



January 21, 2010

REPORT



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**Foundation Investigation and Design Report
Frederick House Bridge Replacement
Highway 11, Site No. 39E-045
Township of Clute, Ontario
Ministry of Transportation, Ontario
GWP 5541-05-00**

Submitted to:

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Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	1
3.0 INVESTIGATION PROCEDURES	1
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	3
4.1 Regional Geology	3
4.2 Subsurface Conditions	3
4.2.1 Organics/Fill/Alluvium	3
4.2.2 Sand	4
4.2.3 Silty Clay to Clay	4
4.2.4 Silt to Sand and Silt	5
4.2.5 Sand and Gravel/Cobbles and Boulders	5
4.2.6 Bedrock	6
4.2.7 Groundwater Conditions	6
5.0 CLOSURE	7

PART B – FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS	9
6.1 General	9
6.1.1 Background Information and Proposed Works	9
6.1.2 Foundation Alternatives	9
6.2 Shallow Foundations	10
6.2.1 Geotechnical Axial Resistance	10
6.2.2 Resistance to Lateral Loads	10
6.2.3 Frost Protection	10
6.3 Steel H-Pile Foundations	11
6.3.1 Geotechnical Axial Resistance	11
6.3.2 Downdrag	12
6.3.3 Set Criteria	12



FOUNDATION REPORT - FREDERICK HOUSE BRIDGE REPLACEMENT HIGHWAY 11, GWP 5541-05-00

6.3.4	Pile Driving Note	12
6.3.5	Resistance to Lateral Loads	13
6.3.6	Frost Protection	14
6.4	Caissons	14
6.4.1	Geotechnical Axial Resistance	14
6.4.2	Downdrag	15
6.4.3	Resistance to Lateral Loads	15
6.4.4	Frost Protection	15
6.5	Site Coefficient	15
6.6	Lateral Earth Pressures for Design	16
6.6.1	Static	16
6.6.2	Dynamic	17
6.7	Approach Embankment Design and Construction	18
6.7.1	Stability	19
6.7.1.1	Methodology	19
6.7.1.2	Parameter Selection	19
6.7.1.3	Results of Analysis	20
6.7.2	Liquefaction Potential and Seismic Analysis	21
6.7.3	Settlement	21
6.7.3.1	Methodology	21
6.7.3.2	Settlement Criteria	21
6.7.3.3	Parameter Selection	22
6.7.3.4	Settlement of Embankment Fill	24
6.7.3.5	Settlement of Foundation Soils	24
6.7.3.6	West Approach	24
6.7.3.7	East Approach	25
6.7.4	Mitigation of Stability/Settlement Issues	25
6.7.4.1	Slope Flattening/Berms	25
6.7.4.2	Lightweight (EPS) Fill	26
6.7.4.3	Preloading	27



FOUNDATION REPORT - FREDERICK HOUSE BRIDGE REPLACEMENT HIGHWAY 11, GWP 5541-05-00

6.7.4.4	Grading	27
6.7.4.5	Sub-excavation	27
6.7.4.6	Staged Construction.....	28
6.7.4.7	Wick Drains/Ground Improvement	28
6.8	Culvert Extension STA 14+285 LT	28
6.8.1	Open Footing Culvert Extension Replacement.....	28
6.8.2	Box Culvert Extension	29
6.8.3	Culvert Bedding, Backfill and Erosion Protection	30
6.8.4	Settlement and Culvert Connection Requirements.....	30
6.9	Subgrade Preparation and Embankment Construction	31
6.10	Design and Construction Considerations	31
6.10.1	Excavations	31
6.10.2	Temporary Shoring.....	32
6.10.3	Groundwater and Surface Water Control	32
6.10.4	Obstructions	32
6.10.5	West Embankment Monitoring	33
6.10.6	Existing Structure Monitoring.....	33
7.0	CLOSURE	33

TABLES

Table 1: Evaluation of Foundation Alternatives – West Abutment and Pier.....	1
Table 2: Evaluation of Foundation Alternatives – East Abutment and Pier.....	2
Table 3: Parameters for Horizontal Subgrade Reaction	3
Table 4: Results of Settlement Analyses	4
Table 5a: Evaluation of Stability/Settlement Mitigation Alternatives – West Approach	5
Table 5b: Evaluation of Stability/Settlement Mitigation Alternatives – East Approach	6
Table 6: Foundation Excavation Details	7

DRAWINGS

Drawing 1	Highway 11, Frederick House River Bridge, Borehole Location and Soil Strata
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FIGURES

Figure 1	Undrained Shear Strength vs. Elevation – West Approach
Figure 2	Critical Stability Sections
Figure 3	Stability Analysis – Section C-C, West Abutment Front Slope, South Side
Figure 4	Stability Analysis – Section E-E, East Abutment Front Slope
Figure 5	Stability Analysis - Section F-F, East Abutment Cross-Section
Figure 6	Stability Analysis - Section A-A, West Abutment Cross-Section with Berms
Figure 7	Stability Analysis – Section I-I, STA 14+320 Cross-Section with Berms
Figure 8	Stability Analysis - Section B-B, West Abutment Front Slope, North Side with Slope Flattening
Figure 9	Stability Analysis – Section D-D, East Abutment Front Slope with Slope Flattening
Figure 10	Stability Analysis – Section H-H, STA 14+300 Cross-Section with Berms and 2.6 m EPS
Figure 11	Stability Analysis –Section G-G, STA 14+280 Cross-Section with Berms and 3.9 m EPS
Figure 12	EPS Details, West Approach STA 14+255 to 14+315
Figure 13	Settlement Analysis – West Approach, North Side, STA 14+280
Figure 14	Settlement Analysis – West Approach, North Side, STA 14+300

APPENDICES

Appendix A Record of Borehole and Drillhole Sheets

List of Symbols and Abbreviations
Lithological and Geotechnical Rock Description Terminology
Record of Borehole and Drillhole Sheets

Appendix B Laboratory Test Results

Figure B-1	Grain Size Distribution – Alluvium (Fill)
Figure B-2	Plasticity Chart – Clayey Silt to Silty Clay (Fill)
Figure B-3	Plasticity Chart – Silty Clay to Clay
Figure B-4	Grain Size Distribution – Clayey Silt
Figure B-5	Oedometer Consolidation Summary – Borehole FR-5, Sample 9
Figure B-6	Oedometer Consolidation Summary – Borehole FR-6, Sample 8
Figure B-7	Oedometer Consolidation Summary – Borehole FR-7, Sample 8
Figure B-8a	Grain Size Distribution – Silt to Sand and Silt
Figure B-8b	Grain Size Distribution – Silty Sand to Sand and Gravel Interlayers
Figure 9	Grain Size Distribution – Sand to Sand and Gravel
Table B-1	Uniaxial Compressive Strength Test Results

Appendix C Non-Standard Special Provisions and Operational Constraints

NSSP	Rigid EPS Installation
OC	Obstructions and Ground Control



PART A

FOUNDATION INVESTIGATION REPORT
FREDERICK HOUSE RIVER BRIDGE REPLACEMENT
HIGHWAY 11, SITE NO. 39E-045
TOWNSHIP OF CLUTE, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5541-05-00



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 Frederick House River Bridge replacement on Highway 11 in the Township of Clute, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P7-1191-0007, dated February 26, 2007, which forms part of the Consultant's Agreement (P.O. Number 5006-E-0015) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated September 18, 2007. The General Arrangement drawing (GA) for the bridge structure was provided to Golder by LEA in April 2009 and revised in June 2009.

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement structure locations by borehole drilling, in situ testing and laboratory testing on selected samples. The boreholes for the current investigation were located in the field by Golder relative to the centreline and offset stakes laid out at the site by LEA, which were based on the April 2009 GA. The location of the investigated area is shown in plan on Drawing 1.

The investigation was supplemented with information contained in the following report from MTO's GEOCRES system:

- Foundation Investigation Report, Highway 11, Frederickhouse River Bridge, Structure Rehabilitation, W.P. 647-90-01, GEOCRES No. 42H-32, by Shaheen & Peaker Limited, dated February 1, 2006.

2.0 SITE DESCRIPTION

The site is situated in the Township of Clute on Highway 11 crossing Frederick House River, approximately 11 km west of Highway 652. The surrounding land is mainly comprised of scattered residences, residential farms and businesses. Grass and tree cover extend beyond the limits of the site, while the banks adjacent to the river are vegetated with grass and small shrubs. The river is up to 5 m deep and is mainly used for recreation. The river banks are relatively steep in the immediate area of the bridge. The river channel is about 60 m wide at the existing bridge location.

