse (i.e. it does not have a control dam).

The existing bridge was constructed in 1951 and has eight main spans with an overall deck length of 50.7 m. The current structure consists of a concrete deck with steel girders supported by concrete-timber hybrid pile caps on timber piles founded at an unknown depth. It is unknown if an earlier structure existed at the site prior to 1951 and details of any older foundations that may be present in the subsurface are not available. We understand that the existing bridge will be replaced with a new three-span structure approximately 54 m long.

The existing highway grade is at about Elevation 312.4 m and 312.3 m at the existing north and south bridge abutments with the grade increasing in elevation away from the bridge abutments. The water level in the river was measured at approximately Elevation 310.2 m in July 2008 and 310.1 m in October 2008.



### 3.0 INVESTIGATION PROCEDURES

The fieldwork at the bridge site was carried out in four stages, with a total of twenty-three (23) boreholes advanced at the site. The borehole locations and groundwater surface elevations are shown on Drawings 1 to 3 and noted on the respective Record of Borehole and Drillhole Sheets in Appendix A.

- Between July 7 and 23, 2008, thirteen (13) boreholes (WC-1 to WC-12 and WC-9a) were drilled for the proposed south and north abutments and approaches. Boreholes WC-1 and WC-11 were drilled on land using a CME 850 track-mounted drill rig supplied and operated by Landcore Drilling Inc. (Landcore) of Sudbury, Ontario. Boreholes WC-2 to WC-10 and WC-12 were drilled over water in the White Clay River using a D25 drill rig mounted on a modular raft supplied and operated by Walker Drilling Ltd. of Barrie, Ontario (Walker).
- On August 28, 2008, an additional one (1) borehole (WC-11a) was drilled on land near the north approach using portable equipment supplied by OGS Inc. of Ottawa, Ontario (OGS).
- Between October 25 and 28, 2008, a further six (6) boreholes (WC-5a, WC-11b, and WC-13 to WC 16) were drilled. Borehole WC-5a was drilled over water using raft-mounted portable equipment supplied and operated by OGS for the proposed south approach. Borehole WC-11b at the north approach and Boreholes WC-13 to WC-16 at the north embankment were drilled on land using portable equipment supplied and operated by OGS.
- On November 5 and 6, 2008, three (3) boreholes (WC-17 to WC-19) were drilled on land through the existing embankment using a CME55 truck-mounted drill supplied and operated by Walker.

The boreholes were advanced using either 108 mm inside diameter (I.D.) continuous flight hollow stem augers, NW casing and wash boring or portable equipment using BW casing and wash boring. Soil samples were obtained, where possible, continuously or at intervals of depth of about 0.75 m to 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99). Shelby tube samples were taken in cohesive deposits at some borehole locations. In each borehole where hollow stem augers or NW Casing was used to advance the hole, N-vane shear tests were conducted in cohesive soils to assess the undrained shear strengths (ASTM D2573-01); in Boreholes WC-5a, WC-11a, WC-11b, WC-13, WC-14, WC-15 and WC-16, B-vanes were used inside the BW Casing. Rock core samples were obtained using an NQ size core barrel.

Boreholes WC-5a, WC-11a and WC-11b were advanced immediately adjacent to Boreholes WC 5, WC-11 and WC-11, respectively, to carry out additional field vane testing in, and/or to obtain Shelby Tube samples of the cohesive soil deposits.

The land-based boreholes were advanced to depths ranging from about 6.5 m to 16.5 m below the existing ground surface and the water-based boreholes were advanced to depths ranging from about 5.5 m to 27.4 m below the water surface at the time of drilling (including rock coring when carried out). Most of the boreholes were advanced to auger, casing, DCPT or sampler refusal (i.e. on inferred bedrock). However, four of the boreholes (WC-5, WC-9a, WC-11 and WC-19) were terminated after penetrating the cohesive deposit and prior to reaching refusal.



A minimum of 3 m of rock core was obtained from four of the boreholes drilled at this site at the proposed pier and abutment foundation units, namely Boreholes WC-6 to WC-9.

The groundwater conditions in the open boreholes were observed during the drilling operations. The water level readings are presented on the Record of Borehole sheets in Appendix A. The boreholes were backfilled with bentonite as per Ontario Regulation 903 (as amended by O. Reg. 372) upon completion of drilling.

In Boreholes WC-9 and WC-13, artesian water conditions were encountered. A seal consisting of granular bentonite (i.e. holeplug) was placed from the bottom of the borehole up to about 3.0 m in Borehole WC-9 and 5.0 m in Borehole WC-13 below the ground surface. Above this seal, the holes were backfilled with cuttings to ground surface to complete the abandonment of the boreholes.

Sediment control procedures as detailed in our Environmental Protection Plan were carried out to minimize sediment entering the river and/or disturbance of the river bottom. The soil cuttings from the land-based boreholes were used for backfill and also distributed in the vicinity of the boreholes. The wash water from the water-based boreholes was allowed to return to the river in a controlled manner; the cuttings from the water-based boreholes were used as backfill and distributed along the river banks.

Traffic protection was carried out for the boreholes drilled within the roadway in accordance with our Traffic Control Plan and MTO Book 7 Temporary Conditions Manual.

The fieldwork was supervised throughout by members of our engineering and technical staff who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil and rock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. Select Shelby tube samples were sent to our Mississauga laboratory for additional laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. A total of five one-dimensional consolidation (oedometer) tests were carried out on Shelby tube samples of the cohesive soil deposit from the boreholes. Uniaxial compressive strength (UCS) testing was carried out on selected specimens of the bedrock core recovered from the boreholes.

The locations of the proposed foundation elements were laid out in the field by Golder relative to the proposed centreline alignment staked in the field by LEA's subconsultant SRQ Geomatics Inc. (SRQ), based on the dimensions shown on the General Arrangement dated May 2008. Golder surveyed the ground surface elevation of the land-based boreholes once completed as well as the water surface elevation for the water-based boreholes. All borehole elevations were then referenced to LEA's centreline alignment survey. The ground surface and water surface elevations are referenced to geodetic datum. The northings and eastings in MTM NAD 83 were determined by plotting the station and offset of the boreholes (relative to the stakes) on the May 2008 General Arrangement and converting to the coordinate system.



## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

Published literature indicates that the site is located in the Abitibi Subprovince of the Superior Province. The bedrock geology forms part of the Huronian Supergroup which consists of conglomerate, sandstone, siltstone and argillite (Geology of Ontario; OGS Special Volume 4)<sup>1</sup>. To the south of the site, the bedrock geology consists of granite-greenstone-gneiss terrane, generally with minor metasedimentary rock overlying the metavolcanic rock.

Based on terrain mapping by the Ontario Department of Lands and Forest<sup>2</sup>, the surficial soils in the vicinity of the site consist of lacustrine deposits comprising varved or massive clays, silts, fine sands and sands. The Ontario Geological Survey<sup>3</sup> (Map 5032) describes the subsurface soils in the vicinity of the site as out-washed plains comprised of gravels and sands, with peat swamps.

### 4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and rock samples, are presented on the attached Record of Borehole and Drillhole sheets in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes is shown on Drawings 1 to 3.

In general, the subsoils at the structure site consist of peat, underlain by deposits of clayey silt to silty clay, silt, silt and sand to sand and gravel and cobbles and boulders. Within the existing embankment, the subsoils are generally comprised of sand embankment fill underlain by clayey silt to silty clay, silt and sand and gravel. The total thickness of overburden (including existing road fill at boreholes WC-17 to WC-19) is variable at the site, ranging from about 5.9 m to 17.4 m. The bedrock surface was confirmed by coring at four locations at depths ranging from 14.3 m to 17.4 m.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

#### 4.2.1 Topsoil

Moist, brown to black sandy topsoil was encountered at ground surface in Boreholes WC-14 and WC-16 having a thickness of 0.3 m and 0.2 m, respectively. The ground surface at these boreholes is at Elevation 310.7 m and 311.1 m, respectively.

The natural water content measured on one sample of the topsoil is about 19 percent.

<sup>1</sup> Geology of Ontario, 1991. Ontario Geological Survey, special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

<sup>2</sup> Northern Ontario Engineering Geology Terrain Study, OGS Electronic Map Reference Number 5465.

<sup>3</sup> Northern Ontario Engineering Geology Terrain Study, OGS Electronic Map Reference Number 5032





#### **4.2.2 Pavement Structure**

In Boreholes WC-17 to WC-19 (drilled on the roadway surface), approximately 200 m to 240 m of asphalt was encountered overlying either 150 m of concrete (WC-17) or 250 m to 300 m of silty sand to sand and gravel road base fill. The ground surface at these boreholes is at Elevation 312.3 m.

#### **4.2.3 Fill**

Fill was encountered in Boreholes WC-1, WC-14, WC-16, WC-17, WC-18 and WC19. In Borehole WC-1 and Boreholes WC-16 through WC-19, the existing embankment fill consists of brown sand to sand and silt to sand and gravel containing trace to some silt, trace clay. Trace organics were noted in the fill in Borehole WC-1. In Borehole WC-14, the fill consists of a brown peat and grey silty clay. The fill was encountered at ground surface in Borehole WC-1, below the pavement structure in Boreholes WC-17 to WC-19 and below topsoil in Boreholes WC-14 and WC-16. The ground surface/top of fill ranges from about Elevation 310.4 m to 312.2 m and the fill ranges in thickness from about 0.6 m to 5.6 m. The bottom of the fill material was encountered up to 6.1 m below ground surface and the fill/native soil interface was encountered between Elevation 306.2 m and 310.1 m.

SPT 'N' values measured within the non-cohesive fill ranges from 4 to 51 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. In Borehole WC-14, one SPT 'N' value within the cohesive fill was measured at 4 blows per 0.3 m of penetration, indicating a firm consistency. Grain size distributions of several samples of the fill are shown on Figures B-1 and B-2 in Appendix B for sand fill and sand and silt fill, respectively.

The natural water content measured on samples of the fill ranges between about 13 percent and 23 percent.

#### **4.2.4 Peat**

A 0.7 m to 3.1 m thick layer of wet, brown to black, very soft to soft peat containing trace sand was encountered underlying the fill in Boreholes WC-14 and WC-16, at the river bed in Boreholes WC-2 to WC-10 and WC-12 and at ground surface in Boreholes WC-11, WC-13 and WC-15. The top of the peat was encountered at depths up to 2.4 m below the ground surface or water surface at between Elevation 311.1 m and 307.8 m.

SPT 'N' values measured within the peat range from 0 (weight of hammer or rods) to 5 blows per 0.3 m of penetration. The higher 'N' values were measured at the north side (Boreholes WC-11 to WC-16) and at the extreme south side (Borehole WC-2) where the peat is considered to have a very soft to firm consistency. Elsewhere, such as in the river, the peat has a very soft consistency.

The natural water content measured on samples of the peat ranged between about 35 percent and 510 percent.



#### 4.2.5 Clayey Silt to Silty Clay

Below the peat and/or fill in all boreholes excluding WC-1 and WC-2, a deposit of wet, grey, varved, clayey silt to silty clay was encountered. The surface of this deposit was encountered between Elevation 306.2 m and 310.2 m and ranged in thickness from 0.8 m to 9.1 m. Boreholes WC-5, WC-10 and WC-11 were terminated within the clayey silt to silty clay deposit and a dynamic cone was driven to refusal adjacent to Borehole WC-10.

SPT 'N' values measured within the clayey silt to silty clay range from 0 (i.e. weight of hammer or weight of rods) to 12 blows per 0.3 m of penetration suggesting a very soft to stiff consistency. In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 13 kPa to 81 kPa, but typically ranging from about 15 kPa to 30 kPa, indicating a predominantly soft to firm consistency. One exception is below the existing embankment where the shear strengths typically ranged from 35 kPa to 50 kPa, indicating a predominantly firm to stiff consistency.

Atterberg limits testing carried out on several samples of the clayey silt to silty clay deposit indicate liquid limits ranging from about 26 percent to 62 percent and plastic limits ranging from about 16 percent to 24 percent, yielding plasticity indices ranging from about 6 percent to 39 percent. The results of the Atterberg limits testing are shown on the plasticity charts on Figures B-3a to B-3c for the clayey silt, clayey silt to silty clay and silty clay for clarity. The results indicate that the stratum ranges from a clayey silt of low plasticity to a silty clay of medium plasticity. The highest results were from the clay varves as shown on Figure B-3d which indicate the clay varves are of high plasticity.

The wide range of plasticity is indicative of the varved nature of the deposit. Where possible, Atterberg limits tests were carried out on samples from the Shelby tubes separated into the clay varved fraction and the 'siltier' varved fraction. The test results confirm that the clay varves are classified as a silty clay to clay of medium to high plasticity and the 'siltier' varves are a silt to clayey silt of low plasticity. It was not possible to separate the varves in some samples as they were too thin and, in these cases, the combined test results were within the range noted above.

In Borehole WC-19, a 1.5 m thick layer of silt was encountered beneath the fill and overlying the clayey silt to silty clay deposit at Elevation 308.0 m. The Atterberg limits testing indicates that the sample tested was non-plastic.

Grain size distribution tests were carried out on several samples of the clayey silt to silty clay deposit and the results are shown on Figures B-4a and B-4b.

The natural water content measured on select samples of this deposit ranges between 26 percent and 76 percent.

Five laboratory consolidation (oedometer) tests were carried out on specimens of the clayey silt to silty clay obtained from Boreholes WC-4, WC-8, WC-13, WC-17 and WC-18 and the test results are shown on Figures B-5, B-6, B-7, B-8, and B-9, respectively. The preconsolidation pressures ( $\sigma_p'$ ) were estimated from the Void Ratio versus logarithmic Pressure plots using the Casagrande method as well as from the Total Work versus Pressure plots. The relevant consolidation test results are summarized below.



Borehole/ Sample Number	Elevation (m)	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	$e_o$	$C_r$	$C_c$	$c_v^*$ (cm <sup>2</sup> /s)
WC-4/6	304.4	25	75	50	3.0	1.43	0.08	0.48	0.0240
WC-8/7	301.5	40	75	35	1.9	1.45	0.06	0.46	0.0088
WC-13/8	303.9	32	70	38	2.2	2.04	0.08	0.63	0.0039
WC-17/7	305.9	87	110	23	1.3	0.75	0.02	0.15	0.0410
WC-18/8	303.0	108	80	-28	0.7	1.25	0.05	0.31	0.0037

\*For stress range of approximately  $80 \leq \sigma_v' \leq 150$  kPa

where:  $\sigma_{vo}'$  effective overburden pressure in kPa

$\sigma_p'$  preconsolidation pressure in kPa

OCR overconsolidation ratio

$e_o$  initial void ratio

$C_c$  compression index (based on void ratio)

$C_r$  recompression index (based on void ratio)

$c_v$  coefficient of consolidation in cm<sup>2</sup>/s in the normally consolidated range

#### 4.2.6 Silt

Beneath the clayey silt to silty clay stratum in Boreholes WC-4, WC-5a, WC-9, WC-11b, WC-13, WC 15, WC-16, WC18 and WC-19, a deposit of silt was encountered. The top of this deposit was encountered from about Elevation 306.5 m to 298.5 m and the thickness ranges from about 0.3 m to 5.2 m.

SPT 'N' values measured within this deposit range from 0 (i.e. weight of rods) to 56 blows per 0.3 m of penetration, indicating a very loose to very dense relative density, typically loose to compact.

The test results of grain size distributions performed on several samples of the silt deposit are shown on Figure B-10.

The natural water content measured on samples of the silt ranged from about 12 percent to 72 percent.

#### 4.2.7 Silt and Sand to Sand and Gravel

A deposit of grey silt and sand to sand and gravel was encountered underlying the peat/fill in Boreholes WC-1 and WC-2, below the clayey silt to silty clay in Boreholes WC-3, WC-14 and WC-17, below the silt in Boreholes WC-5a, WC-9, WC-13, WC15, WC-16, WC-18 and WC19 and below the cobbles and boulders in Borehole WC-6. The silt and sand to sand and gravel was noted to be grey to black in Boreholes WC-6 and WC-19. The deposit contained trace clay and/or trace to some silt and, in some boreholes, the presence of cobbles and boulders were inferred by grinding of augers or during the wash boring. The top of the deposit was encountered between Elevation 294.2 m and 310.0 m and the thickness ranges between 0.8 m and 5.6 m. Boreholes WC-14 and WC-19 were terminated within the silty sand to sand and gravel deposit.



SPT 'N' values measured within the deposit ranged from 8 to 74 blows per 0.3 m of penetration, indicating a loose to very dense relative density.

Grain size distributions on several samples of the silt and sand to silty sand and the sand, gravelly sand and sand and gravel are shown on Figures B-11a and B-11b in Appendix B, respectively.

The natural water content measured on samples of the silt and sand to sand and gravel range between 4 percent and 16 percent.

#### **4.2.8 Cobbles and Boulders**

In Boreholes WC-6 to WC-8, a 0.5 m to 2.6 m thick layer of cobbles and boulders was encountered generally above the bedrock surface. The surface of the cobbles and boulders ranges from Elevation 293.4 m to 297.2 m. In general, the cobbles and boulders were distinctly different from the bedrock below due to the rock fragment types and the presence of a sand and/or gravel matrix.

#### **4.2.9 Bedrock**

Bedrock was encountered and cored in Boreholes WC-6 to WC-9. The bedrock surface was inferred from auger, casing, dynamic cone or sampler refusal in all the remaining boreholes and dynamic cones except Boreholes WC-5, WC-9a, WC-11 and WC-19. The bedrock surface encountered in the boreholes ranged from Elevation 292.8 m to 305.9 m, and was encountered at depths ranging between 5.9 m and 17.4 m below ground or water surface, as presented in Table B-1 in Appendix B. In Boreholes WC-4, WC-11a and WC-12, the bedrock surface was inferred from resistance to dynamic cone penetration of greater than 50 blows per 0.3 m of penetration as well as adjacent boreholes.

Based on a review of the bedrock core samples, the bedrock at the site generally consists of grey, fine grained, moderately to slightly weathered siltstone, which is heavily jointed and fractured in the upper approximately 2.0 m to 3.5 m. In Boreholes WC-6 and WC-7, brownish red, fine to medium, fresh sandstone was encountered below the siltstone at Elevation 291.9 m and 289.5 m, respectively. In Borehole WC-7, a 0.8 m thick seam of sand and gravel was encountered below the siltstone cap rock at a depth of 17.9 m (Elevation 292.3 m).

The Rock Quality Designation (RQD) measured on the core samples ranged from 0 percent to 100 percent. This indicates rock mass of variable quality, ranging from very poor to excellent. The RQD values within the siltstone are generally between about 40 percent and 70 percent, indicating that the bedrock is of poor to fair quality. In the sandstone, RQD values of two rock core runs are 50 percent and 100 percent, indicating the bedrock is of fair to excellent quality. The Total Core Recovery (TCR) during bedrock coring was between 40 percent and 100 percent, generally increasing with depth.

Laboratory UCS testing was carried out on eight core samples of the bedrock from Boreholes WC-6 to WC-9. The UCS results ranged between 41 MPa and 256 MPa, indicating medium strong to extremely strong rock. Typically, the higher values are for the sandstone. The depths and corresponding elevations of the tested samples and results of the UCS testing are presented in Table B-2.



#### 4.2.10 Groundwater Conditions

The water levels were noted during and immediately after the drilling and coring operations in the boreholes. In general, the soil samples taken in the boreholes were noted to be moist to wet with free water evident within most of the non-cohesive materials. Due to the largely water-based drilling program, piezometers were not installed in the boreholes at this site.

In the boreholes advanced on land to the north of the river (i.e. Boreholes WC-11, WC-13 to WC-16), the water levels in the open boreholes were between 0 m and 0.5 m below the ground surface ranging between Elevation 310.2 m and 311.0 m, which is within 0.8 m of the river water level. In the boreholes drilled through the existing embankment (i.e. Boreholes WC-1, WC-17 to WC-19), the water level measured in the open borehole was between 2.1 m and 2.3 m below the ground surface ranging between Elevation 310.0 m and 310.2 m, or at approximately the river water level. The water levels measured in the open boreholes during and upon completion of drilling are summarized below:

Location	Borehole	Depth to Groundwater below Existing Ground Surface (m)	River/Groundwater Elevation (m)
Proposed South Approach	WC-1	2.1	310.1
	WC-2	0	310.2*
	WC-3	0	310.2*
	WC-4	0	310.2*
	WC-12	0	310.2*
	WC-5	0	310.2*
	WC-5a	0	310.1*
	WC-6	0	310.2*
River Channel	WC-7	0	310.2*
	WC-8	0	310.2*
Proposed North Approach	WC-9/9a	0	310.2*
	WC-10	0	310.2*
	WC-11/11a	0	310.2
	WC-11b	0	310.3
	WC-13	0	310.3
	WC-14	0.3	310.4
	WC-15	0.1	311.0
	WC-16	0.5	310.6
Existing South Approach	WC-17	2.3	310.0
	WC-18	2.2	310.1
Existing North Approach	WC-19	2.1	310.2

\* River water level



In Boreholes WC-9 and WC-13, artesian pressures were noted during drilling. In Borehole WC-9 (drilled from the water surface), water was observed to be flowing out of the casing, which had a stick-up of 0.6 m, while advancing the casing through the top of the silt deposit at a depth of 11.7 m (Elevation 298.5 m). After penetrating partially into the silt, the artesian conditions stopped. In Borehole WC-13 (drilled in the low-lying area adjacent to the north bank of the river), the water level was 0.3 m above the ground surface while removing the casing. These boreholes were sealed as discussed in Section 3.0.

The water level of White Clay River was measured at Elevation 310.2 m in July 2008 and at Elevation 310.1 m in October 2008. The high water level and normal water level elevations were not provided to us at the time of this report. The White Clay River is an unregulated watercourse. The 1 in 50 year storm water level is approximately Elevation 311.3 m. Groundwater and river water levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

## **5.0 CLOSURE**

The field personnel supervising the drilling program were Mr. Ed Savard and Mr. Evan Childerhose for the water and land-based boreholes advanced during the summer months, respectively. Mr. Indulis Dumpis and Mr. Tim Rancourt supervised the drilling program during the fall months. This report was prepared by Mr. Tim Rancourt, E.I.T., and the technical aspects were reviewed by Ms. Sarah E.M. Coyne, P.Eng., an Associate with Golder. A quality control review of the report was provided by Mr. Jorge Costa, P.Eng., Golder's Designated MTO Contact for this project.



## Report Signature Page

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# PART B

FOUNDATION DESIGN REPORT  
WHITE CLAY RIVER BRIDGE REPLACEMENT  
HIGHWAY 11, SITE NO. 47-005  
TOWNSHIP OF MAISONVILLE, ONTARIO  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5239-06-00



## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation design aspects of the proposed new Highway 11 bridge structure over the White Clay River and the approach embankments extending from Station 15+300 at the south limit to Station 15+730 at the north limit of the site. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the site.

The interpretation and recommendations presented are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations and embankments. As such, where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

### 6.1 General

The existing bridge carrying Highway 11 over White Clay River was constructed in 1951 and is a 51 m long, eight-span structure with a concrete deck supported on steel girders. The foundations consist of steel reinforced concrete abutments and concrete-timber hybrid pier caps founded on timber piles driven into the clay stratum to unknown depth. The superstructure was rehabilitated in 1986, however, the support substructure was not rehabilitated and the timber piles have deteriorated beyond the point of repair. As indicated in LEA's Preliminary Design Report (PDR) dated June 2008, at some locations the timber piles have displaced from the pier cap beams. Stop-gap measures, such as collars and wooden shims, have been used over the years as temporary repairs to maintain the bridge in service, but such measures are no longer adequate.

It is unknown if an earlier structure existed at the site prior to construction of the current bridge and details of any older foundations that may be existing in the subsurface are not available.

We understand that the proposed bridge will be a 54 m long, 13.5 m wide three-span structure with integral abutments and the proposed centreline will be located about 18 m west of the existing bridge centreline. The proposed abutments are to be located approximately adjacent to the existing abutments.

The existing causeway embankments extend about 140 m and 40 m into the river at the south and north approaches, respectively. The existing ground surface at the abutments is at about Elevation 312.3 m, about 2 m above the existing river water level. The proposed road grade will be at approximately Elevation 313.0 m and 312.7 m at the south and north abutments, respectively, which is between 2.4 m and 2.7 m above the existing river water level. This will result in the new embankments being constructed over native ground and adjacent to the existing causeway embankments with the east side slope abutting onto the existing embankments.

The boreholes drilled through the existing embankment (Boreholes WC17, WC18 and WC-19) encountered generally loose to dense sand and gravel fill to a depth of up to 6.1 m below ground surface underlying the roadway surface asphalt course. Elsewhere, very soft peat was encountered beneath the water surface or beneath a thin veneer of fill. The peat is up to 3.1 m thick and the base of the deposit is up to 4.0 m below the water surface. A deposit of varved, clayey silt to silty clay was encountered below the peat or embankment fill



and is up to 9.1 m thick in the river channel. Below the existing embankment the clayey silt to silty clay deposit typically has a firm to stiff consistency and outside the existing embankment footprint the deposit typically has a soft to firm consistency. Loose to very dense silt, silt and sand to sand and gravel deposits and/or a layer of cobbles and boulders underlie the cohesive deposits. Siltstone/sandstone bedrock of generally poor (highly fractured) to good quality and generally strong to very strong, was encountered between Elevation 295.9 m and 292.8 m, decreasing in elevation towards the north.

The White Clay River water level was measured in July 2008 at Elevation 310.2 m and in October 2008 at Elevation 310.1 m. The water levels in the open boreholes drilled on land were measured at or slightly above the river water level. Artesian pressures were noted when sampling through the silt deposit in Boreholes WC-9 at the north abutment and WC-13 on the embankment north of the north abutment. At these boreholes, the head of water from the artesian conditions rose to about 0.6 m and 0.3 m above the river water level (Elevation 310.8 m and 310.5 m), respectively.

The following sections provide recommendations on the foundation design aspects of the new structure including design and construction of the new embankments. The recommendations take into consideration the implications of the new approach embankments on the existing bridge and existing approach embankments during construction and after construction, after which the existing embankments are to be partially removed to satisfy fisheries requirements. At this site, the critical foundation issues are the presence of the peat and soft to firm clayey silt to silty clay deposit which will cause stability and settlement related concerns for embankment construction, as well as the presence of cobbles and boulders and highly fractured rock which may impede installation of deep foundations to the good quality bedrock.

## **6.2 Bridge Foundation Options**

Given the presence of the peat and soft to firm, varved, clayey silt to silty clay deposit within the overburden at the location of the proposed new bridge abutments, spread footings founded at shallow depth on the soils are not considered feasible, and are not recommended, to support the new bridge structure due to the low geotechnical axial resistance and expected settlement of these strata. Based on the composition and relative density/consistency of the subsurface soils and depth to bedrock encountered at the foundation elements, we recommend that the proposed bridge be supported on deep foundations, consisting of steel H-piles or caissons founded on or into the bedrock.

The details of the recommendations for these options are presented in the following sections. A summary of the advantages, disadvantages, relative costs and risks/consequences of the various foundation alternatives considered for this site is presented in Table 1. We recommend that the bridge foundations be supported on steel H-piles.

## **6.3 Steel H-Piles**

As discussed above, we recommend that the bridge foundations be supported on steel H-piles driven to the bedrock surface. The elevation of the bedrock surface, proposed pile tip elevation and associated length of pile are presented below.



Foundation Element	Representative Borehole	Bedrock Surface/Pile Tip Elevation (m)	Underside of Pile Cap Elevation (m)	Estimated Pile Length below Pile Cap (m)
South Abutment	WC-6	295.9	308.0	12.1
South Pier	WC-7	293.1	307.0	13.9
North Pier	WC-8	292.9	307.0	14.1
North Abutment	WC-9	292.8	308.0	15.2

A layer of cobbles and boulders up to 2.5 m thick is present above the bedrock surface. To enhance the pile penetration through this layer, flange reinforcement of the piles as per OPSD 3000.100 (Driving Shoe) is recommended.

For an integral abutment design, a Non-Standard Special Provision (NSSP) for the installation of CSPs along the upper 3 m of pile will be required; a sample is given in Appendix C.

Piles should be driven from within a cofferdam to facilitate construction of the pile cap (see Section 6.9.2).

### 6.3.1 Geotechnical Axial Resistance

For HP310X110 piles driven to the bedrock surface, a factored geotechnical axial resistance of 2,000 kN at Ultimate Limit States (ULS) may be used for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. In this case, bedrock is an unyielding material and the condition for 25 mm of settlement at Serviceability Limit States (SLS) is typically much higher than the ULS value and therefore the ULS case governs.

### 6.3.2 Downdrag

At both abutments, the loading from the construction of the new approach embankments will cause consolidation settlement of the underlying soft clayey silt to silty clay strata since the deposit will likely not be sub-excavated as part of stability mitigation measures. Settlement of the cohesive deposit relative to the stiff piles will result in the development of downdrag loads (negative skin friction) on the piles if the piles are installed prior to completion of this settlement and if the piles are end-bearing on bedrock. Stability and settlement mitigation alternatives are discussed in Section 6.7.5.

The structural design of the abutment and pier piles should be based on the full downdrag load acting on the piles. The estimated unfactored downdrag load acting on the HP310X110 piles is 125 kN at the south abutment and south pier and 150 kN at the north abutment and north pier. These values may be used for the case where no CSPs are installed along the upper 3 m portion of the pile. Where CSPs are installed over the upper 3 m portion of the pile, which is the case for an integral abutment bridge, the downdrag load may be reduced to 75 kN for the south abutment piles and 100 kN for the north abutment piles.

The downdrag loads noted above are unfactored loads. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section C6.8.4 of the Commentary to the CHBDC for ULS conditions.



### 6.3.3 Pile Driving Note and Set Criteria

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations) and flange reinforcement should be in accordance with OPSD 3000.100 (Driving Shoe) to protect the pile tip from damage during driving through the bouldery deposits. For piles driven to bedrock, Note 5 in Clause 3.3.3 of the Structural Manual (MTO, 2008) should be used on the drawings:

- Piles to be driven to bedrock.

For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to also avoid overdriving and possibly damaging the piles.

Based on our experience, consideration should be given to the following preliminary criteria for piles driven to bedrock. The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. On reaching the required set, the hammer energy should be reduced by about 75 percent and the pile should then be re-driven by increasing the hammer energy slowly in stages up to the maximum rated energy over about 40 blows. This procedure is intended to improve the process of seating the pile on the bedrock surface. A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy.

### 6.3.4 Resistance to Lateral Loads

Lateral loads can be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The evaluation of the piles subjected to lateral loads should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.

The lateral load response of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction,  $k_h$ , (kPa/m) is determined in accordance with Section C6.8.7 in the Commentary to the CHBDC based on the equation for cohesionless soils given below (CFEM, 1992).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of horizontal subgrade reaction (kPa/m)} \\ z \text{ is the depth at any point along the pile (m)} \\ B \text{ is the pile diameter or width (m)} \end{array}$$

and for cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{where} \quad \begin{array}{l} s_u \text{ is the undrained shear strength of the soil (kPa)} \\ B \text{ is the pile diameter or width (m)} \end{array}$$



We understand that an integral abutment foundation design is being considered at this site. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-piles will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

The lateral resistance of the piles should be developed primarily from the passive resistance of the soil. The values of  $n_h$  and  $s_u$  to be used to calculate the coefficient of horizontal subgrade reaction ( $k_h$ ) to be utilized in the structural analysis for the piles at this location are given below:

Soil Unit	Foundation Element	Relevant Borehole	Elevation (m)	$n_h$ (kPa/m)	$s_u$ (kPa)
CSP Backfill (Loose Sand)	Where applicable	WC-6 to W-9	Upper 3 m of pile	4,400	--
Embankment Fill above Water/Groundwater Level (assumed to be compact granular fill)	All	WC-6 to WC-9	above 310.2	6,600	-
Embankment Fill below Water/Groundwater Level (assumed to be loose Granular 'B' Type II)	South abutment South Pier North Pier North Abutment	WC-6 WC-7 WC-8 WC-9	310.2 – 306.2 310.2 – 306.3 310.2 – 307.5 310.2 – 307.6	1,300	-
Soft to Firm Clayey Silt to Silty Clay	South abutment South Pier North Pier North Abutment	WC-6 WC-7 WC-8 WC-9	306.2 – 298.6 306.3 – 298.6 307.5 – 298.6 307.6 – 298.5	-	20 14 14 20
Very Loose to Very Dense Silt	South abutment South Pier North Pier North Abutment	WC-6 WC-7 WC-8 WC-9	298.6 – 297.2 298.6 – 295.7 298.6 – 293.4 298.5 – 294.2	4,400	-
Very Dense Sand to Sand and Gravel with Cobbles and Boulders	South abutment South Pier North Pier North Abutment	WC-6 WC-7 WC-8 WC-9	297.2 – 295.9 295.7 – 293.1 293.4 – 292.9 294.2 – 292.8	11,000	-

At the abutments, the lateral resistance of the piles will be developed from the passive resistance of the soil over the portion of the piles below the CSP liners. The values of  $n_h$  and  $s_u$  to be used to calculate coefficient of horizontal subgrade reaction ( $k_h$ ) to be assumed in the structural analysis for the piles at this location are given below. For a single HP310x110 pile surrounded by a 3 m long CSP liner below the pile cap and extended to the bedrock surface, the estimated maximum lateral resistance at ULS is about 75 kN and at SLS is about 15 kN for 10 mm of deflection. For a single HP310x110 pile without a 3 m long CSP below the pile cap, the estimated maximum lateral resistance at ULS is about 100 kN and at SLS is about 25 kN (for 10 mm of deflection).



Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) in the direction of loading by a reduction factor, R, as follows:

Pile Spacing in Direction of Loading $d$ = Pile Diameter	Subgrade Reaction Reduction Factor
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those listed above.

### 6.3.5 Frost Protection

At this site, the pile caps should be provided with a minimum of 2.4 m of conventional soil cover for frost protection (as per OPSD 3090.100 - Foundation Frost Penetration Depths for Northern Ontario). Alternatively, rigid polystyrene insulation could be used to reduce the required thickness of soil cover. As a guideline for design, it is generally adopted by the MTO that a thickness of 25 mm of rigid polystyrene insulation should be assumed to be equivalent to about 300 mm of conventional soil cover. The insulation, if used, should be placed vertically along the face of the foundation (to the base of the pile cap) and extend horizontally for a distance of 2.4 m beyond the face. A minimum of 1 m of soil cover should be placed over the rigid insulation.

## 6.4 Caissons

Alternatively, caissons could be used for the support of the bridge abutments and piers. Caissons should be socketted a minimum of 2 m into the good quality siltstone or sandstone, typically encountered about 2 m below the bedrock surface elevation; in the case of the south pier, the good quality bedrock was encountered almost 4 m below the surface of the bedrock. The elevation of the base of the caisson socket to be used in design is presented below.

Foundation Element	Representative Borehole	Bedrock Surface Elevation (m)	Elevation of Good Quality Siltstone/Sandstone (m)	Caisson Base Elevation (m)
South Abutment	WC-6	295.9	293.3	291.3
South Pier	WC-7	293.1	289.5	287.5
North Pier	WC-8	292.9	290.9	288.9
North Abutment	WC-9	292.8	290.6	288.6





Given the presence of cobbles and boulders, the fractured nature of the upper portion of the bedrock and the possible sloping bedrock surface, it will be difficult to create a socket and achieve an adequate seal. Temporary liners and tremie concrete will likely be required to install caissons at this site.

#### 6.4.1 Geotechnical Axial Resistance

Caissons at this site will derive their axial resistance mainly from the shaft resistance of the rock socket. The contribution from end-bearing should be neglected due to the difficulties in cleaning and inspecting the base of the sockets. The factored geotechnical axial resistance at ULS for two diameter caissons socketted a minimum of 2 m into the good quality siltstone or sandstone bedrock are given below:

Caisson Diameter (m)	Good Quality Sandstone/Siltstone Bedrock (minimum 2 m socket)	
	ULS (kN)	SLS for 25 mm
1.5	8,000	n/a
1.8	10,000	n/a

The resistance required to achieve 25 mm of settlement is greater than that given for ULS for caissons socketted into the bedrock and, therefore, SLS conditions do not apply.

#### 6.4.2 Downdrag

As discussed in Section 6.3.2, the loading from the new embankment will cause settlement of the underlying soft clayey strata which will result in the development of negative skin friction on the caissons that are socketted into bedrock. As a result, downdrag loads will need to be taken into account in the design of the caissons at the abutments and piers.

The structural design of the abutment and pier caissons should be based on the full downdrag load acting on the caissons. The estimated unfactored downdrag load acting on the caissons for this case, assuming the underside of the caisson cap is at about Elevation 308 m at the abutments, may be taken as 450 kN and 550 kN for 1.5 m and 1.8 m diameter caissons, respectively. These downdrag loads are unfactored loads. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section C 6.8.4 of the Commentary to the CHBDC for ULS conditions.

#### 6.4.3 Resistance to Lateral Loads

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.3.4, using the horizontal subgrade reaction formulas. The recommended maximum lateral resistances for the caissons are as follows:



Caisson Diameter (m)	Factored Lateral Resistance at ULS* (kN)	Lateral Resistance at SLS (kN)
1.5	2,000	500
1.8	2,800	700

\*Note: for the subsurface stratigraphy at this site, the factored lateral resistance at ULS for the caissons is controlled by the UCS of the bedrock socket.

#### 6.4.4 Frost Protection

The pile caps for the caissons at the abutments should be provided with a minimum of 2.4 m of conventional soil cover for protection from frost penetration or sufficient insulation as described in Section 6.3.5.

### 6.5 Seismic Considerations

#### 6.5.1 Site Coefficient

For seismic design purposes, the Site Coefficient,  $S$ , for this site, based on experience and considering the guidelines in Section 4.4.6 of the CHBDC may be taken as 1.5, consistent with Soil Profile Type III.

#### 6.5.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the CHBDC. According to Table A3.1.1 of the CHBDC, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for this area is 0.05. Based on experience, for the subsurface conditions at this site, a 50 percent amplification of the ground motion may occur (i.e. Site Coefficient,  $S=1.5$  for Soil Profile III from Table 4.4 of CHBDC), resulting in an increase in the Peak Horizontal Acceleration (PHA) from 0.05 g to 0.075 g at the ground surface.

We understand, based on Section 4.4.4 of the CHBDC, that this bridge structure is assigned Seismic Performance Zone 1. We further understand from LEA that this bridge is not considered a lifeline structure. Given this, and in accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Zone Performance 1.

### 6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 6.5.2, seismic (earthquake) loading need not be analyzed for this structure.



The following recommendations are made concerning the design of the abutment walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of SP 110S13 Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls Abutment, Backfill) and OPSD 3121.150 (Walls Retaining, Backfill).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive for backfill to structures adjacent to rock fill embankments, dated November 2002. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill) if granular fill is used, or OPSD 3101.200 (Walls Abutment, Backfill Rock) if rock fill is used.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- For restrained structures, the granular fill or rock fill should be placed in a zone with width equal to at least 2.4 m behind the back of the walls (in accordance with Figure C6.20(a) of the Commentary to the CHBDC).  
For unrestrained structures, granular fill or rock fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5H:1V extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b), Case II, of the Commentary to the CHBDC).
- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

	<b>Earth Fill</b>	<b>Rock Fill</b>
Soil unit weight:	21 kN/m <sup>3</sup>	19 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:		
Active, $K_a$	0.31	0.22
At rest, $K_o$	0.47	0.35

- For unrestrained structures, the pressures are based on the rock fill as above or on the granular fill as placed and the following parameters (unfactored) may be assumed:

	<b>Granular 'A'</b>	<b>Granular 'B'</b>
		<b>Type II</b>
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:		
Active, $K_a$	0.27	0.27
At rest, $K_o$	0.43	0.43



If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHDBC.

A restrained structure is typically a concrete box culvert or a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

## 6.7 Approach Embankment Design and Construction

The new highway alignment at the White Clay River crossing will be shifted 18 m west (centreline to centreline distance). In addition, the final grade of the new highway will be about Elevation 313.0 m and 312.7 m at the south and north abutments, respectively, which are 0.7 m and 0.4 m higher than the existing highway grade of about Elevation 312.3 at the existing abutments. The alignment is on a vertical curve with a final grade at about Elevation 313.7 m at STA 15+320, which is about 150 m south of the south abutment. The final grade is at about Elevation 312.2 m at STA 15+620, which is about 100 m north of the north abutment. The existing and new causeway embankments extend about 140 m and 40 m into the river at the south and north approaches, respectively. Given the swampy area over at the northwest side of the river, the north embankment will be approximately 200 m long.

The soils encountered below the proposed approach embankments consisted primarily of peat underlain by sequences of cohesive soil and cohesionless layers. At all areas, the stability and settlement analyses assume that prior to construction of the new embankments, all peat will be removed below the new embankment footprint and that existing fill will be left in place in the transition zone between the new and existing embankments.

Based on the proposed new alignment, new/widened causeways will be required on both the south and north sides of the river. The proposed embankment geometry will result in the addition of up to 4.2 m of fill above the river bed (i.e. above the surface of the peat deposit) at the shoulder of the new embankments. After the peat has been removed, the total thickness of embankment fill above the surface of the clayey silt to silty clay deposit will be up to 6.8 m at the south approach and 5.1 m at the north approach as presented below.

Location	Final Grade (m)	Relevant Borehole(s)	Embankment Height (above the top of peat) (m)	Peat Thickness (m)	Total Fill Thickness (after peat removed) (m)	Clay Thickness (below fill) (m)
STA 15+385 South Approach*	313.4	WC-3 WC-4	4.2	1.4	5.6 (2.4 sub-aqueous)	3.7
STA 15+407 South Approach*	313.3	WC-4	3.8	1.7	5.5 (2.4 sub-aqueous)	6.5
STA 15+435 South Approach*	313.2	WC-4 WC-5	3.8	2.3	6.1 (3.1 sub-aqueous)	7.0
STA 15+468 South Abutment**	313.0	WC-6	3.7	3.1	6.8 (4.0 sub-aqueous)	7.6



Location	Final Grade (m)	Relevant Borehole(s)	Embankment Height (above the top of peat) (m)	Peat Thickness (m)	Total Fill Thickness (after peat removed) (m)	Clay Thickness (below fill) (m)
STA 15+523 North Abutment**	312.7	WC-9	3.6	1.5	5.1 (2.6 sub-aqueous)	9.1
STA 15+573 North Approach*	312.5	WC-11, 11a, 11b	2.3	2.2	4.5 (2.3 sub-aqueous)	8.4
STA 15+600 North Approach*	312.4	WC-13	2.1	2.0	4.1 (2.0 sub-aqueous)	6.5

\* Cross-Section

\*\*Cross-Section and Front Slope

The piezometric conditions were assessed based on the water levels observed in the White Clay River during the foundation investigation at Elevation 310.2 m in July 2008 and at Elevation 310.1 m in October 2008. Based on correspondence with LEA, the lowest water level measured (by others) over the duration of the project to date is Elevation 310.1 m in January 2008. We understand that at this site, the "low water level" has not been defined and that the White Clay River in this area lies between two larger bodies of water (lakes) which contain no dams or monitoring stations. Since the stability analysis is sensitive to low river water levels, for design purposes in the analysis, a river/groundwater level at Elevation 310.1 m has been used.

The following sections present the results of stability and settlement analysis for the new approach embankments, for different fill types, including recommendations for stability and settlement mitigation measures, as required. At the time of preparation of the Contract Documents, MTO identified a source of rock fill for use in construction of the White Clay River approach embankments and the subsequent analyses and design considers rock fill as the preferred construction material for these embankments. The proximity of the existing bridge and approach embankments has been considered as the existing bridge will continue to be used for traffic flow during construction of the new bridge.

### 6.7.1 Embankment Fill Types

Different embankment fill materials (i.e. granular fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils/bedrock), construction cost and time, sub-aqueous placement/densification and ease of construction/availability. Earth (granular) fill was used for construction of the existing embankments.

The main advantage of using granular fill (i.e. sand and gravel, Granular 'B' Type II, etc.) for embankment construction is the ease of construction and the negligible amount of post-construction settlements that will occur within the fill embankment itself above the water level. However, this option will require a larger volume of fill and wider right-of-way because the side slopes (2H:1V) will be flatter than for rock fill slopes. For this project, acceptable granular fill is considered to be well graded, locally available and/or imported, granular material such as Granular 'B' Type II. Further, the use of Granular 'B' Type II for sub-aqueous filling is preferred (when rock fill is not available) since external compaction effort will not be possible below the water level. Granular 'B' Type II fill should be used within a 20 m to 25 m zone behind the abutments due to structure backfilling requirements.



The main advantage of constructing embankments using rock fill is the ability to achieve steeper side slopes (1.25H:1V) thereby reducing the overall quantity of material required for the project as well as reducing the width of the right-of-way required. We understand that MTO has recently identified a local source of rock fill and therefore rock fill may be used for embankment construction (outside the abutment zones) to reduce embankment fill quantities resulting from steeper side slopes. The disadvantage of using rock fill is that some post-construction settlement of the rock fill itself will occur, although mostly within about the first year post-construction. Settlement of the rock fill is discussed further in Section 6.7.3.3. Where rock fill is used to backfill sub-excavated areas under water, settlement of this fill will also occur.

## 6.7.2 Stability

Analyses were performed on the critical (i.e. highest fill and/or thickest clay) sections of the proposed new approach embankments to assess the stability of the approach embankments for the proposed heights and geometries. Critical sections (given in Section 6.7) include those through both the front slopes (into the river) and side slopes (abutment and other cross-sections) of the new approaches, with the abutment front slope geometry being the most critical case. The geometry of the existing approach embankments (including heights and side slope profiles) has been included in the analyses based on the information provided by LEA.

### 6.7.2.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2004 (Version 6.20), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum Factor of Safety of 1.3 is normally adopted for the design of embankment slopes under static conditions. This Factor of Safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum Factor of Safety was achieved for the design embankment height and geometries. In general, circular slip surfaces were analyzed in the design. Wedge analysis was also carried out due to the varved nature of the clay deposit.

### 6.7.2.2 Parameter Selection

For the cohesionless deposits and fill, effective stress parameters were employed in the analysis assuming drained conditions and the shear strength parameters were estimated from empirical correlations using the results of the in situ SPT. The correlations proposed by Peck et al. (1974), Schmertmann (1975) and NAVFAC (1982) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive layers, total stress parameters were employed in the analysis. The total stress parameters (i.e. average mobilized undrained shear strength –  $s_u$ ) for the cohesive soils were assessed based on the results of the in situ field vane tests and estimated from correlations with the SPT results and other laboratory test data. Where appropriate, Bjerrum's (1973) correction factor as a function of the plasticity index of the soil was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests.





Due to the varved nature of the clay stratum, an additional correction factor has been employed to the in situ vane shear strength test results. A reduction factor of 25 percent has been applied to account for the angle of minimum shearing resistance (Milligan and Lo, 1967).

The corrections applied to the in situ field vane shear strength measurements, including Bjerrum's correction for plasticity and the correction to account for the presence of weak horizontal layers in the varved clay, are also appropriate based on a review of available literature [Lo and Stermac (1965), Stermac et al. (1967), Milligan et al. (1962), Quigley and Ogunbadejo (1972) and Milligan and Lo (1967)] and experience on past Golder projects. The Geotechnical Engineering Parameter Summary (Figure 1) also shows that zones of low undrained shear strength are present between about Elevation 305.5 m and 300.5 m and the stability analysis includes consideration of wedge-type slip surfaces at these elevations. The undrained shear strengths measured by the vane tests on the clay stratum underlying the existing embankments are slightly higher than those beside the embankments (Figure 2).

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed approach/abutment areas. The profiles of undrained shear strength versus depth, together with the selected design lines, based on the field and laboratory data are presented on Figure 1. The analysis model geometry and stratigraphy are shown on Figures 3 to 8.

Soil Type	Unit Weight (kN/m <sup>3</sup> )	Corrected Undrained Shear Strength (kPa)	Angle of Internal Friction
Rock Fill	19	--	40°
New Embankment Fill (Granular 'B' Type II)	21	--	35°
Sub-aqueous Fill (Granular 'B' Type II)	20	--	30°
Existing Embankment Fill	20	--	32°
Peat	12	10	--
Clayey Silt to Silty Clay (under existing embankment)	17	25	--
Clayey Silt to Silty Clay (transition zone)	16	14 (see Section 6.7.3.4)	--
Clayey Silt to Silty Clay (under river bed)	16	14	--
Silt	18	--	27°
Cobbles and Boulders	20	--	35°
Silt and Sand to Sand and Gravel	20	--	32°

Note: Groundwater and White Clay River water level assumed to be at Elevation 310.1 m.

### 6.7.2.3 Results of Analysis

The results of the stability analysis are summarized below for the critical embankment sections assuming construction of the embankment to full height in one stage. The minimum Factor of Safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway.





Location		Fill Type	Final Grade (m)	Total Fill Thickness (after peat removed) (m)	Clay Thickness (below fill) (m)	Factor of Safety
South Approach	Cross-Section STA 15+435 (33 m behind abutment)	Rock Fill	313.2	6.1	7.0	< 1.3 (see Fig. 3a)
	Abutment Cross-Section STA 15+468	Granular 'B' Type II	313.0	6.8	7.6	< 1.3 (see Fig. 4a)
	South Abutment Front Slope	Granular 'B' Type II	313.0	6.8	7.6	< 1.3 (see Fig. 5a)
North Approach	North Abutment Front Slope	Granular 'B' Type II	312.7	5.1	9.1	< 1.3 (see Fig. 6a)
	Abutment Cross-Section STA 15+523	Granular 'B' Type II	312.7	5.1	9.1	< 1.3 (see Fig. 7a)
	Cross-Section STA 15+600 (77 m behind abutment)	Rock Fill	312.4	4.1	6.5	> 1.3 (see Fig. 8a)

Note: Results of stability analysis without mitigation.

Limit equilibrium analysis indicates a Factor of Safety of less than 1.3 during construction for the above noted combinations of geometry, embankment type and height, fill type and thickness of underlying clay deposit for the embankments if they are constructed to their final height in one stage. The results indicate that mitigation measures to achieve a Factor of Safety of at least 1.3 for slope stability are required for the front slopes and side slopes, with the exception of the north approach embankment, north of STA 15+600, where the final grade is Elevation 312.4 m and lower.

All of the above analyses incorporate the removal of peat from below the embankment footprint and embankment geometry/configuration (including at the toe of slope) consistent with OPSD 203.020 (Embankments Over Swamp - Existing Slope Excavated to 1H:1V) as discussed in Section 6.8.

### 6.7.3 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the new fills on the compressible foundation soils at this site, including under the existing embankment fill. In addition, settlement of the new fill will also occur, as a portion of this fill will be placed in sub-aqueous conditions.

#### 6.7.3.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using either the commercially available program Settle3D (by Rocscience Inc., Version 2.003), hand and/or spreadsheet calculations. The rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory.

For the settlement analyses, the critical sections were considered to be those with the greatest new embankment height and/or the thickest cohesive deposit, at each approach area as given in Section 6.7.



### 6.7.3.2 Settlement Criteria

Based on MTO's Guideline "Embankment Settlement Criteria for Design" Final Draft, dated March 2, 2010, the following post-construction settlement and differential settlement criteria are considered acceptable within 20 years post-paving for the bridge approach embankments.

Location	Distance from Transition Point (i.e. Abutment)	Total Post-Construction Settlement (mm)
Transition/Taper to Bridge Abutments (non-freeway)	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	100

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

For rock fill that is placed in accordance with SP 206S03 - Earth and Rock Excavation and Grading (i.e. placed versus end dumped), the estimated embedment of the rock fill into the clay deposit below the peat is, on average, about 1 m.

### 6.7.3.3 Settlement of Rock Fill

Where rock fill is used for the construction of the proposed embankments, there will be settlement due to compression of the rock fill itself under self weight, in addition to the settlement of the underlying foundation soils as described above. The magnitude of settlement of the rock fill depends on the following factors:

- type of rock/strength of particles;
- size and shape of rock particles;
- gradation of rock fill;
- total height/thickness of rock fill (stress level); and
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing).

The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e. compacted versus dumped rock fill) as outlined in MTO's "Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates", dated September 2010.

Rock fill should be placed, whenever possible, in a controlled manner (i.e. not end-dumped) in accordance with SP 206S03 (Rock Excavation, Grading). Blading, dozing and 'chinking' the rock fill to form a dense, compact mass is required to minimize voids and bridging and reduce settlements and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e. below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.



### Short-Term Rock Fill Settlement

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO's "Guideline for Rock Fill Settlement" (September 2010), as follows:

Height of Rock Fill, H	Short-Term Rock Fill Settlement (m)	
	Compacted Rock Fill	Dumped Rock Fill
Up to 5 m	0.5% H	1.0% H
>5 m to 10 m	0.75% H	1.5% H
>10 m to 15 m	1.0% H	2.0% H

Approximately 90 percent of the short-term settlement may be expected to occur within the first six (6) months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one (1) year following the completion of embankment construction to full height.

### Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO's "Guideline for Rock Fill Settlement" (September 2010), as follows:

Height of Rock Fill, H	Long-Term Rock Fill Settlement (m)	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15 m	0.1% H	0.2% H

The long-term rock fill settlement is expected to occur from one (1) year following the completion of construction over the life of the embankment.

#### 6.7.3.4 Parameter Selection

The immediate compression of the existing embankment fill material, underlying silt and sand deposit, and sand to sand and gravel deposit, was assessed by estimating an elastic modulus of deformation based on the SPT 'N' values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the clayey silt to silty clay deposit was assessed using the results of the in situ field vane and SPT tests and/or laboratory consolidation tests to estimate the deformation parameters for these soils both under the existing embankment and in the river channel. In addition, the results of the laboratory index testing were also employed to estimate deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967), Kulhawy and Mayne (1990), Azzouz et al. (1976) and Britto and Gunn (1987).



The degree of overconsolidation in the cohesive strata, required in the analyses, was estimated from the results of the in situ field vane tests and the following correlations relating mobilized undrained shear strength to preconsolidation pressure:

$$s_{u(mob)} = 0.22\sigma_p' \text{ (after Mesri, 1975)}$$

where:  $s_{u(mob)}$  = average mobilized undrained shear strength (kPa)  
 $\sigma_p'$  = preconsolidation pressure (kPa)

and

$$s_{u(mob)} = \mu s_{u(FV)} \text{ (after Bjerrum, 1973)}$$

where:  $s_{u(mob)}$  = average mobilized undrained shear strength (kPa)  
 $s_{u(FV)}$  = undrained shear strength from field vane test (kPa)  
 $\mu$  = Bjerrum's correction factor based on Plasticity Index

It is known that some secondary consolidation settlement occurs following the completion of primary settlement. This secondary settlement, or creep settlement, occurs over the long term (i.e. decades) for the normally consolidated clays at this site. The magnitude of secondary (creep) settlement (Mesri 1975 as quoted in Holtz and Kovacs 1981) was estimated using the following:

$$S_c = C_{\alpha\epsilon} \times L_o \times (\Delta \log t)$$

where:  $S_c$  = secondary (creep) settlement (mm)  
 $C_{\alpha\epsilon}$  = modified secondary compression index (%)  
 $L_o$  = initial thickness of compressible deposit (mm) in the normally consolidated portion of the deposit  
 $t$  = time period of interest

Based on Mesri (1975), the following empirical correlation was utilized to estimate  $C_{\alpha\epsilon}$  from water content:

$$C_{\alpha\epsilon} = w_n/100$$

where:  $w_n$  = water content (decimal)

The modified secondary compression index,  $C_{\alpha(\epsilon)}$ , was also estimated directly from the results of the laboratory consolidation tests using the dial reading versus log-time plots.

Summarized below are the simplified stratigraphy, unit weights and deformation parameters employed for the different soils types in the approach areas. The maximum estimated settlement of the foundation soils and fill is presented in the sections below and a discussion on the rate of settlement is included. The embankment geometry and stratigraphy employed in the analyses are the same as those utilized in the assessment of stability.



Soil	Range of Thickness (m)	Unit Weight (kN/m <sup>3</sup> )	Estimated Deformation Properties
Rock Fill	Up to 6.8	19	See Section 6.7.3.3
Granular 'B' Type II Fill (total)	Up to 6.1	21	E' = 25 MPa
Granular 'B' Type II Fill (Sub-aqueous)	Up to 3.1	20	E' = 5 MPa
Existing Embankment Fill	0.6 – 5.6	20	E' = 5 MPa
Clayey Silt to Silty Clay (under river bed)	0.8 – 9.1	16	see Figure 1
Clayey Silt to Silty Clay (under existing embankment)	Up to 7.3	17	see Figure 2
Silt	0.3 – 5.2	18	E' = 2 MPa
Silt and Sand to Sand and Gravel	Up to 5.6	20	E' = 5 MPa
Cobbles and Boulders (with sand and gravel)	Up to 2.6	20	E' = 20 MPa

### Preconsolidation Stress

The consolidation parameters estimated for the clayey silt to silty clay deposit are shown on Figures 1 and 2. These results were compared with values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described previously. Although the value of preconsolidation stress,  $\sigma_p'$ , estimated from the test results (Boreholes WC-4, WC-8 and WC-13) is between 70 kPa and 75 kPa, a review of the consolidation test results and considering the values found in literature for similar varved clay sites in the New Liskeard area (Stermac et al., 1967; Milligan and Lo, 1967), the  $\sigma_p'$  value at this site could be as high as about 100 kPa (also shown on Figure 1). This higher value of  $\sigma_p'$  is also supported by the observation that very little settlement of the foundation soils under the existing embankments appears to have occurred where the grade of the roadway is at about Elevation 312.3 m as evidenced by the lack of pavement distress. However, it is noted that the boreholes drilled through the existing roadway embankment did penetrate up to about 240 mm of asphalt, which although could be indicative of past asphalt padding or overlays, may not necessarily be directly attributable to pavement repair due to, or evidence of, settlement. Further, the results of the stability analyses also indicate that for embankment heights above about Elevation 312 m, the Factor of Safety decreases below 1.3.

For calculating primary settlement, the subsoils will either be in an over-consolidated condition (where the magnitude of settlement is relatively small and settlement occurs relatively quickly - recompression) or a normally consolidated condition (where the magnitude of settlement is larger and the settlement occurs over a longer period of time – virgin compression). Based on the field and laboratory test results and related data in literature, the following values or range of values for  $\sigma_p'$ , were used for estimating the settlement of the cohesive strata, under the proposed approach embankments and immediately adjacent to the abutments (see Section 6.7.3.5):



- Approach Embankments – 67 kPa at Elevation 306 m increasing linearly to 85 kPa at Elevation 298.5 m; and
- At the Abutments – 100 kPa from Elevation 306 m to Elevation 298.5 m.

### Compression Indices

The estimated deformation values of the varved clayey silt to silty clay deposit at this site, based on the results of a limited number of laboratory consolidation tests and empirical correlations employing the results of the laboratory index testing, are compression index,  $C_c$ , of 0.5 and recompression index,  $C_r$ , of 0.05. However, empirical correlations by Koppula (1986) that have been found to be representative of Northern Ontario clays in general, as well as a detailed laboratory study of compression indices for varved clays in the New Liskeard area by Quigley and Ogunbadejo (1972), suggest that design values for  $C_c$  and  $C_r$  equal to 0.7 and 0.07, respectively, are more applicable for this site. Accordingly, the settlement analyses used the values suggested by Quigley and Ogunbadejo (1972).

### Secondary Compression

The value of the modified secondary compression index,  $C_{\alpha(\epsilon)}$ , estimated using the average water content of the varved clay samples and the empirical correlation proposed by Mesri (1975) is about 0.6 percent. Since the water content determinations in the laboratory are typically carried out on combined samples of both the clay and silt laminae, the actual water content (and therefore the value of  $C_{\alpha(\epsilon)}$ ) in the clay laminae will likely be higher than the empirical correlation would suggest. As part of the re-analysis of the embankment settlement, the modified secondary compression index,  $C_{\alpha(\epsilon)}$ , was estimated directly from the results of the laboratory consolidation tests (using the dial reading versus log-time plots), resulting in a higher and likely more representative value of  $C_{\alpha(\epsilon)}$  of about 1.4 percent. The creep settlement calculation considers only the summed thickness of the clay laminae portion of the varved clay-silt deposit, which is estimated to be about two thirds of the total thickness of the deposit.

### Time Rate of Settlement

A review of the laboratory consolidation test data and case studies of embankment settlement on varved clays in Northern Ontario (including the New Liskeard area) in literature including Stermac et al. (1967) and Milligan et al. (1962), suggest that an average value of the coefficient of consolidation,  $c_{v(n/c)}$ , equal to  $5.8 \times 10^{-4} \text{ cm}^2/\text{s}$  is considered appropriate for the normally consolidated varved clay stratum at this site. Further, based on a back-analysis of the embankment settlement monitoring data in the above noted literature, the  $c_{v(n/c)}$  value should be applied to, and the time-rate settlement analysis should consider, two-way drainage of the full varved clay deposit thickness, rather than just the summed thickness of the clay laminae (i.e. two thirds the deposit thickness). In the recompression range (i.e. final stresses at or below the preconsolidation stress), a value of  $c_{v(o/c)}$  of  $7.6 \times 10^{-3} \text{ cm}^2/\text{s}$  is considered appropriate.



### 6.7.3.5 Results of Analysis (without mitigation)

Based on the final embankment geometry, and including peat sub-excavation and backfilling with either rock fill or Granular 'B' Type II, the estimated magnitudes of primary and secondary consolidation of the clayey silt to silty clay deposit estimated to occur if the embankments were constructed in one stage to full height (i.e. to Elevation 313.0 m to 313.3 m at the South Approach and to Elevation 312.3 m to 312.7 m at the North Approach) are presented in Table 2 for the critical sections indicated in Section 6.7. It should be noted that the results presented in Table 2 are based on a preconsolidation stress of 100 kPa as discussed in Section 6.7.3.4.

Based on the discussion in Section 6.7.3.4, presented below are the estimated settlement results based on a range of preconsolidation pressures varying with depth between 67 kPa and 85 kPa, which is the lower bound estimate of preconsolidation pressure based on the laboratory test results, in comparison to a preconsolidation pressure equal to 100 kPa for New Liskeard area clay (Milligan and Lo, 1967; Milligan et al, 1964) for the  $\sigma_p'$  design lines, as shown on Figure 1.

Location	Station	Settlement Type	Estimated Primary Settlement Using Original Design Line for $\sigma_p'$		Estimated Primary Settlement Using $\sigma_p' = 100$ kPa	
			Construct Embankment to El. 312.0 m (S)/ 312.3 m (N)	Construct Embankment to Final Grade <sup>1</sup>	Construct Embankment to El. 312.0 m (S)/ 312.3 m (N)	Construct Embankment to Final Grade <sup>1</sup>
South Approach	STA 15+385	$\delta_{primary}$ $t_{90}$	83 ~2 months	111 ~2 years	76 ~2 months	25 ~2 years
	STA 15+407	$\delta_{primary}$ $t_{90}$	138 ~5 months	203 ~5 years	99 ~5 months	72 ~5 years
	STA 15+435	$\delta_{primary}$ $t_{90}$	178 ~5 months	222 ~5 years	103 ~5 months	108 ~5 years
	STA 15+468 (S. Abut.)	$\delta_{primary}$ $t_{90}$	376 ~7 years	249 ~7 years	114 ~6 months	249 ~7 years
North Approach	STA 15+523 (N. Abut.)	$\delta_{primary}$ $t_{90}$	532 ~9 years	505 ~9 years	282 ~6 months	95 ~9 years
	STA 15+573	$\delta_{primary}$ $t_{90}$	191 ~6 months	59 ~6 years	107 ~6 months	8 --
	STA 15+600	$\delta_{primary}$ $t_{90}$	155 ~5 months	32 ~5 months	98 ~5 months	5 --

1. Not stable. See Section 6.7.2.3.

The estimated magnitude of primary settlement using the design line based on the field and laboratory test data for the new embankment constructed to approximately the same elevation as the existing embankment is up to 0.5 m at the abutments. However, based on the information from the boreholes drilled through the existing embankment, this magnitude of settlement is unlikely and, therefore, the design line for  $\sigma_p'$  based on published literature is used for all subsequent discussion on settlement magnitude and time rate at the abutments. For the estimation of the required extent of EPS fill to mitigate settlement of the approach embankments (see Section 6.7.5.2), the variable-with-depth, more conservative, lower values of preconsolidation pressure are used in the analysis.

For the estimated settlement values given in Table 2, the total settlements exceed the post-construction (i.e. post-paving) settlement criteria given in Section 6.7.3.2 and, therefore, settlement mitigation measures are required.





Although the estimated settlement values have been calculated for the full height of the embankments constructed in one stage, this condition will not be possible as the embankments will not be stable for this case (except for the section north of STA 15+600) as presented in Section 6.7.2.3.

The analysis of the time rate of settlement indicates that approximately 90% of primary consolidation settlement in the normally consolidated range will occur within the period of between about 2 years and 9 years after construction. The creep settlement is typically between 60 mm and 70 mm per log cycle of time (i.e. 9 years to 90 years) and begins after 90% primary consolidation is complete. Therefore, there will typically be less than one log-cycle of creep occurring within the 20-year period design life of the structure.

Differential settlement in the longitudinal direction is estimated to be less than 200:1 and in the cross-sectional (lateral) direction is greater than 200:1 since settlement of the outside (west) shoulder of the embankments will be larger than the settlement of the inside (east) shoulder of the embankments due to the influence of the existing embankment on the subsoils below the east shoulder.

Based on the above results, mitigation of settlement (vertical and differential) will be required at this site and alternatives to mitigate this settlement are discussed in Section 6.7.5.

#### **6.7.4 Liquefaction Potential and Seismic Analysis**

As noted in Section 6.5.2, this site is located in Seismic Zone 1 with a  $PHA < 0.08$ . Further, the bridge structure is not a lifeline structure as confirmed by the structural designer. As such, based on Section 4.4.4 of the CHBDC, the site is assigned a Seismic Zone Performance of 1 and, therefore, in accordance with Section 4.4.5.1 of the CHBDC, no liquefaction analysis is required.

#### **6.7.5 Mitigation of Stability/Time Dependent Settlements**

Given the thickness and consistency of the soft to firm clayey silt to silty clay deposit under the proposed embankment footprints and the height of the new embankments, the results of our analysis indicate that mitigation measures will be required to achieve a target Factor of Safety greater than 1.3 against deep-seated failure for the proposed embankments and to reduce the magnitude of total and differential post-construction settlement of the new embankments. The advantages, disadvantages, relative costs and risks/consequences for stability and settlement mitigation alternatives for the approaches at this site are summarized and ranked in Table 3. We recommend using a combination of mitigation strategies consisting of staged construction and preloading and the use of lightweight Expanded Polystyrene (EPS) fill as the preferred alternatives to enhance stability and reduce post-construction settlements at the site. This and other alternatives are described in more detail below. In all cases, the peat and organic deposits should be sub-excavated from below the embankment footprint prior to embankment construction using the method described in Section 6.8.

##### **6.7.5.1 Staged Embankment Construction and Preloading**

In order to partially mitigate the stability and settlement concerns identified in Sections 6.7.2.3 and 6.7.3.5, it is recommended that a staged construction sequence combined with preloading be implemented to achieve the target Factor of Safety of 1.3 for the final embankments and to minimize the post-construction settlements of the embankment (total and differential), although additional settlement mitigation measures will still be required.



The recommended staged construction sequence is outlined in Table 4. While a second stage of construction could also be suitable to address settlement mitigation, this would result in a further 6 months (minimum) wait period prior to construction of the pavement structure and stability issues could still occur during a second stage of construction; this alternative is not recommended (see Stages 5A ad 5B in Table 4).

We recommend that the embankments be constructed to the maximum stable embankment height and then preloaded for a minimum of 6 months. The maximum stable height is Elevation 312.0 m and 312.3 m on the south and north approach, respectively (see Table 4). For embankments constructed to this height, the clay deposit will be subjected to loading in the overconsolidated range (i.e. stresses not exceeding the preconsolidation pressure) and 90 percent of consolidation is expected to occur within 6 months (using a  $c_{v(o/c)}$  of  $7.6 \times 10^{-3} \text{ cm}^2/\text{s}$ ).

In order to construct the embankments to the final height and achieve a Factor of Safety greater than 1.3, strength gain of the underlying clay soils as a result of compression from the first stage of loading (Stages 1 to 4 in Table 4) must be relied upon. It is estimated that during the first 6-month preload period, the increase in undrained shear strength of the silty clay to clay deposit will be between 0 kPa (stresses not exceeding the preconsolidation pressure) to 2 kPa (clays become normally consolidated and experience virgin compression to about 25 percent completion). Given that the theoretical gain in undrained shear strength cannot be assured and will be less than that required to maintain stability of the embankments at full height at the abutment front slopes, an alternative mitigation method such as lightweight (EPS) fill, will be required for the front slope to achieve a Factor of Safety greater than 1.3, as discussed in further detail in Section 6.7.5.2.

Beyond the abutment front slopes, where stability is less critical, the estimated strength gain of the silty clay to clay deposit during the 6-month preload period is sufficient to achieve an adequate Factor of Safety to allow for construction of the embankments to proceed to the final grade. In this case, and as presented in Table 2 and shown on Figures 9a to 9d, 10a and 10b, a second preload period would be required. However, since the additional loading from the final 0.4 m to 1.3 m of fill will surpass the clay deposit's preconsolidation stress range and will act in the normally consolidated range and since 90 percent of the primary settlement during the second stage of loading (Stages 5a and 6a in Table 4) will occur in 5 to 9 years for a  $c_{v(n/c)}$  of  $5.8 \times 10^{-4} \text{ cm}^2/\text{s}$ , even with a second 6-month wait period the post-construction settlement would be greater than the criteria given in Section 6.7.3.2. Further mitigation will therefore be required to reduce this magnitude of post-construction settlement. This could consist of a longer preload period (not sufficient time in the schedule), a surcharge (embankments would not be stable), or replacement of some of the granular/rock fill with lightweight (EPS) fill which is discussed in further detail in Section 6.7.5.2.

It should be noted that settlement of the existing roadway (up to about 400 mm) will occur while traffic is still flowing during construction of the new roadway embankment, being greatest along the south and west side of the existing embankment. Some maintenance of the existing pavement during construction of the new embankments should be anticipated. A Factor of Safety greater than 1.3 is also achieved for the existing embankment during construction of the new embankment. Since the majority of the existing roadway embankment will be removed after the new bridge is completed, settlement of the portion of the existing embankment remaining under the shoulder/slope of the new embankment after construction will not be a concern.



Given the uneven river bed configuration, some areas will require minor toe berms to achieve a Factor of Safety greater than 1.3 during construction, namely the abutment front slopes (toe berm 3 m long and up to 1.9 m thick; see Figures 5b and 6b) and near STA 15+435 (toe berm 6 m wide and 1.7 m thick with top at Elevation 309.5 m; see Figure 3b). Calculation of fill quantities should also take into consideration the estimated settlement and the penetration of the fill into the subgrade which, for rock fill, could be up to about 0.5 m.

Monitoring of settlement and lateral movement of the new and existing embankments, and the existing bridge itself, should be carried out during construction of the new embankment using a series of surface points established along the top of the embankments and bridge structure. To be able to record as much of the settlement occurring under the new embankment as possible during embankment construction, settlement plates should be installed as each segment of Stage 1 (Table 4) construction is completed to Elevation 310.5 m (i.e. once the embankment is above the river water level). Monitoring points on the existing embankment and bridge consisting of settlement pins (installed immediately outside the live traffic lane for ease of surveying) should be installed prior to the start of construction. In addition to settlement monitoring points, vibrating wire piezometers should also be installed within the cohesive deposit to monitor pore water pressures. In addition to the settlement/lateral movement monitoring, visual observations of the existing embankments and bridge structure should be carried out on a routine basis by the Contract Administrator's Foundation Representative.

This suggested sequence of construction and peat sub-excavation should be included in the Contract using an Operational Constraint (OC), an example of which is given in Appendix C. The instrument locations, types and details will be included in an NSSP, a template of which is also included in Appendix C, which will also include example drawings of the instrumentation installation. A template for the schedule of the frequency of readings and the proposed Review Level(s) and Alert Level(s) for the various instruments is included for the Contract Administration assignment in Appendix C. The final documents should be prepared outside of this report on the basis of concurrence with MTO with the monitoring program and included in the Contract Documents and Contract Administration assignment, as appropriate.

#### **6.7.5.2     *Lightweight (EPS) Fill***

As noted in Section 6.7.5.1, given the estimated small amount of undrained shear strength increase expected during the 6-month preload period and the magnitude of post-construction settlement even with a second 6-month preload period, it is recommended that EPS fill be incorporated into the approach embankments as the most technically preferred option to mitigate settlement and reduce the risk of embankment instability. The main disadvantage of EPS fill is the increased cost of this material. However, given the limited quantities of EPS required at this site and the schedule constraints, the use of EPS is considered the preferred alternative.

In order to achieve a Factor of Safety greater than 1.3 for the south approach embankment front slopes and side slopes and to reduce post-construction settlement to acceptable levels, a 1 m thick zone of EPS is required for a distance of 63 m behind the abutment (STA 15+468 to 15+405) stepping up (decreasing) to 0.3 m for the next 20 m (STA 15+405 to 15+385). For the north approach, a 1 m thick zone of EPS is required within a 10 m distance behind the abutment (STA 15+523 to 15+533) stepping up (decreasing) to 0.5 m of EPS for the next 10 m (STA 15+533 to 15+543) and then stepping up (decreasing) to 0.2 m of EPS for 42 m (STA 15+543 to 15+585). A greater lateral extent and thickness of EPS is required on the south approach due to the higher grades and greater magnitude of settlement anticipated.



The results of the stability analysis for the embankments constructed with EPS fill for the south front slope and side slope are shown on Figures 3b to 7b for the critical sections given in Section 6.7, using EPS, rock fill and Granular 'B' Type II fill as appropriate. While Figure 5b shows a Factor of Safety marginally less than the target 1.3 for the south abutment front slope, as noted in Section 6.7.3.4 a potentially higher preconsolidation pressure (up to 100 kPa) of the clayey silt to silty clay deposit can be assumed for design. This higher preconsolidation pressure and therefore a potentially greater undrained shear strength than assumed in the stability analysis, is supported by the observation of adequate roadway/embankment performance and that very little settlement of the foundation soils under the existing embankments appear to have occurred. In addition, it is probable that some nominal strength gain is likely to occur during the 6-month preload period which, combined with replacement of 1 m of rock fill with EPS to achieve the design height, will enhance the stability of the calculated front slope Factor of Safety to slightly greater than 1.3, as shown on Figure 5c. The estimated total post-construction settlement for the EPS alternative is presented in Table 2 and Figures 9a to 9d, 10a and 10b.

Figures 11 and 12 show the recommended configuration of the EPS behind the abutments in the south and north approaches, respectively. EPS is typically provided in blocks 0.5 m or 1 m thick, but other thickness can also be obtained, to accommodate the steps noted above. A minimum of 1 m of conventional granular cover should be provided over the EPS on the side slopes and the EPS should also taper to a thickness of 0.5 m in these areas, as applicable. Appropriate staggered layout, ties and spacers should be used to ensure the EPS block mass acts as a single unit. The entire top and sides of the EPS mass should be covered with a 6 mil (0.15 mm) thick polyethylene sheet for protection from hydrocarbon-based products that may infiltrate the pavement/shoulder structure. The finished top of the EPS blocks and the polyethylene sheet should be provided with a 125 mm thick concrete slab, for ballast and for protection against differential icing at the roadway structure, and a minimum of 1 m of conventional granular cover (including the pavement structure). It is estimated that a volume of EPS of about 1500 m<sup>3</sup> will be required to satisfy the recommendations for this site. An NSSP should be included in the Contract for Rigid Expanded Polystyrene Embankment Fill; an example is included in Appendix C.

Given the limited thickness of the new embankment above the water level, buoyancy is a potential concern for the EPS. EPS must be placed above the measured (or normal) water level at Elevation 310.2 m (July 2008) and have the appropriate soil and concrete cover. For the 1 m thick zone of EPS recommended above, the underside of the EPS will be at about Elevation 311.0 m and 310.7 m at the south and north abutments, respectively. For a 1 in 50 year flood level at Elevation 311.3 m, the Factor of Safety against buoyancy is greater than 1.5.

### 6.7.5.3 *Change Bridge Alignment*

If the construction schedule cannot accommodate staged construction, then consideration could be given to changing the bridge alignment. If the new bridge was constructed on the existing alignment (rather than 18 m offset to the west), then the proposed grade raise would be less than 1 m and the widening would be minimal. In this case, the stability would be improved and the post-construction settlement of the existing/new roadway would be able to be accommodated using lightweight fill or surcharging. However, construction staging may be difficult or not feasible without a detour depending on the structural concerns with the existing bridge (which is in poor condition and the founding levels of the existing piles are unknown), which could impede traffic flow or require rerouting of traffic to an alternate highway (bypass).



#### 6.7.5.4 Toe Berms (and Surcharging)

Given the limited increase in undrained shear strength of the clay strata under the new embankment expected during the first 6-month preload period, large toe berms would be required to be incorporated into the embankment construction to the final grade for the front slopes and side slopes within an approximately 25 m zone behind the abutments, where Granular 'B' Type II is to be used for embankment construction. In order to achieve a Factor of Safety greater than 1.3 for the final embankment configuration, toe berms for the front slope would have to be sufficiently large such that the backfill below the river water level (after sub-excavation and peat removal) would extend beyond the location of the new piers, well into the river channel, raising environmental, fisheries and navigation concerns. We understand that it is not desirable to extend the filling/toe berm construction beyond the pier locations due to these potential implications. Further, we understand that it is not rapidly possible to obtain additional property (i.e. right-of-way) to accommodate larger toe berms that would be required for the side slopes. Therefore, the use of toe berms to mitigate settlement in these areas is not practical and is not recommended.

As a result of not being able to practically construct toe berms, surcharging is also not a technically feasible option to reduce post-construction settlement due to stability concerns, unless sufficient time is provided to allow for a significantly longer preload period.

#### 6.7.5.5 Other Alternatives

Other alternatives that have been considered, but that are not practical technically or are not feasible, are partial or full sub-excavation of the soft clayey silt to silty clay below the peat, and the use of wick drains.

Sub-excavation of the soft clayey silt to silty clay would require an excavation adjacent to the existing embankment up to about 12 m below the river water level, which is considered impractical without extensive temporary roadway protection along the entire embankment length.

Wick drains may not enhance the rate of settlement because the soil under the new embankment footprint is naturally varved.

### 6.8 Subgrade Preparation and Embankment Construction

Prior to embankment construction, all peat and topsoil/vegetation/organic soils must be removed from below the footprint of the proposed embankments. Where existing fill is encountered in the transition zone between embankments, the fill may remain in place.

In order to maintain stability of the existing embankments and protect the existing bridge, the peat excavation and backfilling should be carried out simultaneously as per OPSS 209 (Embankments Over Swamps and Compressible Soils) and in accordance with OPSD 203.020 (Embankments Over Swamp - Existing Slope Excavated to 1H:1V) using Granular 'B' Type II or rock fill. OPSS 209 allows for backfill material to be placed up to 0.6 m above the water level without compaction. In this case, the work can either start in the centre of each embankment working towards the north/south or from the extreme ends of each embankment towards the abutment areas. Alternatively, the peat excavation and immediate backfilling can be carried out in areas no greater than about 3 m x 3 m working longitudinally and away from the existing embankment and from the new





embankment extremes towards the abutments. This excavation and backfilling technique will control the direction of mud waves created during the backfilling process away from the existing bridge. The constraint to excavate the peat and backfill in small strips or areas only applies until the embankment construction extends to above the water level. An OC is required for this construction sequence and an example is included in Appendix C.

Above the water level, the granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 percent of the standard Proctor maximum dry density in accordance with SP 206S03 (Earth Excavation, Grading). The surface of the rock fill or Granular 'B' Type II, if used for embankment construction in lieu of earth fill or Granular 'B' Type I, should be covered with a layer of Granular 'A' to act as a bedding/levelling pad for the EPS fill. Side slopes should be no steeper than 2H:1V for Granular 'B' Type II or 1.25H:1V for rock fill embankments. The final lift of fill prior to placement of the granular subbase and base courses should be compacted to 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

No other construction activities can take place during the Stage 4 (Table 4) wait period including pile installation (abutments and piers), abutment/pier cap construction or construction traffic/material storage on the partially constructed embankments.

In order to minimize differential settlement between the existing embankment slopes and the newly placed embankment fill, the new fill should be keyed into the existing embankment side slope per the requirements of OPSS 208.010 (Benching of Earth Slopes).

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS 511 (Rip Rap, Granular Sheeting) and SP 511S01 (Rip Rap, Gravel Sheeting). Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Erosion protection could consist of a minimum 0.6 m thick layer of R-10 Rip Rap (300 mm diameter as per OPSS 1004), rock protection or concrete slope paving. The designer should address the potential for scour below the pile caps in the design of the bridge foundations.

To reduce surface water erosion on granular fill embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS 804 (Seed and Cover) should be carried out as soon as possible after construction where earth fill, rather than Granular 'B' Type II, is used. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting as per OPSS 511 (Granular Sheeting) will be required to prevent erosion and to reduce the potential for remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

## **6.9 Design and Construction Considerations**

### **6.9.1 Excavations**

Excavation adjacent to the existing highway will be required to remove the peat from below the new embankment footprint. In the north approach area, excavations up to about 2.6 m below the river water level will be required to remove peat up to about 1.5 m thick. In the south approach area, excavations up to about 3.9 m below the river water level will be required to remove peat up to about 3.0 m thick. The river water level was



measured at Elevation 310.2 m in July 2008. As discussed in Section 6.8, temporary excavation side slopes during peat sub-excavation should be in accordance with OPSD 203.020 (Embankments Over Swamp). Excavations for piles cap construction should be carried out within a temporary or permanent cofferdam.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects and good construction practice. The peat should be classified as Type 4 soil and the existing fill and other native soil should be classified as Type 3, according to the OHSA.

### **6.9.2 Pile Installation and Pile Cap Construction**

Piles can be installed after completion of the 6-month preload period prior to construction of the embankment to final grade. Since EPS is being used to mitigate settlement after completion of the 6-month preload period as identified in Section 6.7.5.1 and Section 6.7.5.2, downdrag loads need not be considered in the design. A typical sequence of construction may be considered as follows:

- Install cofferdam to elevations shown on the contract drawings;
- Excavate soil in cofferdams;
- Install CSP pipes;
- Place tremie concrete within cofferdam;
- Drive piles;
- Place sand in CSPs;
- Unwater cofferdam; and
- Construct pile caps.

It is proposed that the existing roadway will remain open to traffic during construction of the new bridge. Where pile cap construction is adjacent to the existing roadway, the temporary shoring system (i.e. cofferdam) should support and maintain the stability of the existing adjacent roadway embankment.

A temporary excavation support system, if required, should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2, as specified in OPSS 539.

### **6.9.3 Groundwater and Surface Water Control**

Peat excavation up to about 4 m below the water level will not require lowering of the groundwater or river water level. As discussed in Section 6.8, backfill for embankment construction may be placed up to 600 mm above the water level without compaction.

Pile cap construction should take place within an unwatered cofferdam. It is understood that there is a special provision for cofferdams that is typically used in MTO Contracts for this purpose. A NSSP alerting the Contractor that the excavations must be unwatered and kept stable during construction is required and an example is included in Appendix C. The cofferdam should be designed so that the disturbance to the existing embankments is minimized.





#### **6.9.4 Obstructions**

As part of the design and construction of the new foundations, careful consideration should be given to the location of the existing (and possibly older) bridge foundations (i.e. timber piles and/or cribs) relative to the new construction. Specifically, the designer should check that any new piles (batter and orientation) and any temporary shoring do not interfere with the older abandoned piles. This should be checked to the full extent of the pile/shoring length.

A layer of cobbles and boulders, up to 2.6 m thick, was encountered above the bedrock surface in Boreholes WC-6 to WC-8. Further, several instances of grinding of the augers or casing were noted within the silt and sand to sand and gravel deposit in most boreholes, which is indicative of gravelly layers or the presence of cobbles and/or boulders.

An OC should be included in the contract documents to alert the contractor to such potential construction difficulties, such as the existing piles or cobbles/boulders at the site. Example OCs are included in Appendix C for reference.

#### **6.9.5 Monitoring of Existing Structure**

In addition to monitoring of settlement/lateral movement discussed in Section 6.7.5.1, vibration monitoring of the existing bridge during construction of the new bridge should also be carried out. Given the condition of the existing structure, the close proximity of construction of the new structure relative to the existing structure, and the requirement for the existing structure to remain in operation during construction, it is recommended that the existing structure be monitored for excessive vibrations. An example NSSP is included in Appendix C for reference.

#### **6.9.6 Removal of Existing Structure and Embankments**

We understand that after construction of the new bridge and approach embankments are complete, the portion of the existing roadway embankment beyond the east shoulder/slope of the new embankment as well as the existing bridge are to be removed and the site reconstructed for fisheries habitat compensation.

The existing embankment should be removed such that the final side slopes of the new embankment are no steeper than 2H:1V. The existing embankment should be removed to no lower than Elevation 309 m and left in place below this elevation, so that the remaining fill acts as a stabilizing berm for the new embankment.

Where old timber piles exist, it is recommended that they be left in place and not pulled out or removed. During the foundation investigation at this site, artesian conditions were noted in some boreholes and based on these observations and considering the present condition of the existing bridge timber pile foundations, it is possible that extraction of any existing piles from previous foundations may cause disturbance of the foundation soils and loss of ground which could affect the performance of the new structure.

The existing timber piles should be cut off at the river bed level after installation of the new piles to minimize obstructions during subsequent construction of the abutments and to enhance future use of the waterway.



## **7.0 CLOSURE**

This report was prepared by Ms. Sarah E. M. Coyne, P.Eng., Associate and geotechnical engineer with Golder Associates Ltd. The technical aspects were reviewed by Dr. J. Paul Dittrich, P.Eng., Principal. Mr. Jorge M. A. Costa, P.Eng., Principal and the Designated MTO Foundations Contact conducted a quality control review of the report.



## Report Signature Page

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- ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
  - ASTM D1587 Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
  - ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil
- Commercial Software
- GeoStudio (Version 6.20) by Geo-Slope International Ltd.
  - Settle 3D (Version 2.003) by Rocscience Inc.
- Ministry of Transportation Ontario Special Provisions
- SP 110S13 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
  - SP 206S03 Earth Excavation, Grading; Rock Excavation, Grading; Rock Embankment
  - SP 511S01 Rip Rap; Rock Protection, Gravel Sheeting



Ontario Provincial Standard Drawings

OPSD 203.020	Embankments Over Swamp – Existing Slope Excavated to 1H:1V
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.100	Foundation Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3101.200	Walls, Abutment, Backfill, Rock
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

Ontario Provincial Standard Specifications

OPSS 209	Construction Specification for Embankments Over Swamps and Compressible Soils
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover
OPSS 903	Construction Specification for Deep Foundations
OPSS 1004	Material Specification for Aggregate - Miscellaneous

Ontario Water Resources Act

Ontario Regulation 372/97	Amendment to Ontario Regulation 903
Ontario Regulation 903/90	Wells



**REVISED FOUNDATION REPORT - WHITE CLAY RIVER BRIDGE REPLACEMENT  
HIGHWAY 11, GWP 5239-06-00**

**Table 1: Evaluation of Foundation Alternatives**

Options	Ranking	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles Driven to Bedrock Surface	1	<ul style="list-style-type: none"> <li>Standard construction.</li> <li>Allows for integral abutment design.</li> </ul>	<ul style="list-style-type: none"> <li>Possibility of piles “hanging up” on cobble and boulder deposit or in very dense silt and sand/silt to sand and gravel deposit above the bedrock.</li> <li>Cofferdam required for pile cap construction adjacent to and in the river.</li> </ul>	<ul style="list-style-type: none"> <li>Lower relative costs compared with caisson option.</li> </ul>	<ul style="list-style-type: none"> <li>Dewatering within the cofferdam required in order to construct the pile cap.</li> <li>Risk of not penetrating the cobble and boulder deposit overlying the bedrock and achieving design capacity.</li> </ul>
Caissons Socketted into Bedrock	2	<ul style="list-style-type: none"> <li>Reduced number of deep elements compared to steel H-piles.</li> <li>Possible elimination of pile cap.</li> </ul>	<ul style="list-style-type: none"> <li>Temporary liners would be required for groundwater control and support through overburden.</li> <li>Concrete for caissons would have to be placed by tremie methods below the water level.</li> <li>May be difficult advancing through the cobble and boulder deposit and socketting caissons into fractured, strong to very strong siltstone/sandstone bedrock to the design base elevation.</li> </ul>	<ul style="list-style-type: none"> <li>Higher relative costs compared to steel H-piles driven to bedrock.</li> </ul>	<ul style="list-style-type: none"> <li>Difficulties in penetrating the cobble and boulder deposit and the fractured bedrock, resulting in lower capacity.</li> <li>Difficulties achieving seal and drilling large diameter socket into fractured, strong to very strong bedrock to the design base elevation resulting in lower capacity.</li> </ul>
Shallow Foundation	NF		<ul style="list-style-type: none"> <li>Unable to obtain required geotechnical axial capacity at shallow depth or on a prepared pad founded on the near surface soils.</li> </ul>		





# REVISED FOUNDATION REPORT - WHITE CLAY RIVER BRIDGE REPLACEMENT HIGHWAY 11, GWP 5239-06-00

**Table 2: Estimated Consolidation Settlements**

Location	STA	Settlement Type	Estimated Post-Construction Settlement Over 20-Year Period (mm) <sup>1,2</sup>				Post-Construction Settlement Criteria (see Section 6.7.3.2)	Recommendation
			No Foundation Mitigation <sup>2</sup>	Staged Construction with One Preload Period <sup>4</sup>	Staged Construction with Two Preload Periods <sup>4</sup>	Staged Construction with One Preload Period and EPS <sup>4</sup>		
South Approach	STA 15+385	$\delta_{\text{primary}}$ $\delta_{\text{secondary}}$ $\delta_{\text{rock fill}}$ $\delta_{\text{total}}$ $t_{90}$	90 0 40 <b>130<sup>3</sup></b> ~2 months	NF	25 10 0 <b>35<sup>5</sup></b> --	0 10 10 <b>20</b> --	100 mm	Staged construction with one 6-month preload period and 0.3 m EPS (ends at this station) (see Figure 9a).
	STA 15+407	$\delta_{\text{primary}}$ $\delta_{\text{secondary}}$ $\delta_{\text{rock fill}}$ $\delta_{\text{total}}$ $t_{90}$	130 35 40 <b>185<sup>3</sup></b> ~5 years	NF	50 45 0 <b>95<sup>5</sup></b> --	5 10 10 <b>25</b> --	50 mm to 100 mm	Staged construction with one 6-month preload period and 1 m EPS (see Figure 9b).
	STA 15+435	$\delta_{\text{primary}}$ $\delta_{\text{secondary}}$ $\delta_{\text{rock fill}}$ $\delta_{\text{total}}$ $t_{90}$	175 35 50 <b>260<sup>3</sup></b> ~6 years	NF	55 50 0 <b>105<sup>6</sup></b> --	10 10 10 <b>30</b> --	25 mm to 50 mm	Staged construction with one 6-month preload period and 1 m EPS (see Figure 9c).
	STA 15+468 (Abutment)	$\delta_{\text{primary}}$ $\delta_{\text{secondary}}$ $\delta_{\text{total}}$ $t_{90}$	280 35 <b>315<sup>3</sup></b> ~7 years	NF	175 45 <b>220<sup>6</sup></b> --	10 10 <b>20</b> --	0 mm to 25 mm	Staged construction with one 6-month preload period and 1 m EPS (see Figure 9d).
North Approach	STA 15+523 (Abutment)	$\delta_{\text{primary}}$ $\delta_{\text{secondary}}$ $\delta_{\text{total}}$ $t_{90}$	340 25 <b>365<sup>3</sup></b> ~9 years	NF	70 35 <b>105<sup>6</sup></b> --	55 10 <b>65<sup>6</sup></b> --	0 mm to 25 mm	Staged construction with one 6-month preload period and 1 m EPS (see Figure 10a).
	STA 15+573	$\delta_{\text{primary}}$ $\delta_{\text{secondary}}$ $\delta_{\text{rock fill}}$ $\delta_{\text{total}}$ $t_{90}$	110 0 30 <b>140<sup>3</sup></b> ~6 months	NF	NA	10 10 10 <b>30</b> --	50 mm	Staged construction with one 6-month preload period and 0.2 m EPS (ends just beyond this station) (see Figure 10b).
	STA 15+600	$\delta_{\text{primary}}$ $\delta_{\text{secondary}}$ $\delta_{\text{rock fill}}$ $\delta_{\text{total}}$ $t_{90}$	100 0 30 <b>130</b> 5 months	5 0 10 <b>15</b> --	NA	NA	100 mm	Staged construction with one 6-month preload period (see Figure 10c).

**Notes:**

NA – Not applicable, NF – Not feasible

- See Sections 6.7.2 to 6.7.5 for explanation.
- Peat to be sub-excavated and replaced with Granular 'B' Type II or rock fill as per Section 6.8.
- Embankment not stable.
- Refer to Section 6.7.5.1 and 6.7.5.2.
- Meets settlement criteria for  $\sigma_p' = 100$  kPa design line. Conservatively, using the lower range design line  $67\text{kPa} < \sigma_p' < 85$  kPa, based on the laboratory consolidation test results the settlement criteria would be exceeded. See Section 6.7.3.5.
- Does not meet settlement criteria.



## REVISED FOUNDATION REPORT - WHITE CLAY RIVER BRIDGE REPLACEMENT HIGHWAY 11, GWP 5239-06-00

**Table 3: Evaluation of Stability/Settlement Mitigation Alternatives**

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Staged Construction with Preloading (combined with EPS)	1	<ul style="list-style-type: none"> <li>Standard construction operations.</li> <li>Stability improved by slowing construction process and allowing pore pressures to dissipate in varved clayey silt to silty clay deposit.</li> <li>Reduces post-construction settlement.</li> </ul>	<ul style="list-style-type: none"> <li>Operational constraints required in the contract to ensure peat sub-excavation is backfilled immediately as the embankment is raised and to detail staged construction sequence.</li> <li>Primary and secondary consolidation settlement will still occur and may be too large to meet criteria and another mitigation option will be required.</li> <li>Settlement monitoring required.</li> <li>Wait period required between stages to allow for pore pressure dissipation and corresponding strength increase in subsoils.</li> </ul>	<ul style="list-style-type: none"> <li>Cost of lengthening schedule to accommodate construction sequencing.</li> <li>Cost of supervision and monitoring.</li> <li>Additional cost of EPS as replacement for limited design thickness of earth fill.</li> </ul>	<ul style="list-style-type: none"> <li>Risk of contractor not following sequencing properly leading to stability problems.</li> <li>Risk of not achieving sufficient strength gain of underlying soils to allow embankment construction to final grade.</li> <li>Risk of schedule delays if monitoring results indicate a longer preload period is required.</li> <li>Primary and secondary settlement will occur.</li> </ul>
Lightweight (EPS) Fill (combined with staged construction/preloading)	2	<ul style="list-style-type: none"> <li>Reduces load on compressible subsoils thereby improving stability and reducing post-construction settlement of foundation subsoils.</li> <li>Reduced differential settlement.</li> <li>Shorter periods for preloading/staged construction.</li> </ul>	<ul style="list-style-type: none"> <li>Greater volume and high cost for EPS material.</li> <li>Staged construction and one preload period still required for stability and to reduce overall post-construction settlement.</li> <li>Restricted use within embankment as it has to be placed above the groundwater/river water level and requires concrete slab and minimum 1 m of conventional soil cover to mitigate potential for differential icing and buoyancy at high water level.</li> </ul>	<ul style="list-style-type: none"> <li>Greater volume of EPS and cost of EPS being an order of magnitude higher than other fill materials will result in very high construction costs.</li> </ul>	<ul style="list-style-type: none"> <li>Risk of buoyancy of EPS at high water level.</li> </ul>
Change Alignment of Bridge (i.e. new bridge on existing alignment, with minimal widening)	3	<ul style="list-style-type: none"> <li>Magnitude of settlement reduced.</li> <li>Stability improved/maintained.</li> <li>Differential settlement reduced.</li> </ul>	<ul style="list-style-type: none"> <li>Would require a detour bridge/embankment during construction or alternatively would require an alternate bypass during new bridge construction.</li> </ul>	<ul style="list-style-type: none"> <li>Decrease in cost as new additional fill is not required.</li> </ul>	<ul style="list-style-type: none"> <li>Risk of impacting existing bridge/traffic flow due to construction staging.</li> </ul>



**REVISED FOUNDATION REPORT - WHITE CLAY RIVER BRIDGE REPLACEMENT  
HIGHWAY 11, GWP 5239-06-00**

**Table 3: Evaluation of Stability/Settlement Mitigation Alternatives (Continued)**

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Surcharging (and Toe Berms)	NF	<ul style="list-style-type: none"> <li>■ Surcharging reduces time period for long-term consolidation of foundation soils.</li> <li>■ May be combined with staged construction to reduce size of toe berms.</li> <li>■ Reduces post-construction settlement.</li> </ul>	<ul style="list-style-type: none"> <li>■ Large toe berms will be required to maintain stability.</li> <li>■ Toe berms will extend further into river beyond proposed embankments causing environmental issues.</li> <li>■ Toe berms would have to extend on the front slope as well and may impact the pier locations and cause hydraulic and navigational issues.</li> <li>■ Potential for negative impact to the existing piles and approach embankment with existing bridge in operation during construction.</li> <li>■ Would require settlement/stability monitoring.</li> </ul>	<ul style="list-style-type: none"> <li>■ Additional costs for placement and removal of surcharge and for toe berms.</li> <li>■ Additional costs required for instrumentation and monitoring.</li> </ul>	<ul style="list-style-type: none"> <li>■ Risk of embankment instability due to addition of surcharge load.</li> <li>■ Risk to stability and performance of the existing bridge and road embankment.</li> </ul>
Wick Drains	NF	<ul style="list-style-type: none"> <li>■ Decreases the time required for settlement to occur, and may be combined with surcharging.</li> <li>■ Strength gain in foundation soils due to pore water pressure dissipation.</li> </ul>	<ul style="list-style-type: none"> <li>■ Primary and secondary consolidation settlement will still occur.</li> <li>■ Settlement monitoring required.</li> <li>■ Since clay is already varved, wick drains may not significantly reduce the drainage path to allow settlement to occur more quickly.</li> </ul>	<ul style="list-style-type: none"> <li>■ Increased cost of design (additional subsurface investigation), installation and monitoring.</li> </ul>	<ul style="list-style-type: none"> <li>■ Primary and secondary settlement will occur.</li> </ul>
Sub-excavation of Soft Clay (to about 11.7 m below July 2008 river water level)	NF	<ul style="list-style-type: none"> <li>■ Improve stability and reduce settlement due to removal of soft material.</li> </ul>	<ul style="list-style-type: none"> <li>■ Extensive shoring system would be required for depth of excavation if existing structure is to remain in operation during construction.</li> <li>■ Potential for negative impact to the existing piles and approach embankment with existing bridge in operation during construction.</li> <li>■ Requires disposal of large volume of soil and replacement with large volume of imported granular material or rock fill.</li> <li>■ Extensive excavation under water potentially leading to inability to assess extent of clay removal, surface water impacts by sediment locally.</li> </ul>	<ul style="list-style-type: none"> <li>■ Additional costs for sub-excavation, extra fill materials and disposal of excavated material.</li> <li>■ Additional costs for extensive temporary shoring protection system to support existing embankment and structure.</li> </ul>	<ul style="list-style-type: none"> <li>■ Low risk with respect to stability and long-term settlement of new embankment.</li> <li>■ Risk of impacting stability and performance of existing bridge and roadway embankment during new construction operation.</li> </ul>

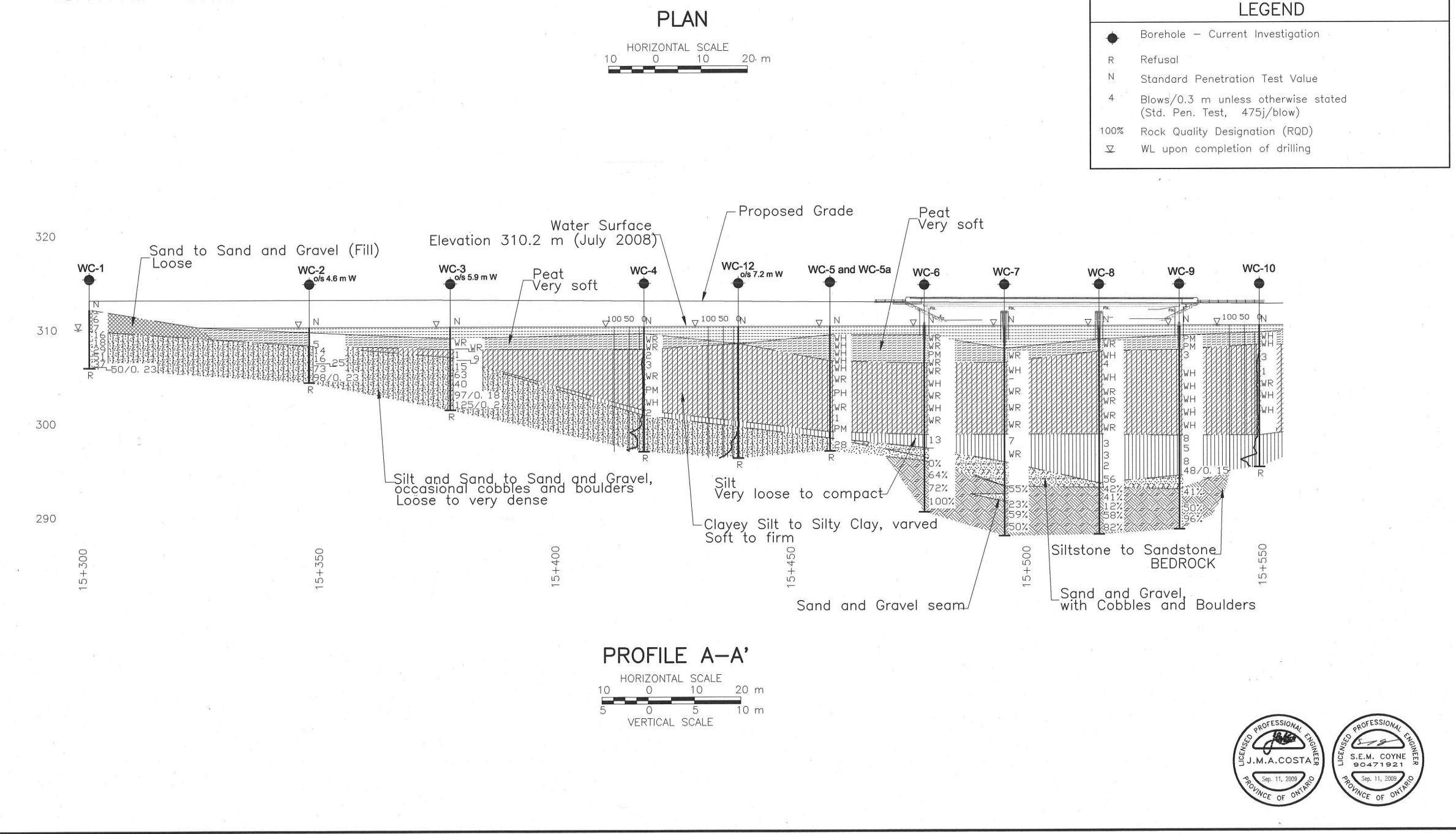
NF indicates that the alternative is not technically feasible or is impractical.



**REVISED FOUNDATION REPORT - WHITE CLAY RIVER BRIDGE REPLACEMENT  
HIGHWAY 11, GWP 5239-06-00**

**Table 4: Staged Construction Details and Sequence**

Stage	Recommended Construction Sequence (North and South Approaches)	Reference Figure Example for Stability FS>1.3/Comments
1	Remove peat adjacent to existing embankment and backfill in strips no wider than 3 m or areas no larger than 3 m by 3 m up to Elevation 310.5 m (0.3 m above water level) using Granular 'B' Type II or rock fill as indicated on the Contract Drawings.	---
2	Install instrumentation.	---
3	Construct the South Approach embankment to Elev. 312.0 m and the North Approach embankment to Elev. 312.3 m using compacted Granular 'B' Type II or rock fill.	Figure 8a (North Approach STA 15+600)
4	After completion of Stage 3 construction along the full length of the embankment, allow for a minimum 6-month preload period (depending on the monitoring results) for dissipation of pore pressures and consolidation of the foundation soils. No other construction activity (pile installation, abutment construction or construction traffic) can proceed during the preload period.	---
<b>5 Recommended</b>	After the preload period, install EPS between STA 15+385 and 15+468 (South Approach) and between STA 15+523 and 15+585 (North Approach). Construct embankments to final grade (Elev. 313.0 m to 313.4 m on South Approach and Elev. 312.2 m to 312.7 m on North Approach). All other construction activities may resume after the preload period is complete.	Figure 3b (South Approach STA 15+435) Figure 4b (South Abutment STA 15+468) Figure 5b (South Abutment Front Slope) Figure 6b (North Abutment Front Slope) Figure 7b (North Abutment STA 15+523)
5A (assessment of 2 <sup>nd</sup> preload period)	After the preload period (Stage 4), the actual duration of which will be determined by the results of the monitoring, construct embankments to final grade south of STA 15+443 (South Approach – final grade between Elev. 313.0 m and 313.4 m) and north of STA 15+543 (North Approach – final grade between Elev. 312.3 m and 312.7 m). Install EPS in the south and north abutment areas (i.e. north of STA 15+443 and south of STA 15+543).	FoS < 1.3 (not stable) and/or does not meet post-construction settlement criteria outside EPS zone.
6A (assessment of 2 <sup>nd</sup> preload period)	Allow for a second 6-month preload period (the actual duration of which will be determined by the results of the monitoring) for dissipation of pore pressures and consolidation of the foundation soils. This preload period applies to the embankment south of STA 15+443 (South Approach) and north of STA 15+543 (North Approach). Other construction activities (except paving and installation of guide rail) can proceed during this preload period.	





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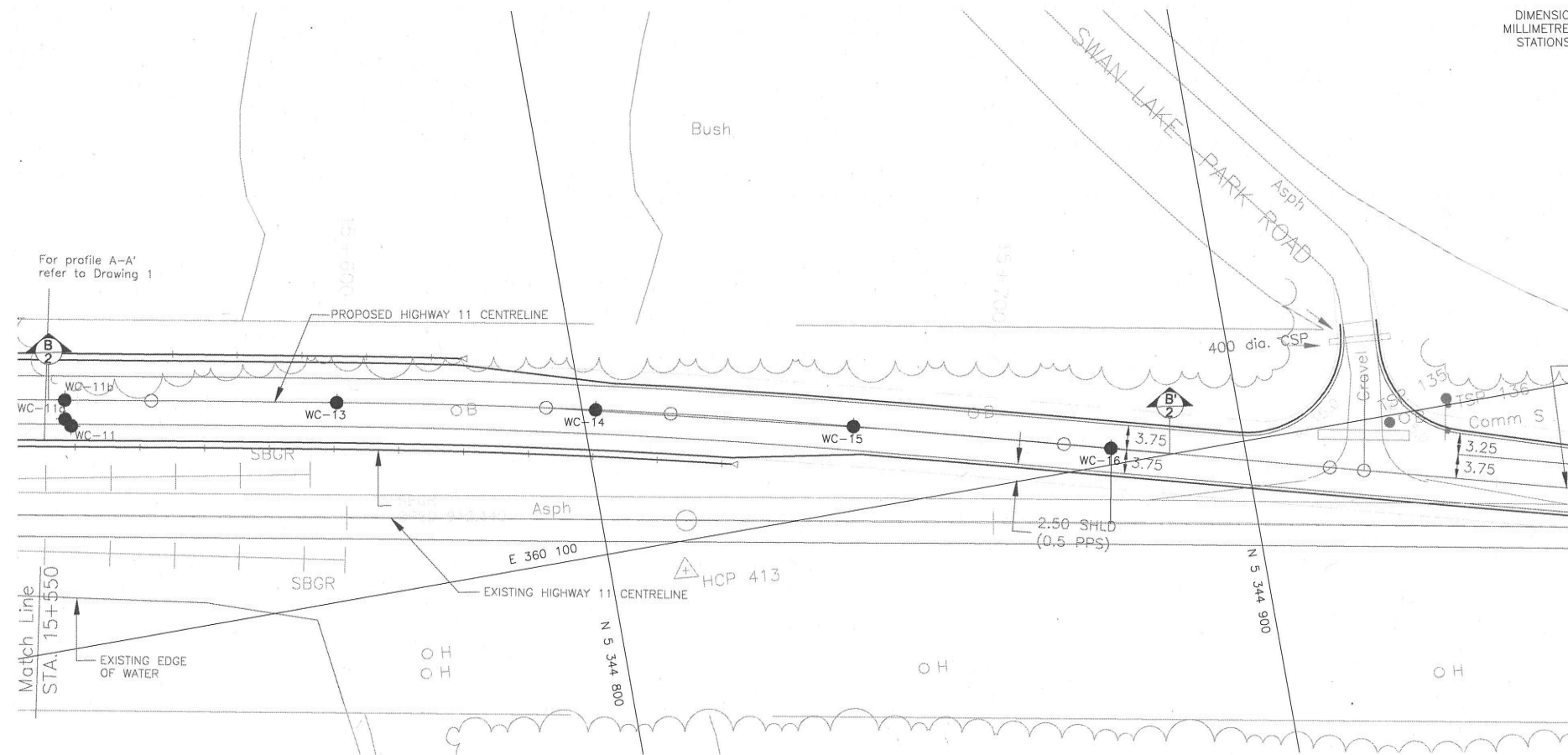
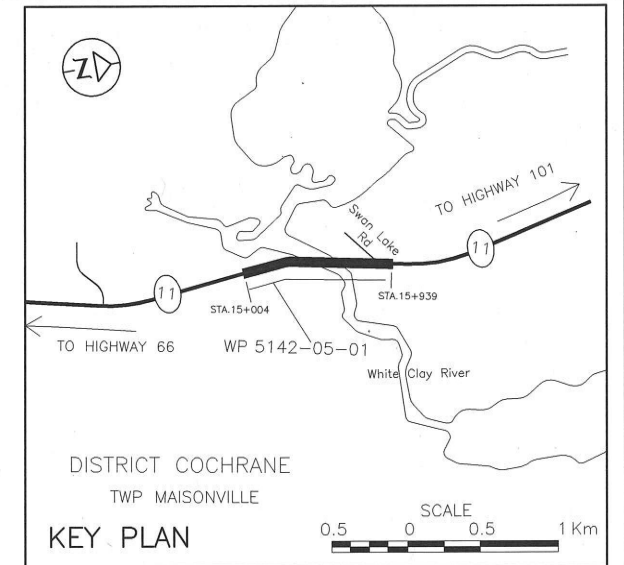


HIGHWAY 11 CROSSING  
WHITE CLAY RIVER  
STA 15+550 TO 15+725  
BOREHOLE LOCATION  
AND SOIL STRATA

SHEET

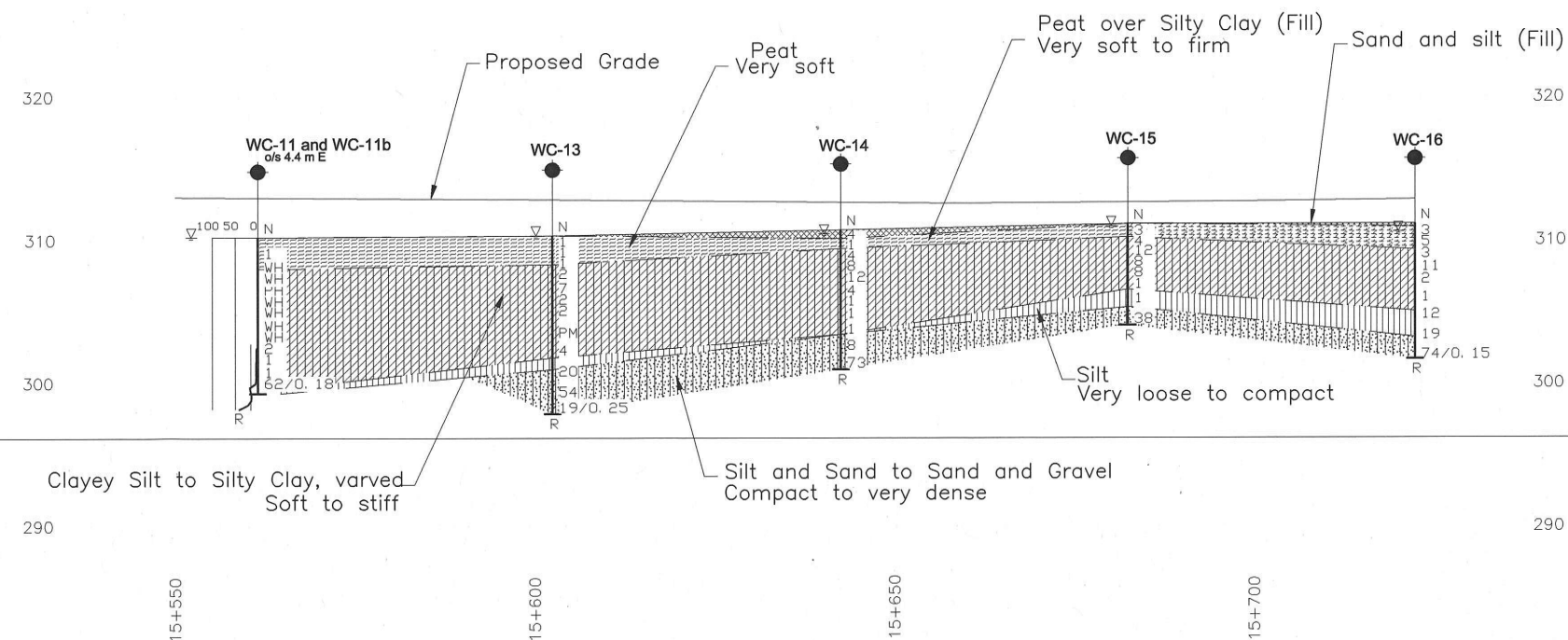


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SUDBURY, ONTARIO, CANADA



PLAN

HORIZONTAL SCALE  
10 0 10 20 m



PROFILE B-B'

HORIZONTAL SCALE  
10 0 10 20 m  
VERTICAL SCALE  
5 0 5 10 m

### LEGEND

- Borehole - Current Investigation
- R Refusal
- N Standard Penetration Test Value
- 4 Blows/0.3 m unless otherwise stated  
(Std. Pen. Test, 475j/blow)
- 100% Rock Quality Designation (RQD)
- ▽ WL upon completion of drilling

No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
WC-11	320.2	5344721.9	360065.7
WC-11a	320.2	5344721.1	360064.5
WC-11b	320.2	5344721.6	360061.5
WC-13	310.3	5344762.9	360069.3
WC-14	310.7	5344802.1	360077.5
WC-15	311.1	5344840.9	360087.1
WC-16	311.1	5344879.5	360097.4

### NOTES

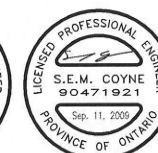
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

### REFERENCE

Base plans provided in digital format by LEA, drawing file name 2007-032-White Clay-Recommended Plan.dwg dated May, 2008 and received September 9, 2008 and Key Plan.dwg received December, 2008.