We understand that the existing Frederick House River Bridge, a five-span steel truss structure, was constructed in 1948 while the proposed bridge will be a three-span structure. The existing bridge has an overall deck length of about 135 m and overall width of about 10 m. We understand from LEA that rehabilitations/repairs have been made over the years to the bridge superstructure and as recently as 2006 to the abutments. The water level of Frederick House River was measured at approximately Elevation 242.3 m in November 2007 by others.

3.0 INVESTIGATION PROCEDURES

A total of eight (8) boreholes were advanced at the site between April 16 and 28, 2009. Six boreholes (FR-1 to FR-6) were advanced for the proposed main bridge abutments, piers and approaches. Two additional boreholes (FR-7 and FR-8) were advanced to address the high embankment fill and culvert extension west of the proposed structure. Three additional boreholes were advanced between July 29 and 30, 2009. Two boreholes (FR-9 and FR-10) were advanced at the west approach toe and one borehole (FR-4A) was advanced at the east pier and taken to bedrock. The locations and elevations of the boreholes are shown on Drawing 1 and presented on the Record of Borehole sheets in Appendix A.



Boreholes FR-1 to FR-3, FR-4A and FR-5 to FR-10 were drilled using a track mounted CME 45-C drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville-Sur-La-Rouge, Quebec. Borehole FR-4 was drilled using portable equipment that was supplied and operated by OGS Inc. of Almonte, Ontario.

The boreholes were advanced to depths ranging from 5.6 m to 29.0 m below the existing ground surface. The machine drilled boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers, with NW casing with wash boring, where necessary. The borehole advanced using portable equipment was advanced using NW and BW casing with wash boring to 5.6 m depth where casing refusal was met probably on the surface of the very dense sand and gravel.

A combination of continuous and non-continuous (at intervals of depth of about 0.75 m to 2.5 m) sampling methods were used in obtaining soil samples. Samples were obtained using a 50 mm outside 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 select borehole depths. Field vane shear tests were conducted in cohesive soils for assessment of undrained shear strengths (ASTM D2573-01) using an MTO standard "N" size vane. Rock core samples were obtained using an 'NQ' size core barrel and the bedrock was cored for a minimum of 3 m.

The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in two boreholes, FR-1 and FR-6, at the west and east approaches, respectively, to allow monitoring of the groundwater level at these locations. The piezometers consisted of 50 mm O.D. rigid PVC tubing with a 1.5 m long slotted screen, sealed within the silt to sand and silt stratum. The boreholes were backfilled with bentonite as per Ontario Regulation 903 (as amended by O. Reg. 372) upon completion of drilling. The two piezometers were decommissioned in a similar manner after the last water level reading was obtained in July 2009. The installation details and water level readings are presented on the Record of Borehole sheets that follow the text of this report.

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. 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. One-dimensional consolidation (oedometer) tests were carried out on Shelby tube samples of the cohesive soil deposit from three boreholes. In addition, uniaxial compressive strength (UCS) testing was carried out on selected samples of the bedrock core recovered from the boreholes.

The proposed boreholes were laid out in the field by Golder relative to the proposed centreline alignment and offset stakes surveyed by LEA and based on the GA supplied by LEA in April 2009. The northings and eastings in MTM NAD 83 coordinate system were determined by plotting the station and offset of the boreholes (relative to the stakes) on the April 2009 GA and converting to the coordinate system. The ground surface elevations were measured relative to the stakes and are referenced to geodetic datum.



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Published literature indicates that the site is located in the Western Abitibi Subprovince of the Superior Province (Geology of Ontario; OGS Special Volume 4)¹. The bedrock of this domain consists of metavolcanic and minor metasedimentary rocks.

Based on terrain mapping by the Ontario Geological Survey², the subsurface soils in the vicinity of the site consist of glaciolacustrine plain deposits comprising of organic peat and clayey silts terrain overlying bedrock.

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 samples, are given on the Record of Borehole and Drillhole sheets attached 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 at the bridge location is shown on Drawing 1.

In general, the subsoils at the structure site generally consist of embankment fill or alluvium, underlain by silty clay to clay and silt to sand and silt. The silt to sand and silt deposit was encountered overlying a sand and gravel deposit or directly overlying bedrock. The groundwater level was generally consistent with the river water level. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Organics/Fill/Alluvium

Between 0.1 m and 0.6 m of organics was encountered in Boreholes FR-1 to FR-4, FR-4A and FR-7 at the ground surface between Elevation 242.9 m and 252.0 m. Embankment fill was encountered at the ground surface or below the organics in all boreholes except FR-3. In Boreholes FR-3, FR-4 and FR-4A, alluvium was encountered below the organics and/or fill. The total thickness of organics/fill/alluvium in the boreholes varied from 1.4 m to 7.6 m.

In Boreholes FR-6 and FR-8, between 0.6 m to 3.8 m of sand and silt to sand and gravel fill containing some clay was encountered. The fill contained clayey silt layers in Borehole FR-6. The surface of the cohesionless fill was encountered between Elevation 257.5 m and 258.5 m. In Borehole FR-6, an obstruction was encountered at a 2.3 m depth within the fill and the borehole was moved as a result.

In Boreholes FR-1, FR-2, FR-4, FR-4A, FR-5, FR-7 and FR-8, 0.6 m to 7.0 m of silty clay to silt fill, containing trace sand, trace gravel and trace organics was encountered between Elevation 242.3 m and 257.9 m.

In Boreholes FR-3, FR-4 and FR-4A, grey, brown or black silty clay to clayey silt to silt alluvium was encountered below the fill or organics and contained some sand and gravel and trace organics.

SPT 'N' values measured within the cohesionless fill layer ranged from 12 blows to 33 blows per 0.3 m of penetration suggesting a very loose to dense relative density. SPT 'N' values measured within the cohesive fill layer varied from 2 to 19 blows per 0.3 m of penetration suggesting a soft to very stiff consistency.

SPT 'N' values measured within the alluvium ranged from 5 blows to 13 blows per 0.3 m of penetration suggesting a very loose to compact relative density. In Borehole FR-3, one 'N' value of 40 was measured near the base of the layer suggesting the alluvium becomes dense with depth.

¹ 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.

² Northern Ontario Engineering Geology Terrain Study, OGS Electronic Map



One grain size distribution test from the alluvium is shown on Figure B-1 in Appendix B. Note that this sample is indicative of the layering within the alluvium as it is classified as a sand and silt.

Atterberg limits testing carried out on samples of the cohesive fill and alluvium as well as a clayey silt seam within the cohesionless fill indicate liquid limits ranging from about 25 percent to 42 percent and plastic limits ranging from about 13 percent to 19 percent, yielding plasticity indices ranging from about 12 percent to 24 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B-2, and indicate that the fill material is classified as a clayey silt of low plasticity to a silty clay of intermediate plasticity.

The natural water content measured on samples of the cohesionless fill ranged between about 6 percent and 19 percent. In the cohesive fill and alluvium, the measured water content ranged between 17 percent and 42 percent.

4.2.2 Sand

A 1.3 m thick layer of sand containing trace gravel and clay pockets was encountered at the ground surface at Elevation 244.7 m and 246.7 m in Boreholes FR-9 and FR-10, respectively.

SPT 'N' values measured within the sand ranged from 4 blows to 8 blows per 0.3 m of penetration suggesting a very loose to loose relative density.

4.2.3 Silty Clay to Clay

A deposit of moist to wet, brown to grey, silty clay to clay was encountered beneath the fill in Boreholes FR-1, FR-2 and FR-5 to FR-10. This deposit was not encountered in the boreholes immediately adjacent to the river. The deposit was distinctly varved in Boreholes FR-1 and FR-5 to FR-10 and somewhat varved in Borehole FR-2. The top of the deposit was encountered at Elevation 243.4 m to 253.7 m and the thickness ranged from 0.8 m to 8.1 m.

SPT 'N' values measured within this deposit ranged from 0 blows (weight of hammer) to 15 blows per 0.3 m of penetration being greatest near the top of the deposit and in Boreholes FR-9 and FR-10. In situ field vane testing carried out within this stratum measured undrained shear strengths that ranged from about 27 kPa to 86 kPa. The results are shown on Figure 1 for the west approach (with the Bjerrum correction for plasticity). The SPT 'N' values combined with the undrained shear strengths indicate that the material had a soft to stiff consistency, typically firm to stiff.