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WP No. 5239-06-00

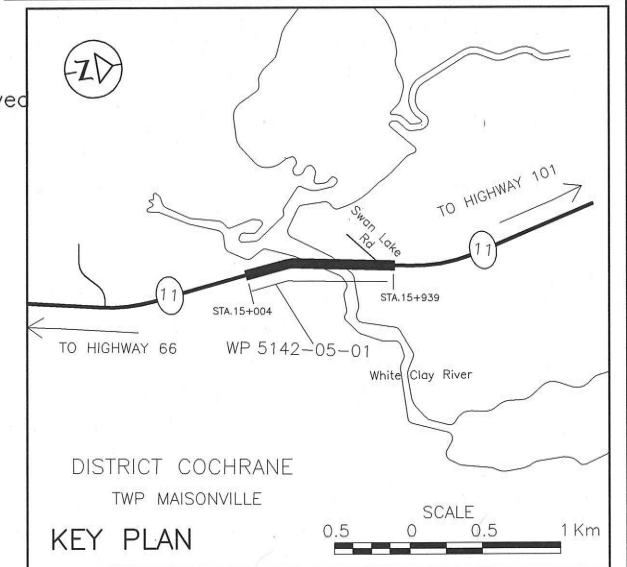


HIGHWAY 11 CROSSING  
WHITE CLAY RIVER  
SOIL STRATA

SHEET



**Golder Associates Ltd.**  
SUDBURY, ONTARIO, CANADA



#### LEGEND

- Borehole - Current Investigation
- R Refusal
- N Standard Penetration Test Value
- 4 Blows/0.3 m unless otherwise stated (Std. Pen. Test, 475j/blow)
- 100% Rock Quality Designation (RQD)
- ∇ WL upon completion of drilling

#### NOTES

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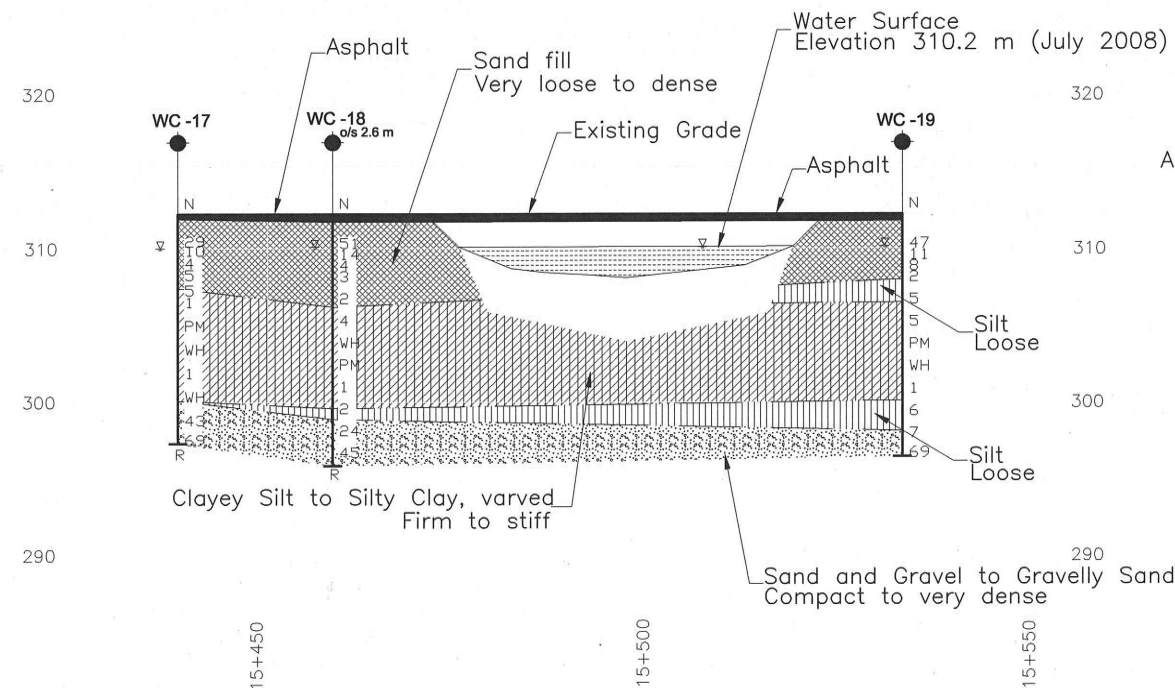
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#### REFERENCE

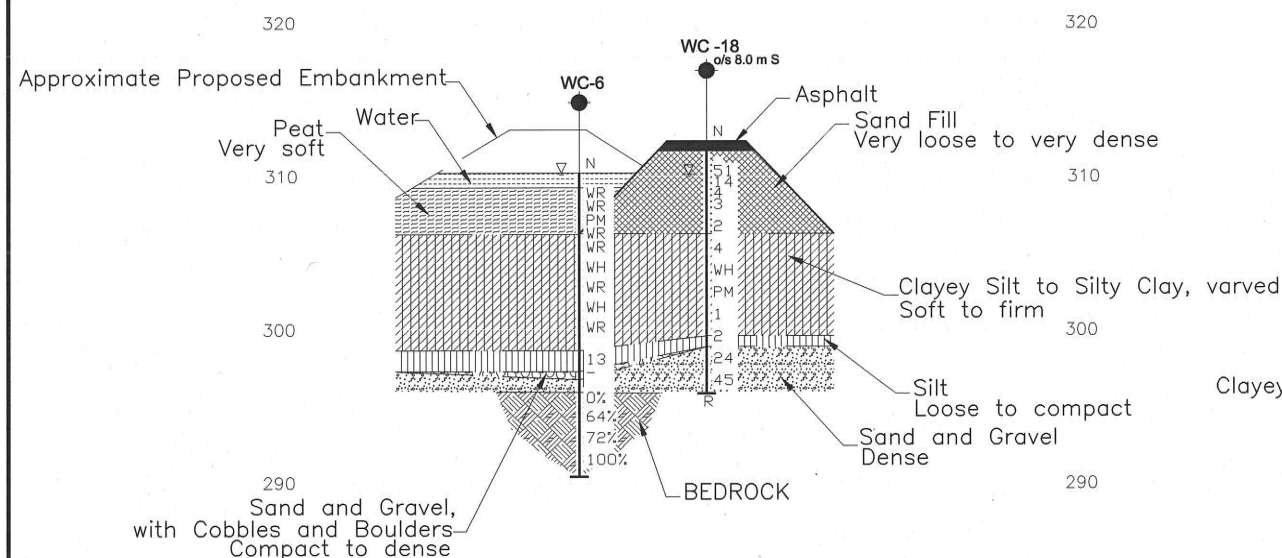
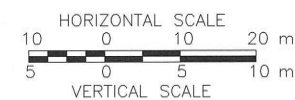
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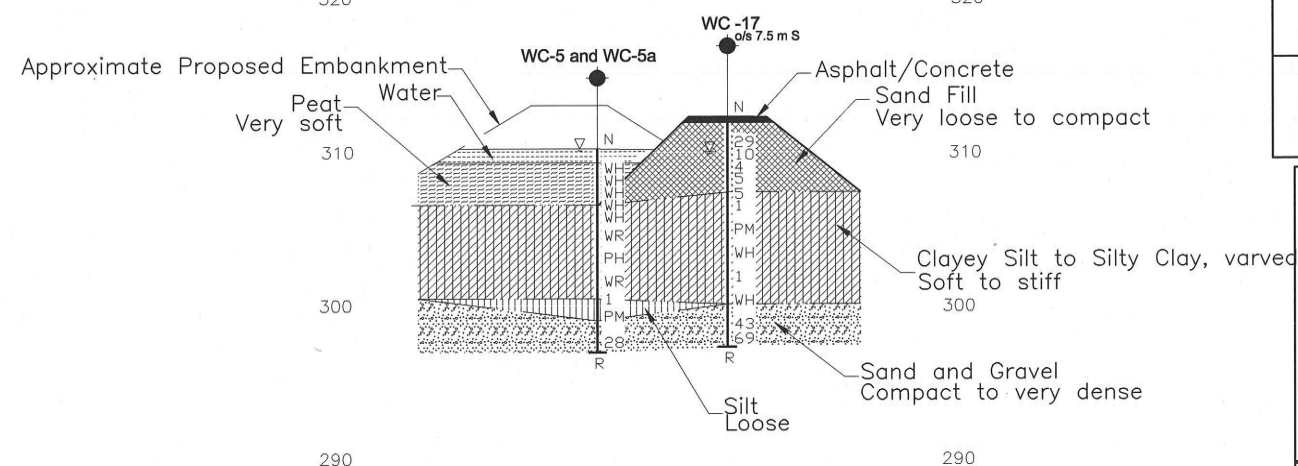
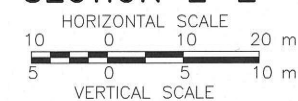
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SUBM'D. AB	CHKD. AB	DATE: Sept. 2009	SITE: 47-005
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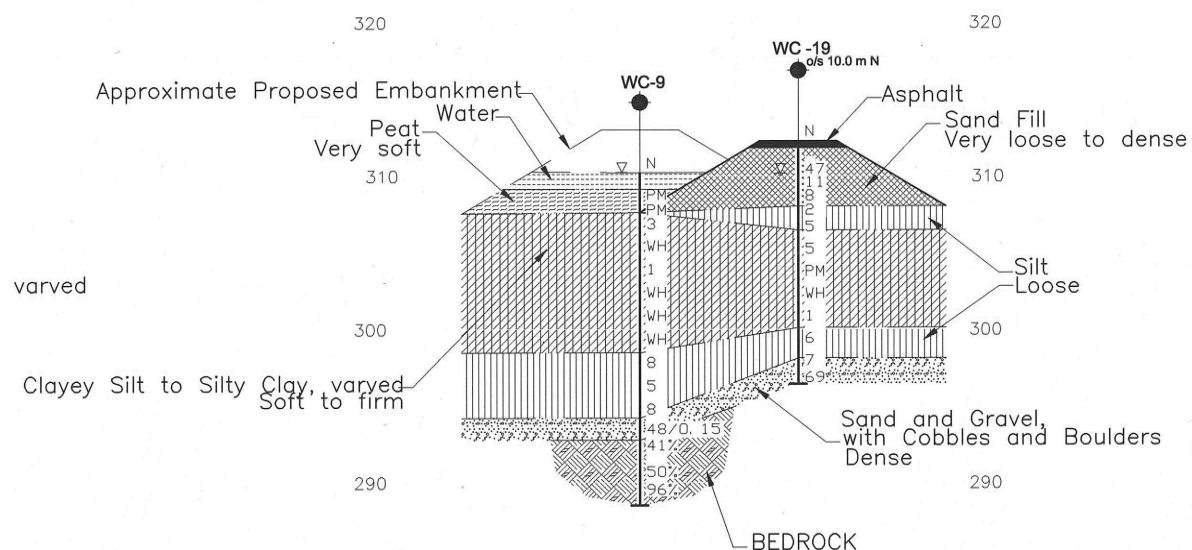
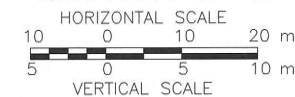
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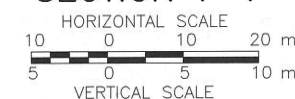
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SECTION D-D'



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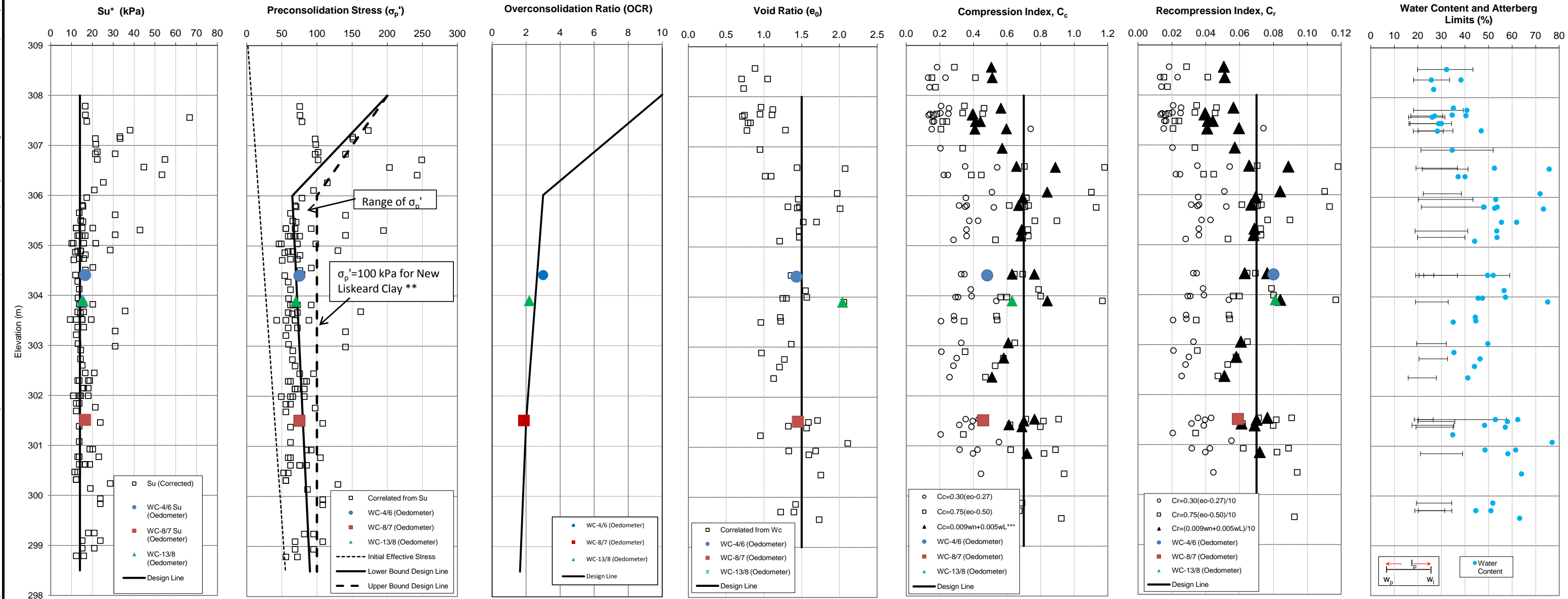




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GEOTECHNICAL ENGINEERING PARAMETER SUMMARY  
PROPOSED EMBANKMENTS  
White Clay River Bridge Replacement

FIGURE 1



\* Corrected for plasticity (Bjerrum, 1973) and for presence of varves (Milligan and Lo, 1967). See Section 6.7.2.2.

\*\* Milligan and Lo (1967) and Milligan et al (1982).

\*\*\*  $C_c = 0.009w_n + 0.005w_L$  based on Koppula (1986).

Golder Associates

Date: October 2011  
Project No: 07-1191-0008

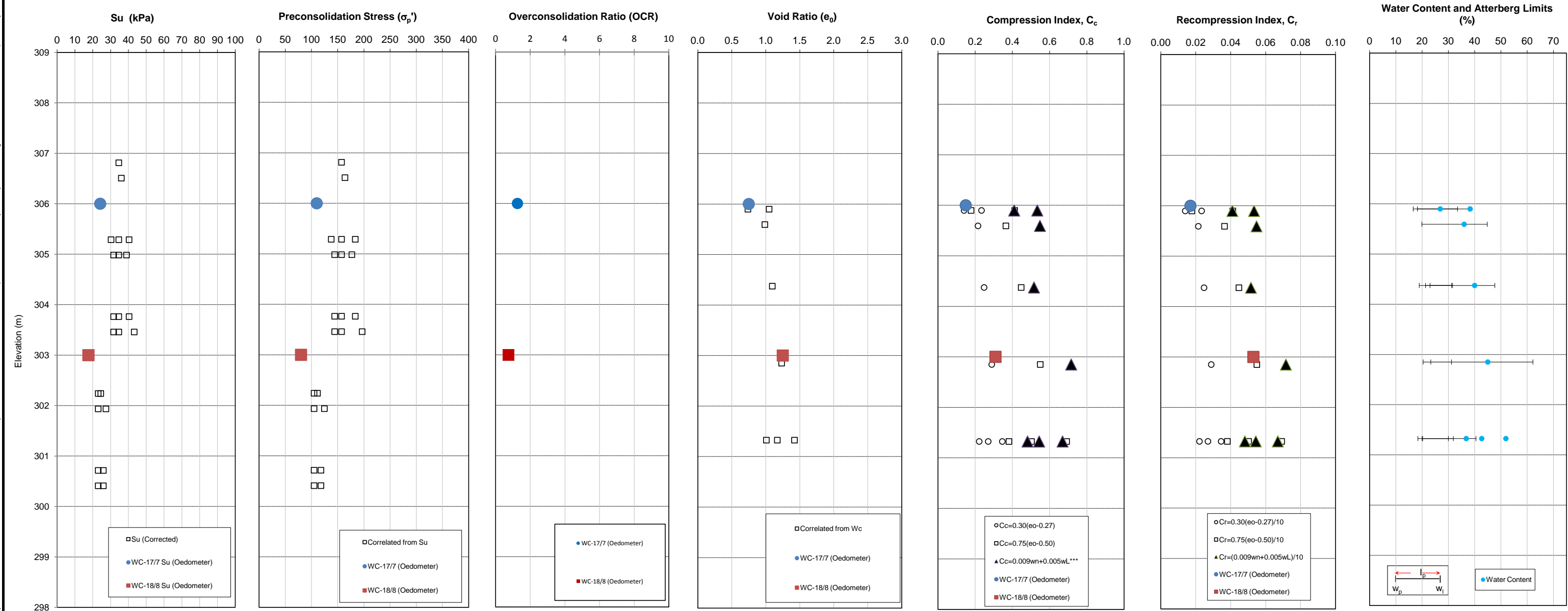
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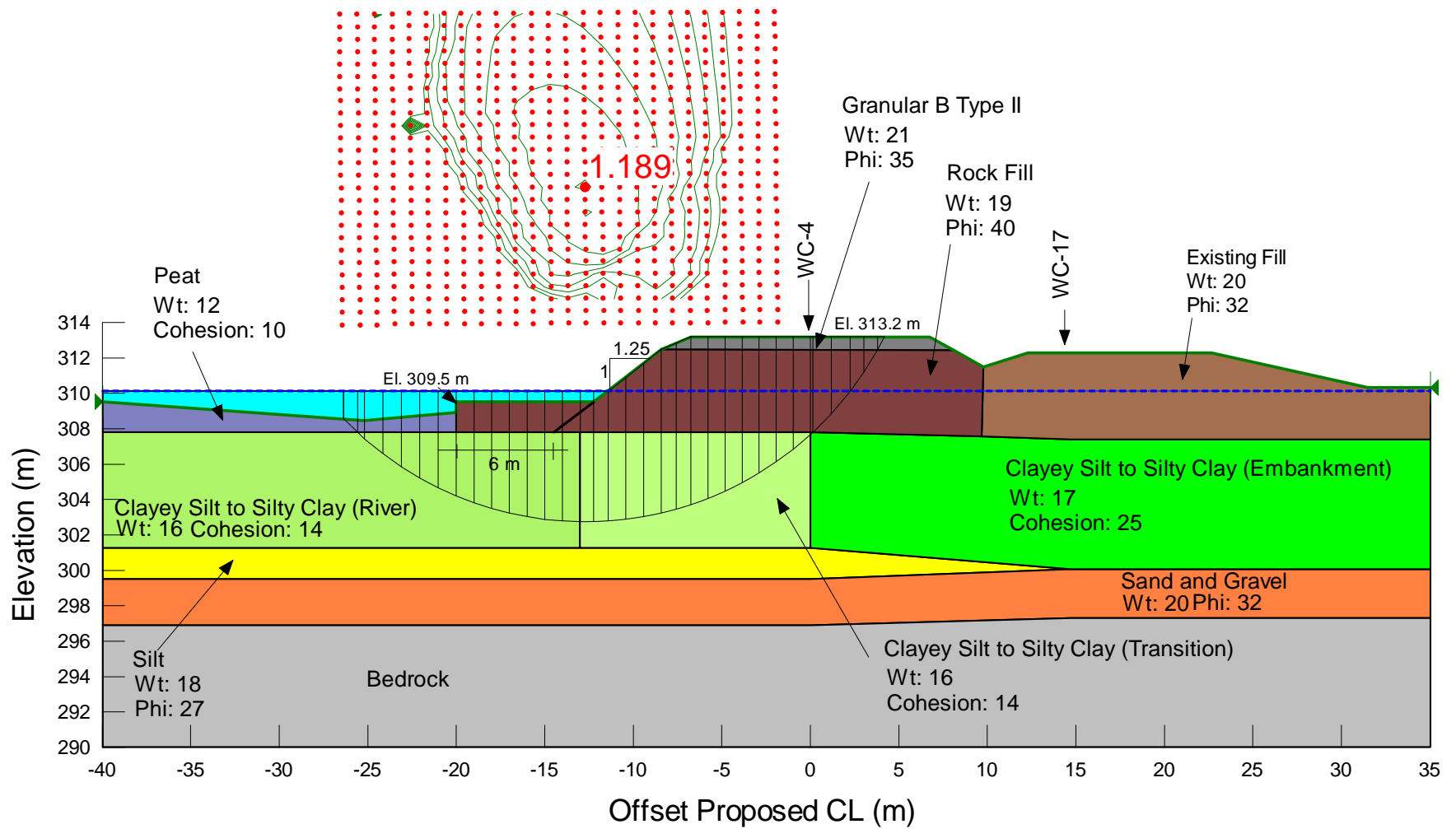
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GEOTECHNICAL ENGINEERING PARAMETER SUMMARY  
EXISTING EMBANKMENTS  
White Clay River Bridge Replacement

FIGURE 2



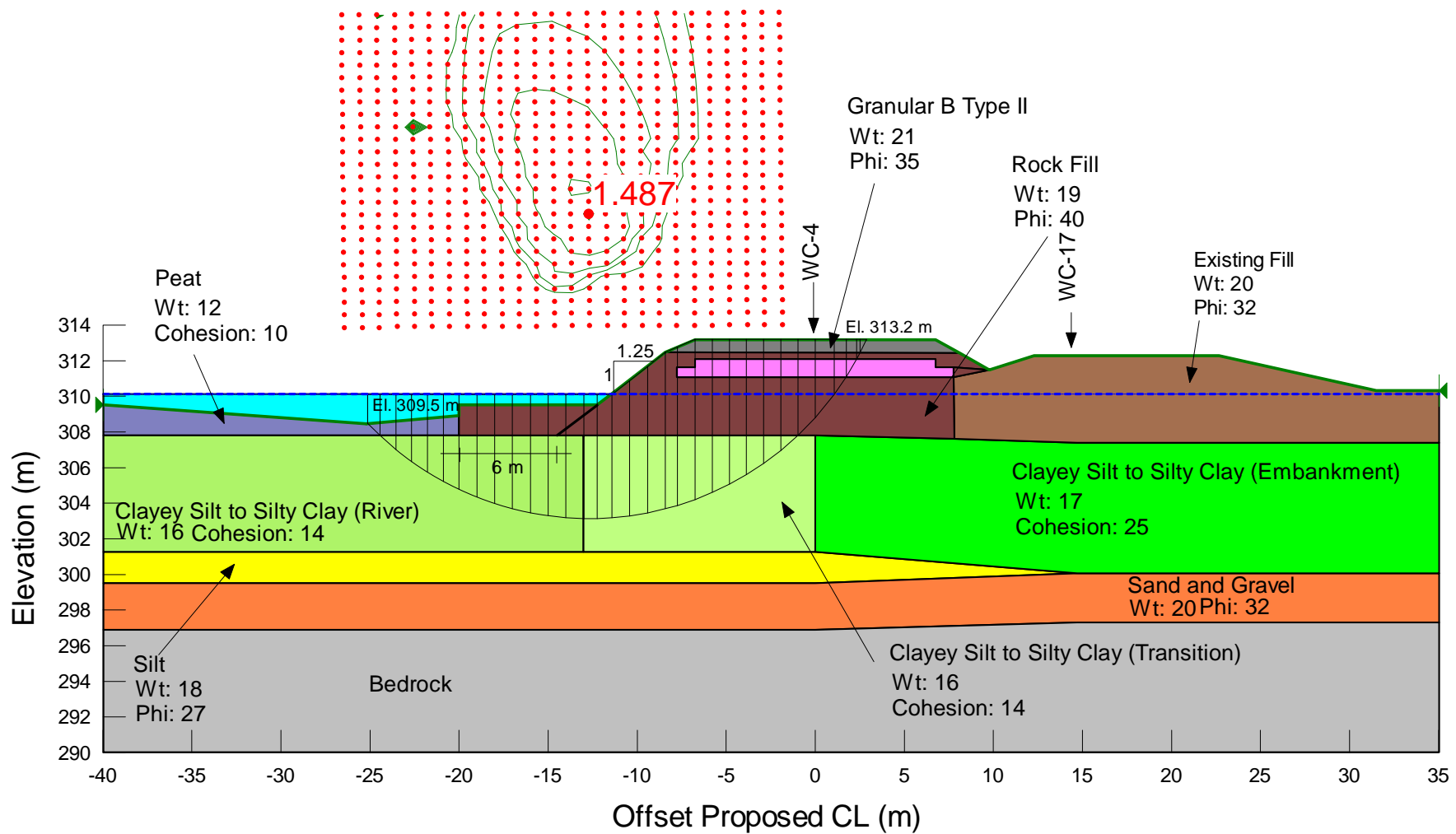
\*\*\* $C_c = 0.009w_n + 0.005w_L$  based on Koppula (1986).




PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE				STABILITY ANALYSIS SOUTH APPROACH STA 15+435 NO MITIGATION			
				PROJECT No. 07-1191-0008	FILE No. ----		
				DESIGN JJL	OCT. 2011	SCALE AS SHOWN	REV.
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				REVIEW JMAC	OCT. 2011		

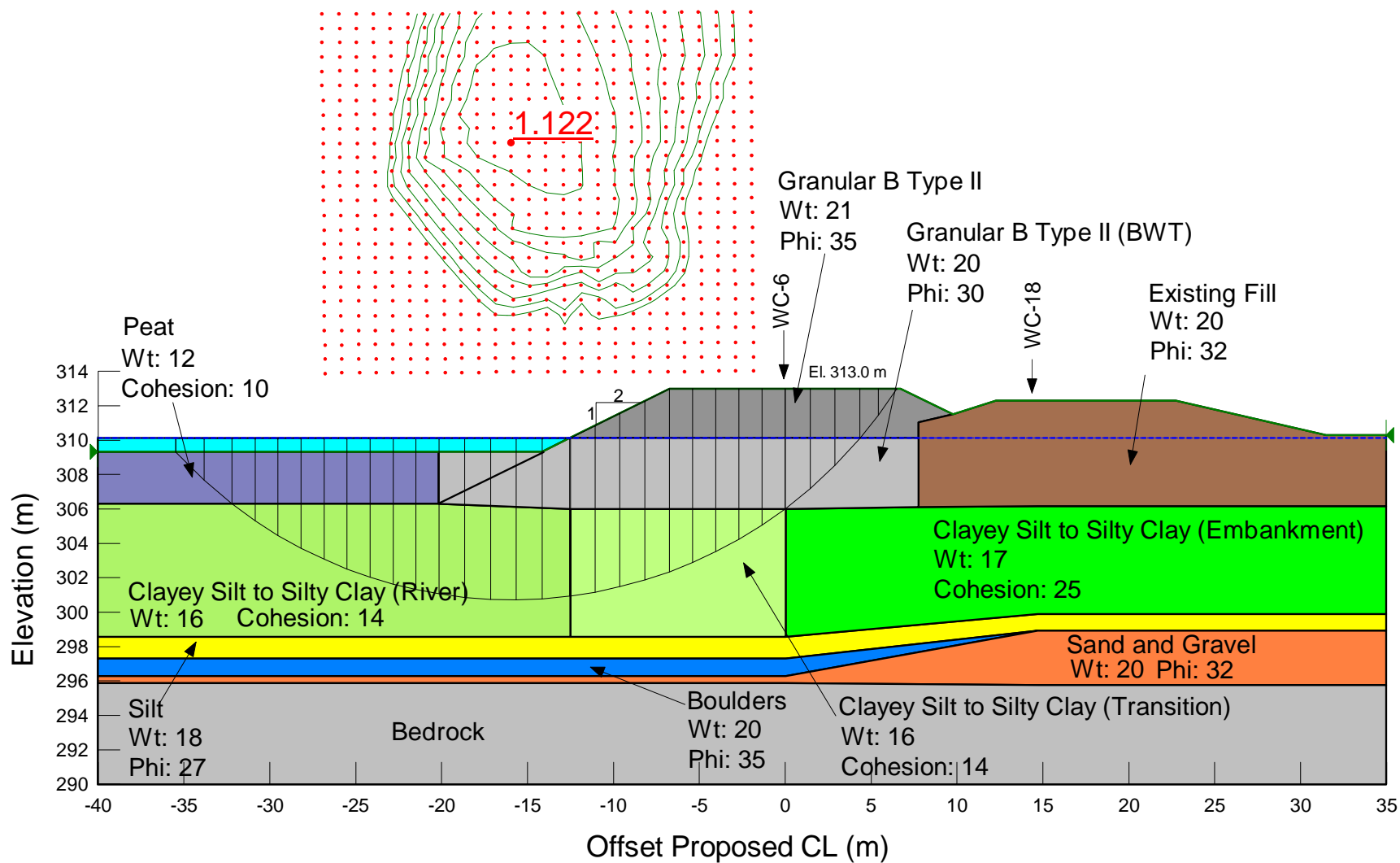


Figure 3a



PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE				STABILITY ANALYSIS SOUTH APPROACH STA 15+435 MITIGATION WITH EPS			
		PROJECT No. 07-1191-0008		FILE No. ----			
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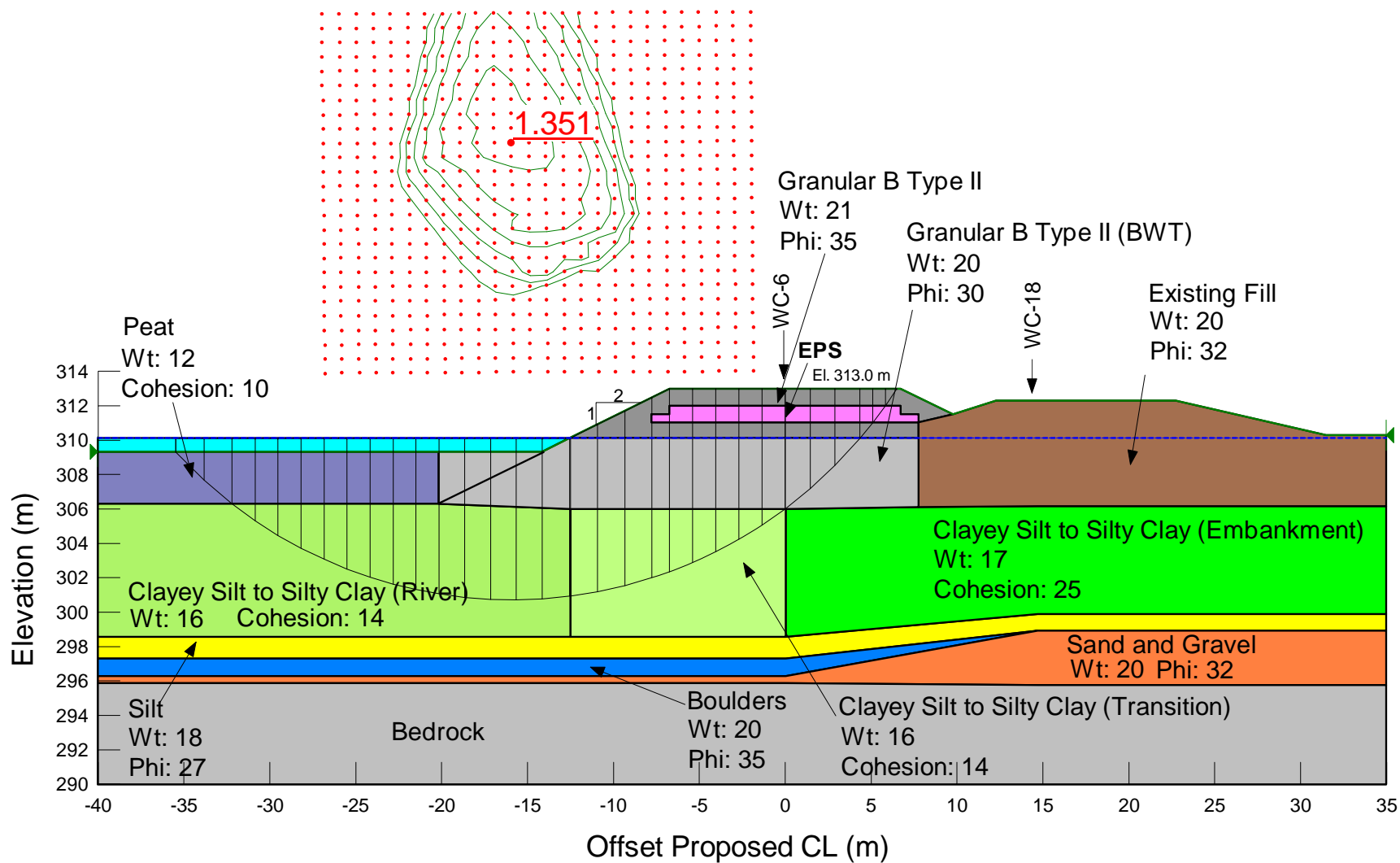
**Figure 3b**



PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
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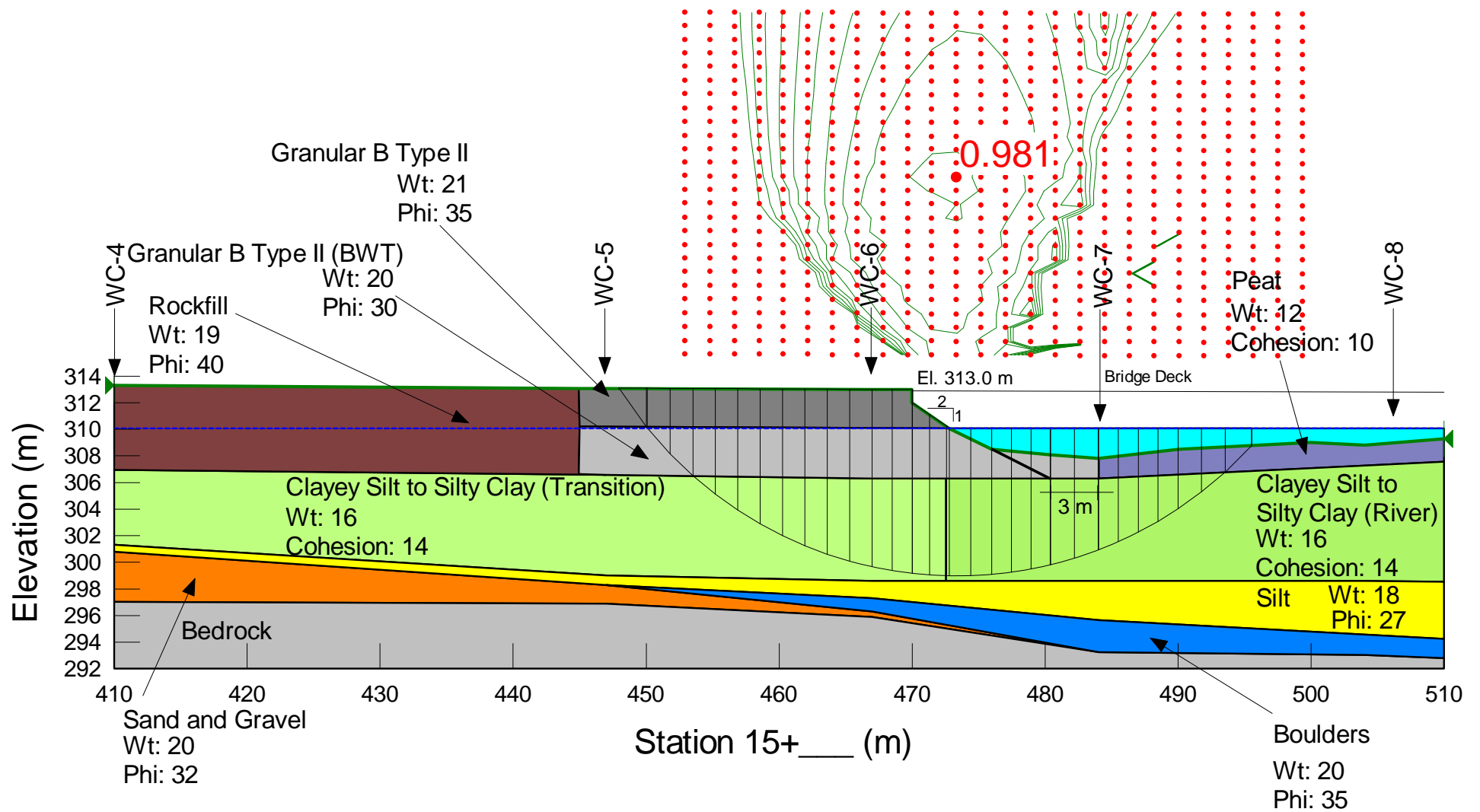
**Figure 4a**




PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
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PROJECT No. 07-1191-0008		FILE No. ----					
DESIGN	JJL	OCT. 2011	SCALE	AS SHOWN	REV.		
CADD	--						
CHECK	SEMC	OCT. 2011					
REVIEW	JMAC	OCT. 2011					



**Figure 4b**

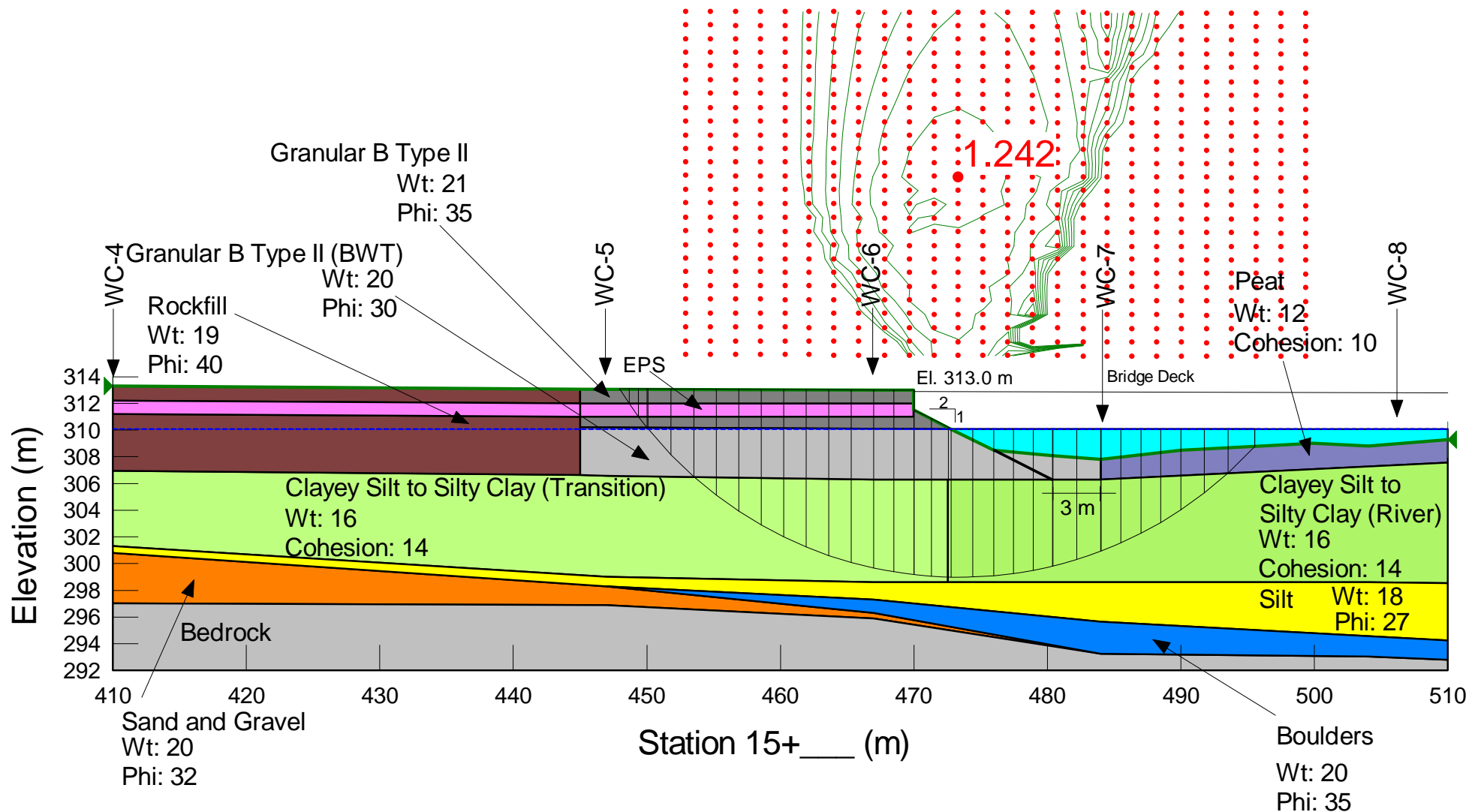


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		DESIGN	JJL	OCT. 2011	SCALE AS SHOWN REV.	
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		Figure 5a				



**Figure 5a**

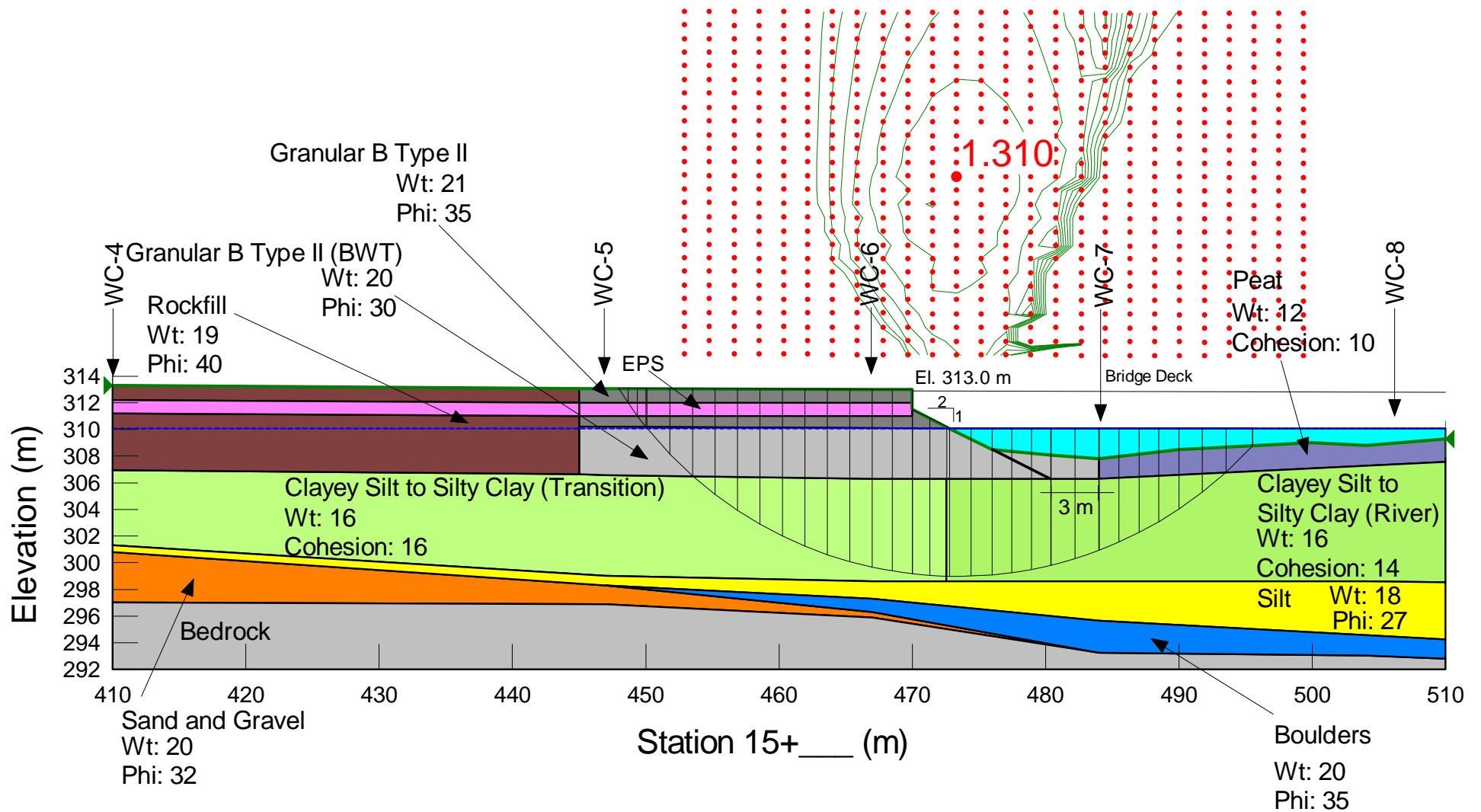





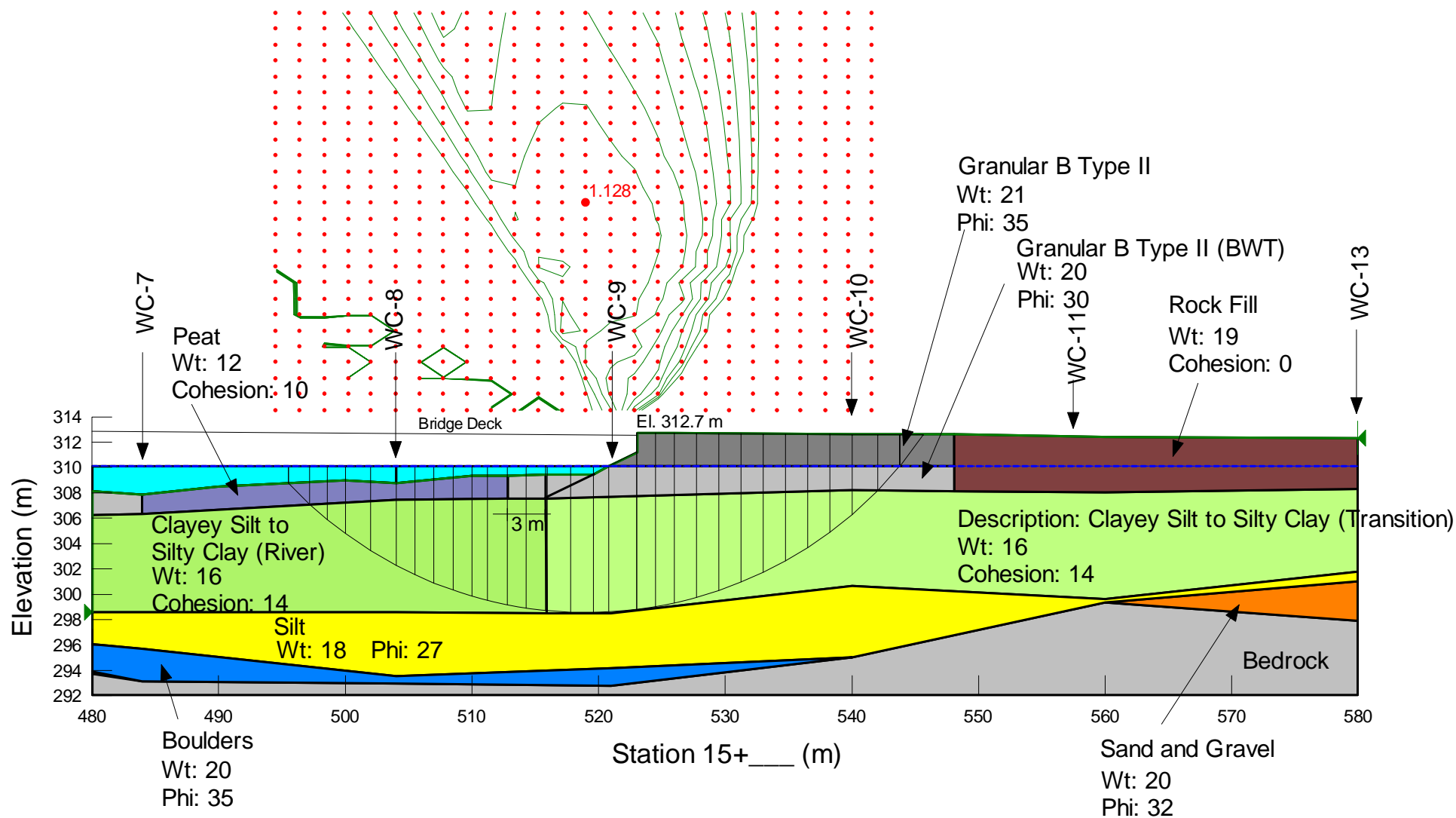
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REVIEW	JMAC	OCT. 2011	



**Figure 5b**



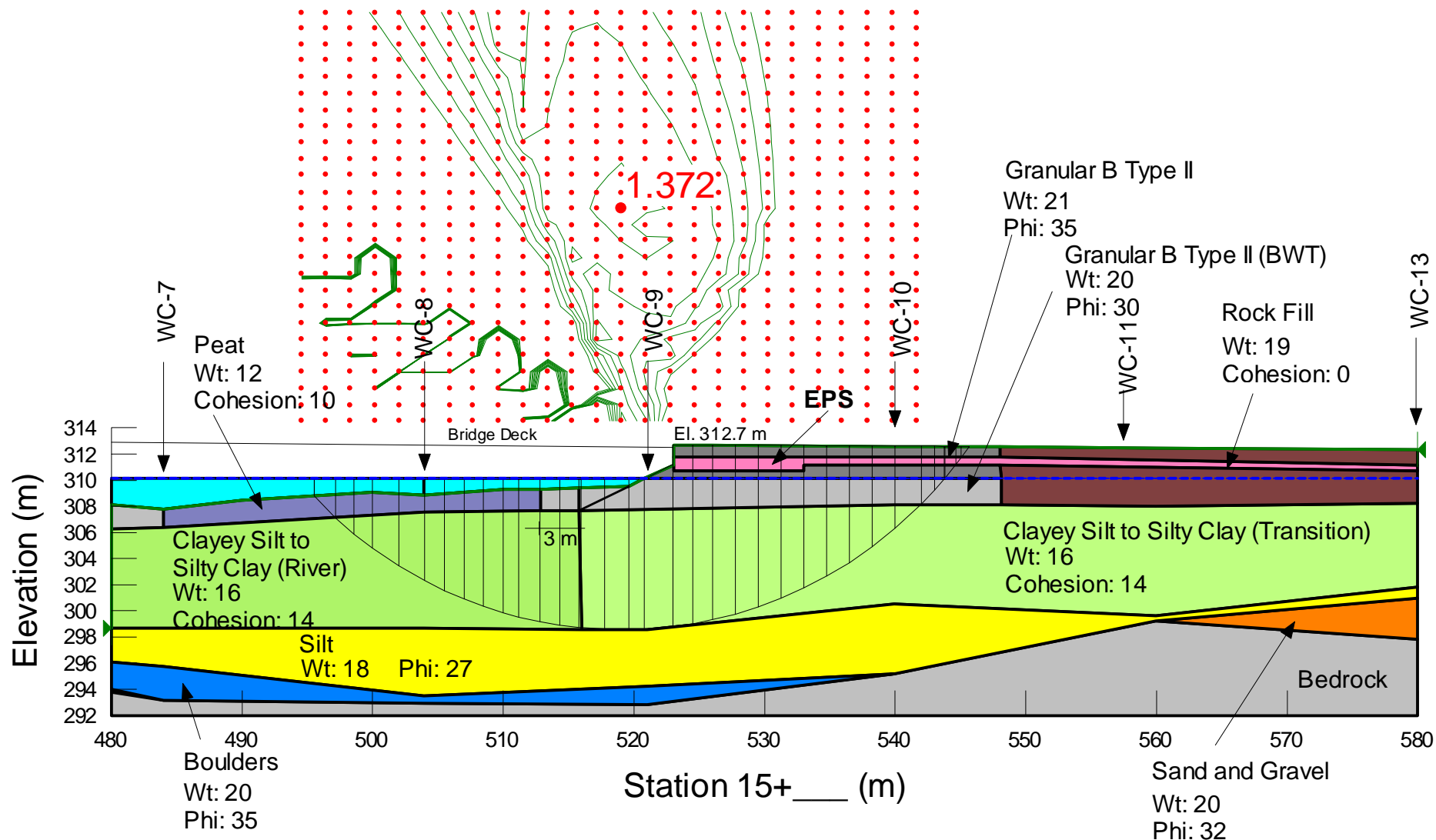
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DESIGN	JJL	OCT. 2011					
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CHECK	SEMC	OCT. 2011					
REVIEW	JMAC	OCT. 2011		Figure 5c			



PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE				STABILITY ANALYSIS NORTH ABUTMENT FRONT SLOPE NO MITIGATION			
PROJECT No. 07-1191-0008				FILE No. ----			
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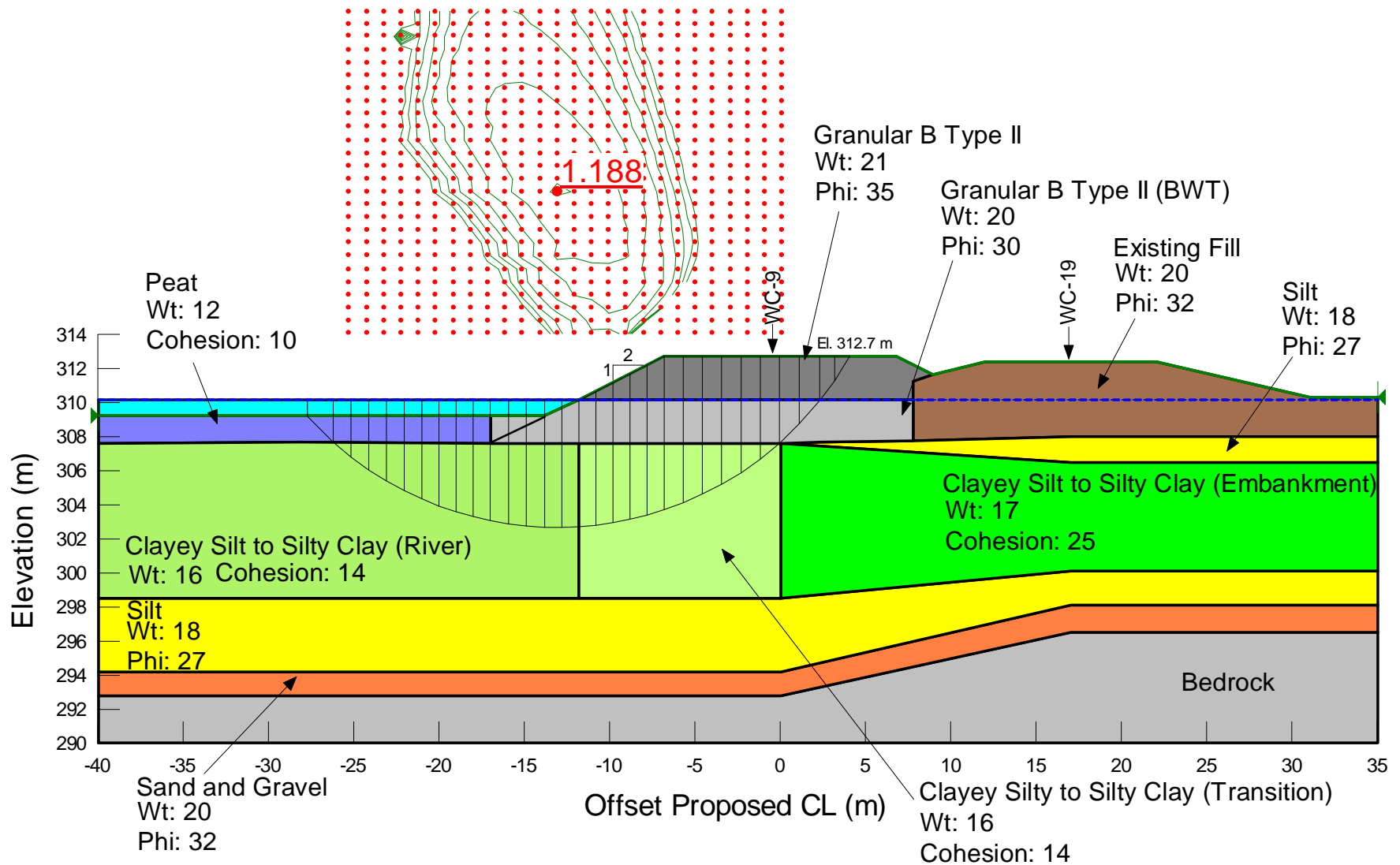
**Figure 6a**



PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE				STABILITY ANALYSIS NORTH ABUTMENT FRONT SLOPE MITIGATION WITH EPS			
PROJECT No. 07-1191-0008				FILE No. ----			
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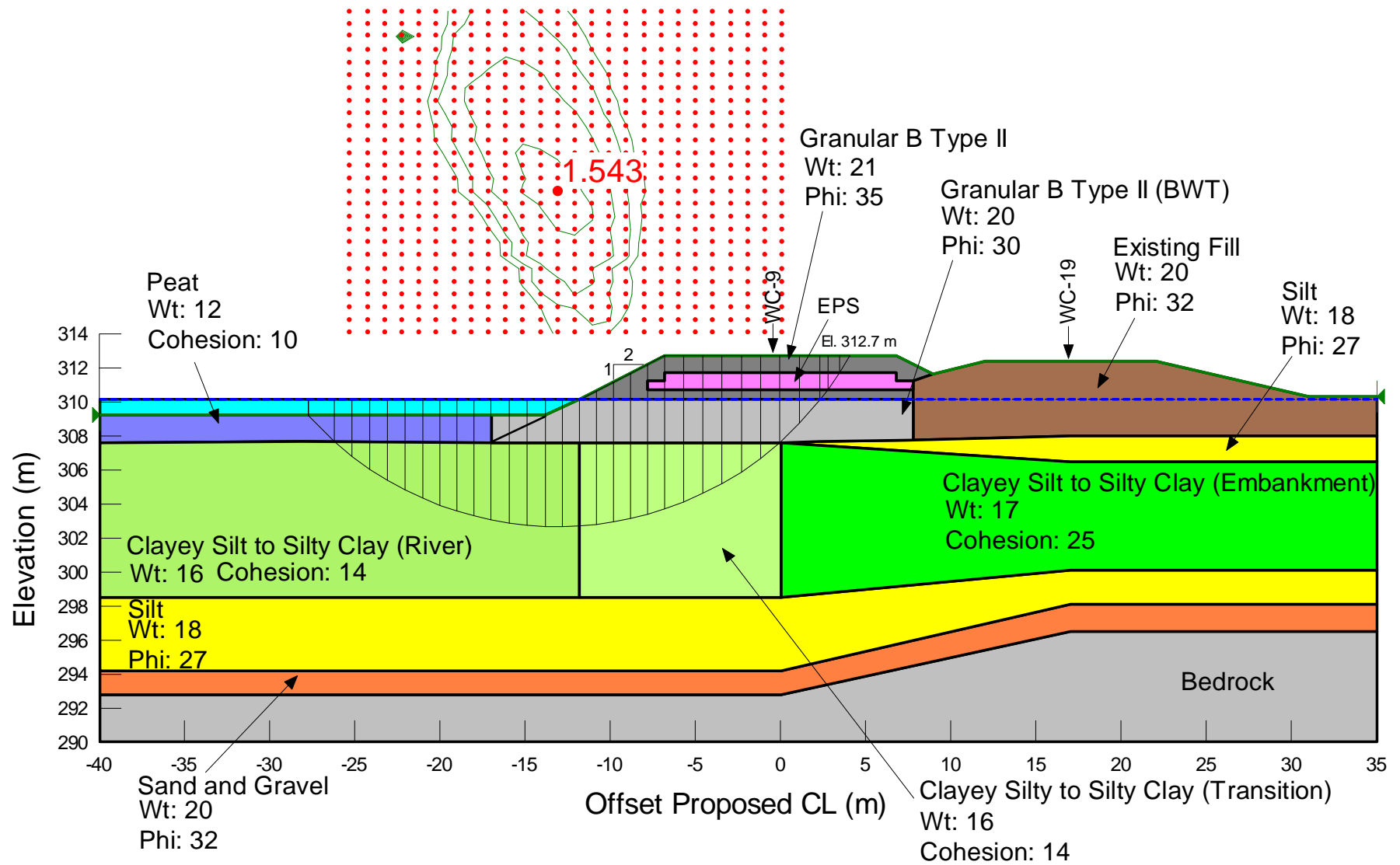
**Figure 6b**



PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE				STABILITY ANALYSIS NORTH ABUTMENT STA 15+523 NO MITIGATION			
PROJECT No. 07-1191-0008				FILE No. ----			
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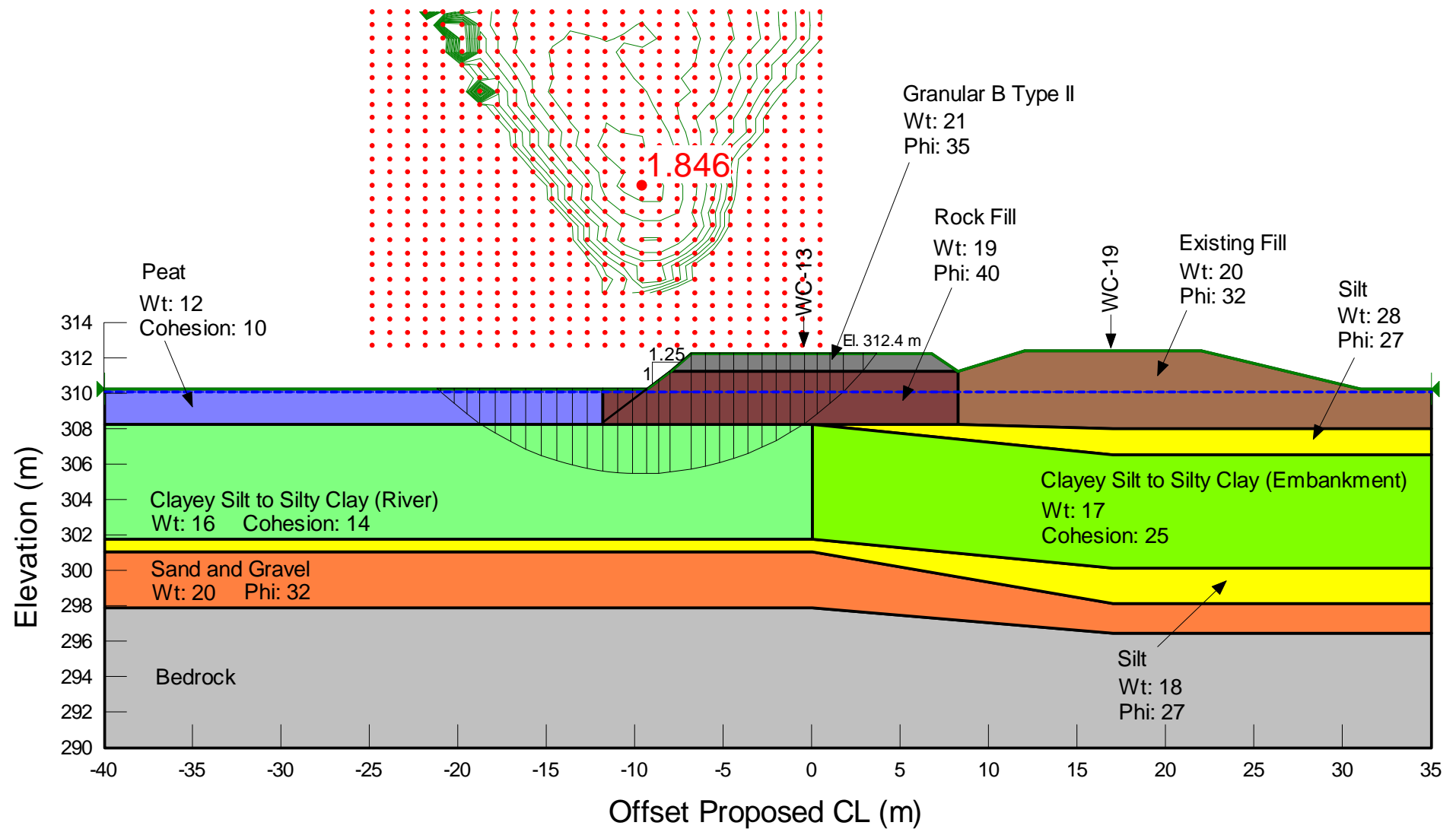
**Figure 7a**




PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
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PROJECT No. 07-1191-0008				FILE No. ----			
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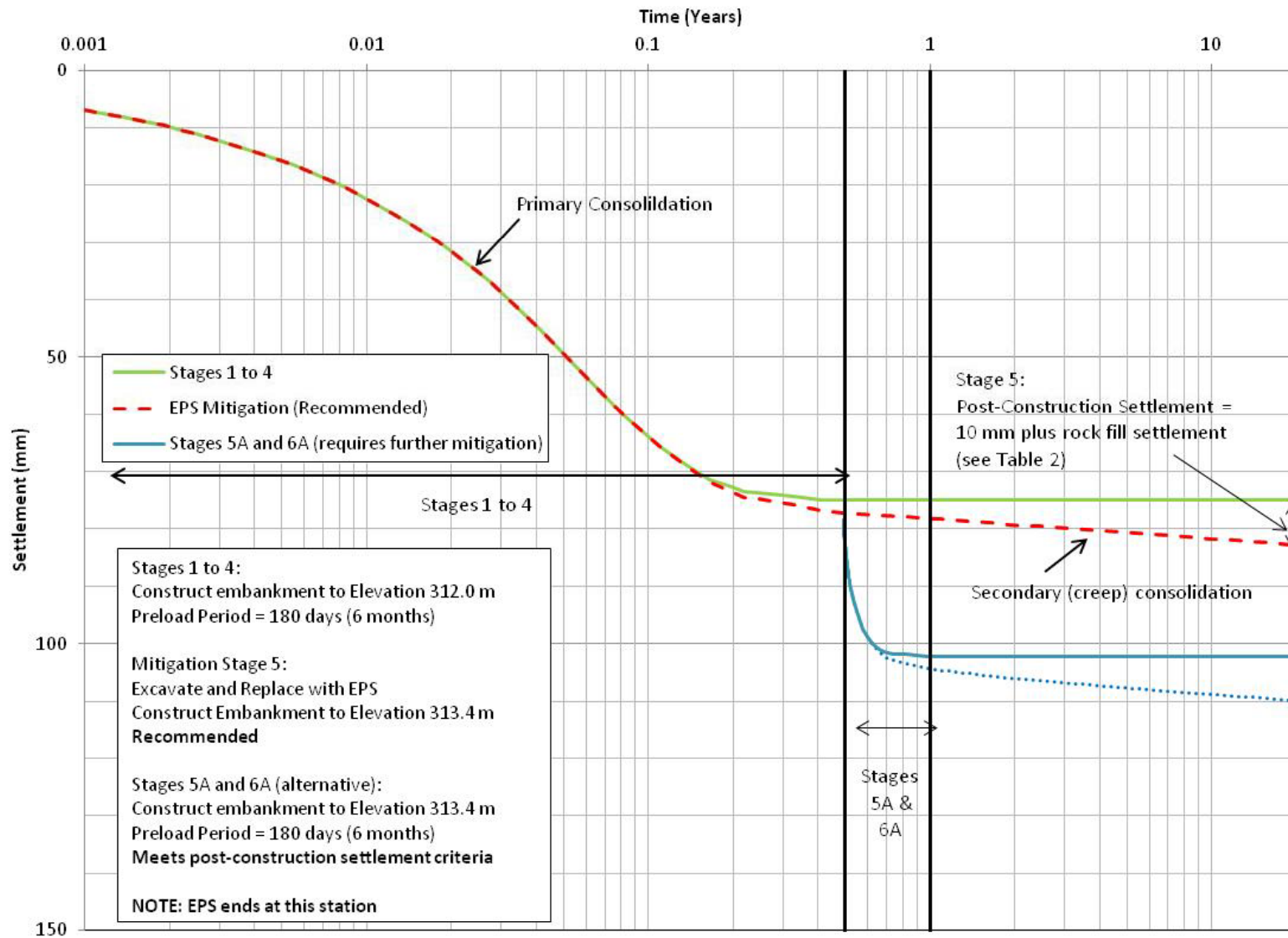
Figure 7b




PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE				STABILITY ANALYSIS NORTH APPROACH STA 15+600 NO MITIGATION			
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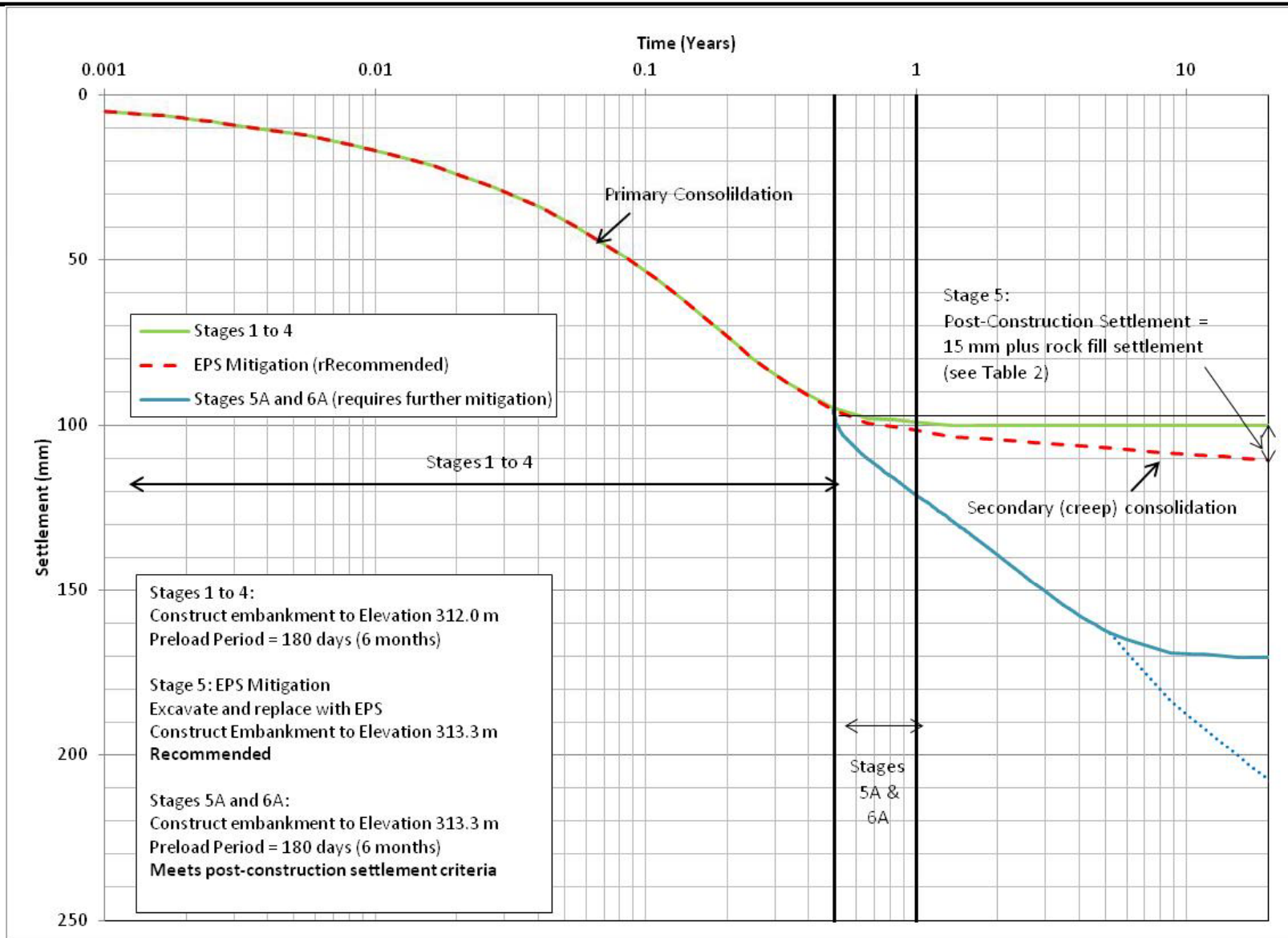
**Figure 8a**





PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE				ESTIMATED CONSOLIDATION SETTLEMENT VS. TIME SOUTH APPROACH STA 15+385			
		PROJECT No. 07-1191-0008		FILE No. ----			
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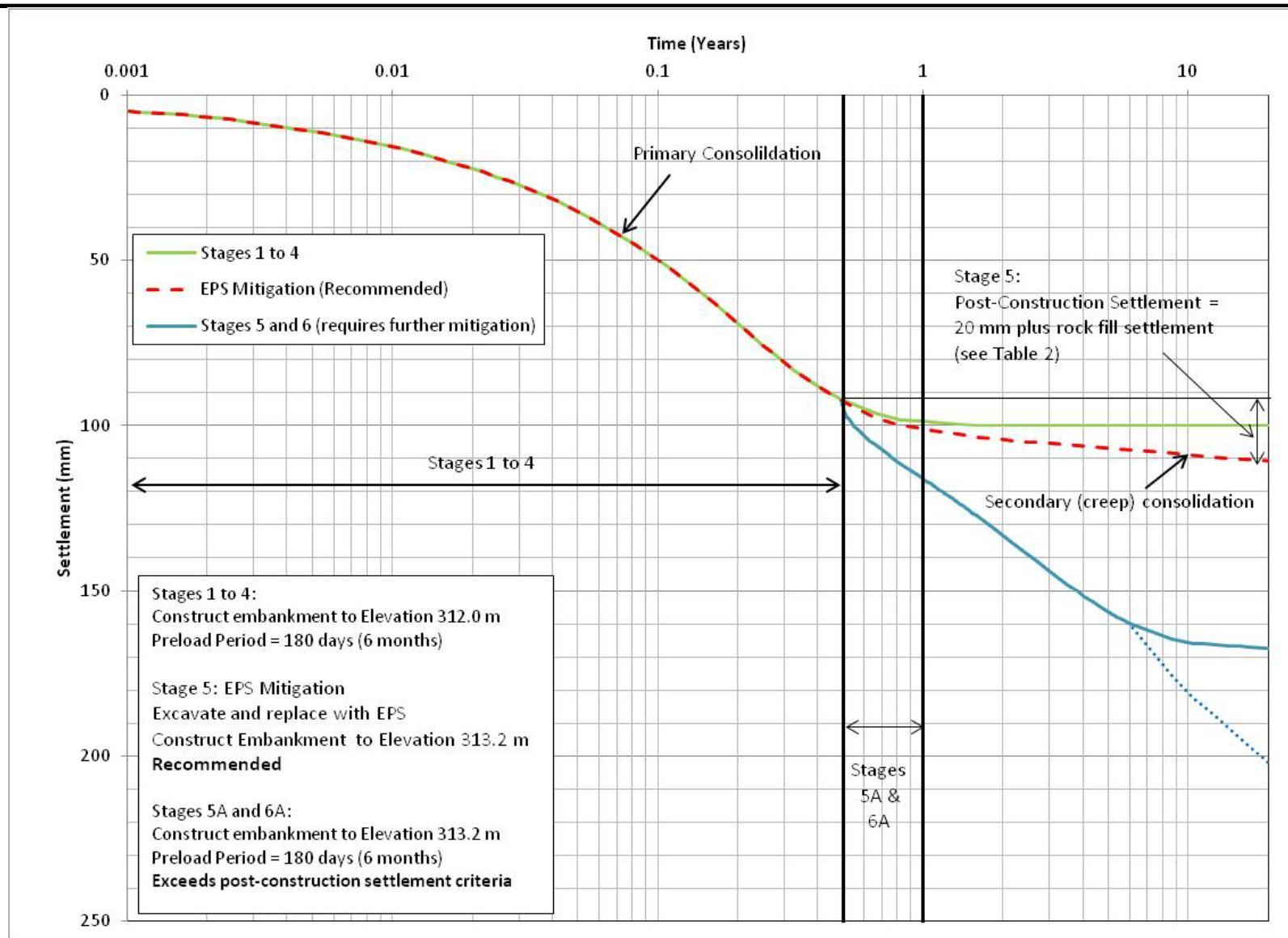
**Figure 9a**



PROJECT				WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE				ESTIMATED CONSOLIDATION SETTLEMENT VS. TIME SOUTH APPROACH STA 15+407			
PROJECT No. 07-1191-0008				FILE No. ----			
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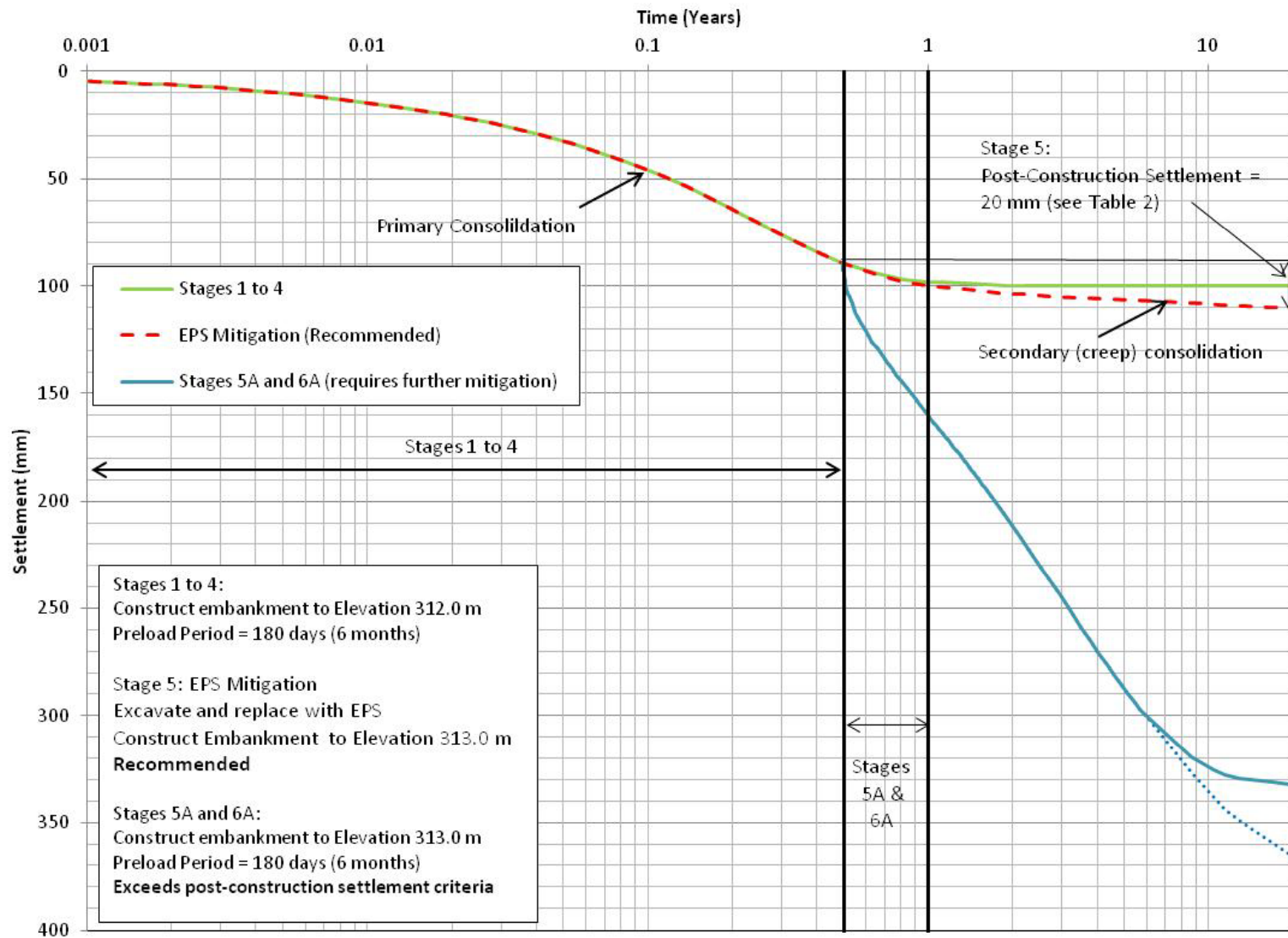
**Figure 9b**



PROJECT		WHITE CLAY RIVER BRIDGE HIGHWAY 11	
TITLE		ESTIMATED CONSOLIDATION SETTLEMENT VS. TIME SOUTH APPROACH STA 15+435	
PROJECT No. 07-1191-0008		FILE No. ----	
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REVIEW	JMAC	OCT. 2011	



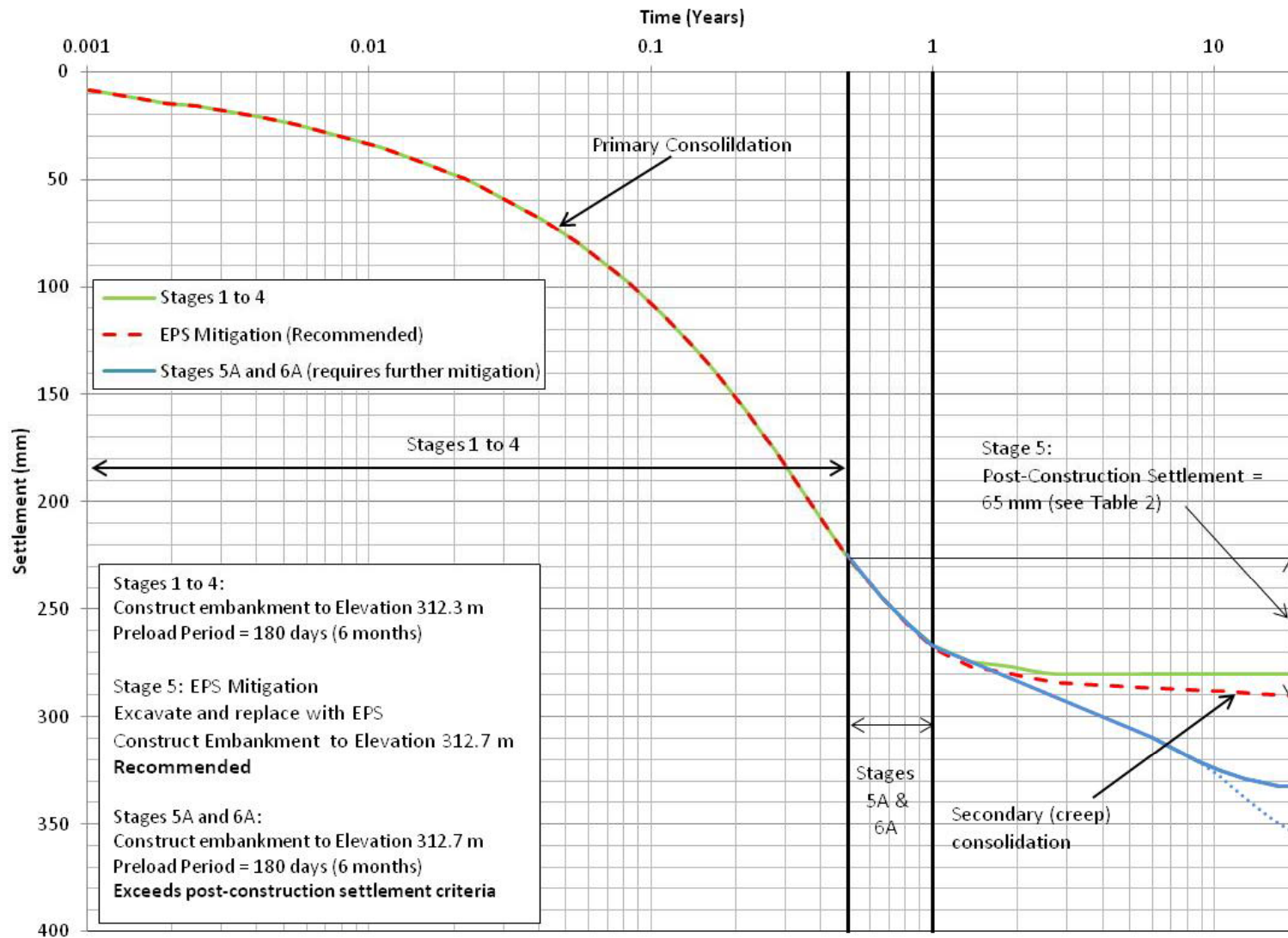
Figure 9c



PROJECT		WHITE CLAY RIVER BRIDGE HIGHWAY 11	
TITLE		<b>ESTIMATED CONSOLIDATION SETTLEMENT VS. TIME SOUTH ABUTMENT STA 15+468</b>	
PROJECT No. 07-1191-0008		FILE No. ----	
DESIGN	JJL	OCT. 2011	SCALE AS SHOWN
CADD	--		REV.
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REVIEW	JMAC	OCT. 2011	



**Figure 9d**

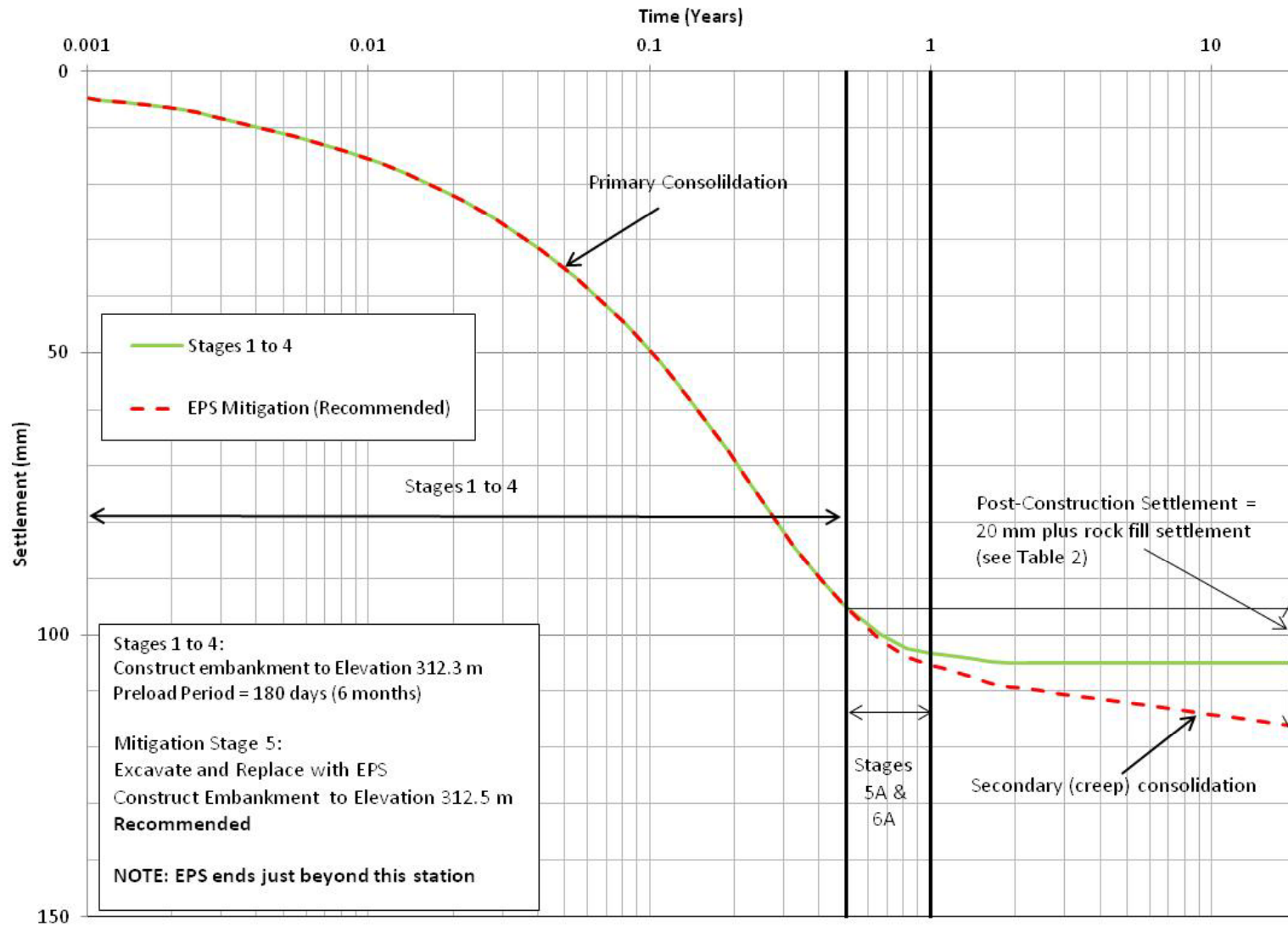



PROJECT		WHITE CLAY RIVER BRIDGE HIGHWAY 11	
TITLE		<b>ESTIMATED CONSOLIDATION SETTLEMENT VS. TIME NORTH ABUTMENT STA 15+523</b>	
PROJECT No. 07-1191-0008		FILE No. ----	
DESIGN	JJL	OCT. 2011	SCALE AS SHOWN REV.
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CHECK	SEMC	OCT. 2011	
REVIEW	JMAC	OCT. 2011	



**Figure 10a**

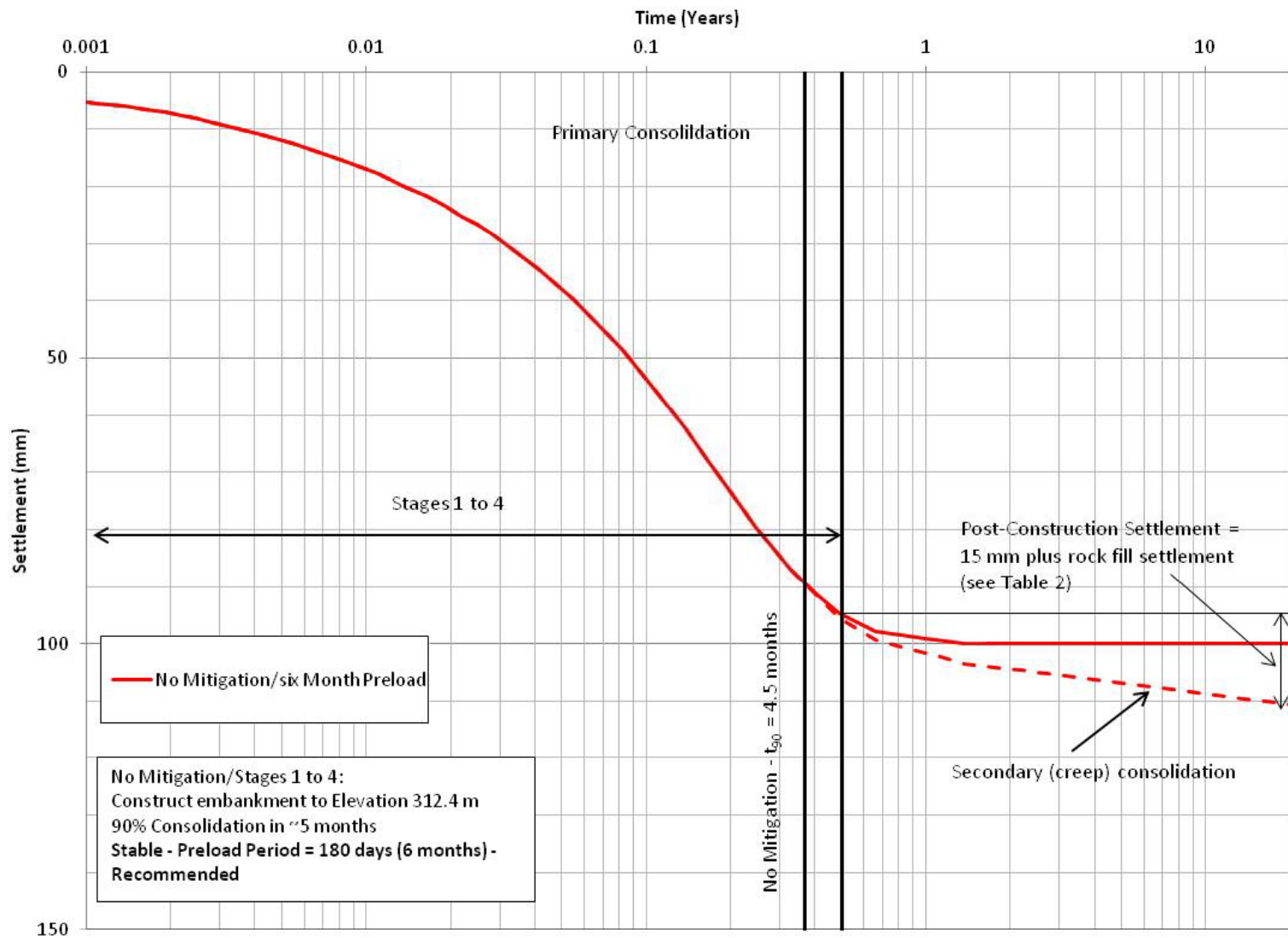




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 <b>Golder Associates</b>	PROJECT No. 07-1191-0008		FILE No. ---		<b>Figure 10b</b>	
	DESIGN	JJL	OCT. 2011	SCALE AS SHOWN		REV.
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	REVIEW	JMAC	OCT. 2011			



**Figure 10b**

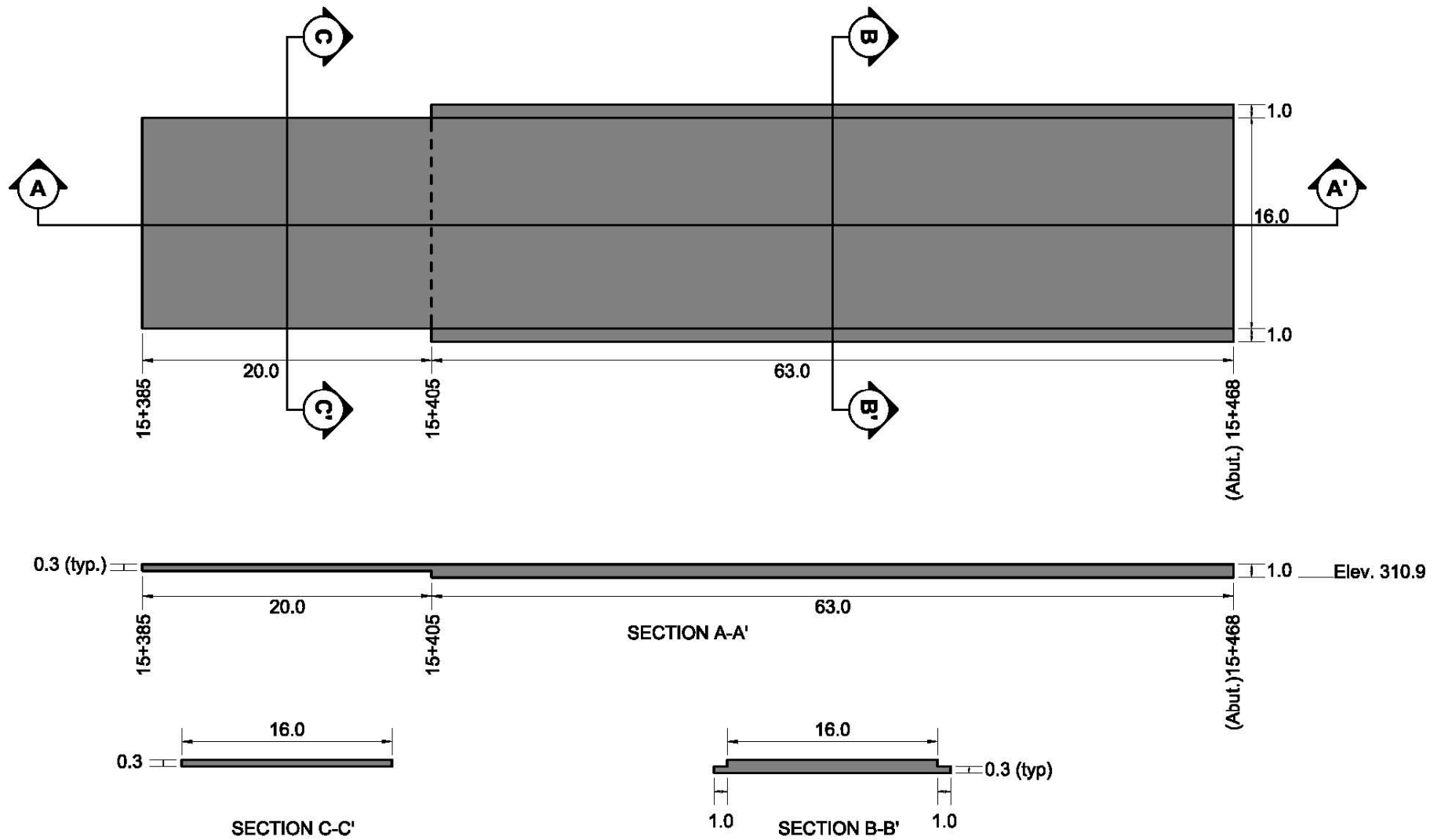


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REVIEW	JMAC	OCT. 2011			



Figure 10c

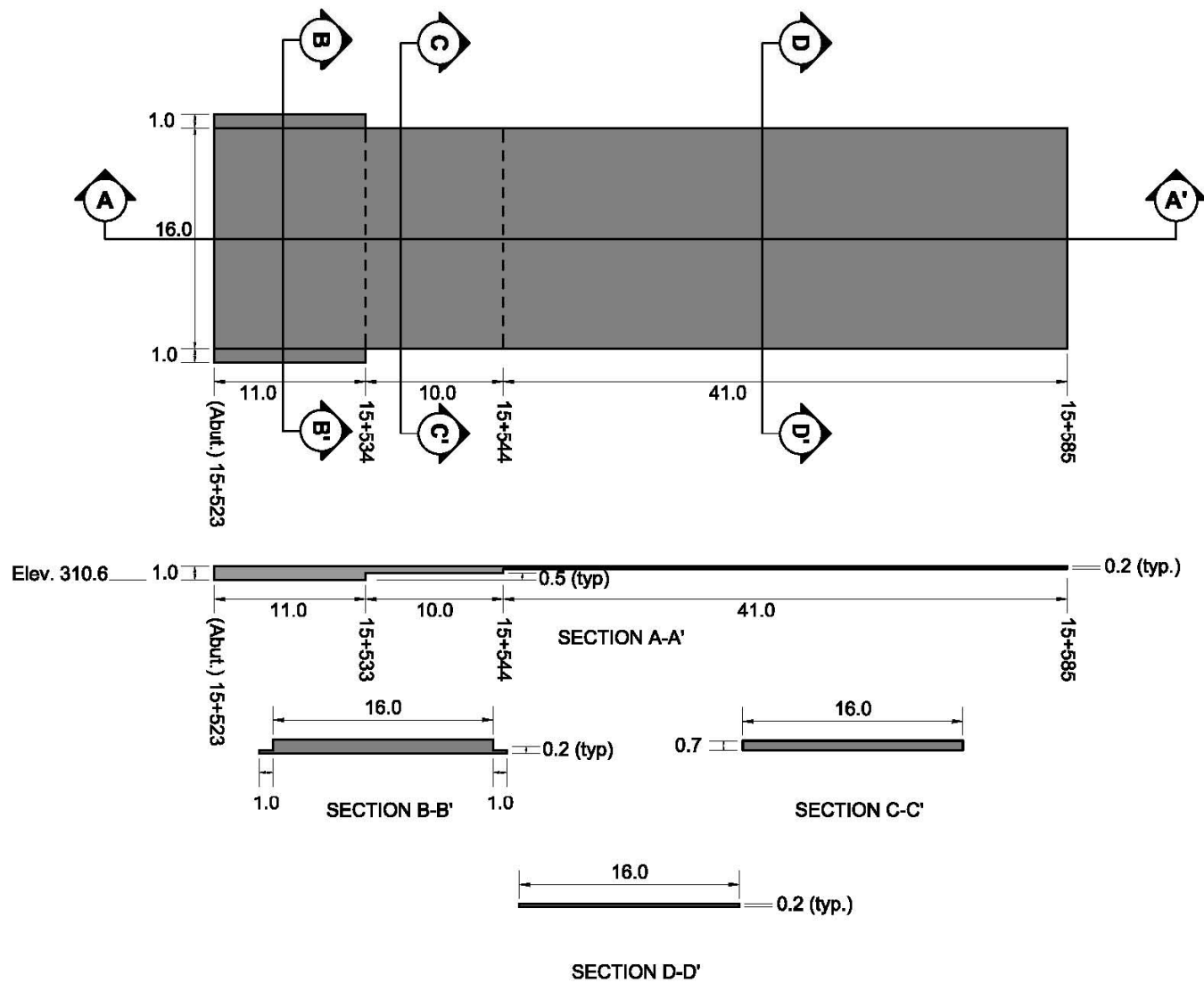




PROJECT		WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE		SOUTH APPROACH STA 15+385 TO STA 15+468 EPS DETAILS			
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		DESIGN JJL	OCT. 2011	SCALE AS SHOWN	REV.
		CADD --			
		CHECK SEMC	OCT. 2011		
		REVIEW JMAC	OCT. 2011		



Figure 11



PROJECT		WHITE CLAY RIVER BRIDGE HIGHWAY 11			
TITLE		NORTH APPROACH STA 15+523 TO STA 15+585 EPS DETAILS			
		PROJECT No. 07-1191-0008		FILE No. ----	
DESIGN	JJL	OCT. 2011	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	SEMC	OCT. 2011			
REVIEW	JMAC	OCT. 2011			



Figure 12



# APPENDIX A

## RECORD OF BOREHOLES AND DRILLHOLES



## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### 1. GENERAL

$\pi$	3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	Factor of Safety
V	volume
W	weight

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. stress: $\Delta\sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s/\rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity).

#### (a) Index Properties (continued)

w	water content
$w_l$	liquid limit
$w_p$	plastic limit
$I_p$	plasticity index $= (w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index $= (w - w_p)/I_p$
$I_c$	consistency index $= (w_l - w)/I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_a$	coefficient of secondary consolidation
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction $= \tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 + \sigma_3)$
$S_t$	sensitivity

Notes: 1  $\tau = c' + \sigma' \tan \phi'$   
2 Shear strength = (Compressive strength)/2



## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

<b>PH:</b>	Sampler advanced by hydraulic pressure
<b>PM:</b>	Sampler advanced by manual pressure
<b>WH:</b>	Sampler advanced by static weight of hammer
<b>WR:</b>	Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils Consistency

	$C_u, S_u$	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
$SO_4$	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note: 1** Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

### V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



## LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

### WEATHERING STATE

**Fresh:** no visible sign of weathering

**Faintly weathered:** weathering limited to the surface of Major discontinuities

**Slightly weathered:** penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

**Moderately weathered:** weathering extends throughout the rock mass but the rock material is not friable.

**Highly weathered:** weathering extends throughout rock Mass and the rock material is partly friable.

**Completely weathered:** rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

### BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

### JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	> 3 m
Wide	1 – 3 m
Moderately close	0.3 – 1 m
Close	50 – 300 mm
Very close	< 50 mm

### GRAIN SIZE

<u>Terms</u>	<u>Size*</u>
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2 mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

\* Note: Grains > 60 microns diameter are visible to the naked eye.

### CORE CONDITION

#### Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

#### Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

#### Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

### DISCONTINUITY DATA

#### Fracture Index

A count of the number of discontinuities (physical separation) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

#### Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90° angle is horizontal.

#### Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separation such as fractures, bedding planes and foliation planes or mechanically induced fractures caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

#### Abbreviations

B - Bedding	⊥ - Perpendicular To
FO - Foliation / Schistosity	- Parallel To
CL - Cleavage	P - Polished
SH - Shear Plane / Zone	K - Slickensided
VN - Vein	SM - Smooth
F - Fault	R - Rough
CO - Contact	ST - Stepped
J - Joint	PL - Planar
FR - Fracture	U - Undulating
MF - Mechanical Fracture	C - Curved



PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-1				1 OF 1		METRIC							
W.P. 5239-06-00			LOCATION N 5344457.7 ; E 360015.3				ORIGINATED BY EC									
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers				COMPILED BY MM									
DATUM Geodetic			DATE July 10, 2008				CHECKED BY AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED								
312.2	GROUND SURFACE															
0.0	Sand, some gravel (FILL) Brown Moist		1	AS	-		312									
311.4																
0.8	Sand and gravel, trace silt and clay, containing organics (FILL) Loose Brown to black Moist		2	SS	6		311									
			3	SS	7											
310.0							310									
2.2	Gravelly SAND Compact Grey Wet		4	SS	16											25 56 (19)
			5	SS	18		309									
308.5																
3.7	SILT and SAND to Silty SAND, trace to some gravel, trace to some clay Compact to very dense Grey Wet		6	SS	29		308									
			7	SS	51											
			8	SS	37		307									8 43 42 7
306.0			9	SS	50/0.23											
6.2	End of Borehole Spoon Refusal (Hammer Bouncing)  Note:  1. Water level at a depth of 2.1 m below ground surface (Elev. 310.1 m) upon completion of drilling.						306									

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-2				1 OF 1		METRIC						
W.P. 5239-06-00		LOCATION N 5344504.9 ;E 360017.2		ORIGINATED BY EHS											
DIST HWY 11		BOREHOLE TYPE NW Casing, Wash Boring		COMPILED BY MM											
DATUM Geodetic		DATE July 22, 2008		CHECKED BY AB											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
310.2	WATER SURFACE														
0.0	WATER														
309.4	0.8														
	PEAT, trace sand Very soft Brown Wet														
308.3	1.9		1	SS	5									165.5	
	Silty SAND, some gravel, trace clay Compact to very dense Grey Wet														
	Occasional cobbles and boulders inferred from grinding of augers.		2	SS	14										
			3	SS	16										15 59 25 1
			4	SS	25										
			5	SS	73										16 52 26 6
304.3	5.9		6	SS	98/0.23										
	End of Borehole Spoon Refusal														

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-3			1 OF 1 METRIC													
W.P. 5239-06-00			LOCATION N 5344534.4 ;E 360020.7			ORIGINATED BY EHS													
DIST HWY 11			BOREHOLE TYPE NW Casing, Wash Boring			COMPILED BY MM													
DATUM Geodetic			DATE July 21, 2008			CHECKED BY AB													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%)			γ			GR SA SI CL		
310.2 0.0	WATER SURFACE WATER						310												
309.0 1.2	PEAT Very soft Brown Wet		1	SS	WR		309												
307.8 2.4	CLAYEY SILT to SILTY CLAY, trace sand, trace organics Very soft Grey Wet		2	SS	WR		308												
307.0 3.2	Silty SAND to SAND and GRAVEL, trace to some clay Loose to very dense Grey Wet		3	SS	1		307												
	Occasional cobbles and boulders inferred from grinding of augers.		4	SS	9		306												
			5	SS	15		305												
			6	SS	63		304												
			7	SS	40		303												
			8	SS	97/0.18		302												
301.4 8.8	End of Borehole Spoon Refusal		9	SS	125/0.2														

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-4			1 OF 1 METRIC											
W.P. 5239-06-00			LOCATION N 5344573.2 ;E 360033.8			ORIGINATED BY EHS											
DIST HWY 11			BOREHOLE TYPE NW Casing, Wash Boring			COMPILED BY MM											
DATUM Geodetic			DATE July 21, 2008			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	10 20 30	W <sub>p</sub> W W <sub>L</sub>	γ	GR SA SI CL				
310.2 0.0	WATER SURFACE WATER						310										
309.5 0.7	PEAT Very soft Brown Wet		1	SS	WR		309										
307.8 2.4	CLAYEY SILT to SILTY CLAY, trace sand Soft to firm Grey Wet  Varved below 3.2 m depth.		2	SS	WR		308										
			3	SS	2		307										
			4	SS	3		306										
			5	SS	WR		305										
			6	TO	PM		304										
			7	SS	WH		303										
301.3 8.9	SILT, trace to some clay, trace sand Very loose Grey Wet		8	SS	2		302										
300.8 9.4	End of Borehole						301										
296.9 13.3	End of DCPT (50 Blows/0.15 m)  Note: 1. Dynamic Cone Penetration Test advanced 3.0 m north of Borehole No. WC-4.						297										

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-5			1 OF 1 METRIC						
W.P. 5239-06-00			LOCATION N 5344612.5 ;E 360041.1			ORIGINATED BY EHS						
DIST HWY 11			BOREHOLE TYPE NW Casing, Wash Boring			COMPILED BY MM						
DATUM Geodetic			DATE July 10, 2008			CHECKED BY AB						
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 × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W <sub>P</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES							
310.2 0.0	WATER SURFACE WATER						310					
309.3 0.9	PEAT, trace sand, trace gravel Very soft Brown Wet		1	SS	WH		309					
			2	SS	WH		308					
			3	SS	WH		307					
306.5 3.7	CLAYEY SILT to SILTY CLAY, varved Soft Grey Wet		4	SS	WH		306					
			5	SS	WH		305					
			6	SS	WR		304					
			7	TO	PH		303					
			8	SS	WR		302					
300.4 9.8	End of Borehole						301					

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-5a			1 OF 1 METRIC													
W.P. 5239-06-00			LOCATION N 5344612.5 ;E 360041.1			ORIGINATED BY ID													
DIST HWY 11			BOREHOLE TYPE BW Casing, Wash Boring, Portable equipment			COMPILED BY MM													
DATUM Geodetic			DATE October 24, 2008			CHECKED BY AB													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%)			γ			GR SA SI CL		
310.1	WATER SURFACE						310												
0.0	WATER						309												
309.0							308												
1.1	For soil stratigraphy refer to Record of Borehole WC-5.						307												
							306												
							305												
							304												
							303												
							302												
							301												
301.0	CLAYEY SILT to SILTY CLAY, varved Very soft to firm Grey Wet		1	SS	1		300	3 +									0 1 43 56		
298.9			2	TO	PM		299												
11.2	SILT Grey Wet						298												
298.2							297												
11.9	SAND and GRAVEL Compact Grey Wet		3	SS	28														
296.8																			
13.3	End of Borehole Casing Refusal																		

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-6			1 OF 2 METRIC													
W.P. 5239-06-00			LOCATION N 5344632.2 ; E 360044.8			ORIGINATED BY EHS													
DIST HWY 11			BOREHOLE TYPE NW Casing, Wash Boring			COMPILED BY MM													
DATUM Geodetic			DATE July 19 and 20, 2008			CHECKED BY AB													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%)			γ			GR SA SI CL		
310.2 0.0	WATER SURFACE WATER						310												
309.3 0.9	PEAT, trace sand Very soft Brown Wet		1	SS	WR		309												
			2	SS	WR		308												
			3	SS	WR		307												
			4	SS	WR		306												
306.2 4.0	CLAYEY SILT to SILTY CLAY, varved Soft to firm Grey Wet		5	SS	WR		305												
			6	SS	WH		304												
			7	TO	WR		303												
			8	SS	WH		302												
			9	SS	WR		301												
298.6 11.6	SILT, trace to some clay, occasional clay seams/layers Compact Grey Wet		10	SS	13		300												
297.2 13.0	BOULDER						299												
296.7 13.5	SAND and GRAVEL Black Wet		11	SC	REC 100%		298												
295.9 14.3			1	RC	REC 71%		297												
							296												

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

Continued Next Page

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

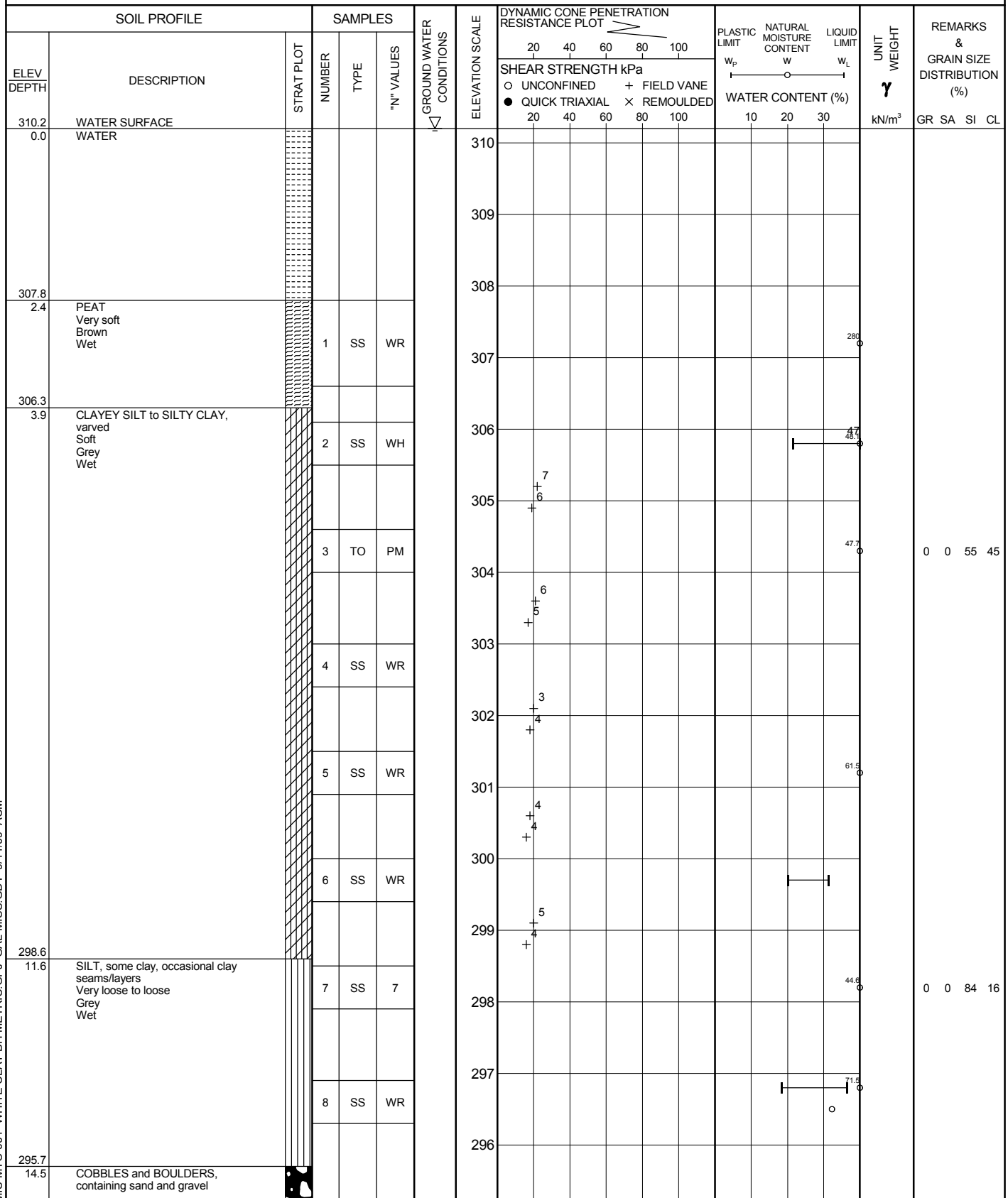


MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE







PROJECT <u>07-1191-0008</u>		<b>RECORD OF BOREHOLE No WC-7</b>		1 OF 2 <b>METRIC</b>	
W.P. <u>5239-06-00</u>	LOCATION <u>N 5344648.9 ;E 360047.9</u>	ORIGINATED BY <u>EHS</u>			
DIST <u>HWY 11</u>	BOREHOLE TYPE <u>NW Casing, Wash Boring</u>	COMPILED BY <u>MM</u>			
DATUM <u>Geodetic</u>	DATE <u>July 17 and 18, 2008</u>	CHECKED BY <u>AB</u>			



MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

Continued Next Page

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

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-7				2 OF 2		METRIC						
W.P. 5239-06-00		LOCATION N 5344648.9 ;E 360047.9		ORIGINATED BY EHS											
DIST HWY 11		BOREHOLE TYPE NW Casing, Wash Boring		COMPILED BY MM											
DATUM Geodetic		DATE July 17 and 18, 2008		CHECKED BY AB											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							
--- CONTINUED FROM PREVIOUS PAGE ---								20 40 60 80 100							
	COBBLES and BOULDERS, containing sand and gravel						295								
293.1							294								
17.1	SILTSTONE (BEDROCK)		1	RC	REC 100%		293								RQD = 55%
292.3							292								
17.9	SAND and GRAVEL						291								RQD = 23%
291.5							290								RQD = 59%
18.7	SILTSTONE to SANDSTONE (BEDROCK)  Bedrock cored from 18.7 m to 22.4 m depth.  For coring details, refer to Record of Drillhole WC-7.		2	RC	REC 100%		289								RQD = 50%
			3	RC	REC 100%		288								
			4	RC	REC 100%										
287.8															
22.4	End of Borehole														

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT: 07-1191-0008

## RECORD OF DRILLHOLE: WC-7

SHEET 1 OF 1

LOCATION: N 5344648.9 ; E 360047.9

DRILLING DATE: July 17 and 18, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D25

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE mm/rev	FLUSH	UN - 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 MB - Mechanical Break	BR - Broken Rock  NOTE: For additional abbreviations refer to list of abbreviations & symbols	NOTES WATER LEVELS INSTRUMENTATION
18	NQ Coring July 18, 2008	Refer to Previous Page		293.10									
		SILTSTONE Fine grained Slightly to moderately weathered Very strong Grey		17.10	1		Grey 75						
19		SAND and GRAVEL layer encountered between 17.9 m depth and 18.7 m depth.		292.30									
				17.90									
20		SILTSTONE Heavily jointed and fractured with broken core zones from 18.7 m to 20.4 m depth and 20.6 m to 20.7 m depth.  Vertical joint between 19.2 m and 19.8 m depth.		291.50			Grey 75						
				18.70	2								
21		SANDSTONE Fine to medium grained Slightly weathered Very strong Brownish red		289.50			Grey 75						
				20.70	3								
22													
					4		Grey 75						
23		End of Drillhole		287.80									
				22.40									
24													
25													
26													
27													

DEPTH SCALE

1 : 50

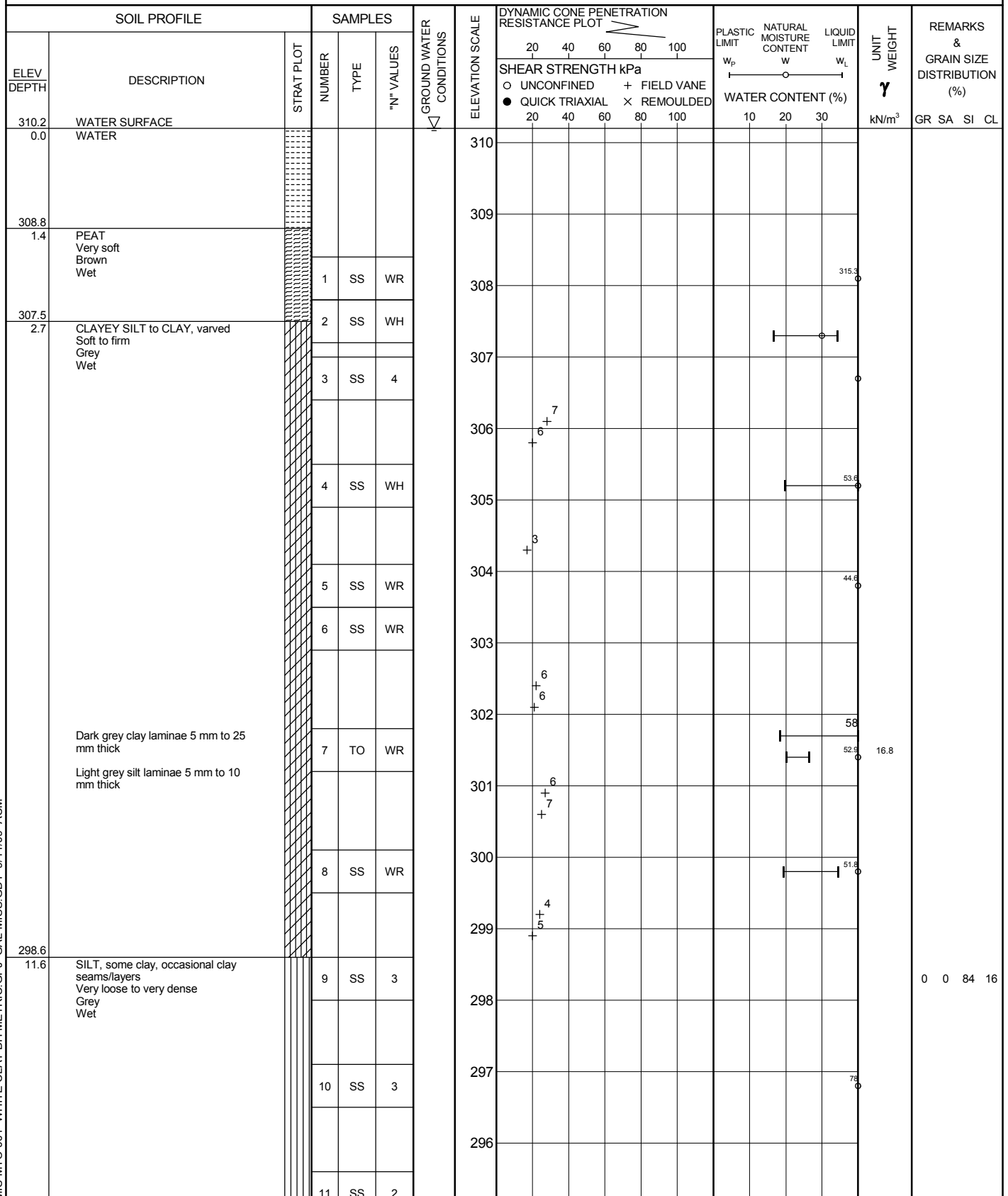


LOGGED: EHS

CHECKED: AB

MIS-RCK 004 WHITE CLAY BH METRIC GPJ GAL-MISS GDT 9/11/09 ACM

PROJECT <u>07-1191-0008</u>		<b>RECORD OF BOREHOLE No WC-8</b>		1 OF 2 <b>METRIC</b>	
W.P. <u>5239-06-00</u>	LOCATION <u>N 5344668.6 ; E 360051.6</u>	ORIGINATED BY <u>EHS</u>			
DIST <u>HWY 11</u>	BOREHOLE TYPE <u>NW Casing, Wash Boring</u>	COMPILED BY <u>MM</u>			
DATUM <u>Geodetic</u>	DATE <u>July 15 and 16, 2008</u>	CHECKED BY <u>AB</u>			



MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

Continued Next Page

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

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-8				2 OF 2		METRIC						
W.P. 5239-06-00		LOCATION N 5344668.6 ; E 360051.6		ORIGINATED BY EHS											
DIST HWY 11		BOREHOLE TYPE NW Casing, Wash Boring		COMPILED BY MM											
DATUM Geodetic		DATE July 15 and 16, 2008		CHECKED BY AB											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
--- CONTINUED FROM PREVIOUS PAGE ---															
293.4	SILT, some clay, occasional clay seams/layers Very loose to very dense Grey Wet		12	SS	56		295								0 6 79 15
16.8	COBBLES and BOULDERS, with gravel						294								
292.9							293								
17.3	SILTSTONE (BEDROCK)  Bedrock cored from 17.3 m to 22.2 m depth.  For coring details, refer to Record of Drillhole WC-8.		1	RC	REC 100%		292							RQD = 42%	
			2	RC	REC 100%		291							RQD = 41%	
			3	RC	REC 40%		290							RQD = 12%	
			4	RC	REC 100%		289							RQD = 58%	
			5	RC	REC 100%		288							RQD = 82%	
288.0	End of Borehole						288								
22.2															

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM



PROJECT: 07-1191-0008

## RECORD OF DRILLHOLE: WC-8

SHEET 1 OF 1

LOCATION: N 5344668.6 ; E 360051.6

DRILLING DATE: July 15 and 16, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D25

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE mm/rev	FLUSH	COLOUR % RETURN	UN - 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 MB- Mechanical Break				BR - Broken Rock  NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION		
									RECOVERY				R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				Diametral Point Load Index (MPa)	RMC -Q' AVG.				
									TOTAL CORE %	SOLID CORE %					B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 10 10 10	10 10 10 10						
									80 60 40 20 0	80 60 40 20 0	80 60 40 20 0	80 60 40 20 0																
		Refer to Previous Page		292.90																								
	NQ Coring July 16, 2008	SILTSTONE Fine grained Moderately weathered Very strong to medium strong Grey  Heavily jointed and fractured with broken core zones from 17.3 m to 19.3 m depth.  Slightly weathered below 20.3 m depth.		17.30	1		Grey	50																				
18				2		Grey	50																					
19				3		Grey	50																					
20				4		Grey	50																					
21				5		Grey	50																					
22																												
23																												
24																												
25																												
26																												
27																												
		End of Drillhole		288.00 22.20																								

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB


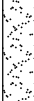

MIS-RCK 004 WHITE CLAY BH METRIC GP J GAL-MISS GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-9			1 OF 2 METRIC							
W.P. 5239-06-00		LOCATION N 5344685.3 ; E 360054.7		ORIGINATED BY EHS									
DIST HWY 11		BOREHOLE TYPE NW Casing, Wash Boring		COMPILED BY MM									
DATUM Geodetic		DATE July 7 to 9, 2008		CHECKED BY AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
310.2 0.0	WATER SURFACE WATER						310						
309.1 1.1	PEAT Very soft Brown Wet		1	SS	PM		309					322.8	
307.6 2.6	CLAYEY SILT to SILTY CLAY, varved Soft to firm Grey Wet  Stiff above 4.0 m depth.		2	SS	PM		308						
			3	SS	3		307						
			4	SS	WH		306					55.8	
			5	SS	1		305					45.6	0 0 55 45
			6	SS	WH		304						
			7	SS	WH		303					46.4	
			8	SS	WH		302					48.5	
			9	SS	8		301						
			10	SS	5		300					51	
298.5 11.7	SILT, trace to some clay, occasional clay seams/layers Loose Grey Wet						299						
							298						
							297						0 0 88 12
							296						

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

Continued Next Page

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

PROJECT <u>07-1191-0008</u>			<b>RECORD OF BOREHOLE No WC-9</b>				2 OF 2 <b>METRIC</b>								
W.P. <u>5239-06-00</u>		LOCATION <u>N 5344685.3 ; E 360054.7</u>		ORIGINATED BY <u>EHS</u>											
DIST <u>HWY 11</u>		BOREHOLE TYPE <u>NW Casing, Wash Boring</u>		COMPILED BY <u>MM</u>											
DATUM <u>Geodetic</u>		DATE <u>July 7 to 9, 2008</u>		CHECKED BY <u>AB</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × REMOULDED 20 40 60 80 100							
294.2	SILT, trace to some clay, occasional clay seams/layers Loose Grey Wet		11	SS	8		295								0 95 (5)
16.0	SAND, trace silt, containing gravel, cobbles and boulders Very dense Grey Wet		12	SS	48/0.15		294								
292.8	SILTSTONE (BEDROCK)  Bedrock cored from 17.4 m to 21.7 m depth.  For coring details, refer to Record of Drillhole WC-9.		1	RC	REC 60%		293								RQD = 41%
17.4			2	RC	REC 100%		292								RQD = 50%
			3	RC	REC 100%		291								RQD = 96%
							290								
288.5							289								
21.7	End of Borehole  Note:  1. On July 8, 2008, casing tip advanced to 11.7 m depth (Elev. 298.5 m) and with top of casing approximately 0.6 m above water (river) surface, water was flowing out of top of casing; when casing was advanced to 13 m depth (Elev. 297.2 m), water was not flowing out of casing.														

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

DATUM: Geodetic

DRILL RIG: D25

DRILLING CONTRACTOR: Walker Drilling

[illegible]

DEPTH SCALE

1 : 50

LOGGED: EHS

CHECKED: AB

PROJECT <u>07-1191-0008</u>			<b>RECORD OF BOREHOLE No WC-9a</b>			1 OF 1 <b>METRIC</b>			
W.P. <u>5239-06-00</u>			LOCATION <u>N 5344685.7 ;E 360052.7</u>			ORIGINATED BY <u>EHS</u>			
DIST <u>HWY 11</u>			BOREHOLE TYPE <u>NW Casing, Wash Boring</u>			COMPILED BY <u>MM</u>			
DATUM <u>Geodetic</u>			DATE <u>July 23, 2008</u>			CHECKED BY <u>AB</u>			
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>P</sub> W W <sub>L</sub> WATER CONTENT (%) 10 20 30 UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.2 0.0	WATER SURFACE WATER						310		
309.3 0.9	For soil stratigraphy, refer to Record of Borehole WC-9.						309	8 + 4 + 3 + 3 +	
							308	3 + 10 +	
							307	4 + 5 +	
							306	4 + 6 + 5 + 5 + 6 + 7 + 6 +	
304.7 5.5	End of Borehole						305	6 +	

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			<b>RECORD OF BOREHOLE No WC-10</b>			1 OF 2 <b>METRIC</b>						
W.P. 5239-06-00			LOCATION N 5344701.3 ; E 360058.8			ORIGINATED BY EHS						
DIST HWY 11			BOREHOLE TYPE NW Casing, Wash Boring			COMPILED BY MM						
DATUM Geodetic			DATE July 7, 2008			CHECKED BY AB						
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 × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub>  WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES							
310.2 0.0	WATER SURFACE WATER						310					
309.4 0.8	PEAT, trace sand Very soft Brown Wet		1	SS	WH		309					
308.1 2.1	CLAYEY SILT to SILTY CLAY, varved Soft Grey Wet  Stiff above 3.5 m depth.		2	SS	WH		308					
			3	SS	3		307					
			4	SS	1		306					0 0 31 69
			5	TO	WR		305					
			6	SS	WH		304					
			7	SS	WH		303					
300.6 9.6	End of Borehole						302					0 1 55 44
							301					
							300					
							299					
							298					
							297					
							296					

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

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

<b>PROJECT</b> 07-1191-0008		<b>RECORD OF BOREHOLE No WC-10</b>		2 OF 2 <b>METRIC</b>	
W.P. 5239-06-00		LOCATION N 5344701.3 ;E 360058.8		ORIGINATED BY EHS	
DIST HWY 11		BOREHOLE TYPE NW Casing, Wash Boring		COMPILED BY MM	
DATUM Geodetic		DATE July 7, 2008		CHECKED BY AB	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED												
295.1 15.1	End of DCPT Cone Refusal (Hammer Bouncing)  Note:  1. Dynamic Cone Penetration Test advanced about 2 m north and 1 m east of Borehole WC-10 on July 21, 2008.					295														

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM



PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-11			1 OF 1 METRIC													
W.P. 5239-06-00			LOCATION N 5344721.9 ; E 360065.7			ORIGINATED BY EC													
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers			COMPILED BY MM													
DATUM Geodetic			DATE July 10, 2008			CHECKED BY AB													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%)			γ			GR SA SI CL		
310.2	GROUND SURFACE							20 40 60 80 100											
0.0	PEAT, some sand Very soft Brown to black Wet		1	AS	-		310												
			2	SS	1		309												
			3	SS	WH		308												
308.0	CLAYEY SILT to SILTY CLAY Soft Grey Wet		4	SS	WH		307												
2.2			5	TO	PH		306												
	Varved below 3.7 m depth.		6	SS	WH		305												
			7	SS	WH		304												
			8	SS	WH		303												
302.9	End of Borehole		9	SS	WH														
7.3	Note: 1. Water level at ground surface (Elev. 310.2 m) upon completion of drilling.																		

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT <u>07-1191-0008</u>			<b>RECORD OF BOREHOLE No WC-11a</b>			1 OF 1 <b>METRIC</b>					
W.P. <u>5239-06-00</u>			LOCATION <u>N 5344721.1 ; E 360064.5</u>			ORIGINATED BY <u>TR</u>					
DIST <u>          </u> HWY <u>11</u>			BOREHOLE TYPE <u>Portable Equipment, 75 mm O.D. Hand Auger</u>			COMPILED BY <u>MM</u>					
DATUM <u>Geodetic</u>			DATE <u>August 28, 2008</u>			CHECKED BY <u>AB</u>					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>p</sub> — W — W <sub>L</sub> WATER CONTENT (%)	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.2 0.0	GROUND SURFACE For soil stratigraphy, refer to Record of Borehole WC-11.						310				
							309	3 + 2 +			
							308				
							307	2 + 3 +			
							306				
							305	4 + 4 + 3 +			
							304	2 + 2 +			
							303	2 + 2 +			
302.7 7.5	Start of DCPT						302				
							301				
							300				
							299				
298.2 12.0	End of DCPT (127 Blows/0.3 m)  Note:  1. Water level at ground surface (Elev. 310.2 m) upon completion of drilling.										

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT <u>07-1191-0008</u>			<b>RECORD OF BOREHOLE No WC-11b</b>			1 OF 1 <b>METRIC</b>											
W.P. <u>5239-06-00</u>			LOCATION <u>N 5344721.6 ;E 360061.5</u>			ORIGINATED BY <u>ID</u>											
DIST <u>HWY 11</u>			BOREHOLE TYPE <u>BW Casing, Wash Boring, Portable equipment</u>			COMPILED BY <u>MM</u>											
DATUM <u>Geodetic</u>			DATE <u>October 25, 2008</u>			CHECKED BY <u>AB</u>											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%)			γ	GR SA SI CL		
								20 40 60 80 100	20 40 60 80 100	10 20 30	10 20 30	10 20 30					
310.3 0.0	GROUND SURFACE For soil stratigraphy, refer to Record of Borehole WC-11.																
304.2 6.1	CLAYEY SILT to SILTY CLAY, varved Firm Grey Wet		1	SS	2												
			2	SS	1												
			3	SS	1												
299.6 11.0	SILT, trace clay, trace sand, trace gravel Loose Grey Wet End of Borehole Spoon Refusal  Note: 1. Water level at ground surface (Elev. 310.3 m) upon completion of drilling.		4	SS	62/0.18												

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT <u>07-1191-0008</u>		<b>RECORD OF BOREHOLE No WC-12</b>		1 OF 1 <b>METRIC</b>						
W.P. <u>5239-06-00</u>		LOCATION <u>N 5344594.2 ; E 360030.6</u>		ORIGINATED BY <u>EHS</u>						
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>NW Casing, Wash Boring</u>		COMPILED BY <u>MM</u>						
DATUM <u>Geodetic</u>		DATE <u>July 22, 2008</u>		CHECKED BY <u>AB</u>						
SOIL PROFILE			SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
310.2 0.0	WATER SURFACE WATER									
308.4 1.8										
300.8 9.4	Start of DCPT									
296.2 14.0	End of DCPT (64 Blows/0.3 m)									

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-13			1 OF 1 METRIC											
W.P. 5239-06-00			LOCATION N 5344762.9 ; E 360069.3			ORIGINATED BY ID											
DIST HWY 11			BOREHOLE TYPE BW Casing, Wash Boring, Portable equipment			COMPILED BY MM											
DATUM Geodetic			DATE October 26, 2008			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) W <sub>P</sub> — W — W <sub>L</sub>			γ	GR SA SI CL		
310.3 0.0	GROUND SURFACE							20 40 60 80 100									
	PEAT Very soft Brown Wet		1	SS	1		310										
			2	SS	1												
			3	SS	1		309										
308.3 2.0	CLAYEY SILT to SILTY CLAY, varved Soft to stiff Grey Wet		4	SS	2		308									0 19 45 36	
			5	SS	7		307										
			6	SS	2		306										
			7	SS	2		305										
			8	TO	PM		304								15.5		
							303										
			9	SS	4		302										
301.8 8.5	SILT Grey Wet						301										
301.0 9.3	Silty SAND to SAND and GRAVEL Compact to very dense Grey Wet		10	SS	20		300										
							299									42 47 (11)	
297.9 12.4	End of Borehole Spoon Refusal (Hammer Bouncing)  Note:  1. During casing removal, water level in casing was noted to be 0.3 m above ground surface.  2. Water level at ground surface (Elev. 310.3 m) upon completion of drilling.		12	SS	19/0.2		298										

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008		<b>RECORD OF BOREHOLE No WC-14</b>		1 OF 1 <b>METRIC</b>	
W.P. 5239-06-00	LOCATION N 5344802.1 ;E 360077.5	ORIGINATED BY ID			
DIST HWY 11	BOREHOLE TYPE BW Casing, Wash Boring, Portable equipment	COMPILED BY MM			
DATUM Geodetic	DATE October 27, 2008	CHECKED BY AB			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							× REMOULDED	
310.7	GROUND SURFACE						20	40	60	80	100	10	20	30				
310.4	Peat (FILL) Very soft Brown Moist		1	SS	4													
310.1	Silty clay (FILL) Firm Grey Wet		2	SS	1									422.8				
309.3	PEAT Very soft Black Wet		3	SS	4													
308.4	CLAYEY SILT to SILTY CLAY, varved Firm to stiff Grey Wet		4	SS	8									43				
		5	SS	12														
		6	SS	4														
		7	SS	1														
		8	SS	1														
307.4																		
306.4																		
305.4																		
304.4																		
303.4	SILT and SAND to SAND and GRAVEL, trace clay Loose to very dense Grey Wet																	
302.4																		
301.4																		
300.9	End of Borehole Casing Refusal																	
9.8	Note:  1. Water level at a depth of 0.3 m below ground surface (Elev. 310.4 m) upon completion of drilling.																	

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-15				1 OF 1 METRIC				
W.P. 5239-06-00		LOCATION N 5344840.9 ;E 360087.1		ORIGINATED BY ID							
DIST HWY 11		BOREHOLE TYPE BW Casing, Wash Boring, Portable equipment		COMPILED BY MM							
DATUM Geodetic		DATE October 27 and 28, 2008		CHECKED BY AB							
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 × REMOULDED 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>p</sub> — W — W <sub>L</sub> WATER CONTENT (%)	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
311.1	GROUND SURFACE										
0.0	PEAT Soft Brown Wet		1	SS	3		311				
310.2			2	SS	4		310				
0.9	CLAYEY SILT to SILTY CLAY, varved Soft to stiff Grey Wet		3	SS	12		310				0 2 37 61
			4	SS	8		309				
			5	SS	8		308				
			6	SS	1		307				
306.5							306				
4.6	SILT, some clay Very loose Grey Wet		7	SS	1		305				0 0 88 12
305.3							304				
5.8	SAND and GRAVEL Dense Grey Wet		8	SS	38						
304.0											
7.1	End of Borehole Casing Refusal  Note:  1. Water level at a depth of 0.1 m below ground surface (Elev. 311.0 m) upon completion of drilling.										

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PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-16			1 OF 1 METRIC															
W.P. 5239-06-00			LOCATION N 5344879.5 ; E 360097.4			ORIGINATED BY ID															
DIST HWY 11			BOREHOLE TYPE BW Casing, Wash Boring, Portable equipment			COMPILED BY MM															
DATUM Geodetic			DATE October 28, 2008			CHECKED BY AB															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%)			γ			GR SA SI CL				
311.1	GROUND SURFACE							20 40 60 80 100													
0.0	TOPSOIL																				
0.2	Brown		1	SS	3													9	40	31	20
	Sand and silt, some clay, trace to some gravel (FILL)																				
310.0	Loose Brown		2	SS	5																
1.1	Wet																				
	PEAT		3	SS	3																
309.3	Soft Brown																				
1.8	Wet																				
	CLAYEY SILT to SILTY CLAY, trace sand, varved		4	SS	11													0	5	63	32
	Firm to stiff																				
	Grey																				
	Wet		5	SS	2																
305.0																					
6.1	SILT, trace sand		7	SS	12																
	Compact																				
	Grey																				
	Wet																				
303.2			8	SS	19																
7.9	SAND and GRAVEL, some silt, trace clay																				
	Compact																				
	Grey																				
	Wet																				
301.7			9	SS	74/0.15																
9.4	End of Borehole																				
	Spoon Refusal																				
	Note:																				
	1. Water level at a depth of 0.5 m below ground surface (Elev. 310.6 m) upon completion of drilling.																				

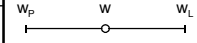
MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-17			1 OF 2 METRIC											
W.P. 5239-06-00			LOCATION N 5344603.0 ; E 360055.1			ORIGINATED BY ID											
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers			COMPILED BY MM											
DATUM Geodetic			DATE November 5, 2008			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)	10 20 30	γ	GR SA SI CL			
312.3	GROUND SURFACE																
0.0	ASPHALT (200 mm) over																
311.9	CONCRETE (150 mm)																
0.4	Sand, trace gravel, trace silt (FILL) Very loose to compact Brown Moist to wet		1	SS	29		312										
			2	SS	10		311							2 93 (5)			
			3	SS	4		310										
			4	SS	5		309										
			5	SS	5		308							3 94 (3)			
307.4	CLAYEY SILT to SILTY CLAY, varved Firm to stiff Grey Wet		6	SS	1		307										
4.9	Dark grey clay laminae 2 mm to 5 mm thick.  Light grey silt laminae 2 mm to 5 mm thick.		7	TO	PM		306						19.4				
			8	SS	WH		305										
			9	SS	1		304										
			10	SS	WH		303										
300.1	SAND and GRAVEL, some silt, trace to some clay Dense to very dense Grey Wet		11	SS	43		302										
12.2			12	SS	69		301										
							300							42 34 18 6			
							299										
							298										

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

Continued Next Page

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

PROJECT <u>07-1191-0008</u>			<b>RECORD OF BOREHOLE No WC-17</b>				2 OF 2 <b>METRIC</b>			
W.P. <u>5239-06-00</u>		LOCATION <u>N 5344603.0 ;E 360055.1</u>		ORIGINATED BY <u>ID</u>						
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>MM</u>						
DATUM <u>Geodetic</u>		DATE <u>November 5, 2008</u>		CHECKED BY <u>AB</u>						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT $w_p$ NATURAL MOISTURE CONTENT $w$ LIQUID LIMIT $w_L$  WATER CONTENT (%)	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE			
15.0	End of Borehole Auger Refusal  Note:  1. Water level at a depth of 2.3 m below ground surface (Elev. 310.0 m) upon completion of drilling.									


MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-18			1 OF 2 METRIC											
W.P. 5239-06-00			LOCATION N 5344622.8 ; E 360058.4			ORIGINATED BY ID											
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers			COMPILED BY MM											
DATUM Geodetic			DATE November 5, 2008			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W <sub>P</sub> W W <sub>L</sub>	WATER CONTENT (%)	10 20 30	γ	GR SA SI CL			
312.3	GROUND SURFACE																
0.0	ASPHALT (240 mm) over crushed sand and gravel (300 mm) (FILL)						312										
311.8							311										
0.5	Sand, trace gravel, trace to some silt (FILL) Very loose to very dense Brown to grey Moist to wet		1	SS	51		310										
			2	SS	14		309										
			3	SS	4		308										
			4	SS	3		307										
			5	SS	2		306										
306.2			6	SS	4		305										
6.1	CLAYEY SILT to SILTY CLAY, varved Firm Grey Wet		7	SS	WH		304										
			8	TO	PM		303										
	Dark grey clay laminae 7 mm to 30 mm thick.  Light grey silt laminae 5 mm to 10 mm thick.		9	SS	1		302										
			10	SS	2		301										
299.6							300										
12.7	SILT Grey Wet						299										
298.9							298										
13.4	Gravelly SAND, trace to some silt Compact to dense Grey Moist to wet		11	SS	24												

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

Continued Next Page

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

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-18				2 OF 2		METRIC								
W.P. 5239-06-00			LOCATION N 5344622.8 ;E 360058.4				ORIGINATED BY ID										
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers				COMPILED BY MM										
DATUM Geodetic			DATE November 5, 2008				CHECKED BY AB										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
295.8	Gravelly SAND, trace to some silt Compact to dense Grey Moist to wet		12	SS	45		297										24 66 (10)
16.5	End of Borehole Auger Refusal						296										
	Note: 1. Water level at a depth of 2.2 m below ground surface (Elev. 310.1 m) upon completion of drilling. 2. Approximately 1.0 m of heave was noted inside augers at 6.0 m depth.																

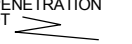
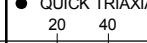
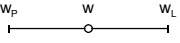

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

PROJECT 07-1191-0008			RECORD OF BOREHOLE No WC-19			1 OF 2 METRIC					
W.P. 5239-06-00			LOCATION N 5344695.1 ; E 360072.8			ORIGINATED BY ID					
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers			COMPILED BY MM					
DATUM Geodetic			DATE November 6, 2008			CHECKED BY AB					
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 × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W <sub>P</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
312.3	GROUND SURFACE										
0.0	ASPHALT (230 mm) over crushed sand and gravel (250 mm) (FILL)						312				
311.8							311				
0.5	Sand, trace gravel, trace to some silt (FILL) Very loose to dense Brown to grey Moist		1	SS	47		310				
			2	SS	11		309				
			3	SS	8		308				
			4	SS	2		307				
308.0							306				
4.3	SILT, containing organics Loose Brown to black Wet		5	SS	5		305				
306.5							304				
5.8	CLAYEY SILT to SILTY CLAY Firm Grey Wet		6	SS	5		303				
	Dark grey clay laminae 5 mm to 10 mm thick.		7	TO	PM		302				
	Light grey silt laminae 5 mm to 10 mm thick.		8	SS	WH		301				
			9	SS	1		300				
300.1							299				
12.2	SILT, some clay, trace sand Loose Grey Wet		10	SS	6		298				
298.1			11	SS	7						
14.2	SAND and GRAVEL Very dense Grey to black Wet										

MIS-MTO 001 WHITE CLAY BH METRIC.GPJ GAL-MISS.GDT 9/11/09 ACM

Continued Next Page

+ <sup>3</sup>, × <sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT <u>07-1191-0008</u>			<b>RECORD OF BOREHOLE No WC-19</b>				2 OF 2 <b>METRIC</b>				
W.P. <u>5239-06-00</u>		LOCATION <u>N 5344695.1 ;E 360072.8</u>		ORIGINATED BY <u>ID</u>							
DIST <u>HWY 11</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>MM</u>							
DATUM <u>Geodetic</u>		DATE <u>November 6, 2008</u>		CHECKED BY <u>AB</u>							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 	PLASTIC LIMIT w <sub>p</sub> NATURAL MOISTURE CONTENT w LIQUID LIMIT w <sub>L</sub>  WATER CONTENT (%)	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
296.5	SAND and GRAVEL Very dense Grey to black Wet --- CONTINUED FROM PREVIOUS PAGE ---		12	SS	69		297				
15.8	End of Borehole  Note:  1. Water level at a depth of 2.1 m below ground surface (Elev. 310.2 m) upon completion of drilling.										



# APPENDIX B

## LABORATORY TEST RESULTS



**TABLE B-1  
REFUSAL/BEDROCK ELEVATIONS  
WHITE CLAY RIVER BRIDGE REPLACEMENT  
GWP 5239-06-00, SITE NO. 47-005  
HIGHWAY 11, TOWNSHIP OF MAISONVILLE**

<b>Borehole</b>	<b>Depth to Refusal/Bedrock Surface (m)</b>	<b>Refusal/Bedrock Surface Elevation (m)</b>	<b>Comments</b>
WC-1	6.2	306.0	Split-Spoon Refusal
WC-2	5.9	304.3	Split-Spoon Refusal
WC-3	8.8	301.4	Split-Spoon Refusal
WC-4*	13.3	296.9	End of DCPT >50 blows/0.3m
WC-5	Refusal not encountered; borehole terminated above bedrock surface		
WC-5a	13.3	296.8	Casing Refusal
WC-6	14.3	295.9	Bedrock Cored
WC-7	17.1	293.1	Bedrock Cored
WC-8	17.3	292.9	Bedrock Cored
WC-9	17.4	292.8	Bedrock Cored
WC-9a	Refusal not encountered; borehole advanced for field vane testing		
WC-10	15.1	295.1	DCPT Refusal
WC-11	Refusal not encountered; borehole terminated above bedrock surface		
WC-11a*	12.0	298.2	End of DCPT >50 blows/0.3m
WC-11b	11.0	299.3	Split-Spoon Refusal
WC-12*	14.0	296.2	End of DCPT >50 blows/0.3m
WC-13	12.4	297.9	Split-Spoon Refusal
WC-14	9.8	300.9	Casing Refusal
WC-15	7.1	304.0	Casing Refusal
WC-16	9.4	301.7	Split-Spoon Refusal
WC-17	15.0	297.3	Auger Refusal
WC-18	16.5	295.8	Auger Refusal
WC-19	Refusal not encountered; borehole terminated above bedrock surface		

\* Bedrock surface inferred from resistance to dynamic cone penetration >50 blows/0.3 m and based on information from the adjacent boreholes.

Compiled by: TR  
Checked by: SEMC  
Reviewed by: JMAC

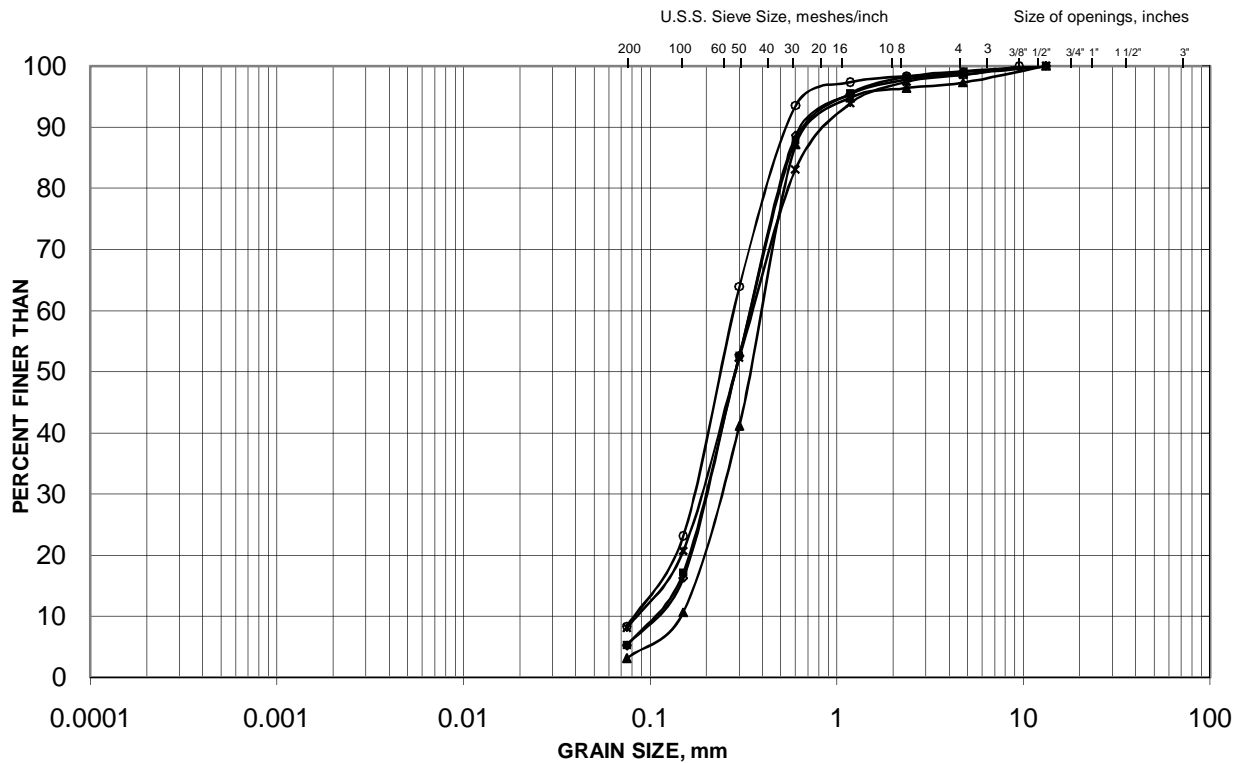
**TABLE B-2**  
**UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS**  
**WHITE CLAY RIVER BRIDGE REPLACEMENT**  
**GWP 5239-06-00, SITE NO. 47-005**  
**HIGHWAY 11, TOWNSHIP OF MAISONVILLE**

<b>Borehole Number</b>	<b>Sample Depth (m)</b>	<b>Sample Elevation (m)</b>	<b>Rock Type</b>	<b>Core Diameter (mm)</b>	<b>Uniaxial Compressive Strength (MPa)</b>
WC-6	17.2	293.0	Siltstone	47	68
WC-6	19.3	290.9	Sandstone	48	256
WC-7	20.3	289.9	Siltstone	48	106
WC-7	22.3	287.9	Sandstone	48	188
WC-8	19.6	290.6	Siltstone	47	132
WC-8	22.1	288.1	Siltstone	47	41
WC-9	20.2	290.0	Siltstone	47	180
WC-9	21.3	288.9	Siltstone	47	138

Compiled by: TR  
Checked by: SEMC  
Reviewed by: JMAC

# GRAIN SIZE DISTRIBUTION Sand (FILL)

**FIGURE  
B-1**



SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—■—	WC-17	2	310.4
—▲—	WC-17	5	308.1
—✱—	WC-18	1	311.2
—◆—	WC-18	3	309.7
—○—	WC-19	2	310.5

Project Number: 07-1191-0008

Checked By: SEMC

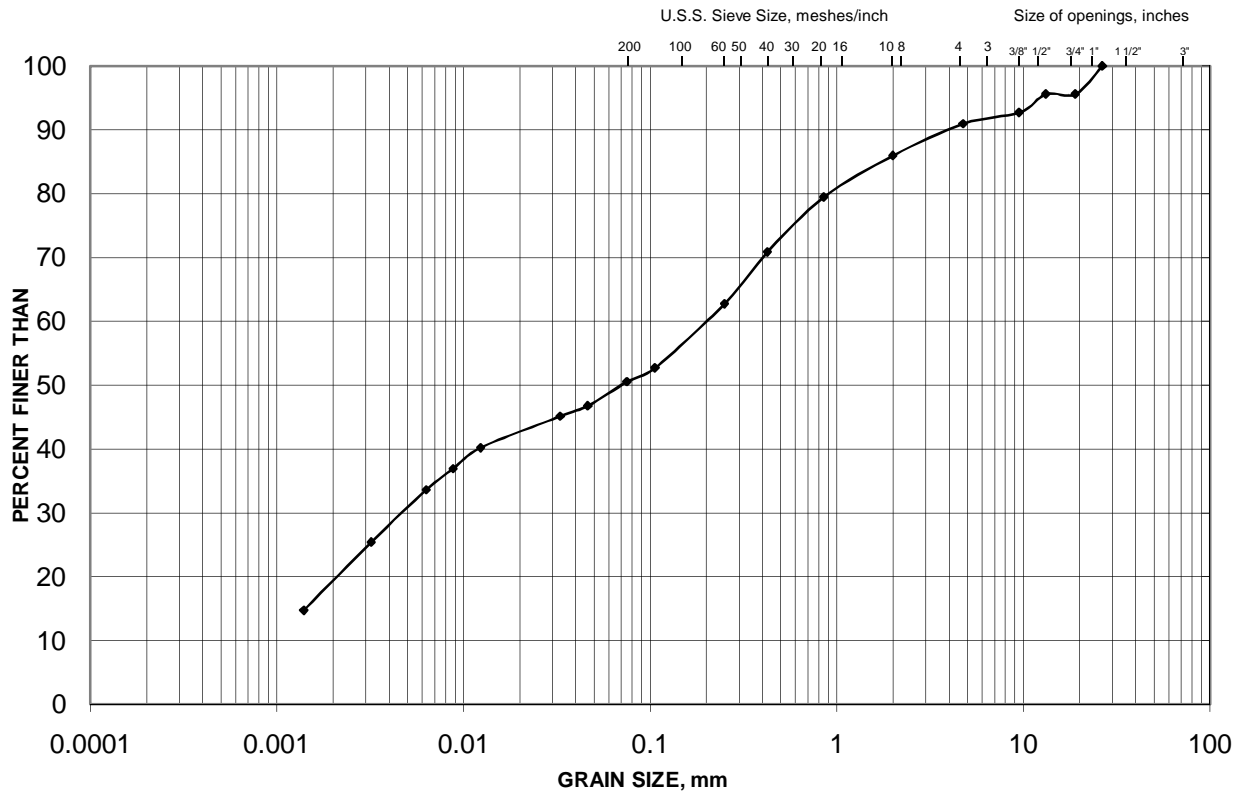
**Golder Associates**

Date: September 2009

# GRAIN SIZE DISTRIBUTION

## Sand and Silt (FILL)

**FIGURE**  
**B-2**



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		

### LEGEND

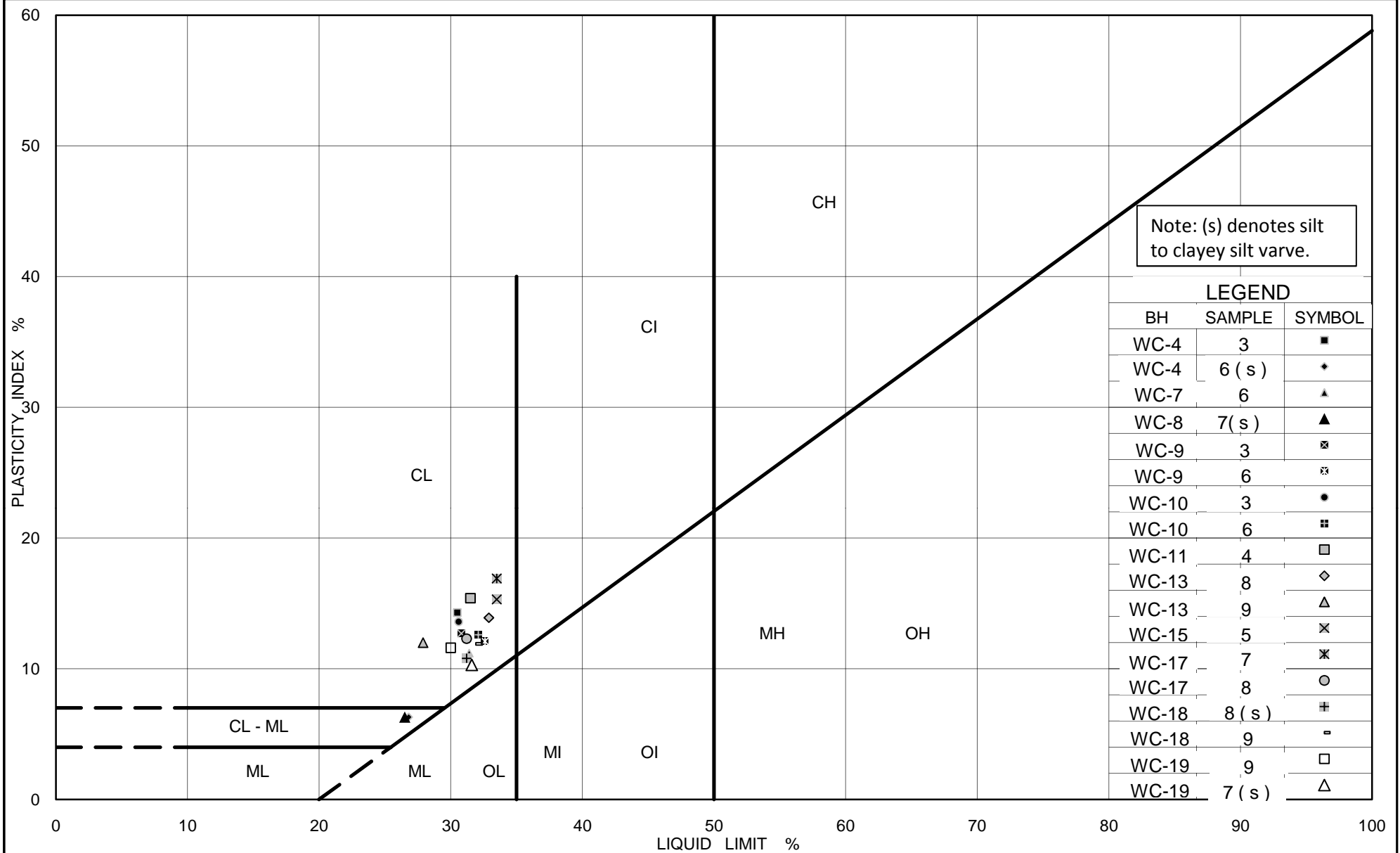
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—●—	WC16	1	310.7

Project Number: 07-1191-0008

Checked By: SEMC

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Date: September 2009



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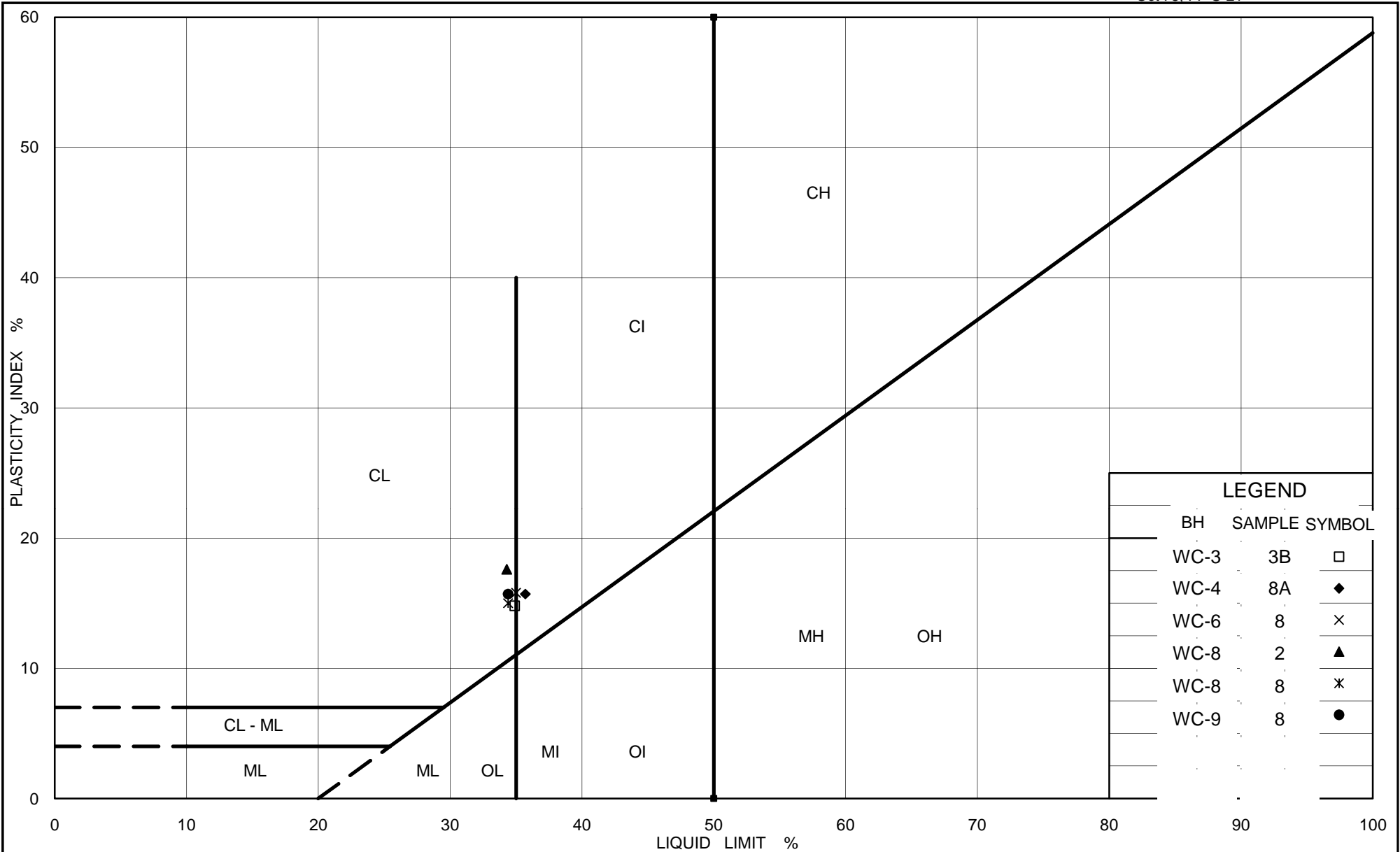
Ontario

# PLASTICITY CHART Clayey Silt

FIG No. B-3a

Project No. 07-1191-0008

Checked by: SEMC



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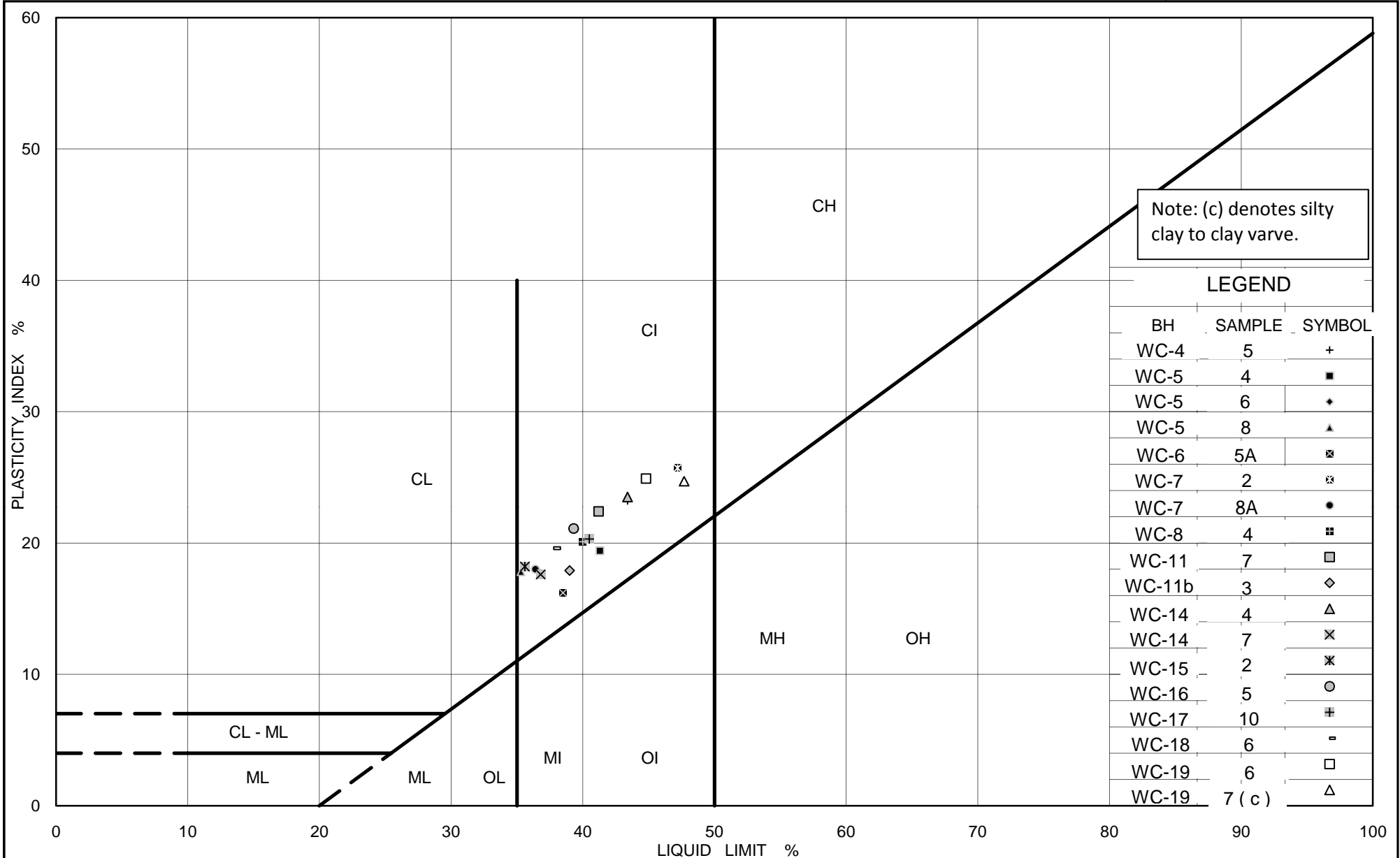
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# PLASTICITY CHART Clayey Silt to Silty Clay

Figure B-3b

Project No. 07-1191-0008

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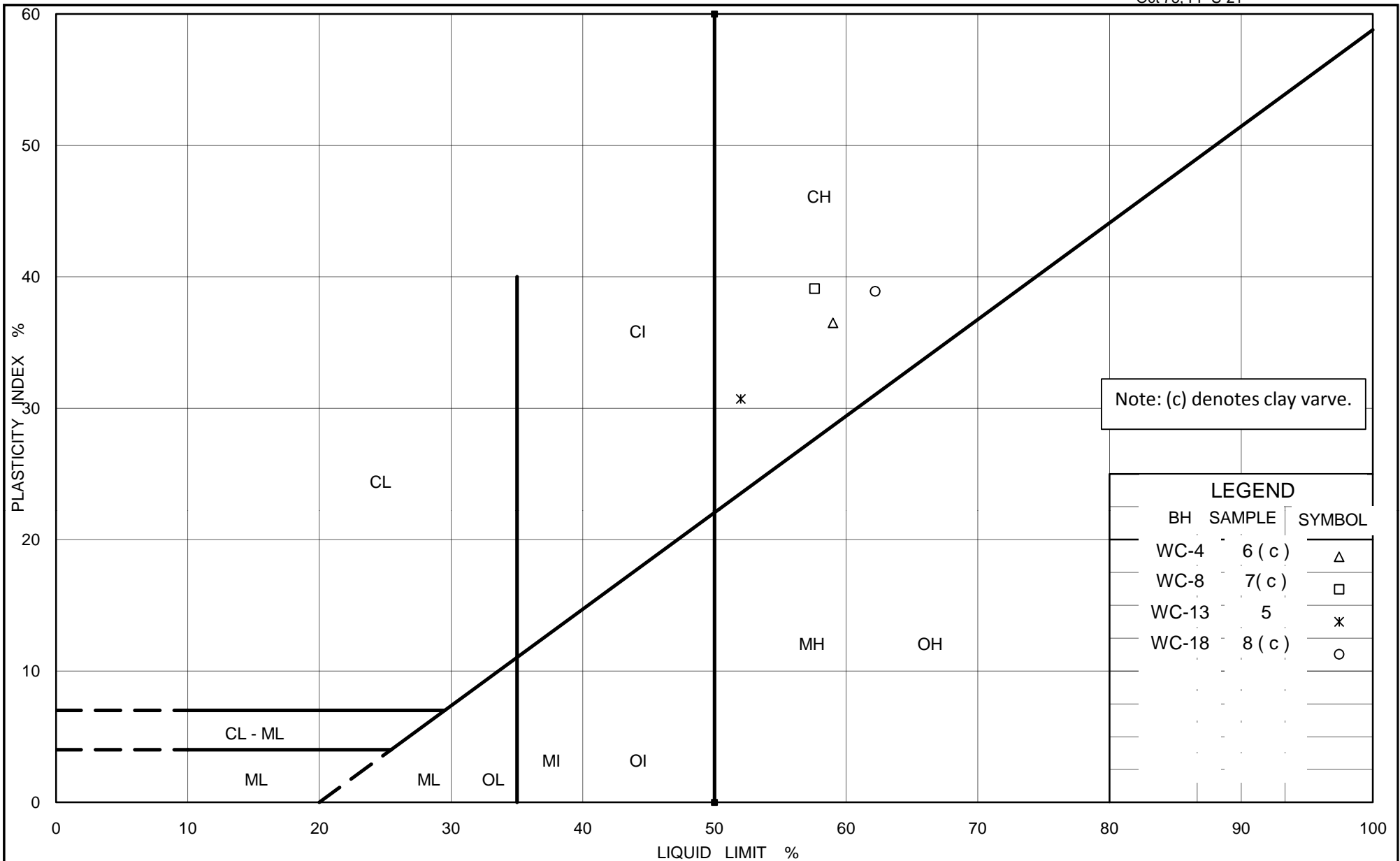
Ontario

# PLASTICITY CHART Silty Clay

FIG No. B-3c

Project No. 07-1191-0008

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## PLASTICITY CHART Clay Varves

Figure B-3d

Project No. 07-1191-0008

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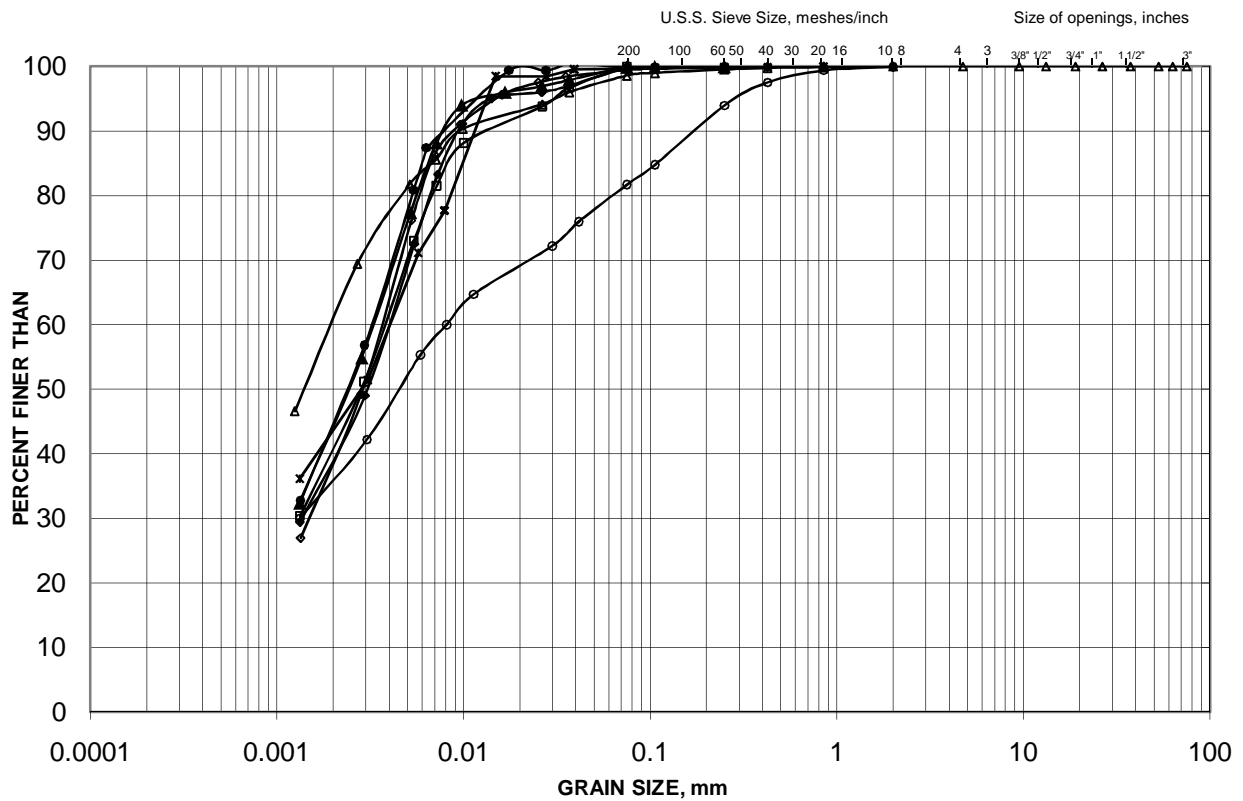










# GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE

B-4a



SILT AND CLAY SIZES				FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED				SAND SIZE			GRAVEL SIZE		
LEGEND	SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)					
		WC-3	3	307.6					
		WC-7	3	304.2					
		WC-9	5	304.2					
		WC-10	7	301.5					
		WC-11	8	303.8					
		WC-13	4b	308.0					
		WC-13	7	305.4					
		WC-16	4	308.6					

Project Number: 07-1191-0008

Checked By: SEMC

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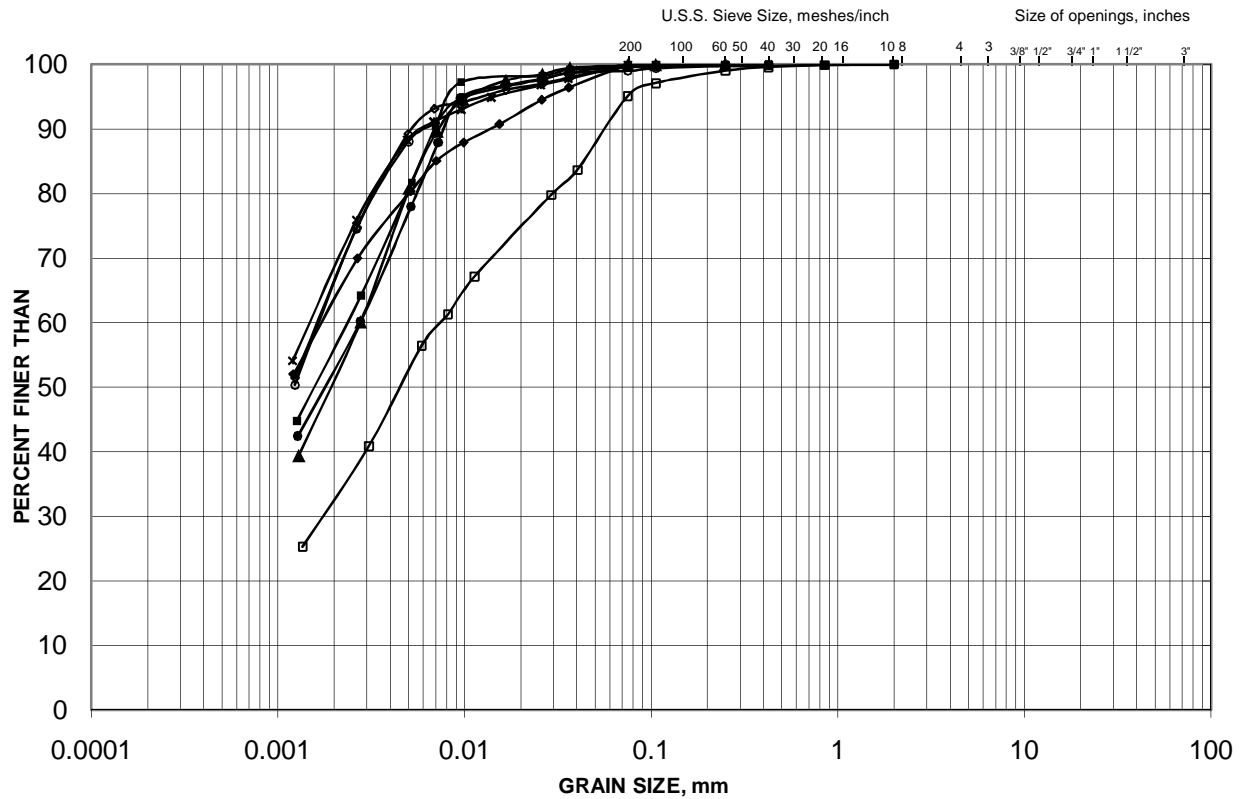
Date: September 2009

# GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE

B-4b



SILT AND CLAY SIZES				FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED				SAND SIZE			GRAVEL SIZE		
LEGEND	SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)					
	—●—	WC-4	4	306.7					
	—●—	WC-5	8	301.6					
	—■—	WC-5a	1	300.7					
	—▲—	WC-6	6	304.4					
	—✕—	WC-10	4	306.1					
	—●—	WC-11	6	306.2					
	—○—	WC-14	5	308.0					
	—□—	WC-15	3	309.6					

Project Number: 07-1191-0008

Checked By: SEMC

**Golder Associates**

Date: September 2009

**OEDOMETER CONSOLIDATION SUMMARY****Figure B-5**

Page 1 of 4

**SAMPLE IDENTIFICATION**

Project Number	07-1191-0008	Sample Number	6
Borehole Number	WC-4	Sample Depth, m	5.5-6.1

**TEST CONDITIONS**

Test Type	Standard	Load Duration, hr	24
Oedometer Number	12		
Date Started	08/13/2008		
Date Completed	09/06/2008		

**SAMPLE DIMENSIONS AND PROPERTIES - INITIAL**

Sample Height, cm	2.55	Unit Weight, kN/m <sup>3</sup>	16.82
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m <sup>3</sup>	11.06
Area, cm <sup>2</sup>	31.47	Specific Gravity, measured	2.74
Volume, cm <sup>3</sup>	80.25	Solids Height, cm	1.050
Water Content, %	52.06	Volume of Solids, cm <sup>3</sup>	33.04
Wet Mass, g	137.66	Volume of Voids, cm <sup>3</sup>	47.21
Dry Mass, g	90.53	Degree of Saturation, %	99.8

**TEST COMPUTATIONS**

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	2.550	1.429	2.550				
4.88	2.539	1.418	2.545	9	1.53E-01	8.84E-04	1.32E-05
9.61	2.532	1.412	2.536	24	5.68E-02	5.80E-04	3.23E-06
19.62	2.520	1.400	2.526	41	3.30E-02	4.70E-04	1.52E-06
39.13	2.497	1.378	2.509	17	7.85E-02	4.62E-04	3.56E-06
77.92	2.456	1.339	2.477	32	4.06E-02	4.14E-04	1.65E-06
155.63	2.231	1.125	2.344	112	1.04E-02	1.14E-03	1.16E-06
311.28	2.022	0.926	2.127	60	1.60E-02	5.27E-04	8.25E-07
622.00	1.895	0.805	1.959	276	2.95E-03	1.60E-04	4.63E-08
1244.88	1.789	0.704	1.842	240	3.00E-03	6.67E-05	1.96E-08
2489.90	1.695	0.614	1.742	66	9.75E-03	2.96E-05	2.83E-08
1244.88	1.707	0.626	1.701				
311.28	1.741	0.658	1.724				
77.92	1.779	0.694	1.760				
19.62	1.816	0.730	1.798				
4.88	1.849	0.761	1.833				

Note:

k calculated using cv based on t<sub>90</sub> values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	1.85	Unit Weight, kN/m <sup>3</sup>	20.06
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m <sup>3</sup>	15.26
Area, cm <sup>2</sup>	31.47	Specific Gravity, measured	2.74
Volume, cm <sup>3</sup>	58.19	Solids Height, cm	1.050
Water Content, %	31.50	Volume of Solids, cm <sup>3</sup>	33.04
Wet Mass, g	119.05	Volume of Voids, cm <sup>3</sup>	25.15
Dry Mass, g	90.53		

Prepared By: LFG

**Golder Associates**

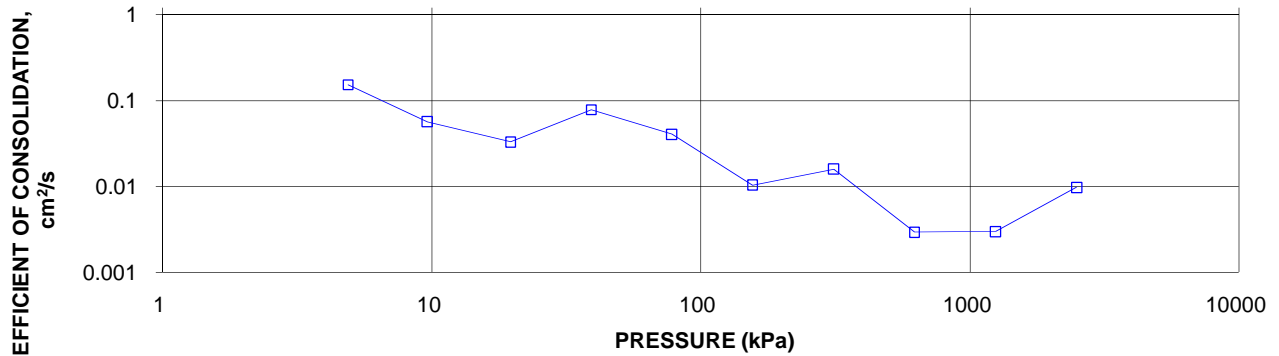
Checked By: MM

# OEDOMETER CONSOLIDATION SUMMARY

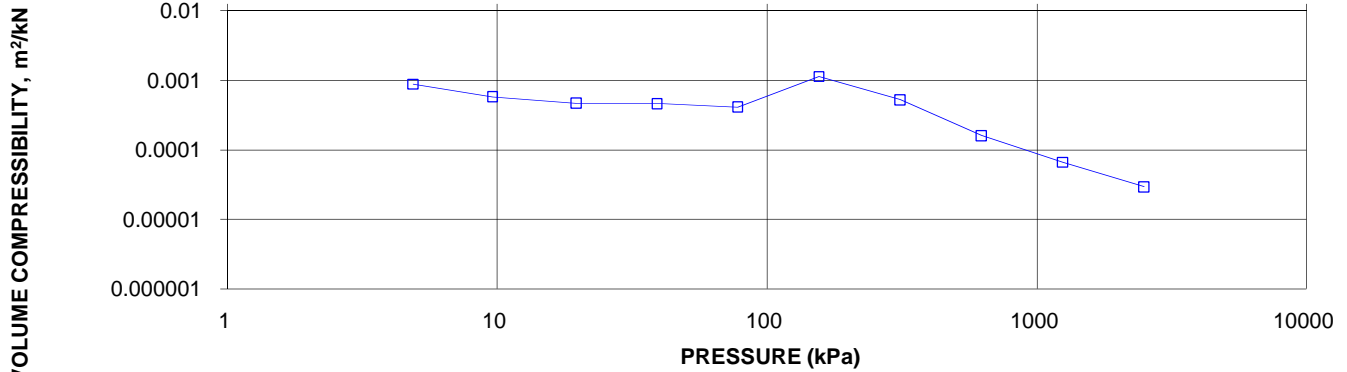
Figure B-5

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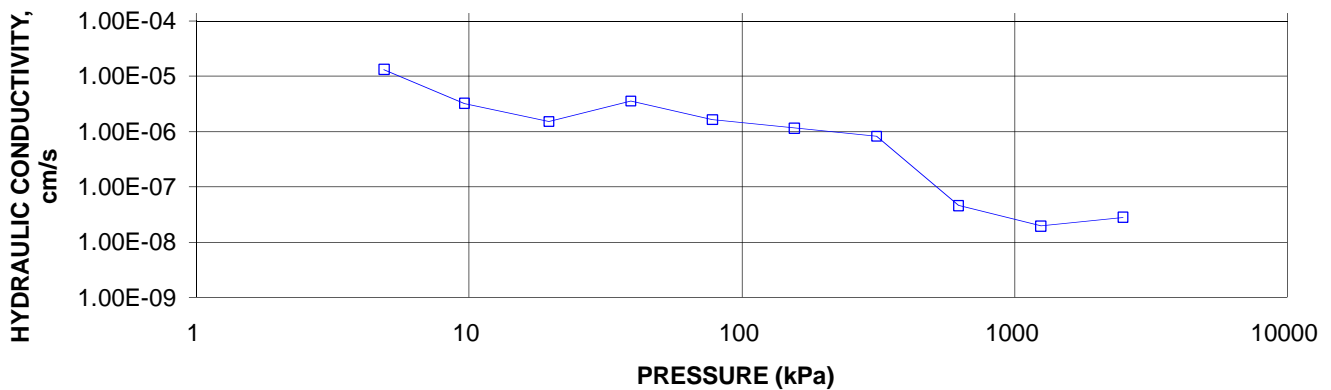
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
BH WC-4 SA 6



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
BH WC-4 SA 6



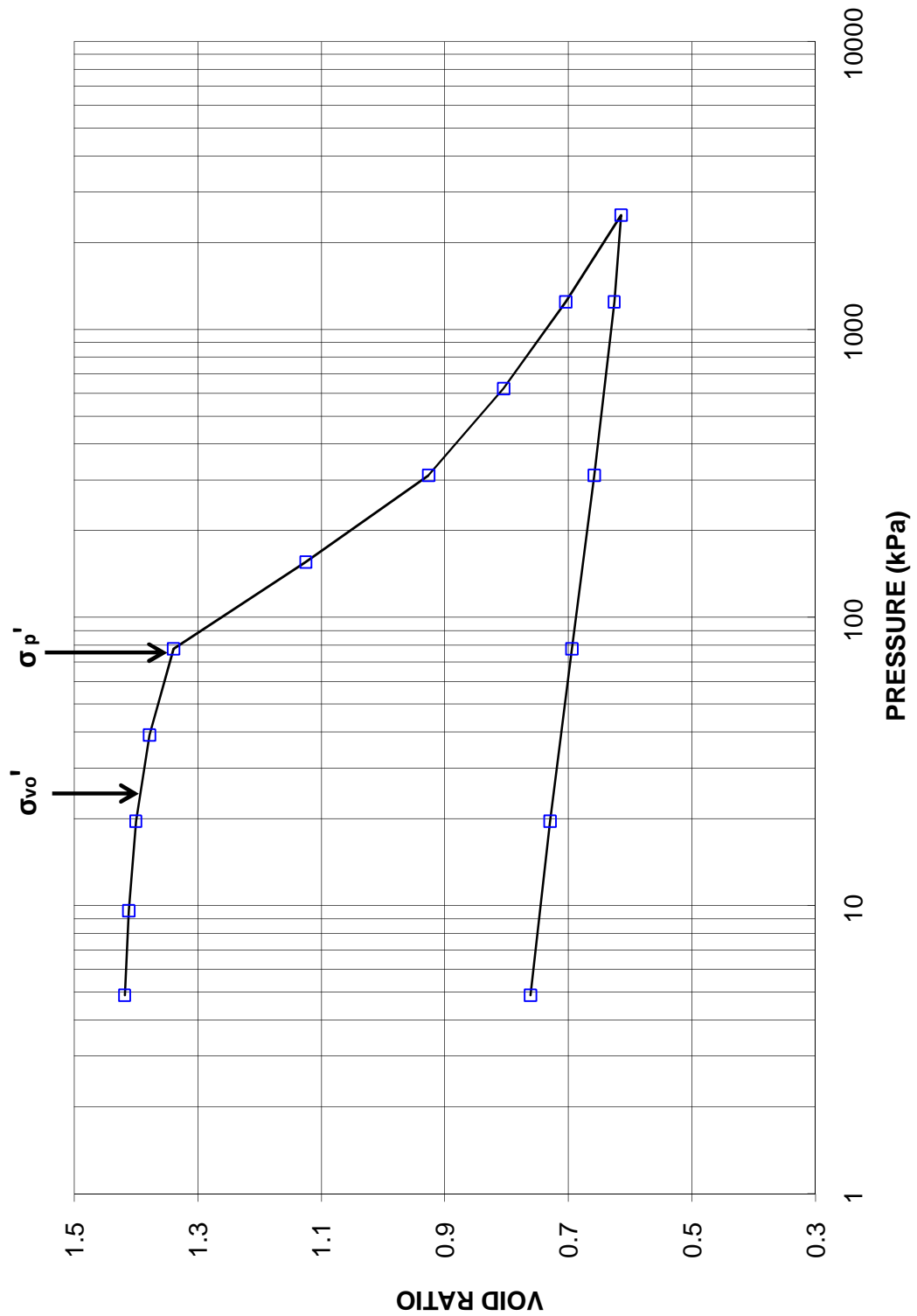
CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH WC-4 SA 6



**CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE**

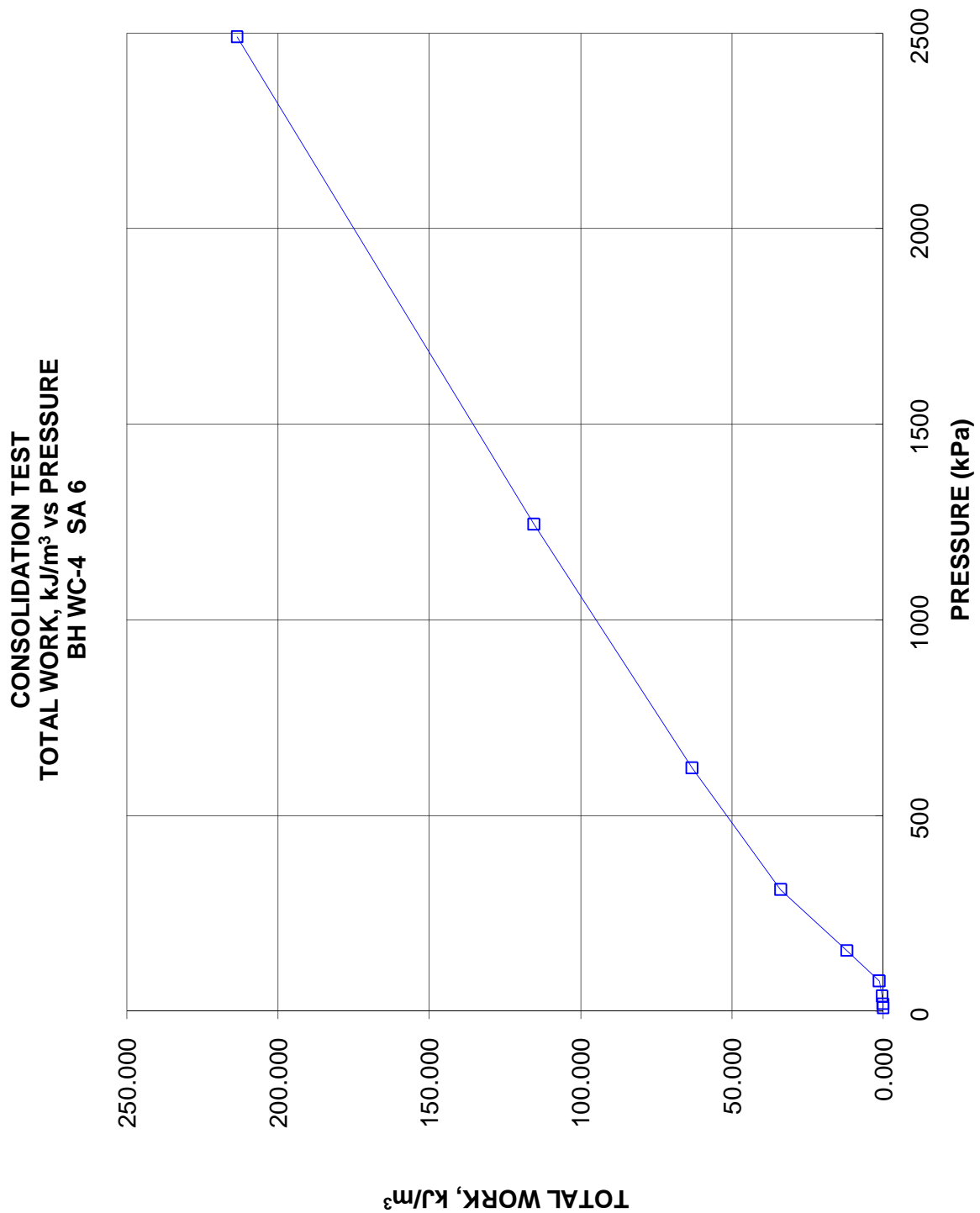
**FIGURE B-5**  
Page 3 of 4

**CONSOLIDATION TEST  
VOID RATIO vs PRESSURE  
BH WC-4 SA 6**



**CONSOLIDATION TEST  
TOTAL WORK VS. PRESSURE**

**FIGURE B-5**  
Page 4 of 4



# OEDOMETER CONSOLIDATION SUMMARY

**Figure B-6**

Page 1 of 4

## SAMPLE IDENTIFICATION

Project Number	07-1191-0008	Sample Number	7
Borehole Number	WC-8	Sample Depth, m	8.4-9.0

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	11		
Date Started	08/13/2008		
Date Completed	09/05/2008		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	16.84
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	11.02
Area, cm <sup>2</sup>	31.57	Specific Gravity, measured	2.75
Volume, cm <sup>3</sup>	80.16	Solids Height, cm	1.037
Water Content, %	52.88	Volume of Solids, cm <sup>3</sup>	32.75
Wet Mass, g	137.68	Volume of Voids, cm <sup>3</sup>	47.41
Dry Mass, g	90.06	Degree of Saturation, %	100.5

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	2.539	1.448	2.539				
4.84	2.512	1.422	2.526	32	4.23E-02	2.20E-03	9.10E-06
9.56	2.502	1.412	2.507	32	4.16E-02	8.34E-04	3.40E-06
19.34	2.485	1.395	2.494	75	1.76E-02	6.85E-04	1.18E-06
38.80	2.458	1.369	2.472	60	2.16E-02	5.46E-04	1.16E-06
77.62	2.407	1.320	2.433	108	1.16E-02	5.17E-04	5.89E-07
155.17	2.174	1.096	2.291	679	1.64E-03	1.18E-03	1.90E-07
310.34	1.999	0.927	2.087	891	1.04E-03	4.44E-04	4.51E-08
619.90	1.879	0.811	1.939	329	2.42E-03	1.53E-04	3.62E-08
1239.81	1.780	0.716	1.830	190	3.73E-03	6.29E-05	2.30E-08
2480.18	1.692	0.631	1.736	175	3.65E-03	2.79E-05	1.00E-08
1239.81	1.699	0.638	1.696				
310.34	1.724	0.662	1.712				
77.62	1.757	0.694	1.741				
19.34	1.792	0.727	1.775				
4.84	1.819	0.753	1.806				

Note:  
k calculated using cv based on t<sub>90</sub> values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.82	Unit Weight, kN/m <sup>3</sup>	19.98
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	15.38
Area, cm <sup>2</sup>	31.57	Specific Gravity, measured	2.75
Volume, cm <sup>3</sup>	57.43	Solids Height, cm	1.037
Water Content, %	29.91	Volume of Solids, cm <sup>3</sup>	32.75
Wet Mass, g	117.00	Volume of Voids, cm <sup>3</sup>	24.68
Dry Mass, g	90.06		

Prepared By: LFG

**Golder Associates**

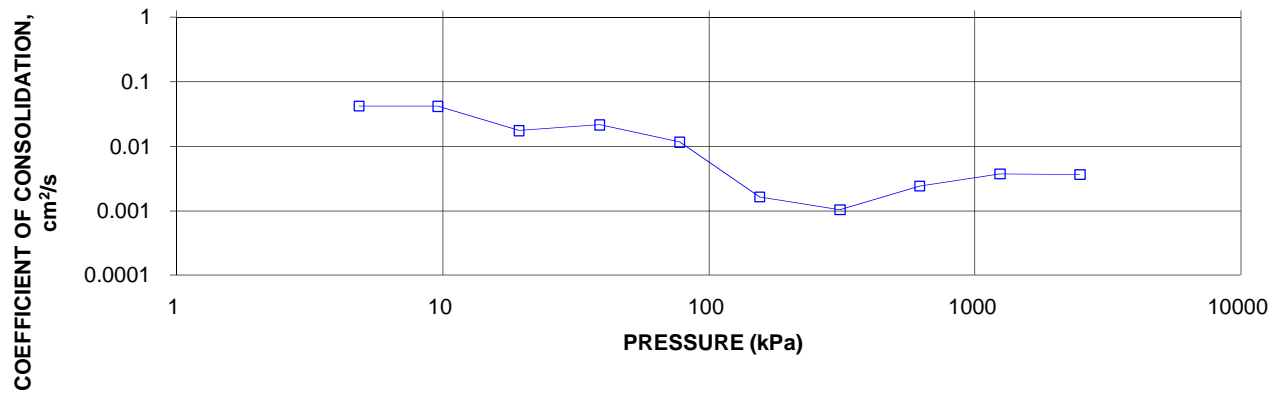
Checked By: MM

# OEDOMETER CONSOLIDATION SUMMARY

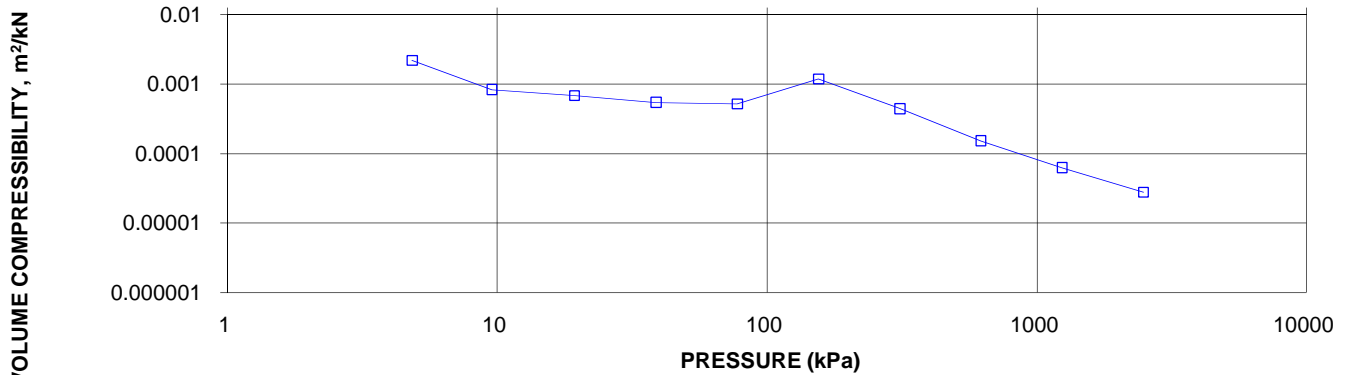
Figure B-6

Page 2 of 4

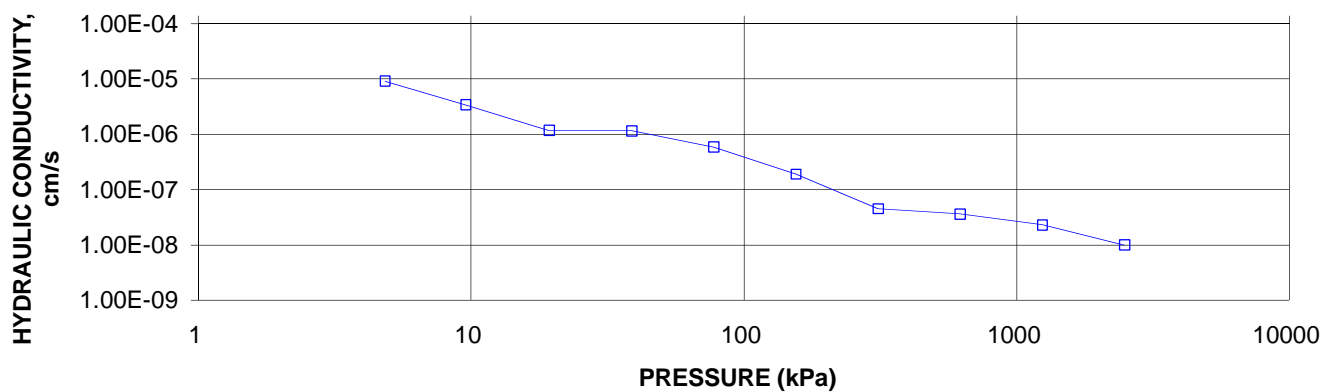
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
BH WC-8 SA 7



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
BH WC-8 SA 7



CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH WC-8 SA 7

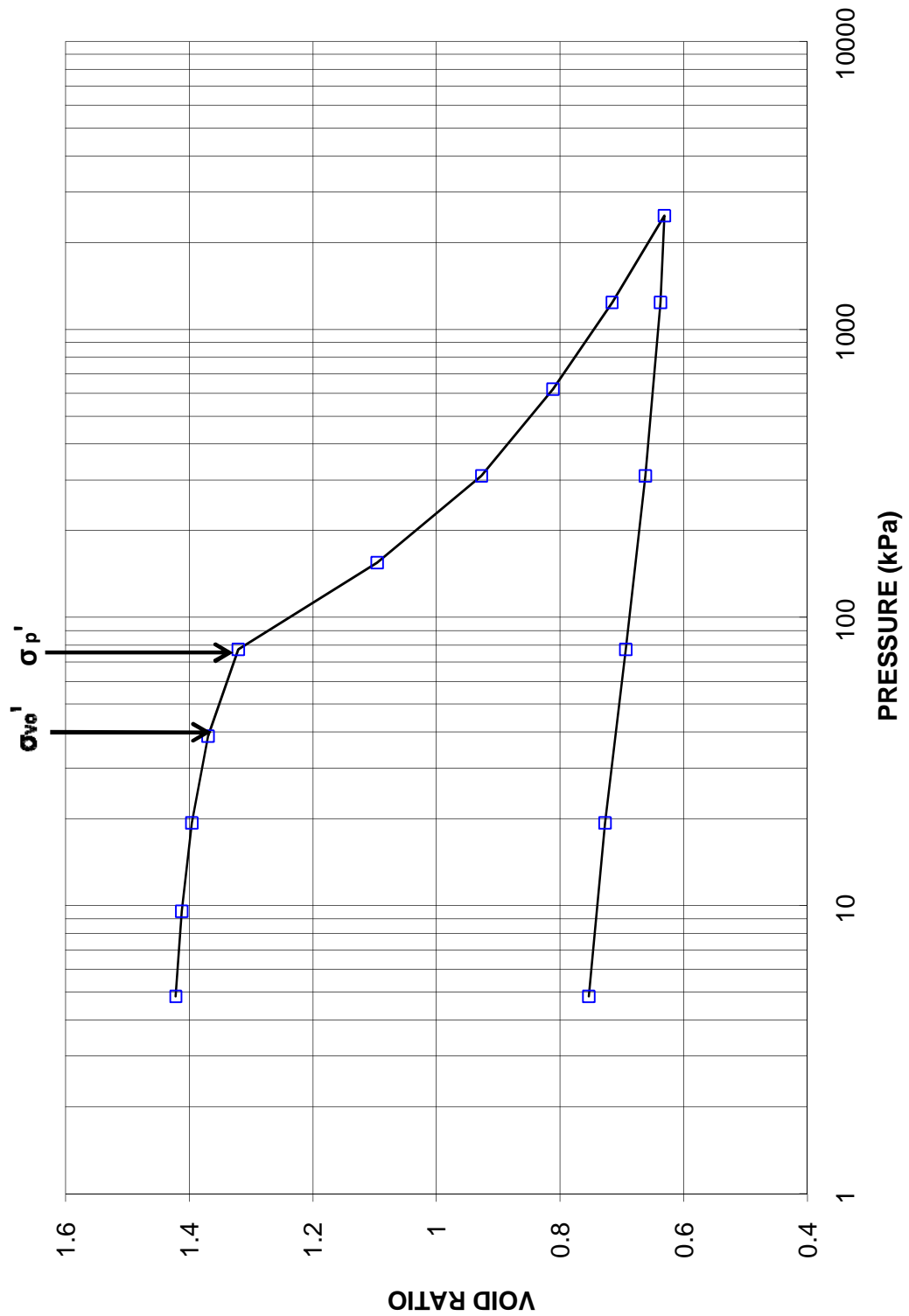




**CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE**

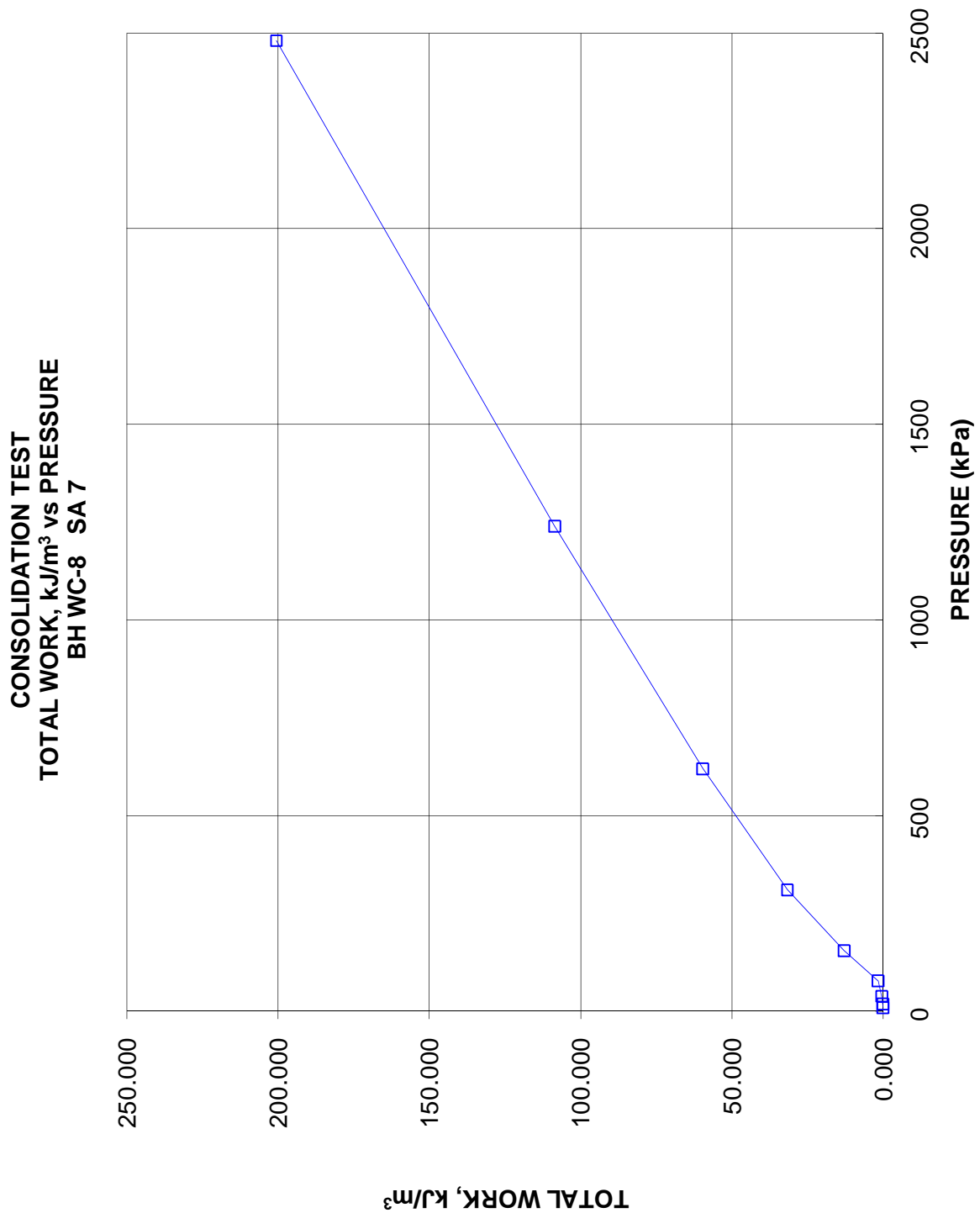
**FIGURE B-6**  
Page 3 of 4

**CONSOLIDATION TEST  
VOID RATIO vs PRESSURE  
BH WC-8 SA 7**



**CONSOLIDATION TEST  
TOTAL WORK VS. PRESSURE**

**FIGURE B-6**  
Page 4 of 4



# OEDOMETER CONSOLIDATION SUMMARY

**Figure B-7**

Page 1 of 4

## SAMPLE IDENTIFICATION

Project Number	07-1191-0008	Sample Number	8
Borehole Number	WC-13	Sample Depth, m	6.1-6.7

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	12/12/2008		
Date Completed	12/31/2008		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.25	Unit Weight, kN/m <sup>3</sup>	15.48
Sample Diameter, cm	4.97	Dry Unit Weight, kN/m <sup>3</sup>	8.84
Area, cm <sup>2</sup>	19.40	Specific Gravity, measured	2.74
Volume, cm <sup>3</sup>	24.31	Solids Height, cm	0.412
Water Content, %	75.08	Volume of Solids, cm <sup>3</sup>	8.00
Wet Mass, g	38.36	Volume of Voids, cm <sup>3</sup>	16.31
Dry Mass, g	21.91	Degree of Saturation, %	100.8

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.253	2.040	1.253				
4.95	1.245	2.021	1.249	19	1.74E-02	1.29E-03	2.20E-06
9.91	1.242	2.012	1.243	69	4.75E-03	5.63E-04	2.62E-07
19.79	1.231	1.987	1.236	60	5.40E-03	8.48E-04	4.49E-07
39.59	1.215	1.948	1.223	21	1.51E-02	6.45E-04	9.54E-07
80.00	1.145	1.778	1.180	39	7.57E-03	1.38E-03	1.03E-06
160.00	0.942	1.284	1.043	803	2.87E-04	2.03E-03	5.72E-08
320.00	0.844	1.048	0.893	386	4.38E-04	4.86E-04	2.09E-08
640.00	0.769	0.866	0.807	165	8.36E-04	1.87E-04	1.53E-08
1280.00	0.707	0.715	0.738	41	2.82E-03	7.73E-05	2.13E-08
2559.95	0.653	0.584	0.680	60	1.63E-03	3.37E-05	5.39E-09
1280.00	0.661	0.604	0.657				
320.00	0.686	0.663	0.673				
80.00	0.712	0.726	0.699				
19.79	0.733	0.778	0.722				
4.95	0.746	0.810	0.740				

Note:  
k calculated using cv based on t<sub>90</sub> values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	0.75	Unit Weight, kN/m <sup>3</sup>	20.28
Sample Diameter, cm	4.97	Dry Unit Weight, kN/m <sup>3</sup>	14.85
Area, cm <sup>2</sup>	19.40	Specific Gravity, measured	2.74
Volume, cm <sup>3</sup>	14.47	Solids Height, cm	0.412
Water Content, %	36.60	Volume of Solids, cm <sup>3</sup>	8.00
Wet Mass, g	29.93	Volume of Voids, cm <sup>3</sup>	6.48
Dry Mass, g	21.91		

Prepared By: LFG

**Golder Associates**

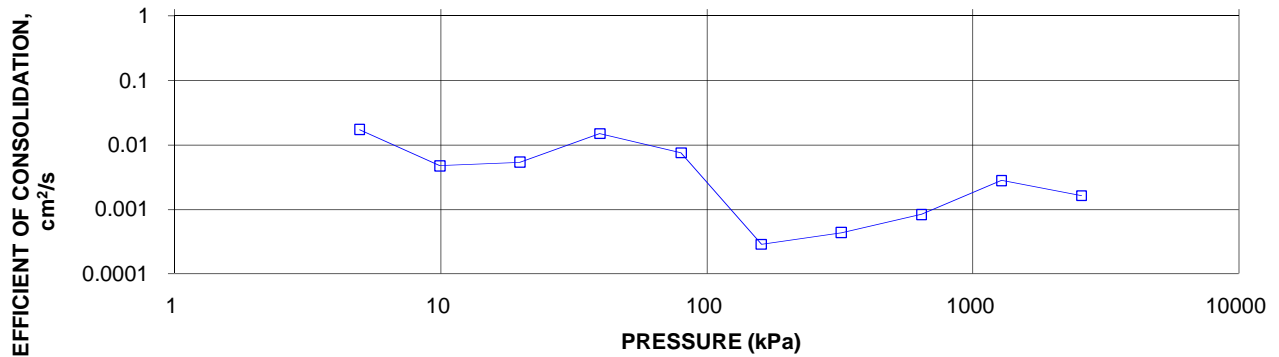
Checked By: MM

# OEDOMETER CONSOLIDATION SUMMARY

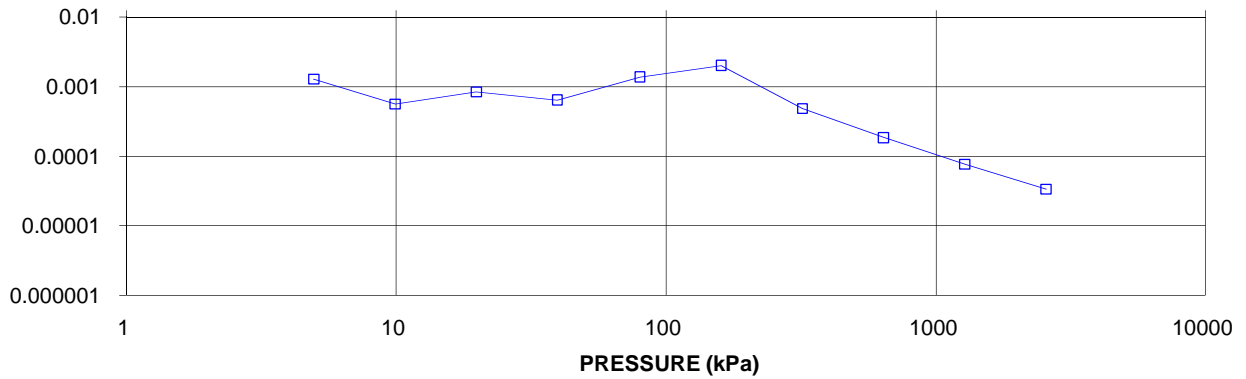
Figure B-7

Page 2 of 4

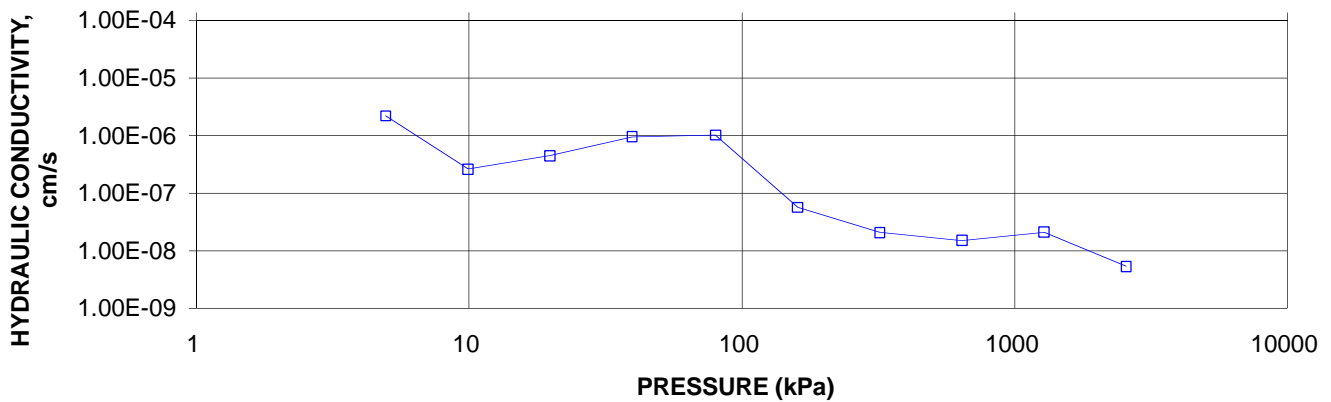
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
BH WC-13 SA 8



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
BH WC-13 SA 8



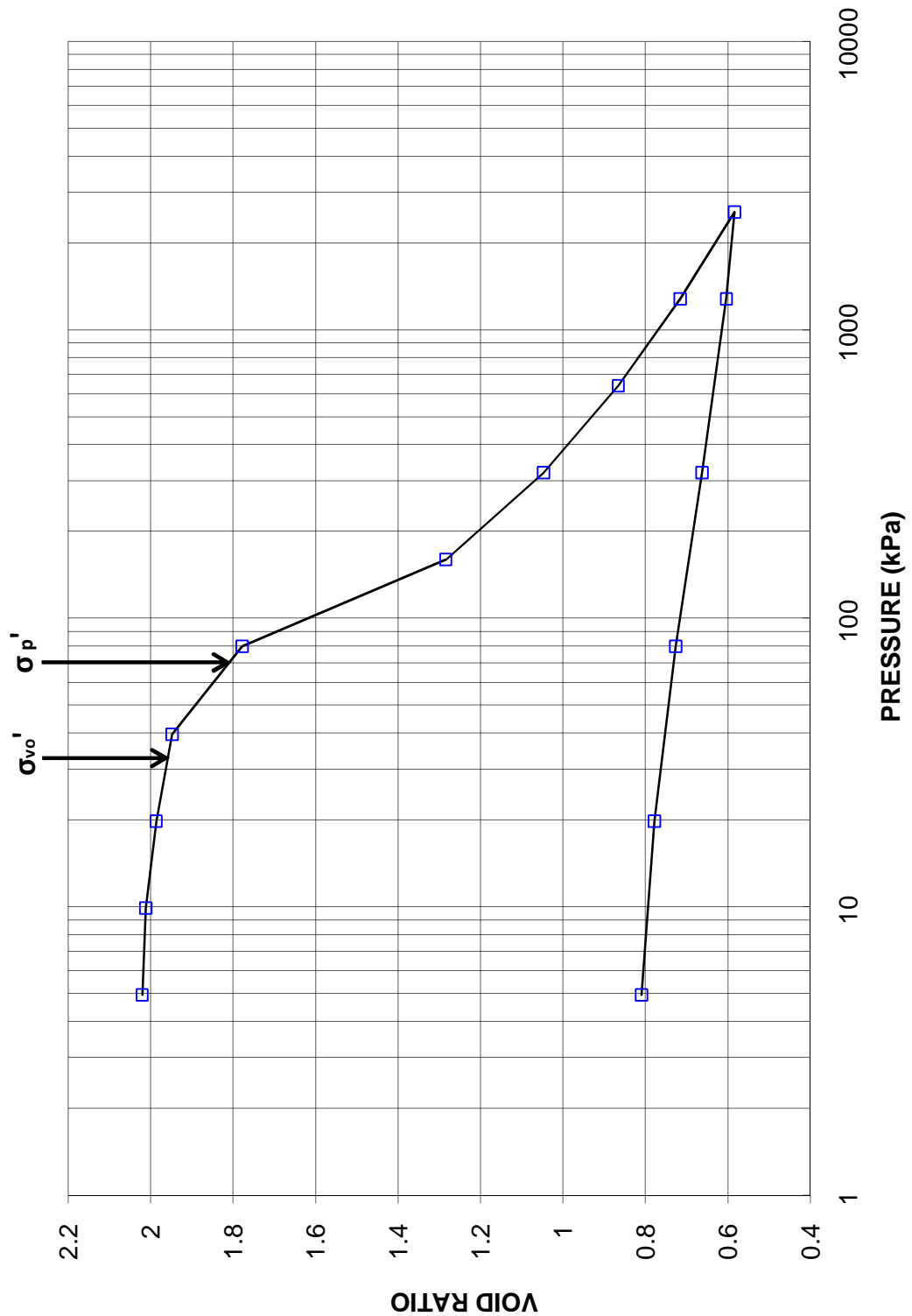
CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH WC-13 SA 8



# CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

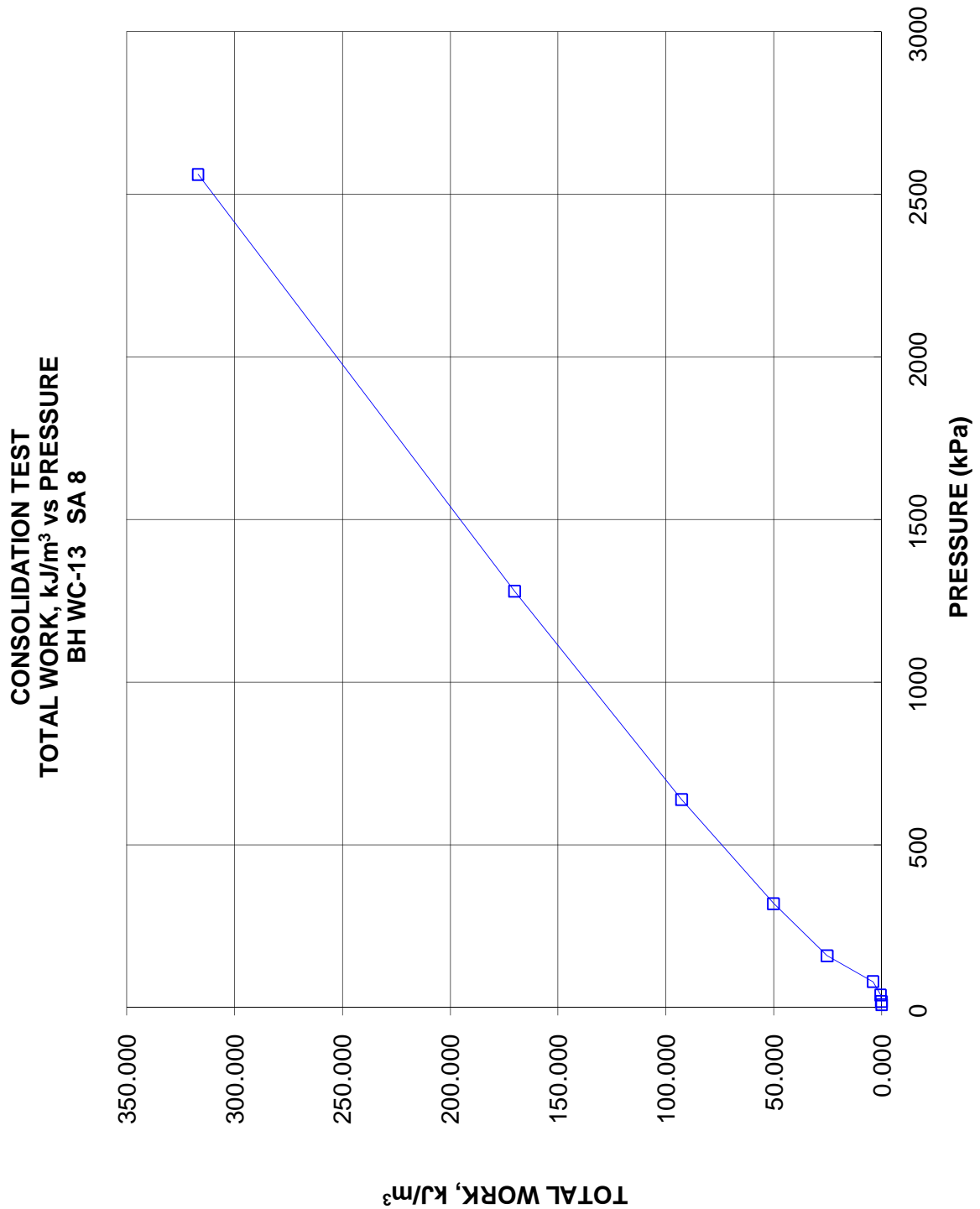
FIGURE B-7  
Page 3 of 4

CONSOLIDATION TEST  
VOID RATIO vs PRESSURE  
BH WC-13 SA 8



**CONSOLIDATION TEST  
TOTAL WORK VS. PRESSURE**

**FIGURE B-7**  
Page 4 of 4



# OEDOMETER CONSOLIDATION SUMMARY

**Figure B-8**

Page 1 of 4

## SAMPLE IDENTIFICATION

Project Number	07-1191-0008	Sample Number	7
Borehole Number	WC-17	Sample Depth, m	6.1-6.7

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	11/24/2008		
Date Completed	12/16/2008		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	19.43
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	15.32
Area, cm <sup>2</sup>	31.57	Specific Gravity, measured	2.74
Volume, cm <sup>3</sup>	80.28	Solids Height, cm	1.450
Water Content, %	26.88	Volume of Solids, cm <sup>3</sup>	45.76
Wet Mass, g	159.10	Volume of Voids, cm <sup>3</sup>	34.52
Dry Mass, g	125.39	Degree of Saturation, %	97.7

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	2.543	0.754	2.543				
4.71	2.533	0.747	2.538	97	1.41E-02	8.35E-04	1.15E-06
9.58	2.529	0.744	2.531	120	1.13E-02	3.55E-04	3.94E-07
19.34	2.518	0.737	2.523	175	7.71E-03	4.47E-04	3.38E-07
38.80	2.502	0.726	2.510	120	1.11E-02	3.19E-04	3.48E-07
77.76	2.479	0.710	2.490	32	4.11E-02	2.34E-04	9.43E-07
155.06	2.439	0.683	2.459	32	4.01E-02	2.01E-04	7.89E-07
310.21	2.388	0.647	2.413	37	3.34E-02	1.31E-04	4.27E-07
619.37	2.324	0.603	2.356	32	3.68E-02	8.03E-05	2.89E-07
1240.78	2.259	0.558	2.291	15	7.42E-02	4.17E-05	3.03E-07
2481.24	2.191	0.512	2.225	40	2.62E-02	2.13E-05	5.48E-08
1240.78	2.194	0.513	2.192				
310.21	2.223	0.533	2.208				
77.76	2.253	0.554	2.238				
19.34	2.279	0.572	2.266				
4.71	2.299	0.586	2.289				

Note:  
k calculated using cv based on t<sub>90</sub> values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.30	Unit Weight, kN/m <sup>3</sup>	20.69
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	16.94
Area, cm <sup>2</sup>	31.57	Specific Gravity, measured	2.74
Volume, cm <sup>3</sup>	72.58	Solids Height, cm	1.450
Water Content, %	22.10	Volume of Solids, cm <sup>3</sup>	45.76
Wet Mass, g	153.10	Volume of Voids, cm <sup>3</sup>	26.82
Dry Mass, g	125.39		

Prepared By: LFG

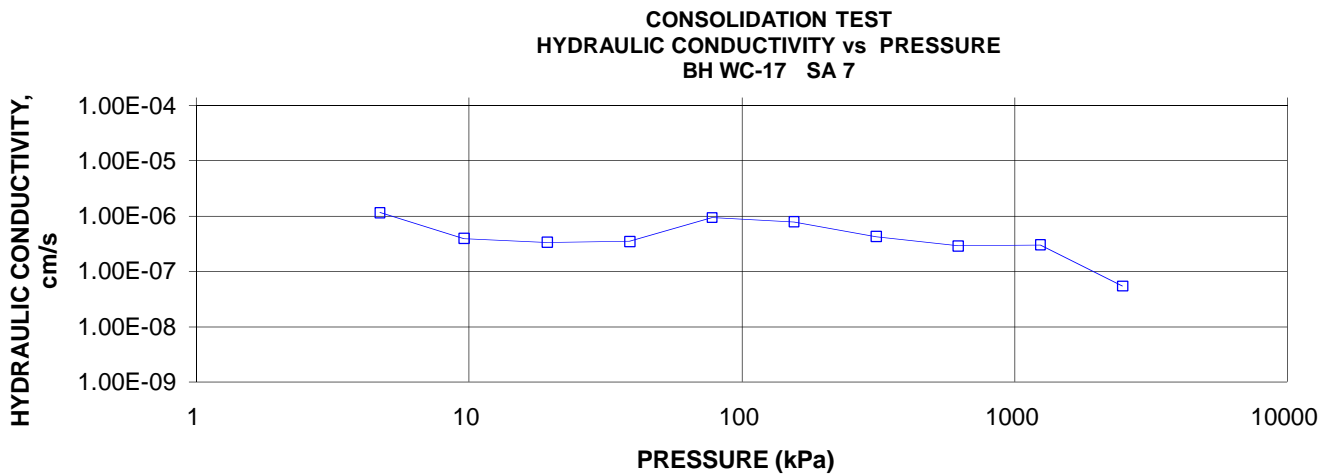
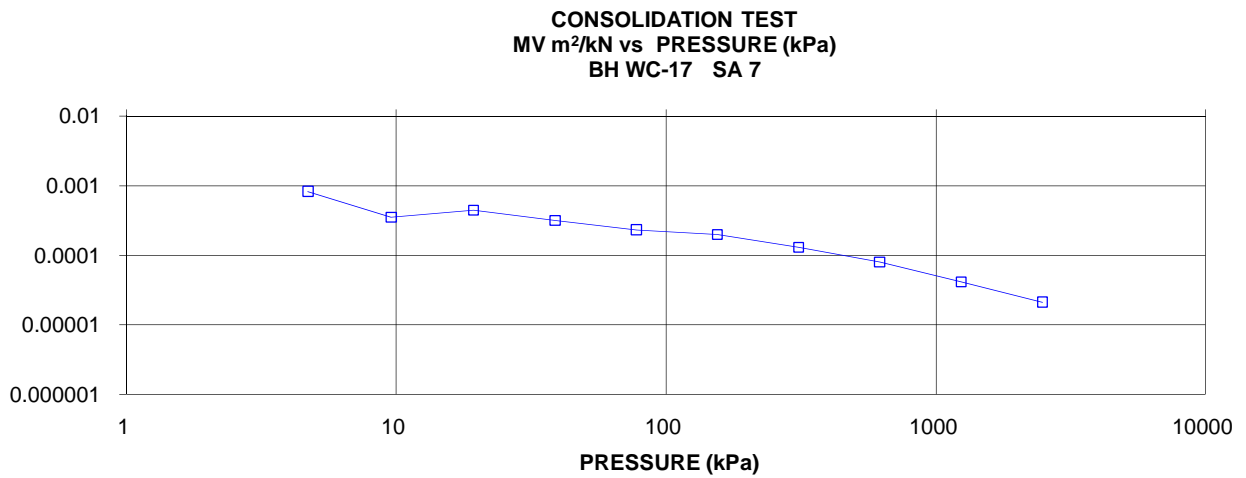
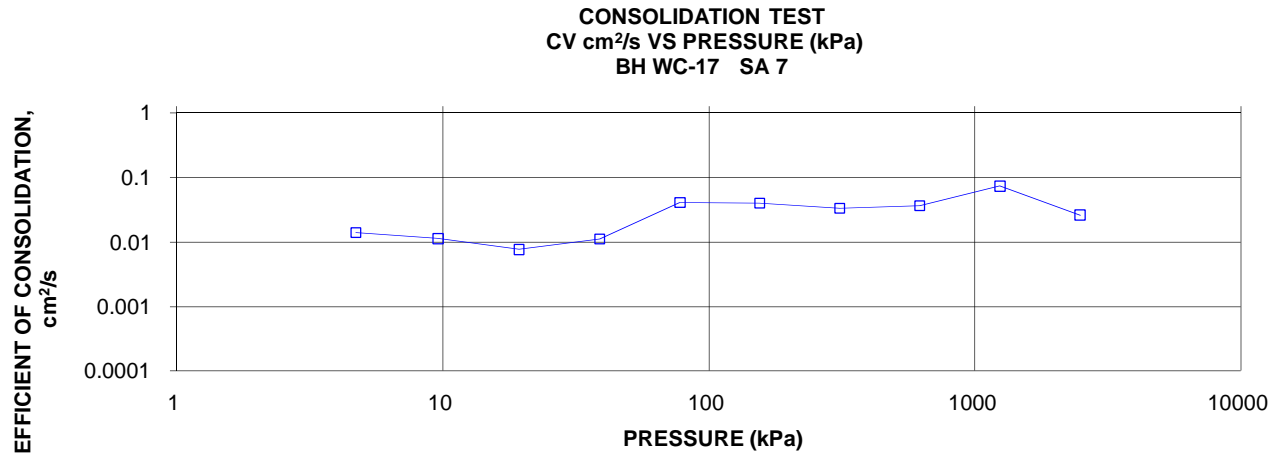
**Golder Associates**