Atterberg limits testing carried out on several samples of the silty clay to clay deposit indicate liquid limits that ranged from about 38 to 68 percent and plastic limits that ranged from about 18 to 25 percent, yielding plasticity indices that ranged from about 19 to 43 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B-3 in Appendix B, and indicate that the deposit ranged from a silty clay of intermediate plasticity to a clay of high plasticity. In two samples, the clay and clayey silt varves were tested separately. The clayey silt varves had liquid limits that ranged from about 27 to 30 percent, plastic limits that ranged from about 16 to 17 percent, yielding plasticity indices that ranged from about 11 to 13 percent. The clay varves had liquid limits that ranged from about 51 to 57 percent, plastic limits that ranged from about 20 to 22 percent, yielding plasticity indices that ranged from about 31 to 35 percent. The results indicate that the varves were a clayey silt of low plasticity as shown on Figure B-3.

The results of Atterberg limits testing in Borehole FR-9 and FR-10, located at the northwest portion of the site, indicated liquid limits of about 28 and 32 percent, plastic limits of about 14 and 18 percent, yielding plasticity indices of about 13 and 14 percent. These results suggest that the deposit could be classified as more of a clayey silt of low plasticity in this area. Further, the SPT 'N' values were the highest in Borehole FR-10 with the lowest liquid limit.



One grain size distribution test of the clayey silt varve is shown on Figure B-4 in Appendix B.

The natural water content measured on samples this deposit ranged from about 24 percent to 59 percent.

Three laboratory consolidation tests were carried out on specimens of the silty clay to clay obtained from Boreholes FR-5, FR-6 and FR-7 and the test results are shown on Figures B-5, B-6 and B-7, respectively. The preconsolidation pressure was 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)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_r	C_c	c_v^* (cm ² /s)
FR-5/9	247.1	133	200	67	1.5	0.707	0.030	0.121	0.0040
FR-6/8	251.1	122	240	118	2.0	1.071	0.084	0.473	0.0038
FR-7/8	245.7	109	230	121	2.1	1.150	0.089	0.360	0.0061

Note: *For approximate stress range of $18 \leq \sigma_v' \leq 285$ kPa

where: σ_{vo}' effective overburden pressure in kPa

σ_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 cm²/s in the normally consolidated range

4.2.4 Silt to Sand and Silt

A deposit of wet, grey, silt to sand and silt containing trace to some gravel and clay was encountered beneath the silty clay to clay deposit in Boreholes FR-1, FR-2 and FR-5 to FR-10 and beneath the alluvium in Borehole FR-3. The top of the deposit ranged from Elevation 241.1 m to 246.5 m and the thickness of the deposit ranged from 1.8 m to 9.9 m. Boreholes FR-1 and FR-7 to FR-10 were terminated within this deposit.

SPT 'N' values measured within this deposit ranged from 3 blows to 27 blows per 0.3 m of penetration indicating a very loose to compact relative density.

Grain size distribution tests were carried out on several samples of the silt to sand and silt and the results are shown on Figure B-8a. One test result from Borehole FR-3 indicates that a gravel size piece was retained on the 26.5 mm sieve causing the results to portray a more gravelly material than was actually the case. This test result is shown on Figure B-8b.

The natural water content measured on samples of the deposit were between about 24 percent and 29 percent.

4.2.5 Sand and Gravel/Cobbles and Boulders

A deposit of wet, grey sand and gravel containing trace to some silt and cobbles and boulders was encountered underlying the silt to sand and silt deposit in Boreholes FR-2, FR-4, FR-4A, FR-5 and FR-6. In Borehole FR-4, this deposit is classified more as a sand to silty sand containing some gravel and cobbles. The top of this deposit was encountered between Elevation 232.7 m and 242.3 m and the thickness of the deposit varied from 3.4 m to 12.1 m. Borehole FR-6 was terminated within this deposit and Borehole FR-4 was also terminated within this deposit due to casing refusal of the portable equipment.



SPT 'N' values measured within this deposit ranged from 12 blows per 0.3 m of penetration to greater than 100, suggesting a compact to very dense relative density. The higher 'N' values are also indicative of the presence of cobbles and boulders. Difficult auger and/or casing advance was observed during drilling through the sand and gravel deposit and, in Boreholes FR-2, FR-4A and FR-5, coring using an NQ core barrel had to be used to advance the borehole through the deposit to reach the bedrock.

Grain size distribution tests were carried out on three samples of this deposit and the results are shown on Figure B-9.

The natural water content measured on samples of the deposit were between about 10 and 13 percent.

In Borehole FR-3, a 0.4 m thick layer of cobbles and boulders was encountered below the silty to sand and silt deposit at Elevation 231.3 m.

4.2.6 Bedrock

Bedrock was encountered and cored for a minimum of 3 m in Boreholes FR-2, FR-3, FR-4A and FR-5. The depth to bedrock below ground surface and corresponding bedrock surface elevation encountered at each borehole is summarized below.

Borehole	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
FR-2	17.9	229.3
FR-3	13.5	231.4
FR-4A	18.3	230.7
FR-5	25.5	229.5

Based on a review of the rock core samples, a grey, fine grained, slightly weathered metasediment bedrock was encountered. In Boreholes FR-2, FR-3, FR-4A and FR-5, silt seams, up to 0.4 m thick, were observed within the bedrock. The Rock Quality Designation (RQD) measured on the core samples typically ranged from 57 percent to 100 percent, with one core sample having an RQD of 9 percent on a 1.2 m core run. This indicates a rock mass quality that generally ranged from fair to excellent. The Total Core Recovery (TCR) during bedrock coring ranged from 65 percent to 100 percent with the lower values indicative of the presence and thickness of silt seams within the bedrock.

Laboratory UCS testing was performed on three samples of the bedrock and the results gave strengths ranging from 89 MPa to 175 MPa. The results of this testing can be found in Table B-1 in Appendix B. Based on the laboratory UCS test results, the estimated intact strength of the bedrock ranges from strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa).

4.2.7 Groundwater Conditions

The water levels were noted during and after the drilling operations in the boreholes. In general, the soil samples taken in the boreholes were noted to be moist to wet. The water levels were noted during and after the drilling and coring operations in the boreholes, and these levels may not have stabilized prior to reading. Piezometers were installed in Boreholes FR-1 and FR-6, with screened sections sealed within the silt to sand and silt deposit. Details of the piezometer installations are shown on the Record of Borehole sheets in Appendix A.



The groundwater level ranged from 1.0 m to 15.9 m below the ground surface in the boreholes. The groundwater level ranged from Elevation 240.7 m to 247.4 m. The water level in Frederick House River was surveyed by others in November 2007 at Elevation 242.3 m. The water levels in the piezometers and open boreholes during and upon completion of drilling are summarized below.

Borehole	Installation	Depth to Groundwater below Existing Ground Surface (m)	Groundwater Elevation (m)	Date
FR-1	Piezometer	4.4	245.4	April 23/09 (after installation)
		5.6	244.2	April 28/09
		6.9	242.9	July 31/09
FR-2	Open Borehole	3.7	244.1	April 23/09
FR-3	Open Borehole	3.8	240.7	April 22/09
FR-4	Open Borehole	1.0	241.9	April 21/09
FR-4A	Open Borehole	10.7 and rising	238.3	July 30/09
FR-5	Open Borehole	7.6	247.4	April 19/09
FR-6	Piezometer	11.6	245.9	April 19/09 (after installation)
		13.9	243.6	April 28/09
		11.9	245.6	July 31/09
FR-7	Open Borehole	8.3	243.7	April 27/09
FR-8	Open Borehole	15.9	242.6	April 28/09
FR-9	Open Borehole	1.5	243.7	July 30/09
FR-10	Open Borehole	4.4 and rising	242.5	July 30/09

It should be noted that groundwater levels in the area are subject to seasonal fluctuations. Although not observed in the boreholes during this investigation, the previous report (Shaheen & Peaker 2006) measured perched water within the embankment fill.

5.0 CLOSURE

The field personnel supervising the drilling program was Mr. Ed Savard, Mr. Trevor Moxam and Mr. Mat Riopelle. This report was prepared by Mr. Evan Childerhose, E.I.T. and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., an Associate with Golder in Sudbury. A quality control review of the report was provided by Mr. Fintan J. Heffernan, P.Eng., Golder's Designated MTO Contact for this project.



Report Signature Page

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PART B

**FOUNDATION DESIGN REPORT
FREDERICK HOUSE RIVER BRIDGE REPLACEMENT
HIGHWAY 11, SITE NO. 39E-045
TOWNSHIP OF CLUTE, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5541-05-00**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed new Highway 11 bridge structure over the Frederick House River. 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 approach embankments including the high fill section to the west and culvert extension. 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

6.1.1 Background Information and Proposed Works

The existing bridge carrying Highway 11 over the Frederick House River was constructed in 1948 and is a five-span steel truss structure with an overall deck length of about 135 m and width of about 10.1 m. The existing highway is between about Elevation 258.1 m and 257.5 m at the west and east abutments, respectively. Rehabilitation of the bridge was carried out in 1959, 1982 and 2006. In 1959, rehabilitation was carried out to address the rotation of the west abutment. In 1982, rehabilitation of the steel and concrete deck took place. Most recently (and ongoing), stabilization of the east abutment was carried out in 2006 by the installation of expanded polystyrene (EPS) blocks behind the abutment to reduce the load and prevent further movements. The existing abutments are constructed on shallow foundations on the native silty clay at about Elevation 247 m. The piers are also founded on shallow foundations on the native material at about Elevation 243 m for the outer piers and 238 m for the central piers.

The new bridge will be located on a new alignment on the north side of the existing bridge. We understand that the proposed bridge will be a three-span structure, 160 m long having a width of 14 m. The final grade of the new Highway 11 in this area will be between Elevation 259.2 m and 258.4 m at the west and east abutments, respectively. This will result in a grade increase of about 1 m which we understand is necessary to correct the vertical curve in this zone.

The Frederick House River is located at the base of a 15 m high valley and is about 60 m to 70 m wide at the proposed crossing location. The water level in the river was measured in November 2007 (by others) at Elevation 242.3 m. The high water level was measured in September 1994 (by others) at Elevation 244.5 m.

The boreholes advanced at the site generally consist of sand and gravel and/or clayey silt to silty clay embankment fill underlain by native silty clay to clay overlying a silt to sand and silt deposit. These deposits are underlain by a sand and gravel deposit containing cobbles and boulders. The bedrock surface was encountered at between Elevation 229.3 m and 231.4 m at the abutments and piers.

6.1.2 Foundation Alternatives

The recommendations on the foundation design aspects of the new structure presented in this report take into consideration the impact of the new foundations and approach embankments on the existing bridge foundations and approach embankments.



We recommend that the bridge foundation elements be founded on steel H-piles, driven to bedrock at the west abutment and pier and to the sand to sand and gravel deposit for the east abutment and pier. Caissons socketted into the bedrock are feasible at this site but not economical. Shallow foundations are not considered feasible for the abutments or the piers. Tables 1 and 2 summarize the advantages, disadvantages, relative costs and risks/consequences of the foundation alternatives for the west and east foundation elements, respectively. Further discussion on the alternatives is given in the sections below.

6.2 Shallow Foundations

Given the poor condition of the existing bridge abutments, the thickness and consistency/relative density of the embankment fill and underlying native soils, spread footings founded on the native soils at depth are not recommended to support the new bridge structure due to the low geotechnical axial resistance and expected settlement of these strata.

Consideration could be given to founding the bridge piers on shallow spread or strip footings on the native silt to sand and silt deposit, however, the axial resistance is likely too low for support of the bridge. The piers, if supported on spread footings founded at about Elevation 240.8 m and 240.5 m at the west and east abutments, respectively, would be constructed at or below the river water level depending on the time of year. At the west pier, the silt to sand and silt at and below founding level has a loose density. At the east pier, the density is compact to dense.

6.2.1 Geotechnical Axial Resistance

Spread footings placed directly on or below the surface of the properly prepared native silt to sand and silt at the west pier may be designed based on a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 200 kPa. A geotechnical resistance at Serviceability Limit States (SLS) of 100 kPa may be used for design, based on 25 mm of settlement and assuming a 2 m to 3 m wide footing. These values are too low for spread footing design for a bridge, much less the major bridge at this site. For spread footings at the east pier, higher values may be used but differential settlement between the east pier and east abutment would be a concern.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*.

6.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the base of the mass concrete and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.35 between the base of the concrete footings and the native silt to sand and silt, constructed in-the-dry. This value represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.2.3 Frost Protection

All footings should be provided with a minimum of 2.5 m of soil cover or equivalent thickness of insulation for frost protection (OPSD 3090.100).



6.3 Steel H-Pile Foundations

Based on the borehole information obtained at this site, steel H-piles driven to bedrock are recommended for support of the west abutment and pier. At the east abutment and pier, we recommend that the structure be supported on pile end-bearing within the sand to sand and gravel deposit containing cobbles and boulders, as it is unlikely that piles will reach the bedrock surface. We understand that the abutment will be semi-integral since an integral abutment design is not possible due to the length of the bridge (160 m).

For design, the estimated tip elevations for the piles are presented below. The elevations are based on the depth to bedrock encountered in the boreholes at the west abutment and pier. At the east abutment and pier, the elevations are based on the elevation within the sand to sand and gravel deposit where difficult auger/casing advance was encountered, indicative of the presence of cobbles and boulders. The approximate underside of pile cap elevations as shown on the latest GA drawing.

Foundation Unit	Relevant Borehole	Elevation of Cobbles and Boulders	Bedrock Surface Elevation (m)	Recommended Tip Elevation (m)	Underside of Pile Cap Elevation (m)	Approximate Design Pile Length (m)
West Abutment	FR-2	232.7	229.3	229.3	252	22.7
West Pier	FR-3	231.8	231.4	231.4	242	10.6
East Pier	FR-4A	234.0	230.7	234.0	240	6.0
East Abutment	FR-5	241.6	229.5	238.6	251	12.4

We recommend increasing the level of the pile cap at the east pier to increase the pile length. For an underside of pile cap at Elevation 242 m, the piles would be 8 m in length. In this case, filling around the pile cap to achieve the frost capacity would be required.

The proposed west abutment is located about 20 m west of the existing west abutment. The existing west abutment is founded on spread footings at about Elevation 247 m. Therefore, the underside of pile cap for the new west abutment will be about 5 m higher than the existing footing. The proposed east abutment is located roughly adjacent to the existing east abutment which is founded at about Elevation 246.3 m, resulting in the new underside of pile cap being about 4.7 m higher. This separation distance will be beneficial in regards to excavation (discussed in Section 6.10.1) and with respect to the impact of the new piling operations on the existing bridge, although we recommend monitoring of the existing bridge during construction (discussed in Section 6.10.5).

6.3.1 Geotechnical Axial Resistance

For HP310X110 piles at the west abutment and pier, driven to practical refusal on the metasediment bedrock which contains silt seams, a factored axial resistance at ULS of 2,000 kN may be used. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type. It is possible that some west abutment piles could “hang up” in the cobble and boulder deposit encountered within 3.4 m of the bedrock surface.

At the east abutment and pier, a factored axial geotechnical resistance at ULS of 1,600 kN and an axial geotechnical resistance at SLS of 1,400 may be used assuming minimum tip elevations given above.



6.3.2 Downdrag

At the abutments, soft to stiff silty clay to silt fill and native soft to stiff silty clay to clay deposits are present. Downdrag loading may be induced as a result of the addition of approach embankment fill after pile installation is complete and the resulting settlement of the cohesive layers relative to the piles.

At the west abutment, between 3.2 m and 10.2 m of new embankment fill will be placed above the existing ground surface at the abutment location. Settlement of the cohesive fill and native deposits relative to the stiff pile will result in the development of downdrag (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. In this case, downdrag loads will need to be taken into account for design of the piles supporting the west abutment. Downdrag loads are anticipated as a result of the settlement of the cohesive fill deposit and the native silty clay to clay deposit relative to the piles, specifically on the north side, where the load is highest. At the west abutment, the structural design of the abutment pile should be based on an estimated unfactored downdrag load acting on the HP310X110 of 100 kN per pile.

Downdrag loads are not anticipated at the east abutment due to the low height of additional fill, less than 2.4 m. Downdrag is not a consideration at the piers since minimal or no fill will be added, and since the silty clay to clay deposit was not encountered at the pier locations.

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 Set Criteria

For piles to be 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. A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap a minimum of 10% piles to confirm the set after adjacent piles have been driven.

All pile installation/driving should be in accordance with the latest Special Provision, SP903S01. The piles should be provided with flange reinforcement (i.e. driving shoes) to assist in penetration through the deposits containing cobbles and boulders and avoid damage to the pile tip. Driving shoes shall be in accordance with OPSD 3000.100.

6.3.4 Pile Driving Note

The pile driving note to be added to the drawings for the west abutment and pier piles is Note 4 in Clause 2.5.11 of the Structural Manual:

“Piles to be driven to bedrock”.