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# OEDOMETER CONSOLIDATION SUMMARY

Figure B-8

Page 2 of 4



Project No. 07-1191-0008

Prepared By: LFG

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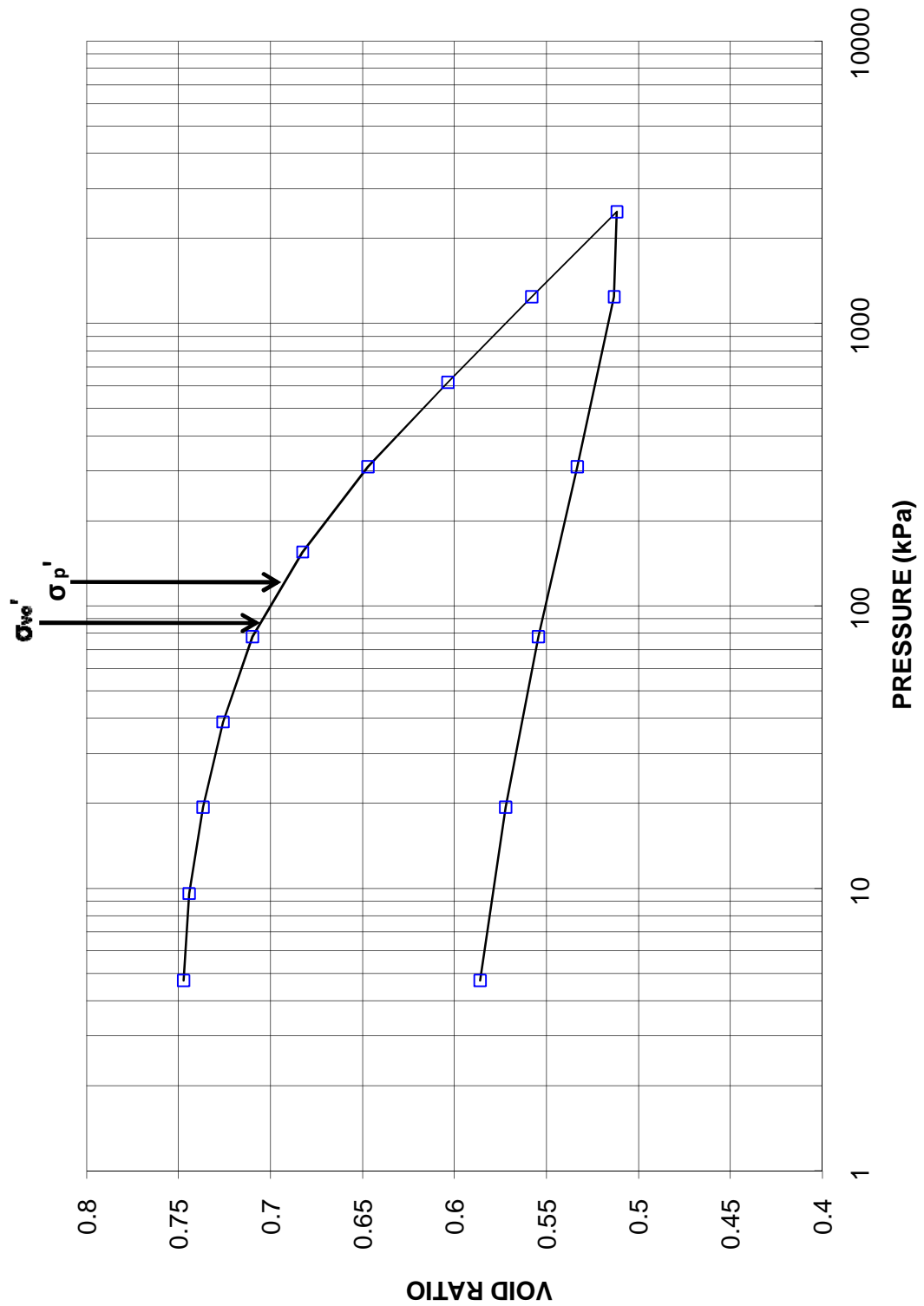
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# CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

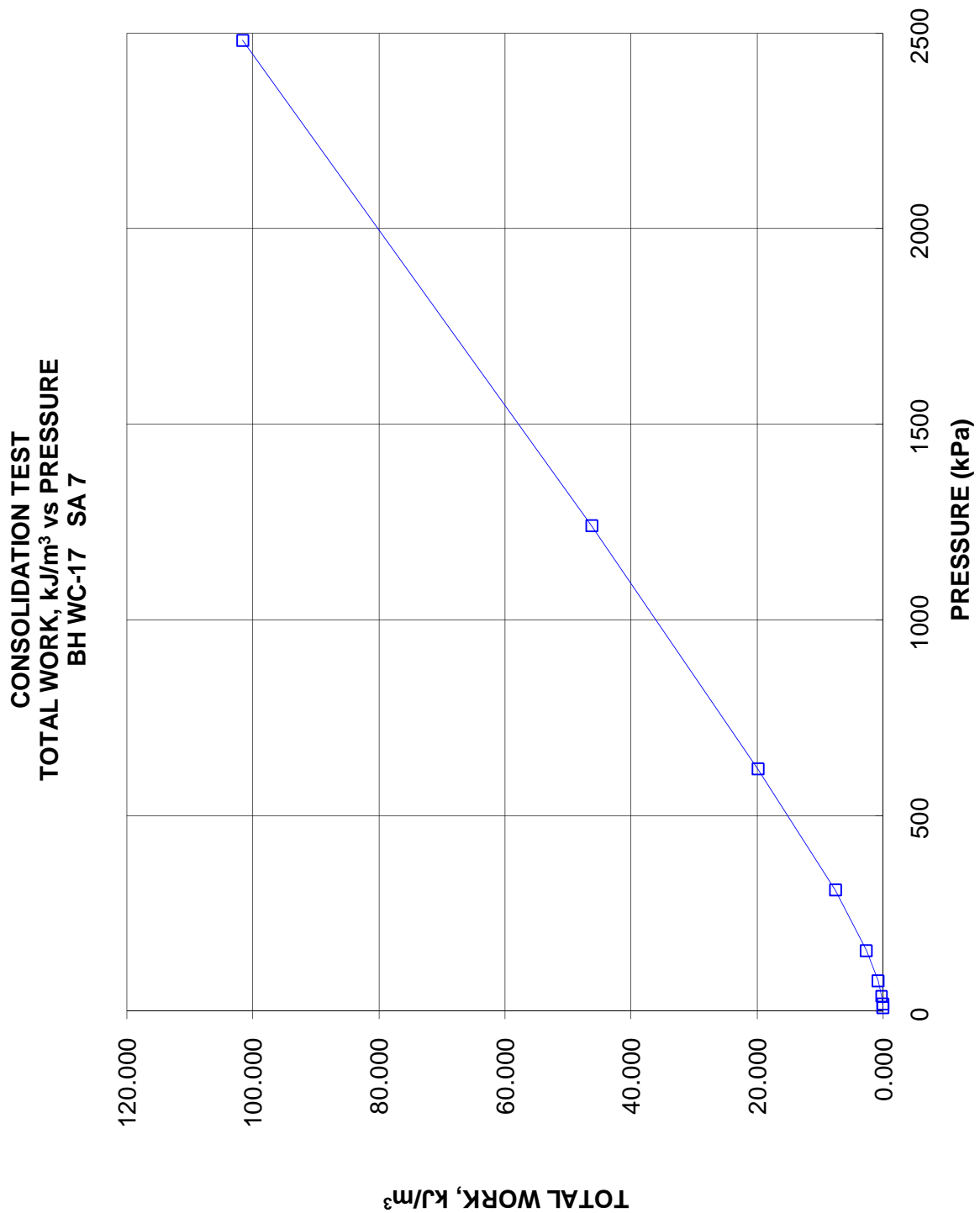
FIGURE B-8  
Page 3 of 4

CONSOLIDATION TEST  
VOID RATIO VS. PRESSURE  
BH WC-17 SA 7



**CONSOLIDATION TEST  
TOTAL WORK VS. PRESSURE**

**FIGURE B-8**  
Page 4 of 4



# OEDOMETER CONSOLIDATION SUMMARY

Figure B-9

Page 1 of 4

## SAMPLE IDENTIFICATION

Project Number	07-1191-0008	Sample Number	8
Borehole Number	WC-18	Sample Depth, m	8.8-9.8

## TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	11/24/2008		
Date Completed	12/13/2008		

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m <sup>3</sup>	17.42
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	12.01
Area, cm <sup>2</sup>	31.55	Specific Gravity, measured	2.75
Volume, cm <sup>3</sup>	59.76	Solids Height, cm	0.843
Water Content, %	45.03	Volume of Solids, cm <sup>3</sup>	26.61
Wet Mass, g	106.12	Volume of Voids, cm <sup>3</sup>	33.15
Dry Mass, g	73.17	Degree of Saturation, %	99.4

## TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s
0.00	1.894	1.246	1.894				
4.84	1.881	1.230	1.888	7	1.08E-01	1.42E-03	1.50E-05
9.54	1.877	1.226	1.879	60	1.25E-02	4.27E-04	5.22E-07
19.48	1.862	1.208	1.870	120	6.18E-03	8.07E-04	4.89E-07
38.83	1.841	1.183	1.852	89	8.17E-03	5.65E-04	4.52E-07
77.73	1.804	1.139	1.822	79	8.91E-03	5.13E-04	4.48E-07
155.24	1.703	1.019	1.753	567	1.15E-03	6.87E-04	7.73E-08
310.47	1.617	0.917	1.660	540	1.08E-03	2.93E-04	3.10E-08
621.61	1.540	0.826	1.578	46	1.15E-02	1.31E-04	1.47E-07
1243.21	1.473	0.746	1.506	69	6.97E-03	5.67E-05	3.88E-08
2484.35	1.412	0.675	1.443	30	1.47E-02	2.57E-05	3.70E-08
1243.21	1.413	0.675	1.413				
310.47	1.445	0.713	1.429				
77.73	1.481	0.756	1.463				
19.48	1.513	0.794	1.497				
4.84	1.539	0.825	1.526				

Note:  
k calculated using cv based on  $t_{90}$  values.

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.54	Unit Weight, kN/m <sup>3</sup>	19.26
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	14.77
Area, cm <sup>2</sup>	31.55	Specific Gravity, measured	2.75
Volume, cm <sup>3</sup>	48.57	Solids Height, cm	0.843
Water Content, %	30.34	Volume of Solids, cm <sup>3</sup>	26.61
Wet Mass, g	95.37	Volume of Voids, cm <sup>3</sup>	21.96
Dry Mass, g	73.17		

Prepared By: LFG

**Golder Associates**

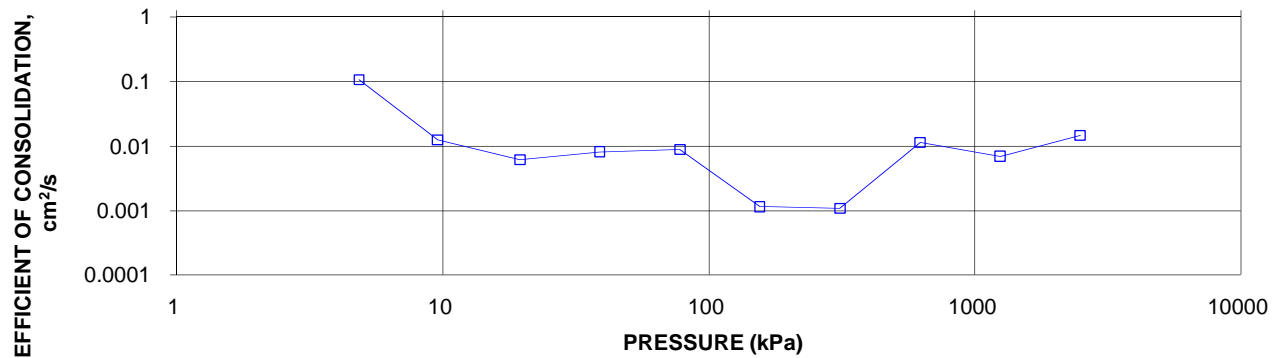
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# OEDOMETER CONSOLIDATION SUMMARY

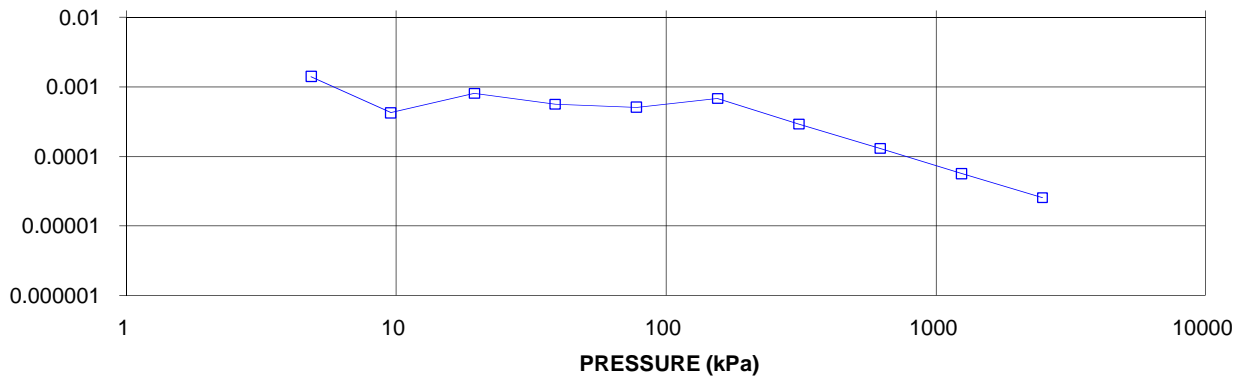
Figure B-9

Page 2 of 4

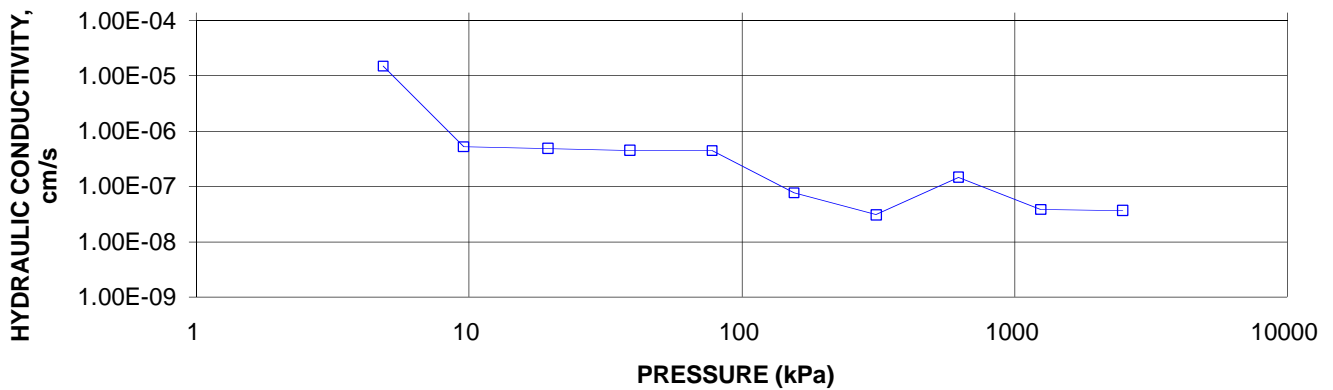
CONSOLIDATION TEST  
CV cm<sup>2</sup>/s VS PRESSURE (kPa)  
BH WC-18 SA 8



CONSOLIDATION TEST  
MV m<sup>2</sup>/kN vs PRESSURE (kPa)  
BH WC-18 SA 8



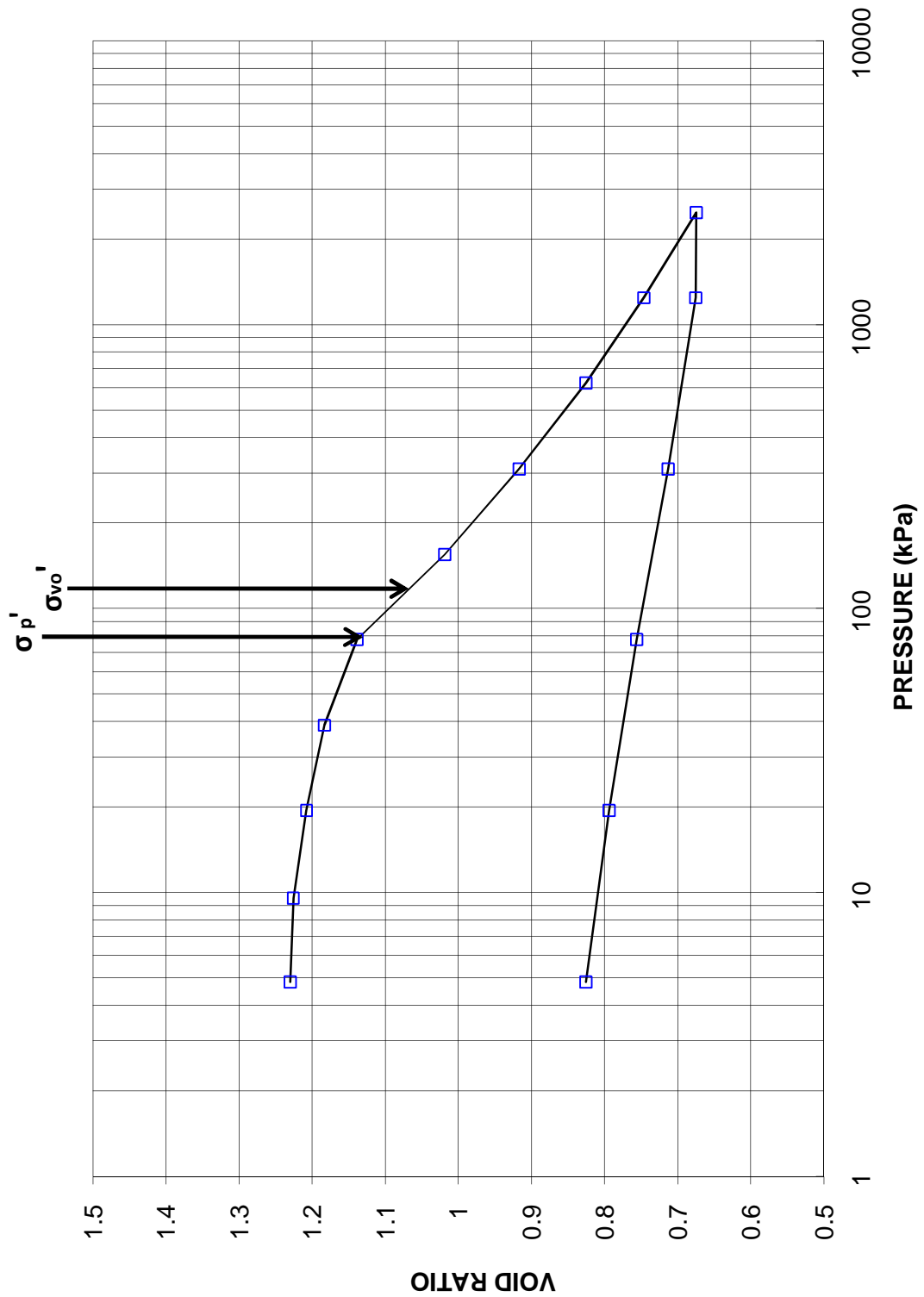
CONSOLIDATION TEST  
HYDRAULIC CONDUCTIVITY vs PRESSURE  
BH WC-18 SA 8



# CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

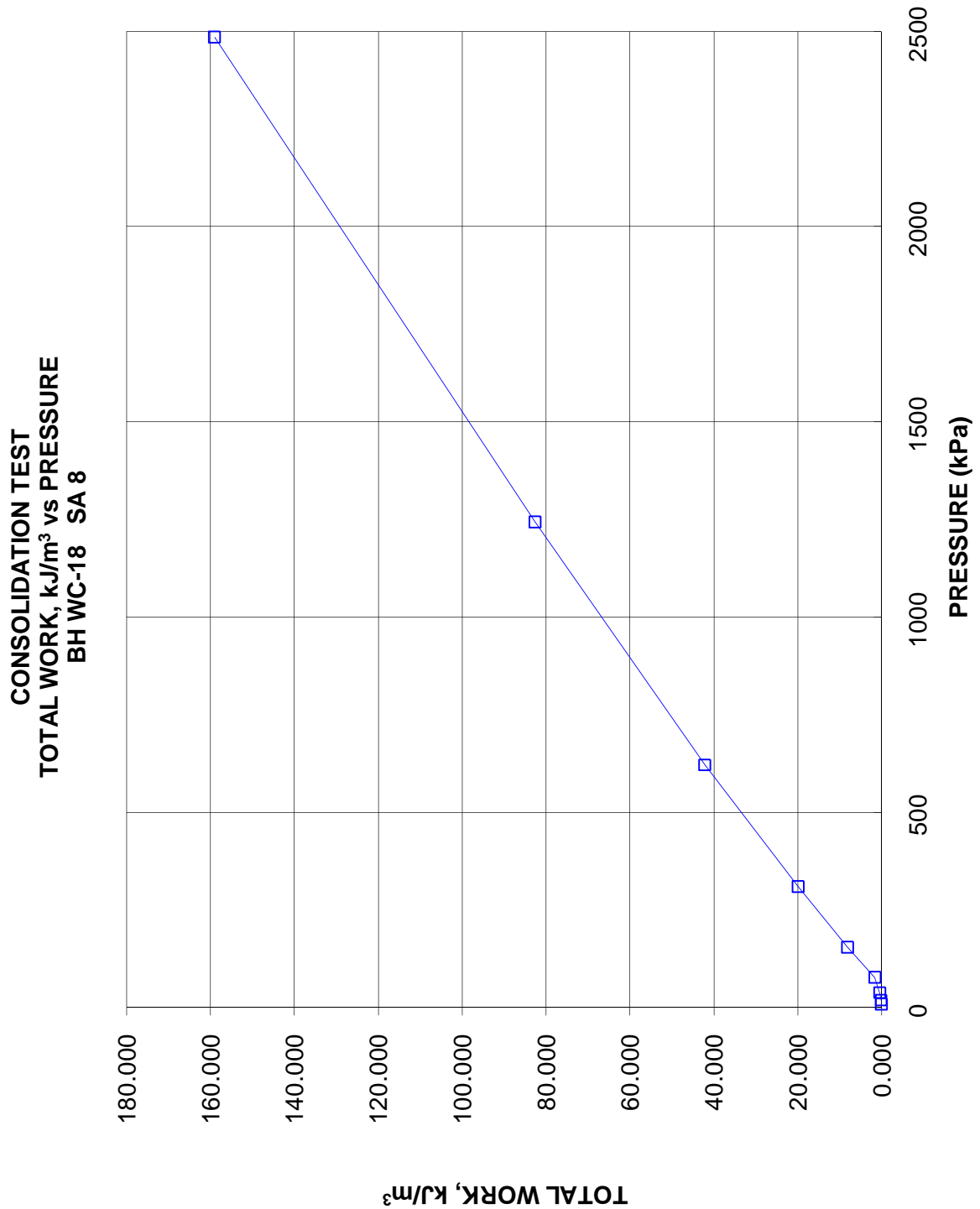
FIGURE B-9  
Page 3 of 4

CONSOLIDATION TEST  
VOID RATIO vs PRESSURE  
BH WC-18 SA 8



**CONSOLIDATION TEST  
TOTAL WORK VS. PRESSURE**

**FIGURE B-9**  
Page 4 of 4

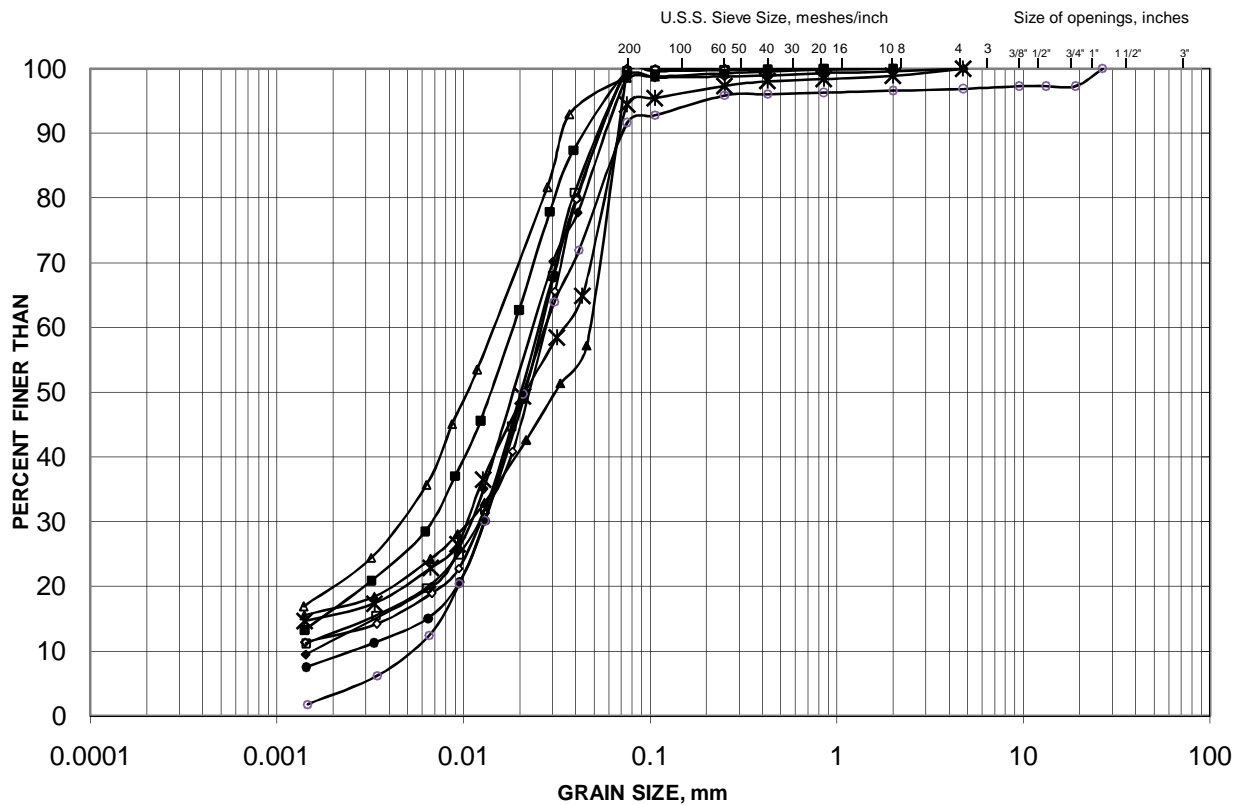











# GRAIN SIZE DISTRIBUTION

Silt

FIGURE

B-10



SILT AND CLAY SIZES				FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED				SAND SIZE			GRAVEL SIZE		
LENGEND	SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)					
		WC-4	8b	301.2					
		WC-6	10	298.3					
		WC-7	7	298.1					
		WC-8	9	298.3					
		WC-8	12	293.8					
		WC-9	10	296.7					
		WC-11b	4	299.4					
		WC-15	7	306.1					
		WC-19	10	299.8					

Project Number: 07-1191-0008

Checked By: SEMC

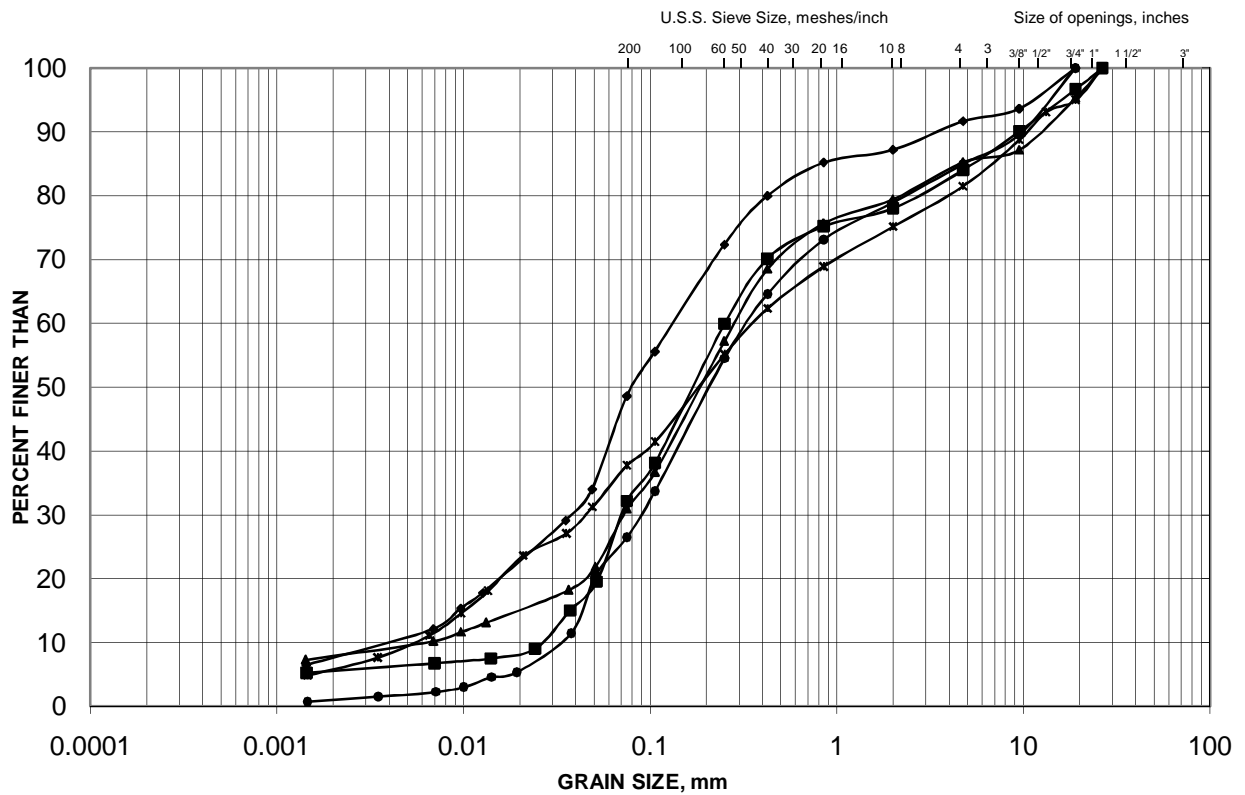
**Golder Associates**

Date: September 2009

# GRAIN SIZE DISTRIBUTION

## Silt and Sand to Silty Sand

**FIGURE**  
**B-11a**



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—●—	WC-1	8	306.5
—●—	WC-2	3	306.7
—■—	WC-2	5	305.1
—▲—	WC-3	8	303.0
—*—	WC-14	10	302.8

Project Number: 07-1191-0008

Checked By: SEMC

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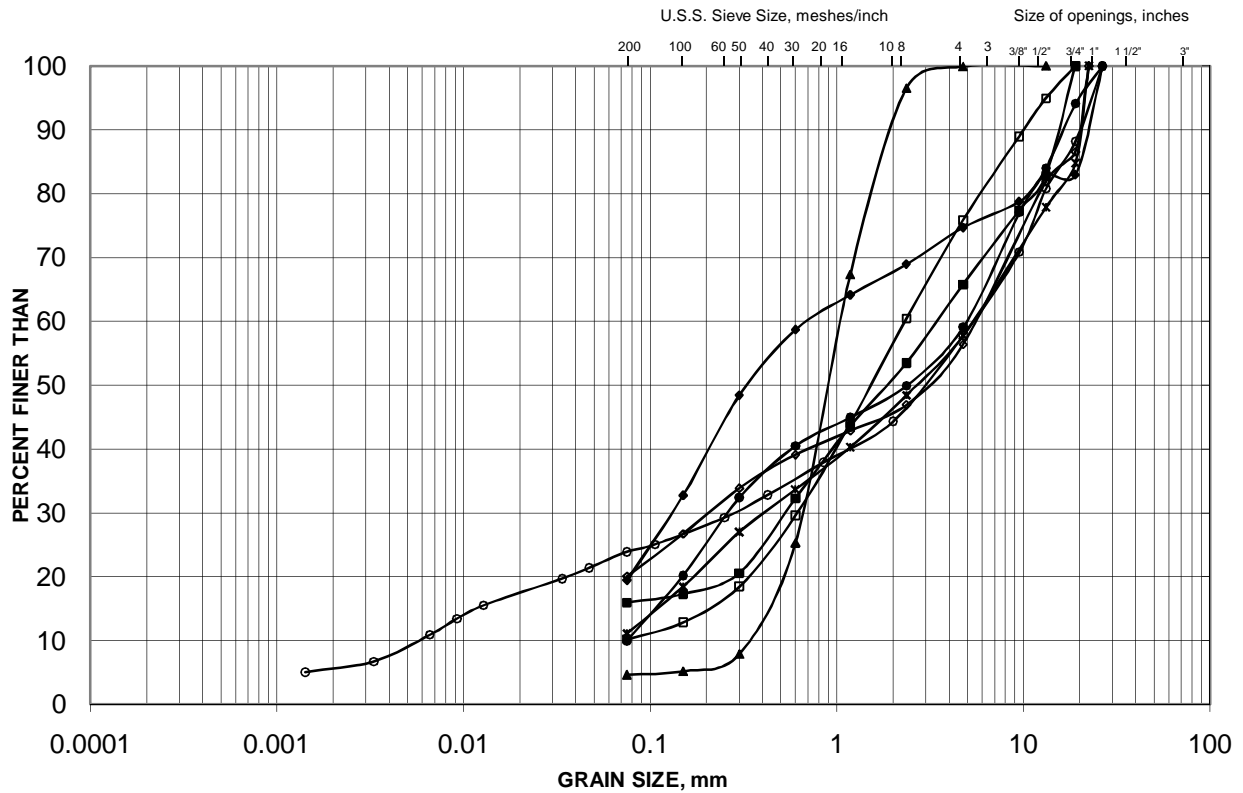
Date: September 2009











# GRAIN SIZE DISTRIBUTION

## Sand, Gravelly Sand, Sand and Gravel

**FIGURE**  
**B-11b**



SILT AND CLAY SIZES				FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE	
FINE GRAINED				SAND SIZE			GRAVEL SIZE			
LEGEND	SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)						
		WC-1	4	309.5						
		WC-3	6	305.1						
		WC-6	11b	296.5						
		WC-9	12	293.9						
		WC-13	11	299.3						
		WC-16	8b	303.0						
		WC-17	11	299.8						
		WC-18	12	296.9						

Project Number: 07-1191-0008

Checked By: SEMC

**Golder Associates**

Date: September 2009



# APPENDIX C

## NON-STANDARD SPECIAL PROVISIONS OPERATIONAL CONSTRAINTS AND CONTRACT ADMINISTRATION ASSIGNMENT

**Special Provision**

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**SCOPE**

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

**SUBMISSION AND DESIGN REQUIREMENTS**

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

**MATERIAL**

**Corrugated Steel Pipe**

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

**Special Provision**

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CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

**Sand Fill**

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

**Table 1 – Sand Fill Gradation Requirements**

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 µm	#30	80% to 100%
425 µm	#40	40% to 80%
250 µm	#60	5% to 25%
150 µm	#100	0% to 6%

**CONSTRUCTION**

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Install piles by driving to bedrock.
4. Place loose sand into 600 diameter CSP.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

## **CSP FOR INTEGRAL ABUTMENTS – Item No.**

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### **Special Provision**

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The CSP at each pile shall be constructed to the following tolerances:

<b><u>Criteria</u></b>	<b><u>Tolerance</u></b>
<b>Maximum deviation of CSP from pile centroid</b>	<b>+/- 50 mm</b>
<b>Maximum deviation of any point on the top perimeter of the CSP from the specified elevation</b>	<b>+/- 10 mm</b>

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

### **BASIS OF PAYMENT**

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

## **OPERATIONAL CONSTRAINT – Peat Sub-excavation and Staged Construction**

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### **Special Provision**

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#### **Peat Sub-excavation and Backfilling**

This operational constraint outlines the procedure(s) to be used for sub-excavation of peat (muskeg) and backfilling of the excavation for construction of the embankment. The work shall be sequenced to ensure that the flow of traffic is maintained through this area. In no way will this sequencing or methodology relieve the Contractor of his responsibility to ensure safety for the existing embankment.

The excavation work shall be carried out in areas or strips with limited width to provide stability of the existing highway embankment and to protect the existing bridge. The peat removal and backfilling shall be carried out as follows:

- The limits and elevation of the peat removal is shown elsewhere on the Contract Drawings;
- The excavation and backfilling shall be carried out in accordance with geometry as per OPSD 203.020 and in accordance with OPSS 209;
- The excavation shall be backfilled with Granular B Type II or rock fill as specified elsewhere on the Contract Drawings;
- Work shall be carried out starting from the furthest point away from the abutments towards the abutments; and
- The excavator shall be maintained a distance of 3 m back from the leading edge of the excavation for stability during the portion of the excavation below the water level. The Contractor shall be responsible for maintaining stability of the excavation at all times.

#### **Staged Embankment Construction and Preloading**

##### **General**

Baseline readings for the Settlement Pins (Ss) shall be conducted on the fourth, fifth and sixth days following notification of completion of instrument installation and receipt of the required installation information. If the monitoring on these days shows that the baseline has been established (i.e. consistent readings reflecting initial conditions obtained over the three (3) day period), embankment construction may continue on the seventh day. If the baseline is not established within this three (3) day period, additional daily readings shall be completed until three (3) consecutive consistent readings have been obtained prior to commencement of embankment construction.

For the Vibrating Wire Piezometers (VWPs), embankment construction above Elevation 310.5 m within 50 m of the VWP location shall not continue sooner than seven (7) working days following completion of installation of each VWP instrument, including notification and submission of required information to the Contract Administrator. Embankment construction shall not continue before establishment of baseline readings. Baseline readings for VWPs shall be established by others based on the readings taken on the fourth, fifth and sixth days following installation.

For the Settlement Plates (SPs), embankment construction above Elevation 310.5 m shall not continue sooner than three (3) working days following completion of installation of each SP. The baseline will be

established by others based on the readings taken on the first, second and third days after completion of instrument installation.

The Contractor shall confirm that the elevation of the top of each stage of filling is within 100 mm of the planned elevation. Elevations shall be provided to the Contract Administrator within seven days of placement of each stage of filling. The Contractor shall keep records of the filling operations including the thickness of each lift of fill and provide these records to the Contract Administrator within seven days of reaching the top of each stage of filling.

Embankment construction may proceed in the following manner:

- Install monitoring instrumentation on existing embankment (i.e. Settlement Pins (Ss)) and survey instruments to establish a baseline as detailed elsewhere in the contract documents.
- Remove the peat and backfill using Granular B Type II or Rock Fill as indicated on the Contract Drawings to Elevation 310.5 m.
- Install remaining monitoring instrumentation (Vibrating Wire Piezometers and Settlement Plates) and establish baseline as noted above.
- Construct embankment to Elevation 312.0 m from STA 15+350 to STA 15+468 and to Elevation 312.3 m from STA 15+523 to STA 15+725 using Granular B Type II or Rock Fill as indicated on the Contract Drawings.
- Allow for a “wait period” of 6 months – no other construction activities may take place during this period. The Contractor shall not proceed with additional filling or other construction activities during the six-month period or until approval has been given by the Contract Administrator.
- For the embankments between STA 15+350 and STA 15+468 and STA 15+523 to STA 15+725, construct embankment to final grade including abutments, backfill and EPS fill as described elsewhere in the Contract Documents. Allow for a second “wait period” of 5 months as indicated elsewhere in the contract documents. Other construction activities may take place during this wait period, however, the Contractor shall not proceed with paving or installation of guide rail until approval has been given by the Contract Administrator.

## **SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT - Item No. 68**

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### **Special Provision**

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#### **1.0 SCOPE**

This Special Provision contains the requirements for the supply and installation of Settlement Plates (SP), Settlement Pins/Stakes (S) and Vibrating Wire Piezometers (VWP) to monitor settlements and porewater pressures in the foundation soils during construction of the Highway 11 White Clay River bridge approach embankments between:

STA 15+350 and STA 15+468  
STA 15+523 and STA 15+725

The purpose of the Settlement Plates is to monitor settlements of the embankment relative to the installation at Elevation 310.5 m. The settlement readings shall help to establish the timing for construction of the abutments and piers, construction of the embankments and final paving. Settlement is measured by survey of the top of the rod attached to the plate with reference to stable, non-settling Benchmarks.

The purpose of the Settlement Pins/Stakes is to directly monitor settlement of the existing highway embankment. Settlement is measured by survey of the top of the pin/stake with reference to stable, non-settling benchmarks.

The purpose of the Vibrating Wire Piezometers is to monitor piezometric head at depth within the foundation soil. The piezometer readings shall help to establish the timing for construction of the abutments and piers, construction of the embankments and final paving.

The rate of fill placement for construction of the embankments and the timing for construction of the bridge abutments and piers and final paving shall be controlled by the instrumentation readings.

#### **2.0 REFERENCES**

This Special Provision refers to the following standards, specifications or publications:

##### **Ontario Provincial Standard Specifications, Construction**

OPSS 905                      Steel Reinforcement for Concrete

##### **Ontario Provincial Standards Specifications, Material**

OPSS1010                      Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1350              Concrete – Materials and Production

OPSS 1205                      Clay Seal

OPSS 1301                      Cementing Materials



**Ontario Water Resources Act RRO 1990:**

Regulation 903                      Wells

**3.0                                      DEFINITIONS**

**Geotechnical Engineering Consultant** means a consultant with MTO classification of “Geotechnical (Structures and Embankments) - High Complexity”, to undertake the supply and installation of geotechnical instruments.

**Benchmark** means a non-yielding, deep-seated survey reference point.

**Monitoring Program** means the monitoring readings conducted by others as part of the Contract Administration Assignment.

**Settlement Pin/Stake** means a bolt or stake embedded in a concrete plug for the purposes of settlement monitoring.

**Settlement Plate** means a plate installed at the defined level with a series of rods attached to a plate for the purposes of settlement monitoring.

**Vibrating Wire Piezometer** means a sensor attached to a cable installed in a borehole for the purposes of measuring pore pressure response.

**Equal** shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

**4.0                                      DESIGN AND SUBMISSION REQUIREMENTS**

**4.01                                      Design Requirements**

**4.01.01                                      Underground Utilities**

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractor’s work shall be repaired by the Contractor, at no cost to the Contract Administrator.

**4.01.02                                      Boreholes**

The Contractor shall document subsurface conditions at the locations of instruments and prepare Record of Borehole sheets (borehole logs).

**4.01.03                                      Survey Benchmarks**

The Contractor shall provide non-yielding temporary benchmarks relative to the existing Benchmark on site (as shown on the Contract Drawings as GBM #934781) as necessary such that direct sighting is possible from all Settlement Pins (S) and Settlement Plates (SP) to at least one temporary benchmark or the Benchmark.

The locations of the temporary benchmarks are to be approved by the Contract Administrator prior to installation of the monitoring instruments.

#### **4.01.04                      Marking and Labelling**

The location of any above-ground monitoring fixture shall be made clearly visible to nearby traffic before, during and after embankment construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

Instruments or their data cables shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible for at least 2 years.

#### **4.01.05                      Protection of Instruments**

All instruments shall be adequately protected by the Contractor such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced at no cost to the Contract Administrator.

#### **4.02                              Submission Requirements**

##### **4.02.01                      Notification**

The Contract Administrator shall be notified a minimum of 15 working days in advance of commencing the installation of instruments.

##### **4.02.02                      Installation Methods**

The Contractor shall submit details of proposed installation methods, including location and types of data-acquisition system, monitoring enclosure, temporary benchmarks and installation schedule. to the Contract Administrator a minimum of 15 days before the start of instrument installation.

#### **5.0                              MATERIALS**

##### **5.01                              General**

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless noted otherwise.

##### **5.02                              Settlement Plates (SP)**

###### **5.02.01                      Plate**

The Contractor shall supply a steel plate with thickness of at least 6.35 mm. The plate shall be at least 0.5 m wide by 0.5 m long.

###### **5.02.02                      Rod**

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4mm (1"), supplied in lengths as required to complete the installation.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and repeated.

#### **5.02.03 Friction Reducing Sleeve**

The friction reducing sleeve shall consist of Schedule 40 - 50.8mm (2") O.D. PVC pipe cut perpendicular to the axis of the pipe.

#### **5.02.04 Protective Surround**

The protective surround for the portion of the rod within the embankment shall consist of 300 mm diameter corrugated steel pipe (CSP - OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the Friction Reducing Sleeve (PVC pipe) shall be filled with medium to coarse sand.

#### **5.03 Settlement Pins (S)**

##### **5.03.01 Concrete**

Concrete (OPSS.PROV 1350) for anchoring the Pin shall be minimum 25 MPa compressive strength and set time sufficient to secure the settlement pin within two days of pouring.

##### **5.03.02 Pin**

The pin shall be a 25.4 mm minimum diameter reinforcing steel bar (OPSS 905) cut 0.4m long.

The top of the reinforcing steel bar shall be angled or rounded in such a way that a single survey point can be clearly identified and repeated.

#### **5.04 Vibrating Wire Piezometers (VWP)**

##### **5.04.01 Vibrating Wire Piezometer Sensors**

The vibrating wire piezometer sensors shall be:

- Slope Indicator model 52611020 (-5 to 50 psi); or
- RST model VW2100-0.35; or
- Equal.

The VWPs shall be compatible with the Slope Indicator VW Minilogger, model 52613310, or equal. All VWPs shall be of the same make/supplier.

##### **5.04.02 Signal Cable**

The signal cable shall be:

- Slope Indicator model 50613524 cable; or

- RST model EL380004 cable; or
- Equal.

The length of cable for each piezometer shall be carefully estimated from the construction drawings to ensure that there is sufficient length of signal cable for each piezometer to provide enough slack in the borehole and along the trenches until each cable is out of the embankment footprint area where they shall be protected from earthmoving equipment and extended to the monitoring station.

#### **5.04.03 Bentonite**

Bentonite to form borehole plugs as required shall be in accordance with OPSS 1205 in pellet form in sufficient quantity.

#### **5.04.04 Filter Sand**

Sand for filters around VWP sensors shall be clean washed sand, such as “Sakcrete” washed general-purpose sand; or similar.

#### **5.04.05 Grout**

Grout shall be cement-bentonite mix consisting of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU - OPSS 1301).

#### **5.04.06 Trench Burial and Conduit**

The signal cable for each piezometer shall be buried in a shallow trench as shown in the Contract Drawings, and taken out of the embankment footprint area if possible and/or to an area that will not be impacted by construction operations. Conduits to protect the signal cables in the trenches and above ground surface shall consist of Schedule 40 – 75 mm - 3" - steel pipe or Schedule 80 – 75 mm - 3" - rigid PVC pipe. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench.

#### **5.04.07 Data Acquisition System (VW Miniloggers)**

One VW Minilogger (data acquisition system) shall be supplied for each of the VWPs beneath the embankment for this project (4 VW Miniloggers total). The data acquisition systems shall be from the same supplier as the VWPs and shall consists of:

- Slope Indicator Model 52613310; or
- Equal.

The VW Minilogger shall be programmed according to the following:

- Recording Software: VWP data shall be recorded a minimum of four times per day (one reading every 6 hours);
- Test Software: once this program is transferred to the VW Minilogger, the system shall be tested

and data manually recorded on site.

The real-time data shall be retrieved on site by direct wire (i.e. RS232 Cable) with a portable laptop computer.

#### **5.04.08                      Portable Laptop Computer**

The Portable Laptop Computer (with a three year warranty) shall be:

- Intel Pentium M or IV (1.6 GHz or higher) with Windows XP Professional Operating System, 1GB memory, Network Card: 10/100 Integrated Ethernet LAN, a minimum of 80GB hard drive storage, a DVD/CD-RW Rom and Microsoft Office Standard 2003, to retrieve, read and store the VWP readings, supplied with an extra battery pack and vehicle charger, or equal.

#### **5.04.09                      Wooden Posts**

Wooden posts for the support of the data acquisition system enclosures shall be:

- 100 mm x 100 mm (4"x4"), minimum 3 m (10') long pressured treated lumber.

### **6.0                              EQUIPMENT**

#### **6.01                              Equipment Operation and Weather Conditions**

All installation and monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The instruments shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. Monitoring of instruments shall be feasible all year round.

#### **6.02                              Laptop Computer**

The portable laptop computer will become property of the MTO and shall be handed to the Contract Administrator after the installation of the instruments for the Monitoring Program.

The calibration factors for the VWPs shall be entered into the portable laptop computer by the Contractor for initialization of the instruments.

### **7.0                              CONSTRUCTION**

#### **7.01                              Subsurface Conditions**

The subsurface conditions at the site are described in:

- Foundation Investigation Report – White Clay River Bridge Replacement, Highway 11, Site No. 47-005, Township of Maisonville, GWP 5239-06-00, Geocres No. 42A-77, dated September 11, 2009, by Golder Associates Ltd.

#### **7.02                              Drawings**

Reference shall be made to the following drawings:

- Monitoring Plan and Typical Section – Sheet 46; and
- Monitoring Instrument Details – Sheet 47.

### 7.03 Instrumentation Installation

#### 7.03.01 Instrument Locations

The quantity and location of instruments are as shown in the Contract Documents and in Table 1 below.

**Table 1 – Instrument Quantities and Locations**

Location		Number of Instruments		
Station	Approach	SP	S	VWP
15+388	South Approach	0	1	0
15+413		2	1	1
15+438		2	1	0
15+463		2	1	1
15+528	North Approach	2	1	1
15+553		2	1	0
15+578		0	1	0
15+603		2	1	1
15+628		0	1	0
15+653		0	1	0
15+678		0	1	0
Total		12	11	4

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground surface elevation at each instrument location.

The locations of the monitoring instruments should be adjusted in the field such that they will not be damaged by the sub-excavation procedures for the new embankment, by highway maintenance equipment on the existing highway, or by earth moving equipment.

#### 7.03.02 Installation Program

Instrument installation shall commence after the sub-excavation and backfilling process reaches Elevation 310.5 m. Embankment construction above this level may not proceed any closer than 50 m to an individual instrument until the instrument is fully installed and baselined. Table 2 provides a summary of the installation schedule requirements.

**Table 2 - Installation Program**

Type	Start Installation	Finish Installation
SP	after embankment construction reaches Elevation 310.5 m	at completion of embankment construction
S	before any embankment	before any embankment construction

	construction	
VWP	after embankment construction reaches Elevation 310.5 m	before proceeding with any embankment construction above Elevation 310.5 m

### 7.03.03 Settlement Plates

#### 7.03.03.01 General

The Settlement Plates shall be installed at Elevation 310.5 m. As embankment construction proceeds the rods shall be extended above the new top of embankment.

The locations of the Settlement Plates are as shown in the contract documents and in Table 3 below.

**Table 3 - Approximate Settlement Plate Locations**

Location	Instrument Number	Station	Offset from New Centreline	Estimated Final Thickness of Embankment (m)*
South Approach	SP#1	15+413	6 m LT	2.7
	SP#2	15+413	6 m RT	2.7
	SP#3	15+438	6 m LT	2.6
	SP#4	15+438	6 m RT	2.6
	SP#5	15+463	6 m LT	2.5
	SP#6	15+463	6 m RT	2.5
North Approach	SP#7	15+528	6 m LT	2.2
	SP#8	15+528	6 m RT	2.2
	SP#9	15+553	6 m LT	2.0
	SP#10	15+553	6 m RT	2.0
	SP#11	15+603	6 m LT	1.8
	SP#12	15+603	6 m RT	1.8

\* Above Elevation 310.5 m.

The elevation, easting and northing of the centre of the base of the plate and top of the rod shall be surveyed after installation.

The total distance from the base of the plate to the top of the rod shall be measured to an accuracy of  $\pm 2$  mm or better.

#### 7.03.03.02 Plate

The plate shall be installed horizontally on a minimum 100 mm thick pad of Granular B Type II (OPSS 1010) placed over the chinked rock fill surface at Elevation 310.5 m.

#### 7.03.03.03 Rod

The rod shall be fixed to the centre of the plate and perpendicular to the plate.

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

#### **7.03.03.04 Friction Reducing Sleeve**

The friction reducing sleeve shall be extended over the entire length of the rod that is below ground and within the embankment fill except that the cap on top of the rod shall extend 25 mm above the top of the friction sleeve at all times.

#### **7.03.03.05 Extension of Rod**

The Settlement Plate rods shall be extended upwards as the embankment is constructed so that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill.

#### **7.03.03.06 Protective Surround**

The CSP, Friction Reducing Sleeve and sand protective surround shall be extended with the rods. The rod shall be in the centre of the CSP and friction reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

#### **7.03.04 Settlement Pins**

##### **7.03.04.01 General**

The elevation, easting and northing of the top of the pins shall be surveyed after installation.

The locations of the Settlements Pins are shown in the Contract Documents and in Table 4 below.

**Table 4 - Approximate Settlement Pin Locations**

<b>Location</b>	<b>Instrument Number</b>	<b>Station</b>	<b>Offset from New Centreline*</b>
South Approach	S#1	15+388	11.0
	S#2	15+413	11.0
	S#3	15+438	11.5
	S#4	15+463	12.3
North Approach	S#5	15+528	12.7
	S#6	15+553	11.8
	S#7	15+578	11.2
	S#8	15+603	11.0
	S#9	15+628	10.3
	S#10	15+653	9.5
	S#11	15+678	8.4

\* Location to be field adjusted as described in Section 7.03.01.

##### **7.03.04.02 Settlement Pins**



The Settlement Pins shall be cast into a concrete plug placed in a hole at the top of the existing embankment.

### **7.03.05 Vibrating Wire Piezometers**

#### **7.03.05.01 General**

Installation of the Vibrating Wire Piezometers shall be as per the manufacturer's recommendations in addition to what is stated or emphasised below.

The locations of the Vibrating Wire Piezometers are as shown in the Contract Documents and in Table 5 below.

**Table 5 – Approximate Vibrating Wire Piezometer Locations**

<b>Location</b>	<b>Instrument Number</b>	<b>Station</b>	<b>Offset from New Centreline</b>	<b>Tip Elevation (m)</b>
South Approach	VWP#1	15+413	0	304.6
	VWP#2	15+463	0	302.7
North Approach	VWP#3	15+528	0	303.1
	VWP#4	15+603	0	305.1

The piezometers shall be installed in boreholes after embankment construction has reached Elevation 310.5 m.

The VWP signal cables shall be extended to the VW Minilogger enclosure areas through a metal or plastic conduit buried in trenches, as shown in the Contract Drawings.

#### **7.03.05.02 Borehole Installation**

The borehole shall be advanced to 300 mm below the tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

#### **7.03.05.03 VW Miniloggers**

The signal cables from the VWPs shall be connected to the VW Minilogger via the burial trench which shall be situated adjacent to the nearest settlement plate right-of-centreline.

The VW Miniloggers shall be securely attached inside two (2) enclosures secured to wooden posts, one for the instruments under the embankment on either side of the river. The posts shall be installed with a minimum 1.2 m embedment. A single post can be used to secure the enclosure for the VW Miniloggers and signal cables on either side of the river.

The VW Miniloggers shall be fastened to the inside of an insulated wooden "box" enclosure attached to the posts. The box enclosure should be large enough to contain the two VW Miniloggers required for each enclosure. The access to the box enclosure should be lockable.

The Contractor shall ensure access to the box enclosures at all times, including but not limited to snow clearing in the winter.