For the east abutment and pier piles, the pile driving note is Note 2 which shall read:

“Piles to be driven in accordance with Standard SS103-11 using an ultimate capacity of 4,000 kN per pile but must be driven below Elevation 234.0 m at the east pier and 238.6 m at the east abutment.



6.3.5 Resistance to Lateral Loads

Lateral loads can be resisted fully or partially by the use of battered steel H-piles. If vertical piles only are used, such as in integral abutments, 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 resistance to lateral loading in front of a vertical pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the equation for cohesionless soils given below.

$$k_h = \frac{n_h z}{B}$$

where: n_h = the constant of horizontal subgrade reaction (kPa/m)
 z = the depth (m)
 B = the pile diameter or width (m)

and for cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

where: s_u = the undrained shear strength of the soil (kPa)
 B = the pile diameter or width (m)

At both 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 in Table 3. The values reflect the variability in the subsurface conditions below the abutments. For a single HP310X110 pile at the abutments, the estimated maximum lateral resistance at ULS is about 115 kN and at SLS, for 10 mm of deflection, is about 30 kN (for a steel yield strength of 300 MPa).

Based on the above discussion, it is considered that both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting abutments (CHBDC Commentary C6.8.7.1).

The upper zone of soil (down to a depth below the pile cap equal to about $1.5 \times B$ after Broms 1964, where B = pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2), 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 spacing in between those listed above.

6.3.6 Frost Protection

The pile caps should be provided with a minimum of 2.5 m of conventional soil cover for frost protection (per OPSD 3090.101). If the required soil cover cannot be provided, consideration could be given to the use of rigid polystyrene foam insulation below the footings. As a guideline, 25 mm of rigid polystyrene insulation is assumed to be equivalent to about 300 mm of soil cover.

6.4 Caissons

Consideration could be given to the use of caissons for support of the abutments and piers although caissons are not considered to be economical at this site. The high axial capacity of the caissons would result in fewer units being required to support the abutments and piers than that required for the H-piles. It should be noted, however, that there may be difficulty in socketting the large diameter caissons within the strong to very strong metasediment bedrock and achieving an adequate seal, since silt layers were also encountered within the rock. Temporary liners through the overburden and tremie concrete will likely be required to install caissons at this site. As well, the sand and gravel deposit containing cobbles and boulders may also cause difficulties in caisson installation.

6.4.1 Geotechnical Axial Resistance

If caissons are considered as a founding alternative, the caissons at this site will derive their axial resistance mainly from the shaft resistance of the rock socket. The contribution from end-bearing will be neglected due to the difficulties in cleaning and inspecting the base of the sockets. The bedrock surface is defined in Section 6.3 and the factored geotechnical axial resistance at ULS for two different caisson diameters socketted a minimum of 2 m into the bedrock are given below.

Caisson Diameter(m)	Metasediment Bedrock (minimum 2 m socket)	
	ULS (kN)	SLS for 25 mm
1.0	5,000	n/a
1.5	8,000	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.

It should be noted that blow-up of the base of the caisson could occur during installation through the overburden below the groundwater level and a sufficient head of water should be maintained at all times to balance the hydrostatic pressures.

6.4.2 Downdrag

As discussed in Section 6.3.2, the loading from the new fill at the west approach embankment will cause settlement of the cohesive fill which will result in the development of downdrag 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 west abutment. The estimated unfactored downdrag load acting on the west abutment caissons for this case (assuming the underside of the caisson cap is at Elevation 224.9 m) may be taken as follows.

Caisson Diameter (m)	Unfactored Downdrag Load (kN)
1.0	200
1.5	300

The downdrag loads noted above 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.

Downdrag loads at the east abutment and piers can be neglected.

6.4.3 Resistance to Lateral Loads

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.4.5 and Table 3, using the horizontal subgrade reaction formulas. Maximum lateral resistances for the caissons can be provided if caissons are being considered for support of the bridge.

6.4.4 Frost Protection

The pile caps for the caissons at the abutments should be provided with a minimum of 2.5 m of conventional soil cover for frost protection or sufficient insulation as described in Section 6.3.6.

6.5 Site Coefficient

For seismic design purposes, the Site Coefficient, *S*, for this site, in accordance with Section 4.4.6 of the *CHBDC* may be taken as 1.5, consistent with Soil Profile Type III.



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, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

6.6.1 Static

The following recommendations are made concerning the design of the walls. It should be noted that 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 Ontario Provincial Standard Specifications (OPSS) 1010 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. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 percent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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 Ontario Provincial Standard Drawings (OPSD) 3101.150 and 3121.150.
- 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 3101.200.
- 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. Compaction equipment should be used in accordance with OPSS 501.06 or SP 105S10. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 2.5 m behind the back of the wall stem (as outlined on Figure C6.20(a), Case I, of the Commentary to the *CHBDC*) or within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the footing/pile cap (as outlined in Figure C6.20(b), Case II, of the Commentary to the *CHBDC*).
- For Case I, 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 earth fill or rock fill:



	Earth Fill	Rock Fill
Soil unit weight:	21 kN/m ³	19 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.31	0.22
At rest, K_o	0.47	0.36

- For Case II, 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:

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
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, may be taken as the following (in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the *CHDBC*):

- rotation (i.e. ratio of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
- horizontal translation of 0.001 times the height of the wall; or
- a combination of both.

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.

6.6.2 Dynamic

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 *CHDBC*. In this regard, the following should be taken into account in the lateral earth pressures.

- Seismic loading may result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.7 of the *CHBDC*, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for the Cochrane 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, $S=1.5$), resulting in an increase in the ground surface acceleration from 0.05 g to 0.075 g (PHA).



We understand that this highway route/bridge is not designated as a lifeline bridge. As such, based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned to Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, structures located in Seismic Performance Zone 1 need not be analysed for seismic loads.

6.7 Approach Embankment Design and Construction

The new centreline of Highway 11 will be located about 18 m north of the centreline of the existing Highway 11 alignment in the area of the Frederick House River structure. The existing approaches were built out into the river valley and up to 4.6 m and 4.0 m of fill was encountered above the native surface at the west and east abutments, respectively. The existing ground slopes away from the existing abutments to the north and towards the river channel at the west approach. The final grade of the highway will be Elevation 259.2 m and 258.4 m at the west and east approaches, respectively. Therefore, the new embankments will be up to about 10.2 m above the existing ground surface at the north edge of the new bridge at the west abutment. At the east approach, the existing ground slopes upward towards the north and downwards toward the river. The height of new fill at the east abutment will be up to about 2.4 m. The river banks are quite steep, up to about 2 m above the water level, as they have probably been scoured during periods of high water.

The soils encountered below the proposed approach embankments consisted primarily of silty clay to granular fill underlain by native silty clay, silt to sand and silt and sand and gravel deposits overlying bedrock. Based on the results of our subsurface investigation, consolidation of the silty clay deposit has occurred in the past due to prior fill placement.

At all areas, the analyses assume that prior to construction of the new embankments, all surficial organic soils will be removed below the new embankment footprint. We do not recommend disturbance of the existing embankments under and to the south of the existing bridge. Further, given the depth of the existing silty clay fill and the potential depth of excavation adjacent to the existing bridge, we do not recommend removal of the existing fill except where required for pile cap construction.

The piezometric conditions were assessed based on the water levels measured in Frederick House River (by others), on the groundwater levels noted during drilling of the boreholes and the water levels measured in the piezometers. The river water level was measured (by others) at Elevation 242.3 m in November 2007 and the high water level was measured (by others) at Elevation 244.5 m in September 1994. The water level in the piezometers installed at the west and east approaches in Boreholes FR-1 and FR-6, respectively, were measured at Elevation 244.2 m and 243.6 m on April 28, 2009, approximately 1 week after installation and at 242.9 m and 245.6 m on July 31, 2009. For design purposes for our analysis, the groundwater level has been assumed to be the November 2007 water level, Elevation 242.3 m, at the river rising away from the river to about Elevation 244 m at the abutments. The analyses were also run using the high water level where appropriate.

For the purpose of analysis, granular fill has been considered for the construction of the approach embankments and the berms as indicated below using side slopes at 2H:1V. Given that earth fill was used for the original construction, rock fill has not been considered in the analysis.

The following sections present the results of stability and settlement analysis for the new approach embankments including recommendations for settlement and stability mitigation measures, as required. The proximity of the existing bridge and approach embankments has been considered as the bridge will still be used for traffic flow during construction of the new bridge.



6.7.1 Stability

Analyses were performed on the critical sections of the proposed approach embankments to assess the stability and liquefaction potential for the proposed embankment height, existing geometry and soil stratigraphy. The critical embankment sections at this site were located at the proposed abutments and include both the front slopes (towards the river), the side slopes of the new approaches and other critical sections as shown on Figure 2. The geometry of the proposed approach embankments, existing ground surface and existing riverbed included in the analyses is based on the contour information from the GA drawing provided by LEA. The analysis also considers the proximity of the existing bridge embankments.

6.7.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.13), 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 analysed in the design. Block and wedge surfaces were also analyzed in some cases.

The incorporation of a 2 m wide mid-height bench (or berm) into the uniform side slope profile is required where the embankment will exceed a height of 8 m (OPSD 202.010). Given that the north side of the west approach embankment is greater than 8 m, this bench must be incorporated. Where berms are used, the embankment slopes were analyzed at 2H:1V above and below the berm.

6.7.1.2 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 US Navy (1971) 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. The existing cohesive fill was modelled assuming drained conditions, given that this fill is above the water level and will not become fully saturated.

Where appropriate, Bjerrum's correction factor (1973) 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.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types including EPS fill in the proposed approach/abutment areas. The slope stability analyses model geometry and stratigraphy are shown on Figures 3 to 11 for the critical sections indicated on Figure 2. For the silty clay to clay at the west approach, the parameters used in the analysis are shown graphically on Figure 1.



Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Angle of Internal Friction
New Granular* Fill	21	--	35°
EPS Fill	0.5	15	--
Existing Sand to Gravelly Sand Fill	20	--	30°
Existing Silty Clay to Silt Fill	17	--	25°
Alluvium	17.5	--	27°
Silty Clay to Clay West Approach	17.5	See Figure 1	--
Silty Clay to Clay East Approach	17.5	40	27°
Silt to Sand and Silt	18	--	27°
Sand to Sand and Gravel	19	--	32°

*Granular B Type I or II.

6.7.1.3 Results of Analysis

The results of the stability analyses are summarized below for the critical approach embankments slope sections. The minimum Factor of Safety is based on a deep-seated, global trial failure surface that would impact the operation of the roadway. Where the factor of Safety is greater than 1.3, the relevant figure is referenced.

Location	Section on Figure 2	Station	Relevant Boreholes	Maximum New Fill Height	Factor of Safety
West Approach	A	14+338 (W. Abut)	FR-2, FR-9 BH-4, BH-5 (S&P 2006)	3.2 m south side 10.2 m north side	1.23 (mitigation required)
	B	Profile	FR-1, FR-2, FR-3	10.2 m	1.15 (mitigation required)
	C	Profile	FR-1, FR-2, FR-3	3.2 m	<1.0 at over-steepened toe 1.62 deep seated (Figure 3) (overall slope mitigation not required, only at river bank)
	G, H	14+280 (G) 14+300 (H)	FR-7, FR-8	1.2 m south side 7.2 m north side	<1.0 (mitigation required)
	I	14+320	FR-1, FR-10	2.7 m south side 9.7 m north side	1.25 (mitigation required)
East Approach	D	Profile	FR-4, FR-6 BH-2 (S&P 2006)	2.4 m	<1.0 through existing fill (mitigation required)
	E	Profile	FR-4, FR-5, FR-6	2.4 m	<1.0 through the lower fill 1.61 deep seated (Figure 4) (overall slope mitigation not required, only at river bank)
	F	14+498 (E. Abut)	FR-5BH-2 (S&P 2006)	2.4 m	>1.48 (Figure 5) (mitigation not required)



The minimum realistic Factors of Safety for each section are presented above. The results of the analyses typically also give shallow surficial slip surfaces with FoS less than stated above; however, these surfaces are not realistic or representative of true failure conditions. Selection of the minimum Factor of Safety involves engineering judgement on the results generated by the computer program and selection of a realistic failure surface that would impact the operation of the roadway.

Limit equilibrium analysis indicates a Factor of Safety of less than 1.3 for most of the critical embankment sections for the combination of embankment geometry and height, water level and stratigraphy. Specifically, the east approach embankment (Figure 5) is stable without mitigation and in general, the front slopes in front of the abutments towards the river are stable (Figures 3 and 4), with minor regrading required at the toe of the slope. Elsewhere and in particular at the west approach, where the Factor of Safety is less than 1.3, stability mitigation measures will be required as discussed in detail in Section 6.7.4.

6.7.2 Liquefaction Potential and Seismic Analysis

As noted in Section 6.6.2, this site is located in Seismic Zone 1 with a PHA<0.08. Further, the bridge structure is not a lifeline structure. As such, based on Section 4.4.4 of the *CHBDC*, the site is assigned a Seismic Performance of 1 and, therefore, in accordance with Section 4.4.5.1 of the *CHBDC*, liquefaction analysis is not required.

6.7.3 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the new fills on the existing fill and compressible foundation soils at this site. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the embankment fill itself.

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 hand and spreadsheet calculations. The model geometry and stratigraphy is shown on Figures 3 to 11.

Based on the proposed embankment heights and the undrained strength of the silty clay deposit, time-dependent consolidation settlement of the silty clay to clay deposit is expected at the west approach. At the east approach, the embankment heights are much lower and the settlement is expected to be elastic, occurring rapidly during construction. The rate of settlement of the cohesive soils at the west approach was assessed using Terzaghi's one-dimensional consolidation theory.

The silty clay to silt fill is variable across the site. For design, it is assumed that half the thickness of the deposit will be analyzed using consolidation settlement theory using parameters similar to that of the native silty clay to clay. The other half of the deposit has been analyzed using elastic theory.

6.7.3.2 Settlement Criteria

Based on information obtained from MTO Foundations, the following settlement criteria are considered acceptable to occur within 10 years post-paving for the approach embankments of bridges.



Location	Settlement Tolerance
Abutment to 25 m behind abutment	10 mm to 25 mm
25 m to 75 m behind abutment	25 mm to 75 mm
75 m to 150 m behind abutment	75 mm to 150 mm

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

6.7.3.3 Parameter Selection

The immediate compression of the fill and cohesionless soils were 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 silty clay deposit was assessed using the results of the in situ field vane and SPT tests and the laboratory consolidation test to estimate the deformation parameters for these soils. 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 over-consolidation 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)
 σ_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)
 μ = Bjerrum's correction factor based on Plasticity Index

It is known that some 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 following equations for secondary (creep) settlement from Holtz and Kovacs (1981) were employed in the analysis:

$$S_c = C_{\alpha\epsilon} \times L_o$$

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

where: S_c = secondary (creep) settlement (mm)
 $C_{\alpha\epsilon}$ = modified secondary compression index (%)
 L_o = initial thickness of compressible deposit (mm)



FOUNDATION REPORT - FREDERICK HOUSE BRIDGE REPLACEMENT HIGHWAY 11, GWP 5541-05-00

The following simplified stratigraphy, unit weights and deformation parameters have been employed in the settlement analysis of the proposed approach embankments. The thickness and elevation of the materials vary greatly across the site and the relevant boreholes have been used in the settlement estimates.

Deposit	Unit Weight (kN/m ³)	Deformation Properties
New Granular B Fill	21	See Section 6.7.3.3
Existing Sand to Gravelly Sand Fill	20	E' = 25 MPa
Existing Silty Clay to Silt Fill	17	E' = 5 MPa
Silty Clay to Clay	17.5	(see below)
Silt to Sand and Silt	18	E' = 5 MPa
Sand to Sand and Gravel	19	E' = 15 MPa

The following consolidation parameters were estimated for the clay deposit based on the results of laboratory consolidation tests performed on specimens of the clay obtained from Boreholes FR-5 to FR-7 (as shown below) and compared with values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described previously.

Location	Borehole/Sam ple No.	Elevation (m)	σ_{v0}' (kPa)	σ_p' (kPa)	OCR	e_0	C_r	C_c	c_v (cm ² /s)
East Abutment	FR-5/9	247.1	133	200	1.5	0.707	0.030	0.121	0.0040
	FR-6/8	251.1	122	240	2.0	1.071	0.084	0.473	0.0038
West Abutment	FR-7/8	245.7	109	230	2.1	1.150	0.089	0.360	0.0061

Due to the varved nature of the clay, the range of c_v from the consolidation test data and the correlations of c_v to other laboratory test data for normally consolidated soil (US Navy, 1971), a range of c_v and drainage path lengths have been used to calculate the time rate of primary settlement. Based on the observations and measurements from the thickness of the clay and clayey silt/silt varves in the Shelby tube samples, the clayey silt/silt comprises about one-third to half of the total thickness of the deposit, while the clay comprises about two-thirds to half of the total thickness. While it is expected that the silt varves will provide for short drainage paths for dissipation of pore water from the clay varves, it has been conservatively assumed in the analyses that two-way drainage of the deposit thickness will occur with an average c_v for the clay of 4.6×10^{-3} cm²/s.