#### **7.03.05.04 Completion of Installation**

It is known that the process of installing VWPs can temporarily alter the pore water pressure acting on the piezometer tip. Normally, the installation of a VWP shall not be considered to be complete until the pore pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. Since the piezometers at this site will be installed after loading on the subsoils is in progress, the stabilization of readings may not be possible. Initial readings shall be taken by the contractor within the first three (3) days following installation to confirm the instrument is functioning properly. The baseline reading will be taken by others starting no sooner than the fourth day following construction, for three consecutive days. The Contractor shall not continue with embankment monitoring above Elevation 310.5 m before the seventh day following installation, dependant on approval from the Contract Administrator.

## **7.04                      Coordination with Monitoring Program**

### **7.04.01                      Notification**

The Contractor shall notify the Contract Administrator no later than 3 days the completion of installation of Settlement Plates, Settlement Pins and Vibrating Wire Piezometers

### **7.04.02                      Reporting**

The Contractor shall supply the information outlined in the following sections to the Contract Administrator within 3 days of completion of installation of each instrument.

#### **7.04.02.01                      Settlement Plates (SP)**

- SP location, northing and easting (NAD83 MTM coordinates);
- Geodetic elevation of plate and top of rod;
- Distance between base of plate and top of rod;
- Dates of installation;
- Installation notes / sketches; and
- Details of Settlement Plate, sleeve and plate.

Adjustments in the length of any rod shall be coordinated with the Contract Administrator to allow surveying by others of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

#### **7.04.02.02                      Settlement Pins (S)**

- S location, northing and easting (NAD83 MTM coordinates);
- Geodetic elevation of top of pin;
- Dates of installation; and

- Installation notes / sketches.

#### **7.04.02.03                      Vibrating Wire Piezometers**

- VWP location, northing and easting (NAD83 MTM coordinates);
- Geodetic elevations of VW sensors;
- Stratigraphic log of subsurface conditions, including drilling method notes;
- Dates of installation;
- Installation notes / sketches; and
- Model, make and serial numbers of VW sensors, readout unit and signal cable.

All piezometers shall be calibrated prior to installation and the calibration data for each piezometer shall be provided to the Contract Administrator.

#### **7.04.03                      Monitoring**

##### **7.04.03.01                      Settlement Plates**

Monitoring of the Settlement Plates shall be done by others. Monitoring shall be conducted during and after the embankment construction. The Contractor shall provide installation information as specified above and provide access to the Settlement Plates for monitoring, including, but not limited to a scaffolding platform and ladder if required and snow clearing in the winter. The contractor shall provide electric power and general area lighting as needed for reading the instruments.

##### **7.04.03.02                      Settlement Pins**

Monitoring of the Settlement Pins shall be done by others. Monitoring shall be conducted during and after sub-excavation and backfilling and embankment construction. The Contractor shall provide installation information as specified above and provide access to the Settlement Pins for monitoring.

##### **7.04.03.03                      Vibrating Wire Piezometers**

Monitoring of the Vibrating Wire Piezometers shall be done by others. Monitoring shall be conducted during and after the embankment construction. The Contractor shall provide installation information as specified above and provide access to the VW Miniloggers for monitoring.

The Contractor shall transfer the Portable Laptop Computer to the Contract Administrator, including all the data-logging software and hardware, operation instructions and calibration constants. The contractor shall also transfer the keys for the locks of the VWP monitoring enclosure boxes. The Contractor shall be available for one site meeting with the Contract Administrator to transfer and explain about any questions from the Contract Administrator regarding the data-logging system.

#### **7.05                      Decommissioning of Instruments**

**7.05.01****General**

The Contractor shall decommission all the Settlement Plates (SP), Settlement Pins (S), and Vibrating Wire Piezometers (VWP) at the end of the Monitoring Program following construction, unless advised otherwise by the Contract Administrator. Decommissioning of instrumentation shall be carried out according to the Ontario Water Resources Act, R.R.O. 1990, Regulation 903.

The Settlement Plates in Table 6 shall be decommissioned after completion of the first wait period and just prior to installation of the EPS fill.

**Table 6 - Approximate Settlement Plate Locations**

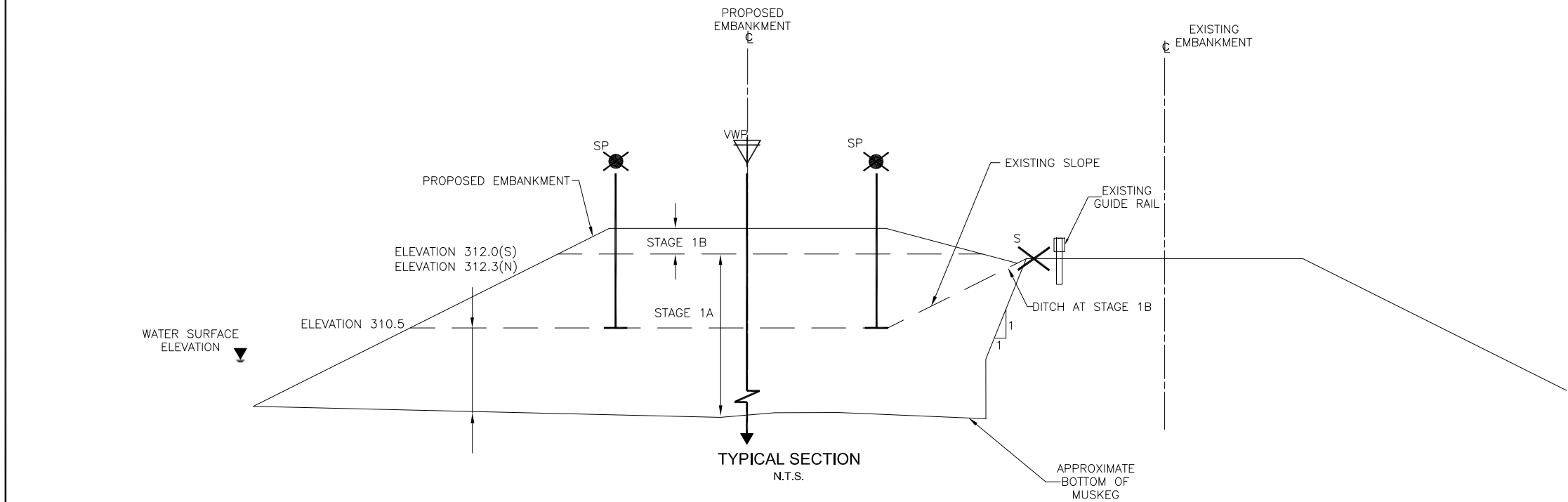
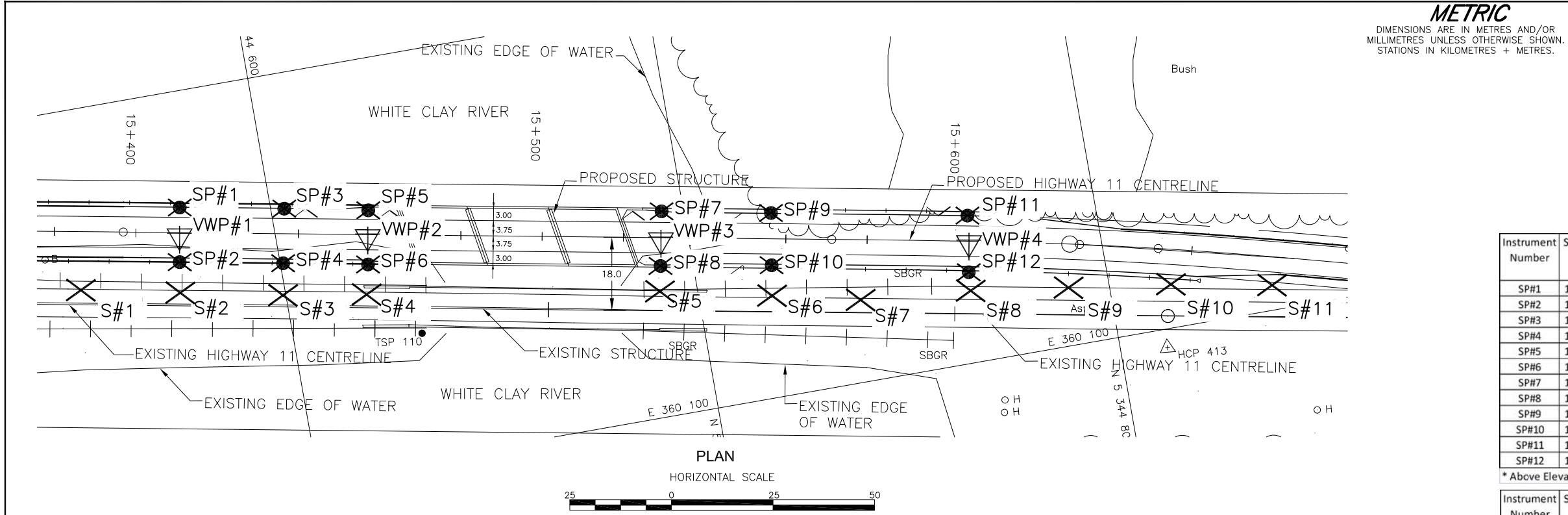
<b>Location</b>	<b>Instrument Number</b>	<b>Station</b>	<b>Offset from New Centreline</b>
<b>South Abutment</b>	SP#5	15+463	6 m LT
	SP#6	15+463	6 m RT
<b>North Abutment</b>	SP#7	15+528	6 m LT
	SP#8	15+528	6 m RT

**8.0****MEASUREMENT FOR PAYMENT**

Measurement will be made of the number of units of Settlement Plates, Settlement Pins and Vibrating Wire Piezometers installed.

**9.0****BASIS OF PAYMENT**

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, supply of monitoring instruments and equipment and material to complete the work.



CONT No. 2011-5110  
WP No. 5239-06-00

HIGHWAY 11 CROSSING  
WHITE CLAY RIVER  
MONITORING PLAN  
AND TYPICAL SECTION



SHEET  
46



**Golder Associates Ltd.**  
SUDBURY, ONTARIO, CANADA

Instrument Number	Station	Offset from New CL	Estimated Thickness of Embankment (m)*
SP#1	15+413	6m LT	2.7
SP#2	15+413	6m RT	2.7
SP#3	15+438	6m LT	2.6
SP#4	15+438	6m RT	2.6
SP#5	15+463	6m LT	2.5
SP#6	15+463	6m RT	2.5
SP#7	15+528	6m LT	2.2
SP#8	15+528	6m RT	2.2
SP#9	15+553	6m LT	2.0
SP#10	15+553	6m RT	2.0
SP#11	15+603	6m LT	1.8
SP#12	15+603	6m RT	1.8

\* Above Elevation 310.5 m

Instrument Number	Station	Offset from New CL*
S#1	15+388	11.0
S#2	15+413	11.0
S#3	14+438	11.5
S#4	15+463	12.3
S#5	15+528	12.7
S#6	15+553	11.8
S#7	15+578	11.2
S#8	15+603	11.0
S#9	15+628	10.3
S#10	15+653	9.5
S#11	15+678	8.4

\*Location to be field adjusted as described in the specifications.

Instrument Number	Station	Offset from New CL	Tip Elevation (m)
VWP#1	15+413	0	303.5
VWP#2	15+463	0	302.7
VWP#3	15+528	0	302.2
VWP#4	15+603	0	303.9

#### LEGEND

SP	SETTLEMENT PLATES
S	SETTLEMENT PINS
VWP	VIBRATING WIRE PIEZOMETERS

#### NOTES

- Refer to NSSP - "Supply and Installation of Embankment Monitoring Equipment" for instrument installation.
- This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.
- SP#5, SP#6, SP#7 and SP#8 to be decommissioned immediately prior to installation of EPS.
- Stage 1b applies to 25m south, of south abutment and 20m north, of north abutment only.

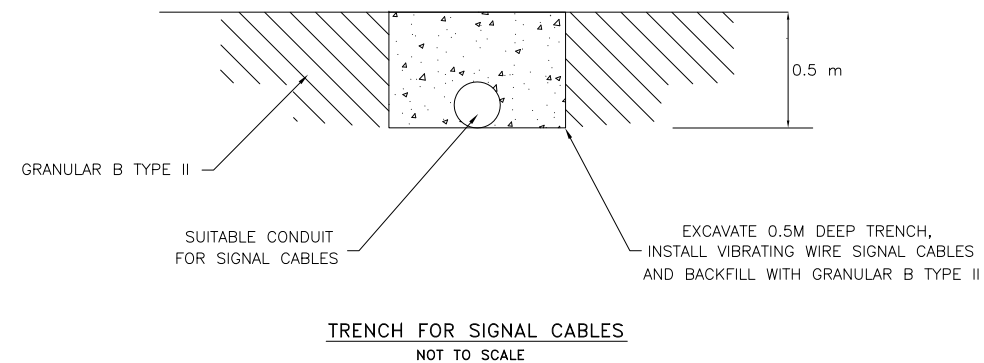
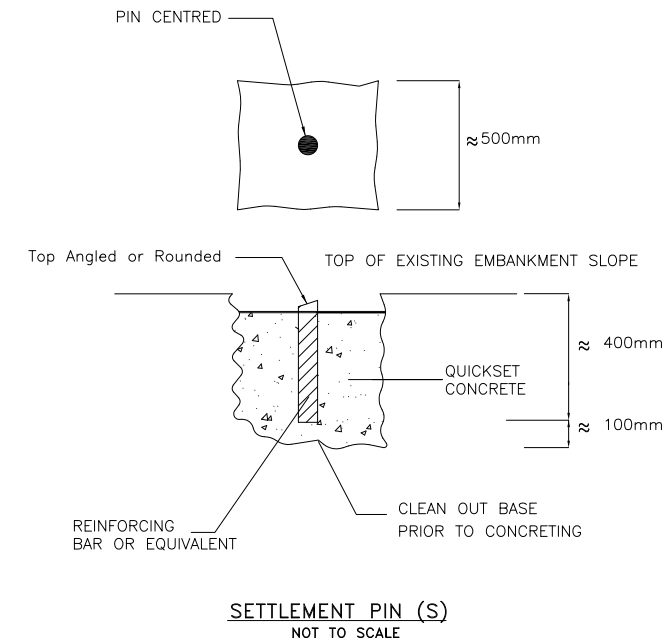
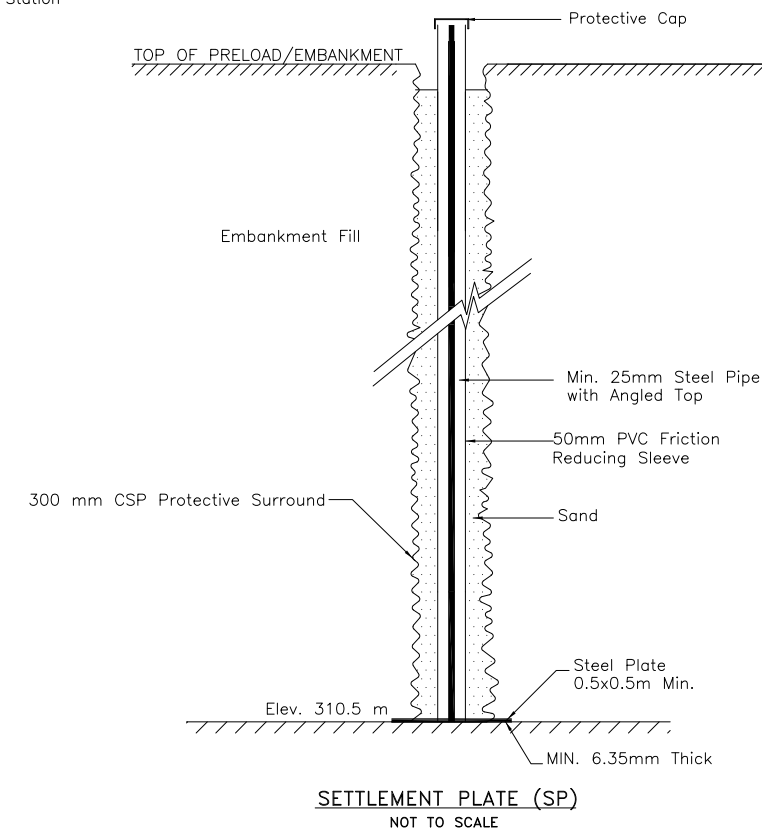
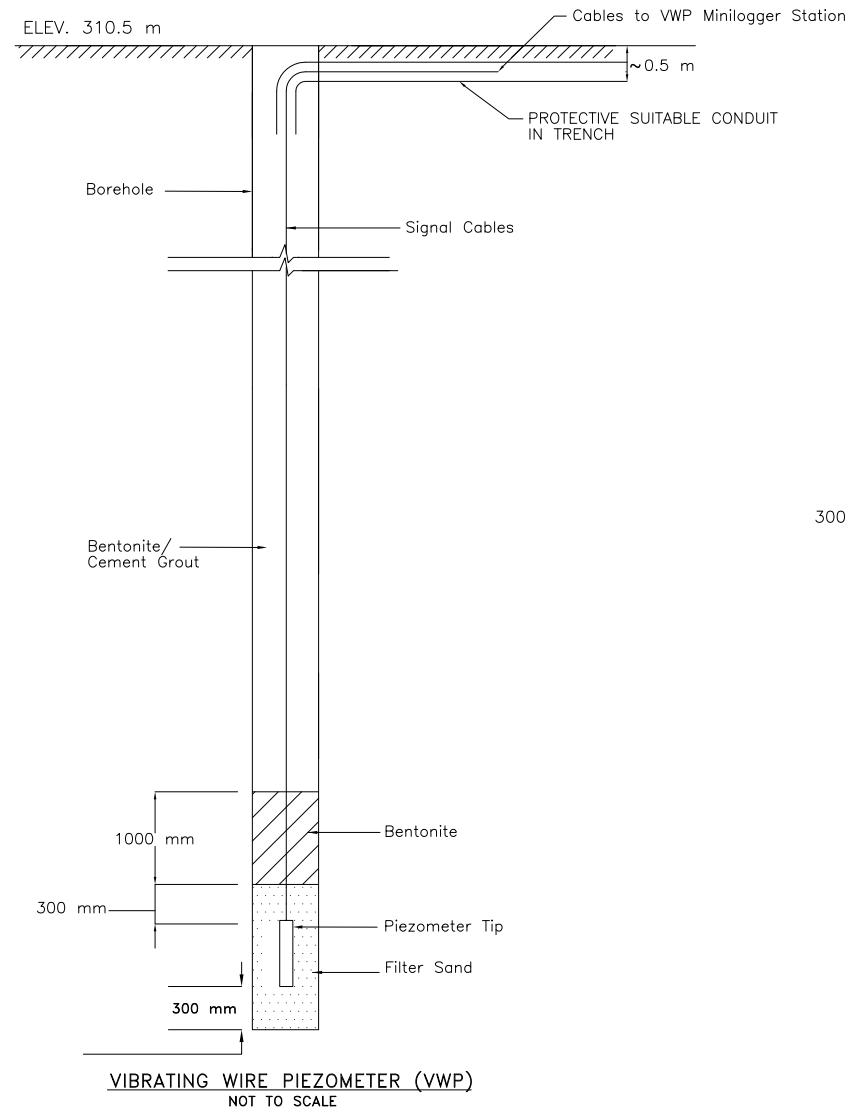
#### REFERENCE

Base plans provided in digital format by LEA, drawing file name 2007-032-White Clay-Recommended Plan.dwg dated May, 2008 and received September 9, 2008 and Key Plan.dwg received December, 2008.

**BENCH MARK**  
GBM # 93U781  
EL. 311.695  
TABLET IN TOP OF BOULDER  
STA. 15+322.599 RT. 38.841



NO.	DATE	BY	REVISION
Geocres No. 42A-77			
HWY. 11	PROJECT NO. 07-1191-0008	DIST.	
SUBM'D. SEMC	CHKD. SEMC	DATE: MAR 2011	SITE: 47-005
DRAWN: MM	CHKD.	APPD. JMCA	DWG. C-1



NO.	DATE	BY	REVISION
Geocres No. 42A-77			
HWY. 11	PROJECT NO. 07-1191-0008		DIST.
SUBM'D. SEMC	CHKD. SEMC	DATE: MAR 2011	SITE: 47-005
DRAWN: MM	CHKD.	APPD. JMAC	DWG. C-2

## **RIGID EXPANDED POLYSTYRENE EMBANKMENT FILL - Item No. 46**

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### Special Provision

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#### **1. SCOPE**

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene backfill and associated works as shown on the Contract Drawings.

As part of the work under this item, the Contractor shall supply and place 100 mm of mortar sand, polyethylene sheeting, and reinforced concrete top slab above the EPS at the abutments as shown on the Contract Drawings.

#### **2. REFERENCES**

This special provision refers to the following standards, specifications or publications.

##### **National Standards of Canada**

CAN/CGSB - 51.20 M87

##### **ASTM**

ASTM D1621 Test Method for Compressive Properties of Rigid Cellular Plastics

ASTM C203 Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation

ASTM C177 Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus

ASTM D2842 Test Method for Water Absorption by Rigid Cellular Plastics

ASTM D2863 Test Method for Measuring the Minimum Oxygen Content

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

##### **OPSS - Ontario Provincial Standard Specification**

OPSS 212 Borrow

OPSS 501 Compaction

OPSS 517 Dewatering

OPSS 904 Concrete in Approach Slabs

OPSS 1010 Aggregates – Granular A,B,M, and Selected Subgrade Material

OPSS 1605      Expanded Extruded Polystyrene Pavement Insulation

OPSS 1860      Geotextiles

### **3. SUBSURFACE CONDITIONS**

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

### **4. DEFINITIONS**

For the purpose of this special provision, the following definitions apply:

#### **Rigid Expanded Polystyrene**

Molded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

#### **Rigid Extruded Expanded Polystyrene**

Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot

The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

#### **Quality Verification Engineer**

An Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

### **5. QUALIFICATION**

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

### **6. SUBMISSION AND DESIGN REQUIREMENTS**

#### **6.1 Submission of Shop Drawings**

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality



Verification Engineer that provides full details of materials and construction procedure.

## **6.2 Delivery, Storage, Handling and Protection**

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturers' requirement.

## **6.3 Construction**

The contractor shall submit full details of the following:

- a. The method of foundation excavation and preparation.
- b. Construction of leveling/drainage layer (granular base and mortar sand) as shown on the Contract Drawings.
- c. The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer by layer basis.
- d. The method and limits of placement of polyethylene sheeting.
- e. The method of placement of the reinforced 30MPa concrete top slab.
- f. The method of placement of subbase material.
- g. The method of placement of side slope cover.

## **6.4 Quality Verification Engineer**

1. The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments. The Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
2. The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Backfill the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in

conformance with the installation procedures and specifications of the contract documents.

## **7.0 MATERIALS**

### **7.1 Granular Leveling Pad**

The leveling pad shall consist of mortar sand with gradation and physical requirements as specified in OPSS 1004.

### **7.2 Rigid Expanded Polystyrene**

#### **7.2.1 General**

##### **7.2.1.1 The Contractor shall submit:**

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
  - 1) Geometry
  - 2) Nominal Density
  - 3) Compressive Strength
  - 4) Flexural Strength
  - 5) Thermal Resistance
  - 6) Dimensional Stability
  - 7) Flammability
  - 8) Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

##### **7.2.1.2 Production Lots**

Each block of the same production lot shall be stamped with the same production code showing plant

identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

### 7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

**Table 1 – Material Properties**

Property	Unit	Requirements	Test Procedure
Geometry - Linear - Flatness - Squareness - Thickness	mm	1200 x 600 x 300 with tolerances $\pm 1\%$ 10 mm in 3 m $\pm$ 0.5% -3, +5	
Compressive Strength	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m <sup>2</sup> .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

#### 7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm .

The maximum deviation from the specified linear dimensions shall be  $\pm 1\%$ . The flatness of the block faces shall be within  $\pm 10$  mm of a line formed by a 3 m straight edge.

The maximum difference in corner to corner dimensions (squareness) shall be 0.5%. The thickness shall be within -3 to +5 mm.

#### 7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

#### 7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be

determined in accordance to ASTM C203, method 1, Procedure B.2.7.4.

#### **7.2.2.4 Dimensional Stability**

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

#### **7.2.2.5 Thermal Resistance**

The thermal resistance shall be  $0.7 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$  for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25\text{mm}} = \frac{R_{\text{measured}}}{\text{thickness (mm)}} \times 25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

#### **7.2.2.6 Flammability**

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

#### **7.2.2.7 Water Absorption**

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

#### **7.2.2.8 Chemical Resistance**

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. A table identifying the chemical resistance as either resistant, limited, or not resistant shall be submitted.

#### **7.2.2.9 Biological Resistance**

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

#### **7.2.2.10 Environmental**

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

### **7.3 Polyethylene Sheeting**

The plastic sheeting shall be 6 mil polyethylene sheeting or equivalent.

#### **7.4 Concrete Top Slab**

The concrete top slab shall consist of 30 MPa reinforced concrete as shown on the Contract Drawings.

### **8.0 DELIVERY, STORAGE AND HANDLING**

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

### **9.0 CONSTRUCTION**

#### **9.1 Foundation Excavation**

Foundation excavation shall be carried out to the design elevations shown on the drawings as earth excavation or excavation for structure. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' Type II material in accordance with OPSS 1010.

#### **9.2 Leveling Pad**

Place, level and compact a 100 mm thick layer of mortar sand material in accordance with OPSS 501 to within  $\pm 30$  mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground. The leveling pad must be placed in-the-dry.

#### **9.3 Installation of Blocks**

- (1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary. Contractor shall ensure all trimmed material is disposed of in accordance with all applicable regulations and that no trimmed debris enters the watercourse.

- (2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers except at the vertical construction joints.
- (3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- (4) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- (5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- (6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- (7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.
- (8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- (9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- (10) The top surface and side surfaces of the expanded polystyrene shall be covered with 6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

#### **9.4 Concrete Top Slab**

The concrete top slab shall be poured after the polyethylene sheeting is fixed in place. Place 125 mm thick layer of concrete in accordance with OPSS 904 to within  $\pm 30$  mm of the design elevation.

#### **9.5 Backfill**

Backfill over the top of the concrete slab and on the sides of the embankment shall be as shown on the Contract Drawings.

## **10. EQUIPMENT**

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

## **11. QUALITY ASSURANCE**

### **11.1 General**

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

### **11.2 Sampling Frequency**

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. At a minimum, three blocks shall be tested.

### **11.3 Acceptance/Rejection**

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

## **12. MEASUREMENT FOR PAYMENT**

### **12.1 Actual Measurement**

Measurement will be by volume in cubic metres of rigid expanded polystyrene backfill material measured in its original position based on theoretical dimensions.

## **13. PAYMENT**

### **13.1 Basis of Payment**

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above.



## **Foundation Monitoring Speciality Plan – White Clay River Bridge**

### **1.0 GENERAL**

Requirements specified for Specialist Qualifications; Services, Deliverables and Records; and the Foundation Monitoring Plan apply to all the Instrumentation Monitoring. Instrumentation monitoring is required for the following geotechnical instruments:

- Settlement Pins (Ss);
- Settlement Plates (SPs); and
- Vibrating Wire Piezometers (VWPs).

The instrumentation monitoring services include: data collection, data reduction and reporting; adherence to criteria used to assess the embankment performance based on the monitoring data collected from the instrumentation installed by others.

#### **1.0.1 Specialist Qualifications**

The Foundation Engineering Consultant services required for this assignment have been categorized as Geotechnical Specialty - High Complexity.

The Foundation Engineering Consultants that are registered in MTO's Consultant Registry Appraisals and Qualifications System (RAQS) at the complexity rating in the required specialty that meets the identified complexity requirement for this assignment are eligible to provide Foundation Engineering services for this project. The Foundation Consultant shall not be the same Instrument Installation Consultant retained by the Contractor for the supply and installation of embankment monitoring equipment.

The Foundation Engineer shall have a minimum of five (5) years experience in the monitoring of vibrating wire piezometers, settlement plates and pins and survey benchmarks, or alternatively demonstrate expertise through providing satisfactory services associated with the supply, installation and monitoring services for the instrumentation specified for a minimum of two (2) projects in which the work was of similar scope to that in the Contract.

#### **1.0.2 Services, Deliverables and Records**

The Foundation Engineering Consultant shall:

- Review the monitoring program and, if deemed necessary, submit in writing to the Contract Administrator recommendations for modifications to the Monitoring Program;
- Meet with the Contractor in order to receive the Portable Laptop Computer used for monitoring vibrating wire piezometers and to receive reports with details about installation of instruments installed by the Contractor and calibration certificates, as specified in the Special Provision titled, "Supply and Installation of Embankment Monitoring Equipment", included in the contract documents;
- The Foundation Engineering Consultant is required on site to establish the baseline readings. The Contract Administrator staff may take all other required

readings provided they are immediately forwarded to the Foundation Engineering Consultant.

- With the exception of the Portable Laptop Computer referred to above and all instruments installed by the Contractor, supply all materials and equipment that are required for the Monitoring Program;
- Calibrate and maintain monitoring equipment;
- Take instrument readings, reduce data, prepare reports;
- Provide transmittal of instrumentation readings and reports to the Contract Administrator;
- Interpret instrumentation readings as needed for the purpose of ongoing construction;
- Notify the Contract Administrator of required modifications to the construction procedures accordingly, if necessary. Interpretation shall include making correlations between instrumentation data and specific construction activities; and
- Notify the Contract Administrator if critical instrument readings, as specified herein, for any instrumentation are reached. Discuss as soon as possible (within 48 hours) with the Contract Administrator response action(s), and submit a plan of actions, to prevent the critical instrument readings (i.e. Review/Alert levels) from being exceeded.

Progress Reports shall be submitted to the Contract Administrator, the Ministry's Contract Control Officer and Foundations Engineer. Weekly reports shall be issued from the beginning of construction monitoring to the end of the one month period immediately after the top of preload is reached. Thereafter, one report shall be submitted after each set of readings is taken. The Progress Reports shall discuss the Contractor's operations with respect to the installation of instrumentation and/or a summary of the monitoring that was completed.

The Foundation Engineering Consultant shall maintain a Foundation Monitoring diary. The diary shall document original conditions, work in progress, including any unusual or problem situations that arise, record of actions taken by the Contractor to rectify the situation, and restored conditions. The diary shall be supported by photographs of these conditions.

### 1.0.3 Submission of Foundation Monitoring Plan

The Foundation Engineering Consultant shall, in a brief narrative, discuss the applicable experience and qualifications of specialist staff, the role each will play in administration of the Contract, the authority to be assumed, and the reporting relationships with the construction administration staff.

The Consultant shall also complete the Foundation Monitoring Plan table in the format provided below.

<b>Foundation Monitoring Plan</b>		
<b><i>Major Inspection Tasks</i></b>	<b><i>Level of Inspection</i></b>	<b><i>Deliverable Record(s)</i></b>

List major inspection tasks associated with foundation monitoring.	State frequency/level of inspection.	List associated Deliverable Records for each task.
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#### 1.0.4 Purpose

The purpose of the Monitoring Program is to monitor settlements and pore water pressures in the foundation soils at selected locations during the construction activities associated with the White Clay River bridge replacement.

The timing for embankment construction, the preload duration and the placement of EPS at the approaches shall be controlled by the instrument readings. The instrumentation shall not be decommissioned unless instructed by the Contract Administrator after discussion with, concurrence from and unless instructed by MTO.

#### 1.0.5 Drawings

Reference shall be made to the following drawings included in the Contract Documents:

- Monitoring Plan and Typical Section (Sheet 46); and
- Typical Instrument Details (Sheet 47).

#### 1.0.6 Subsurface Conditions

The subsurface Conditions at the site are described in the following report:

- Foundation Investigation Report – White Clay River Bridge Replacement, Highway 11, Site No. 47-005, Township of Maisonville, Ontario, GWP 5239-06-00, Geocres No. 42A-77, dated September 11, 2009, by Golder Associates Ltd.

#### 1.0.7 Equipment Operation

- Monitoring shall be conducted year round. All monitoring equipment shall be maintained and rendered operational throughout the monitoring period.
- Any equipment malfunction shall be investigated and attempts shall be made to remedy the malfunction. Notification of any equipment malfunction and equipment that cannot be repaired shall be made to the Contract Administrator. Documentation of the possible causes and suggested remedial measures shall be forwarded to the Contract Administrator.

#### 1.0.8 Reading Schedule and Frequency

- The Foundation Engineering Consultant shall save and archive raw data in electronic and hard copy format.
- Monitoring shall commence immediately after the Instrumentation Installation Consultant has confirmed proper functioning of the instrumentation (i.e. has

satisfied himself that a proper baseling has been established). Monitoring is to be carried out during sub-excavation and backfilling activities (i.e. Ss) and during embankment construction and continue through the preload periods (Ss, SPs and VWPs) in accordance with the project staging. The actual length of the monitoring period depends on the construction schedule and on the results of the monitoring.

- The minimum monitoring frequencies along with the anticipated number of readings for the monitoring areas in this contract are given in Tables 1a to 1c. The monitoring frequency is the same for each individual instrument indicated in the tables. Instruments shall be read more or less frequently if judged to be required by the Contract Administrator.
- It should be noted that the number of readings given in Tables 1a to 1c are estimates and may vary depending on the actual construction schedule.

**Table 1a - Minimum Monitoring Frequency of Ss**

STAGE	FREQUENCY	ANTICIPATED NO. OF READINGS/SITE VISITS (**)
Baseline Readings (*) (After installation and prior to start of sub-excavation)	3 readings on 3 consecutive days, no sooner than 3 days following installation	3
Immediately Prior to Sub-excavation and backfilling operations for new embankment	Once	1
During backfilling and embankment construction to Elevation 310.5 m within 50 m of the monitoring instrument	Twice a day	40
During instrument installation period (VWPs and SPs on new embankment)	Once per day	15
During embankment construction to Elevation 312.0 m (new south embankment) and Elevation 312.3 m (new north embankment)	Twice a day	14
Preload period after end first stage of new embankment construction (anticipated duration about 6 months)	Daily	10
	-For 2 weeks	
	Weekly	6
	-For 6 weeks	
	Bi-weekly	2
	-For 1 month	
	Monthly	3
	-To end of wait period	

Immediately Prior to Filling above Elevation 312.0 m (new south embankment) and Elevation 312.3 m (new north embankment)	Once	1
Period after end of second stage of embankment construction	Weekly -For 6 weeks	6

(\*) Baseline readings: Value of instrument readings taken prior to construction to provide a baseline against which all subsequent readings are compared to assess movements of the ground.

(\*\*)Number of readings may vary.

**Table 1b - Minimum Monitoring Frequency of SPs**

STAGE	FREQUENCY	ANTICIPATED NO. OF READINGS/SITE VISITS (**)
Baseline Readings (*) (After installation once the excavation and backfilling has reached Elevation 310.5 m)	3 readings on 3 consecutive days, no sooner than 3 days following installation	3
Immediately Prior to Filling above Elevation 310.5 m within 50 m of the monitoring instrument	Once	1
During embankment construction to Elevation 312.0 m (south embankment) and Elevation 312.3 m (north embankment)	Twice a day	14
Preload period after end first stage of embankment construction (anticipated duration about 6 months)	Daily -For 2 weeks	10
	Weekly -For 6 weeks	6
	Bi-weekly -For 1 month	2
	Monthly -To end of wait period	3
Immediately Prior to excavating for EPS placement	Once	1

(\*) Baseline readings: Value of instrument readings to provide a baseline against which all subsequent readings are compared to assess movements of the ground. The baseline reading shall be considered the average of the three readings as the instrument is not likely to stabilize due to the filling that will have taken place prior to instrument installation.

(\*\*)Number of readings may vary.

**Table 1c - Minimum Monitoring Frequency of VVPs**

STAGE	FREQUENCY	ANTICIPATED NO. OF READINGS/SITE VISITS (**)
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Baseline Readings (*) (After installation once the excavation and backfilling has reached Elevation 310.5 m)	3 readings on 3 consecutive days, no sooner than 3 days following completion of installation	3
Immediately Prior to Filling above Elevation 310.5 m within 50 m of the monitoring instrument	Once	1
During embankment construction to Elevation 312.0 m (south embankment) and Elevation 312.3 m (north embankment)	Twice a day	14
Preload period after end first stage of embankment construction (anticipated duration about 6 months)	Daily	
	-For 2 weeks	10
	Weekly	
	-For 6 weeks	6
	Bi-weekly	
	-For 1 month	2
	Monthly	
	-To end of wait period	3
Immediately Prior to excavating for EPS placement	Once	1
Period after end of second stage of embankment construction	Weekly -For 6 weeks	6

(\*) Baseline readings: The baseline reading shall be considered the average of the three readings as the instrument is not likely to stabilize due to the filling that will have taken place prior to instrument installation. The readings are required to determine that the instrument is functioning correctly. The VWP readings will be compared to the static background river water level to assess changes in the piezometric head.

(\*\*) Number of readings may vary. Readings at VWPs to be recorded by the data logger at the frequency outlined in the Contract Specifications.

## 2.0 INSTRUMENTATION SPECIFIC REQUIREMENTS

### 2.0.1 Settlement Pins

#### Surveying

The elevations of the settlement pins (Ss) shall be surveyed to an accuracy of plus/minus 2 mm, or better, and shall be reported to the nearest millimetre.

Surveying for settlement monitoring shall be conducted by a surveyor with appropriate equipment and experience to meet the required accuracy requirements.

#### Reporting

A brief interpretation of the updated monitoring data shall be reported to the Contract Administrator within one (1) working day during construction and on a weekly basis thereafter, after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Progress Reports.

As a minimum the following shall be submitted to the Contract Administrator in the Progress Reports based on the readings collected from the Ss.

- A plot of settlement of the existing embankment versus time;
- Fill height of the new embankment within 20 m of the instruments versus time;
- Plan view, cross section and profile sketches showing the top of fill on new embankment while the Ss are being surveyed.

#### Review Levels and Alert Levels

Typically, embankment failures result in an acceleration of settlements after placement of a lift of fill. If such a condition is observed or the maximum settlement measured exceeds the respective Review Level in Table 2a, the Foundation Engineering Consultant shall immediately inform the Contract Administrator and discuss response action(s) to prevent the Alert Levels being reached. All construction work shall be continued such that instrument Alert Levels are not reached.

If the maximum settlement measured exceeds the respective Alert Level in Table 2a, the Foundation Engineering Consultant shall immediately inform the Contract Administrator and the Contract Administrator shall instruct the Contractor to stop all construction activities on and within the new embankment. No construction shall take place on the new embankment until all the following conditions are satisfied:

- The cause of the accelerated settlement has been identified and analyzed by the Foundation Design Engineer;
- Any corrective action deemed necessary by the Foundation Design Engineer has been implemented;
- The Contract Administrator deems it safe to proceed.

**Table 2a – Review Levels and Alert Levels for Settlement Pins**

Location	Instrument Number	Station / Offset (*)	Settlement Response Levels (mm)	
			Review	Alert
Existing South Approach	S#1	15+388 / 11.0 m RT	25	40
	S#2	15+413 / 11.0 m RT	25	40
	S#3	15+438 / 11.5 m RT	25	40
	S#4	15+463 / 12.3 m RT	25	40
Existing North Approach	S#5	15+528 / 12.7 m RT	25	40
	S#6	15+553 / 11.8 m RT	25	40
	S#7	15+578 / 11.2 m RT	25	40
	S#8	15+603 / 11.0 m RT	25	40
	S#9	15+628 / 10.3 m RT	25	40
	S#10	15+653 / 9.5 m RT	25	40

	S#11	15+678 / 8.4 m RT	25	40
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(\*) Offset from centreline of new Highway 11.

## 2.0.2 Settlement Plates

### Surveying

The elevations of the top of the settlement plates (SPs) shall be surveyed to an accuracy of plus/minus 2 mm, or better, and shall be reported to the nearest millimetre.

Surveying for settlement monitoring shall be conducted by a surveyor with appropriate equipment and experience to meet the required accuracy requirements.

### Reporting

A brief interpretation of the updated monitoring data shall be reported to the Contract Administrator within one (1) working day during construction and on a weekly basis thereafter, after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Progress Reports.

As a minimum the following shall be submitted to the Contract Administrator in the Progress Reports based on the readings collected from the SPs:

- A plot of settlement of the base of the preload versus time;
- Fill height within 20 m of the instruments versus time;
- Plan view, cross section and profile sketches showing the top of preload while the SPs are being surveyed.

### Review Levels and Alert Levels

Typically, embankment failures result in an acceleration of settlements after placement of a lift of fill. If an acceleration in the rate of settlement is observed or the maximum settlement measured exceeds the respective Review Level in Table 2b, the Foundation Engineering Consultant shall immediately inform the Contract Administrator and discuss response action(s) to prevent the Alert Levels being reached. All construction work shall be continued such that instrument Alert Levels are not reached.

If the maximum settlement measured exceeds the respective Alert Level in Table 2b, the Foundation Engineering Consultant shall immediately inform the Contract Administrator and the Contract Administrator shall instruct the Contractor to stop all construction activities on and within the embankment. No construction shall take place on the affected embankment until all the following conditions are satisfied:

- The cause of the accelerated settlement has been identified and analyzed by the Foundation Design Engineer;
- Any corrective action deemed necessary by the Foundation Design Engineer has been implemented;
- The Contract Administrator deems it safe to proceed.



**Table 2b – Review Levels and Alert Levels for Settlement Plates**

Location	Instrument Number	Station / Offset (*)	Settlement Response Levels (mm)		Estimated Settlement Prior to SP Installation (mm)**
			Review	Alert	
South Approach	SP#1	15+413 / 6 m LT	140	170	30
	SP#2	15+413 / 6 m RT	140	170	30
	SP#3	15+438 / 6 m LT	180	215	30
	SP#4	15+438 / 6 m RT	180	215	30
	SP#5	15+463 / 6 m LT	130	160	30
	SP#6	15+463 / 6 m RT	130	160	30
North Approach	SP#7	15+528 / 6 m LT	130	160	25
	SP#8	15+528 / 6 m RT	130	160	25
	SP#9	15+553 / 6 m LT	190	230	25
	SP#10	15+553 / 6 m RT	190	230	25
	SP#11	15+603 / 6 m LT	155	185	20
	SP#12	15+603 / 6 m RT	155	185	20

(\*) Offset from centreline of new Highway 11.

(\*\*) Estimated settlement expected to occur prior to installation of SPs after embankment construction reaches Elevation 310.5 m. Refer to attached settlement plots for embankment construction to Elevation 312.0 m and 312.3 m for the south and north approach embankments, respectively.

The measured rate of settlement shall be compared to the predicted rates shown on the settlement versus time plots, as appropriate. If the measured rate of settlement is greater than that predicted, notify the Contract Administrator.

### 2.0.3 Vibrating Wire Piezometers

#### Readout Unit

The vibrating wire piezometers (VWPs) shall be read using the Portable Laptop Computer supplied by the Contractor.

#### Coordination of Readings

The VWP data reduction (calculation of excess pore pressure – EPP: pore pressure in excess of hydrostatic) requires the groundwater level elevation at the time the VWPs are read. Therefore, the elevation of the reference point (i.e. water level in the White Clay River) shall be obtained by surveying on the same day when the VWPs are monitored.

#### Surveying

The elevation of the river water level shall be surveyed to an accuracy of plus/minus 2 mm, or better, reported to the nearest millimetre.

Surveying of the river water level shall be conducted by a surveyor with appropriate equipment and experience to meet the required accuracy requirements.

### Reporting

A brief interpretation of the updated monitoring data shall be reported to the Contract Administrator within one (1) working day during construction and on a weekly basis thereafter, after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Progress Reports.

As a minimum the following shall be submitted to the Contract Administrator in the Progress Reports:

- Plots of piezometric elevation versus time for VWPs and the river water level reference;
- Plots of piezometric EPP and embankment vertical stress versus time for VWPs;
- Plan view, cross section and profile sketches showing the top of fill location while the VWP readings were being taken;
- Completed embankment fill height and date of completion on the piezometric head plots; and
- Review and Alert Levels on the EPP plots.

### Review Levels and Alert Levels

The increase in pore pressure in the foundation soils associated with the placement of fill lifts should be equal to or lower than the increase in total vertical stress due to the fill placement, relative to the timing of installation of the VWPs (i.e. after fill placement to Elevation 310.5 m).

The failure of embankments founded on soft soils is usually associated with increases in pore pressure in excess of the increase in embankment total stress as described above.

If such a condition is observed or the maximum excess pore pressure measured exceeds the respective Review Level in Table 2c, the Foundation Engineering Consultant shall immediately inform the Contract Administrator and discuss response action(s) to prevent the Alert Levels being reached. All construction work shall be continued such that the instrument Alert Levels are not reached.

If the maximum excess pore pressure measured exceeds the respective Alert Level in Table 2c, the Foundation Engineering Consultant shall immediately inform the Contract Administrator and the Contract Administrator shall instruct the Contractor to stop all construction activities on and within the embankment until all the following conditions are satisfied:

- The cause of the excess pore pressure has been identified and analyzed by the Foundation Design Engineer;
- Any corrective action deemed necessary by the Foundation Design Engineer has been implemented;
- The Contract Administrator deems it safe to proceed.

**Table 2c - Review Levels and Alert Levels for Excess Pore Pressures for VWPs**

Location	Instrument No.	Station / Offset (*)	Tip Elevation (m)	Embankment Construction Stage**	Excess Pore Pressure (EPP) - Response Levels (kPa)***		
					Review	Alert	Max EPP before following stage****
South Approach	VWP#1	15+413 / 0 m	303.5	1a	30	35	5
	VWP#2	15+463 / 0 m	302.7	1a	25	30	5
North Approach	VWP#3	15+528 / 0 m	302.2	1a	10	15	5
	VWP#4	15+603 / 0 m	303.9	1a	30	35	5

(\*) Offset from centreline of new Highway 11.

(\*\*) Staging of embankments required to maintain stability, see Contract Documents for details.

(\*\*\*) EPP values are calculated referenced to a static background water level Elevation 310.1 m.

(\*\*\*\*) Waiting period prior to Stage 1b construction shall be determined based on the specified Excess Pore Pressures (EPP). EPS/Fill placement following the waiting period shall not take place until the EPP drops to below the specified values. These values should be used in conjunction with other monitoring data to determine when preloading is complete.

### 3.0 CONTROL MONITORING LEVELS

#### 3.0.1 General

The monitoring program will provide input for the rate of placement of preload fill and preload duration.

##### Stabilization of Settlements due to Primary Consolidation

Settlement data monitored at the SPs allow for an assessment of the approximate total settlement due to primary consolidation and the approximate time required for settlements due to primary consolidation to stabilize.

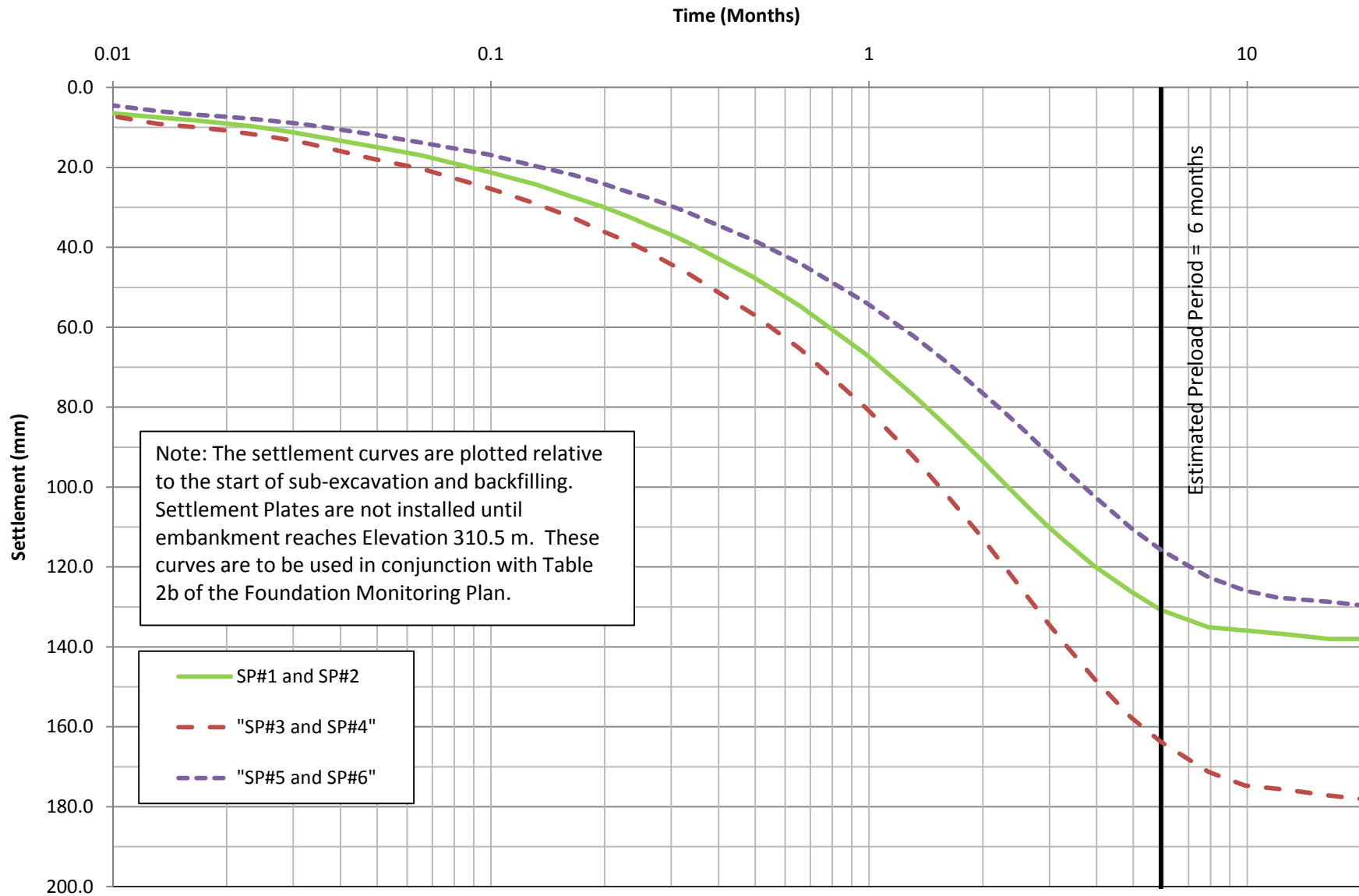
The anticipated magnitude of total settlement and the required time for settlements due to primary consolidation to stabilize shall be assessed for each of the SPs using an appropriate method.

Settlement data monitored at the Ss allow for an assessment of the approximate settlement occurring on the existing highway while in operation during construction of the replacement bridge.

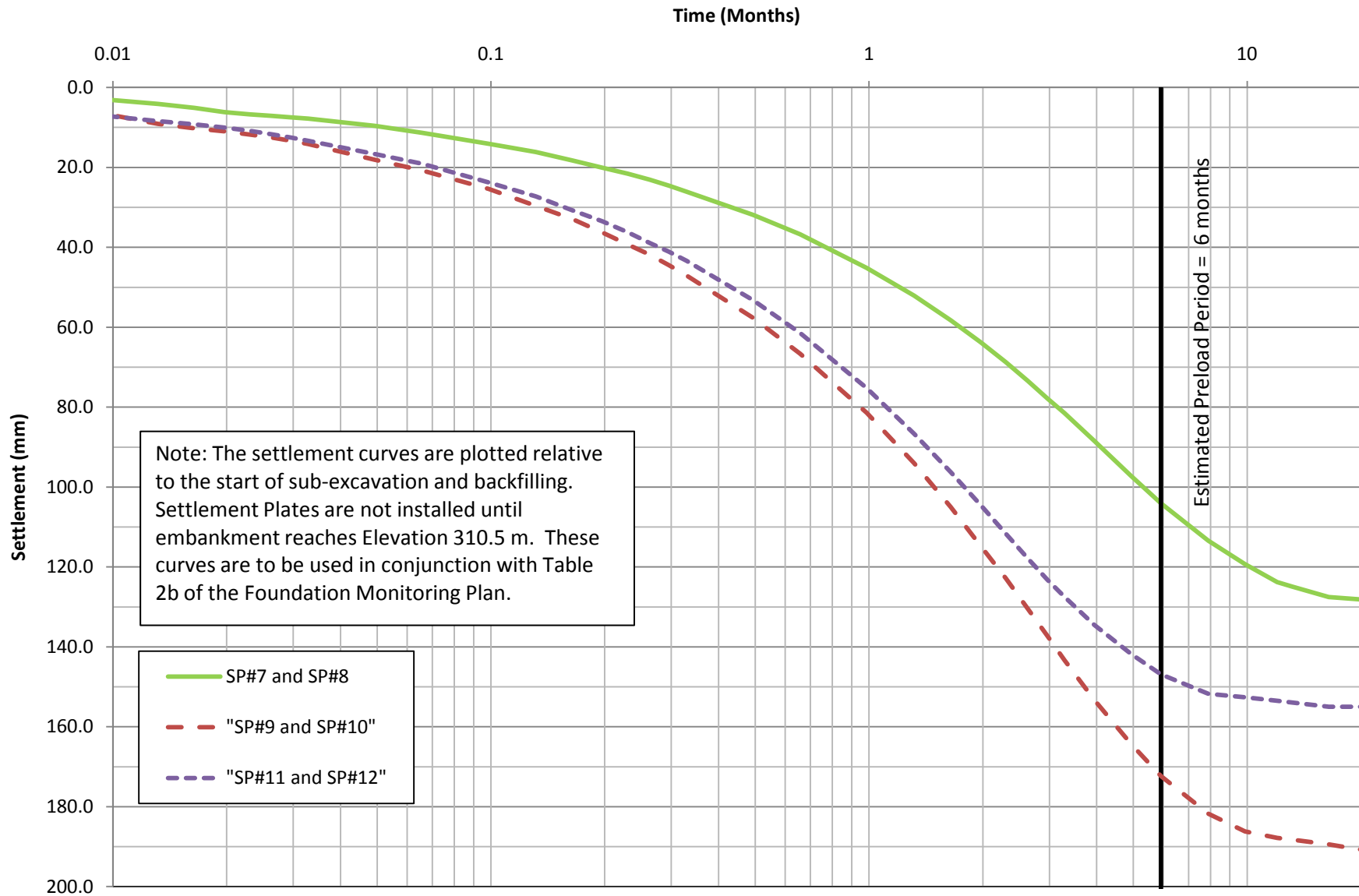
### 4.0 FINAL REPORT

At the completion of the monitoring program, a final monitoring report shall be issued to the Contract Administrator. The monitoring results shall be presented in tabular and graphical form as described above for each instrument type. An interpretation of the monitoring readings shall be included in the report.

# White Clay River Bridge Replacement - South Approach Embankment Constructed to Elevation 312.0



# White Clay River Bridge Replacement - North Approach Embankment Constructed to Elevation 312.3



**UNWATERING - Item No.**

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**Non-Standard Special Provision**

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Pile caps construction below the groundwater and/or river water levels must be carried out in-the-dry. The excavation shall be kept stable during the work.

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**Operational Constraint – Obstructions – Timber Piles**

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As part of the work for the installation of piles and excavations for pile caps for the structure foundation elements, the Contactor shall be alerted that timber piles and timber cribbing from the existing bridge structure are present at the site.

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**Operational Constraint – Obstructions - Boulders**

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The Contractor shall be alerted that the fill and slope materials may contain cobbles and/or boulders. Cobbles and boulders may be encountered within the silt and sand to sand and gravel deposit. A layer of cobbles and boulders overlies the bedrock.



## **VIBRATION MONITORING - Item No.**

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### **Non-Standard Special Provision**

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#### **1.0 GENERAL**

##### **1.1 Scope**

This special provision describes requirements for vibration monitoring of the existing structure during pile driving at the site.

#### **2.0 DEFINITIONS**

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificate(s) of conformance.

#### **3.0 SUBMISSION REQUIREMENTS**

The Contractor shall submit three (3) copies of the vibration monitoring plan to the Contract Administrator at least 3 weeks prior to the piling operations. The vibration monitoring shall satisfy the specifications and at a minimum contain the following specific information:

Name of Firm/QVE responsible for monitoring including qualifications of vibrations monitoring specialist;  
Proposed instrumentation;  
Proposed location of instruments on existing structure;  
Proposed frequency of readings; and  
Proposed methods for adjusting piling methods if readings show excess vibrations.

#### **4.0 PROCEDURES**

##### **4.1 Locations of Vibration Monitoring Equipment**

The vibration monitoring equipment shall be placed directly on the concrete foundations of the existing bridge abutments as close as possible to the pile driving operations.

#### **5.0 MONITORING**

##### **5.1 Vibration Limits**

The vibrations on the existing footing shall not exceed 50 mm/s (peak particle velocity).

## **5.2 Frequency of Readings**

- 5.2.1 The Contractor shall take readings on the first pile in each pile group (i.e. at each corner of the abutment), starting with the pile furthest away from the existing structure. As a minimum, the readings should be taken and recorded during the first 3 m of driving and continuously during driving through the bouldery deposits and during seating of the pile onto the bedrock.

## **5.3 Submission of Results**

- 5.3.1 The results shall be submitted to the Contract Administrator prior to continuing with the remaining piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results. Additional submissions may be required at the discretion of the Contract Administrator. The results shall be immediately reviewed by the QVE and submitted to the Contract Administrator prior to the Contractor continuing with the remaining piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with the vibration monitoring results.
- 5.3.2 If the results are acceptable, the Contractor may continue installing the remaining piles with vibration monitoring readings being taken during driving of each pile during bedrock seating. The results of subsequent piles should be submitted to the Contract Administrator at the end of each day.
- 5.3.3 If the readings are not within the limits stated above, the Contractor must alter his driving procedures until the vibrations on the existing structure are within acceptable levels. The above process must be repeated for each pile.

## **6.0 CERTIFICATE OF CONFORMANCE (COC)**

Upon completion of the work in each area of pile driving, the Contractor shall submit to the Contract Administrator a CoC sealed and signed by the QVE. The certificate shall state that the vibrations on the existing structure were below the limits stated above, and where the levels were exceeded, what procedures were used to reduce the vibrations to below the limits stated above.

## **7.0 BASIS OF PAYMENT**

Payment at the contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

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