The modified secondary consolidation index, $C_{\alpha\epsilon}$, used in the analysis to calculate creep is 0.42 percent.

The maximum estimated settlement of the fill and foundation soils in these areas (due to the loading imposed by the new embankment fill) is presented below and a discussion on the rate of settlement is included.



6.7.3.4 Settlement of Embankment Fill

Existing Fill

In order to limit excavation depths adjacent to the existing approach embankments, the majority of the existing fill will be left in place at this site. Settlement of the existing fill will occur under the new embankment loading. The sand to sand and silt fill will occur rapidly and is expected to be less than 25 mm.

The silty clay to silt fill will also settle under the new embankment loading but will be time dependent. Given the variable nature of the fill, it is not possible to accurately estimate the magnitude and time rate of settlement. In general, it is considered that settlement of the fill, both cohesive and cohesionless will generally occur during construction of the embankment.

The estimated total and post-construction settlement for the fill are presented in Table 4. The values given in Table 4 take into account that half of the silty clay fill was analyzed using elastic theory and half was analyzed using consolidation theory.

New Granular Fill

We recommend the use of granular fill for the new embankment construction at this site. Granular fill could consist of Granular B Type I or II or Select Subgrade Material (SSM). In this regard, the additional settlement from the properly compacted granular fills is expected to be less than about 25 mm and will occur during construction.

6.7.3.5 Settlement of Foundation Soils

Based on the embankment geometry at the critical sections indicated above, settlement of the subsoils should be anticipated. The estimated initial settlement of the cohesionless soils and the time-dependent primary consolidation and secondary (creep) settlement under the existing and new embankments is summarized below and in Table 4. Settlement of the subsoils at each of the approach embankments are discussed separately below.

6.7.3.6 West Approach

Along the south side of the approach embankment, the silty clay to clay is thicker but the new fill height is lower. Along the north side, the clay is thinner but the new fill is much higher. The height of fill and thickness of silty clay at the north side of the embankment in the area of Borehole FR-7 (STA 14+280) will increase the effective stress beyond the preconsolidation pressure resulting in large, time-dependent settlements. All along the south side of the embankment (between STA 14+260 and STA 14+338) and the north side up to about 20 m behind the abutment, the effective stress will not go beyond the preconsolidation pressure and therefore the settlement will occur rapidly during construction.

Settlement of the west approach embankment will occur differentially between the north and south sides of the embankment with settlement being greatest in magnitude (as well as time-dependent) on the north side. Further, differential settlement will also occur from east to west, being greatest in magnitude (and time-dependent) near STA 14+280 and decreasing easterly.

As presented in Table 4, the post-embankment construction settlement along the south side of the embankment is less than 25 mm, which meets the criteria given in Section 6.7.3.2. At the north side of the embankment at STA 14+280 and STA 14+300, the settlement is greater than 25 mm and therefore mitigation is required in these areas. Further, at the north side of the embankment at the west abutment, the settlement is also greater than 25 mm, although a greater portion of the magnitude is due to the compression of the existing cohesive fill.



Where settlement is time dependent (between about STA 14+280 and STA 14+300), it is estimated that about 95 percent of the consolidation settlement will be completed in about 4 to 8 months.

The magnitude of creep settlement for the cohesive strata at this site is expected to be about 25 mm in 10 years after completion of primary consolidation west of about STA 14+310.

Based on the results in Table 4 and the discussion above, mitigation of settlement (vertical and differential) will be required at the west approach and alternatives to mitigate this settlement are discussed in detail in Section 6.7.4.

6.7.3.7 East Approach

The magnitude of settlement of the subsoils at the east approach is given in Table 4. Although some of the existing fill requires removal to construct the pile cap, the net effective embankment height will be less than about 2.5 m. For the low height of new fill at the east approach, settlement is expected to occur rapidly during construction. Given that the existing ground surface is relatively level in the area of the approach, differential settlement is not a concern at the east approach.

Given that the magnitude of settlement is less than 25 mm, mitigation of settlement at the east approach is not required.

6.7.4 Mitigation of Stability/Settlement Issues

Based on the presence of the silty clay fill and native silty clay deposit under the proposed west approach embankment footprint and the variable height of the new embankments (due to the sloping ground surface), 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 potential magnitude of post-construction settlement of the new and existing embankments. The results of the analysis are discussed in Sections 6.7.1 and 6.7.3.

At the east abutment, the new fill height is minimal and minor mitigation measures are required.

The advantages, disadvantages, relative costs and risks/consequences for stability and settlement mitigation alternatives at this site are summarized and ranked in Tables 5a and 5b for the west and east approaches, respectively. At the west approach, we recommend the use of mid-height stability berms, slope flattening and/or lightweight (EPS) fill to mitigate stability. Preloading, and in some areas the use of lightweight (EPS) fill, will be required to mitigate settlement at the west approach. At the east approach, we recommend slope flattening to achieve the target factor of safety for stability; mitigation of settlement is not required.

6.7.4.1 Slope Flattening/Berms

The mitigation measures described below should be used in conjunction with the settlement mitigation requirements detailed below.

West Approach

At the west approach, 2H:1V side slopes are not stable at this site given the subsurface conditions and site geometry. We recommend the use of mid-height berms on the north side slope where the embankment is highest and where sufficient property is available. Although OPSD 202.010 indicates that the berm should be placed 8 m below the final grade, we recommend constructing the berm at Elevation 254 m to achieve consistency along the length of the west approach. Berms 4 m and 2 m wide at Sections A-A and I-I will result in a Factor of Safety of greater than 1.3 as shown on Figures 6 and 7.



At Sections G-G and H-H, the size of berm is limited by the property requirements. The edge of the slope/berm can be no more than 27 m from the centreline at Section G-G and 38 m at Section H-H. Therefore, at Section H-H, an 8.6 m berm is required at Elevation 254 m and no berm is possible at Section G-G. In both cases, the Factor of Safety is still below the target of 1.3. Therefore, additional mitigation measures, such as lightweight fill, are required to mitigate stability and are discussed in Section 6.7.4.2 below.

The existing ground surface in the location of the proposed front slope of the west abutment is highly variable between the north and south sides of the new bridge. Sections B-B and C-C were analyzed through the north and south front slopes, respectively. The front slope should be constructed at no steeper than 3H:1V down towards the river on the north side. On the south side, the existing slope angle is currently at about 5H:1V, with an approximately 4 m high bank at the river. This “over-steepened” toe results in localized failure surfaces less than 1.3, while the overall slope has a factor of safety greater than 1.3. It appears that the vegetation and/or old cribbing at the bank is aiding stability. We recommend flattening the entire slope at 3H:1V or flatter (as at the north side) or providing some stabilizing measures at the toe, if flattening is not possible. The results of the analyses for the north and south sides are shown on Figure 8 (Section B-B with slope flattening) and Figure 3 (Section C-C with mitigation not required), respectively.

In general, the 4 m berm at the abutment (STA 14+338) should increase to 8.6 m at STA 14+300 and then, for property reasons, taper to 0 m at STA 14+280. The berm should also taper towards the 3H:1V front slope accordingly.

East Approach

At the east approach, the critical stability section through Section D-D requires flattening of the slope such that it is maintained at about 4H:1V starting from the river's edge (at low water level) up to the front face of the east abutment. At section E-E, which is the profile through the alignment centreline and front slope, the existing slope with the minimal filling at the crest is generally stable; however, some surficial failures through the fill at the toe of the slope are possible as presented in Section 6.7.1.3. We recommend flattening the entire slope at about 4H:1V or flatter (as at Section D-D) or providing some stabilizing measures at the toe, if flattening is not possible. The results of the stability for Sections D-D and E-E are shown on Figure 9 (with slope flattening) and Figure 4 (with mitigation not required), respectively, for the high water level case. Stability mitigation measures are not required for the east approach cross-section.

6.7.4.2 Lightweight (EPS) Fill

As discussed above, due to property restrictions, it is not feasible to achieve the size of berms required for stability in the area of Sections G-G and H-H at the west approach. In order to achieve a Factor of Safety of greater than 1.3, a minimum of 3.9 m and 2.6 m of EPS is required at Sections G-G (STA 14+280) and H-H (STA 14+300), respectively. The results of the analysis showing the required berms and EPS fill are shown on Figures 10 and 11.

The EPS should extend from STA 14+255 to 14+315. EPS comes in blocks typically 0.5 or 1 m in depth. Therefore, the EPS thickness should be 4 m thick between STA 14+275 and 14+285, stepping to 2.5 m between STA 14+275 and 14+265 and to 1.0 m between STA 14+265 and 14+255. The EPS should step up in 1 m increments for every 10 m of distance east of STA 14+285. The EPS limits along the north side of the new embankment should taper at 2H:1V and be provided with about 2 m of soil cover. On the south side, the EPS should similarly tie into the existing embankment. Appropriate ties and spacers should be used to keep the EPS block mass acting as a single unit. The top of the EPS blocks should be provided with a minimum of 1 m of soil cover, plus a 125 mm thick concrete slab, for ballast and for protection against differential icing. Buoyancy of the EPS blocks is not an issue at this site.



EPS is not required at Section A-A (at the abutment) or I-I (STA 14+320) for stability. However, if preloading to the final grade for the specified 4 months is not possible, then EPS may also be required for settlement mitigation.

A volume of about 1,700 m³ would be required in this case. The cost of EPS is substantially higher compared to other alternatives, however, given the property constraints at this site, may be the most practical solution. An NSSP for the supply and installation of EPS fill should be included in the Contract Documents; an example is provided in Appendix C. The EPS requirements are detailed on Figure 12. Note, the granular levelling pad below the EPS (as noted in the NSSP) should extend below all the blocks, including as they step up towards the east and west as well as on the south side where the block tie in to the existing embankment.

6.7.4.3 *Preloading*

We recommend the use of preloading to reduce the magnitude of post-construction settlement (including differential settlement) at the west approach. In order to reduce post-construction settlement to less than 25 mm, preloading is required for at least 4 months to the maximum height that stability permits. The magnitude of settlement without mitigation, after pre-loading (i.e. post-construction), and in consideration of the EPS required for stability, is given in Table 4 and shown on Figures 13 and 14 for the west approach at STA 14+280 and 14+300.

The maximum preload height is Elevation 256 m at STA 14+280 increasing to Elevation 257.5 m at STA 14+300 and to Elevation 259.2 m at STA 14+320 and easterly to the abutment.

Although preloading is not required in the abutment area since it is anticipated that settlement will generally occur during construction, we recommend preloading to the final grade in the abutment area to reduce uncertainties of differential settlement due to the presence of existing cohesive fill left in place.

Surcharging is not feasible for reasons of stability without an increase of the stability berms discussed in Section 6.7.4.1 above.

Monitoring of the west approach embankment will be required and will determine whether sufficient settlement has occurred to achieve the post-construction settlement criteria. Further discussion of monitoring is given in Section 6.10.5.

6.7.4.4 *Grading*

We understand that the grades at the new bridge location were increased in order to correct a vertical highway curve. This resulted in a grade raise (over that of the existing bridge) of 1.1 m and 0.8 m at the west and east abutments, respectively. If the grade was lowered at the west side (and therefore at the east side), the requirement for toe berms at the west approach may decrease as well as the thickness of the EPS fill, and corresponding property requirements. Note that lowering the grade may also result in the need for deeper excavations, which will require more extensive shoring.

6.7.4.5 *Sub-excavation*

Sub-excavation of the cohesive fill and/or native silty clay to clay at this site is too deep to be carried out in open cut. Extensive shoring would be required adjacent to the existing roadway beyond what is already required for abutment construction. Therefore, sub-excavation is not recommended as an alternative to mitigate settlement or stability.



6.7.4.6 *Staged Construction*

Given the variable embankment height (due to the sloping ground surface) across the west approach area, staged construction is not considered practical. Since the pre-consolidation pressures are only exceeded on the north side of the new embankment, the strength gain from staged preloading will be minimal. Further, essentially no strength gain will occur at the toe of the slope, which typically controls stability. Therefore, there would be little or no stability improvement or reduction of post-construction settlement by the use of staged construction.

6.7.4.7 *Wick Drains/Ground Improvement*

The use of wick drains may not enhance the rate of settlement beyond that discussed above since the silty clay to clay soil under the embankment footprint is naturally varved and in many areas of limited thickness. Further, extensive pre-augering would be required to reach the clay deposit through the existing fill material. Other ground improvement techniques would similarly involve extensive excavation and cost for mobilization of equipment and may not significantly reduce the magnitude or time required for settlement.

6.8 **Culvert Extension STA 14+285 LT**

The existing culvert consists of a 1.2 m wide by 1.2 m high rigid frame open (RFO) footing structure. The existing culvert is proposed to be extended by about 15 m to accommodate the alignment for the new Frederick House River Bridge.

The invert of the culvert is sloping towards the north, where the culvert currently outlets mid-slope at about Elevation 252 m. The invert of the extension ranges from about Elevation 252 m at the connection to 250.8 m at the new outlet.

At the extension location, the new highway embankment will result in the addition of about 7.2 m of fill. Settlement of the founding soils below the culvert will take place as discussed in detail in Section 6.7.3.5 and the preferred mitigation alternative in this area is the use of stabilizing berms and EPS fill. This would increase the length of the culvert extension.

Since the invert of the culvert extension will be above the groundwater level at this site, groundwater control will not be required. However, surface water should be directed away from the excavations during construction. Further information regarding construction considerations are provided in Section 6.10.

Either a box culvert or an "open footing" (shallow foundation) culvert is feasible for the replacement of the existing RFO culvert. Deep foundations are not considered practical for foundation of the culvert at this site. We recommend extension of the culvert with a box culvert.

Recommendations for both a shallow foundation (open footing) culvert extension and a box culvert extension are provided in the following sections.

6.8.1 **Open Footing Culvert Extension Replacement**

An open footing culvert extension, and any associated retaining walls, can be supported on strip footings founded below the topsoil and existing fill on a compacted Granular 'A' pad placed over the native soft to firm silty clay to clay. The granular pad should be a minimum of 1 m thick, and the total soil cover should be a minimum of 2.5 m, to provide adequate protection against frost penetration. The minimum elevation for the base of the pad should be at Elevation 249.8 at the outlet, rising to 250.6 m at the connection.



Strip footings placed on the properly prepared Granular 'A' pad, at or below the elevation identified above, should be designed based on the following factored geotechnical resistances at ULS and geotechnical resistances at SLS.

Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
0.6 m	150 kPa	100 kPa
0.9 m	225 kPa	150 kPa

* For 25 mm of total settlement for the given footing width.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs significantly from those given above.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.2 of the *CHBDC*.

Resistance to lateral forces/sliding resistance between the concrete footings for the culvert extension replacement and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The following values for the coefficient of friction, $\tan \phi'$ or $\tan \delta$, can be used for cast-in-place and pre-cast concrete footings founded on the minimum 1 m thick Granular 'A' pad placed over the native silty clay to clay deposit:

Footing Type	Coefficient of Friction
Cast-in-place concrete footing on compacted granular pad	$\tan \delta = 0.60$
Pre-cast concrete footing on compacted granular pad	$\tan \delta = 0.50$

The above values are unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.8.2 Box Culvert Extension

Given the presence of soft to firm silty clay to clay within the zone of influence of the culvert foundation, we recommend founding the culvert on a 1 m thick, compacted Granular 'A' pad placed over the native material. The base of the pad should therefore be at Elevation 249.8 m at the outlet, rising to 250.6 m at the connection. All fill material should be removed. The pad should be placed and compacted in accordance with MTO's Special Provision SP105S10. This pad will also provide sufficient bedding for the culvert.

A box culvert extension placed on the properly prepared Granular 'A' pad, at or below the elevation identified above, should be designed based on a factored geotechnical resistance at ULS of 350 kPa and geotechnical resistance at SLS of 175 kPa for 25 mm of settlement and assuming a footing width of 1.2 m.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the culvert span or founding elevation differs significantly from those given above.



The geotechnical resistances provided above are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.2 of the *Canadian Highway Bridge Design Code (CHBDC)*.

Resistance to lateral forces/sliding resistance between the base slab for the culvert extension/replacement and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For design, the coefficient of friction ($\tan \delta$) between a pre-cast concrete box culvert extension/replacement and the granular pad should be taken as 0.5. These values are unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.8.3 Culvert Bedding, Backfill and Erosion Protection

For a box culvert extension, the bedding levelling pad and backfill requirements should be in accordance with OPSS 422 for pre-cast rigid frame culverts. Box culvert extensions should be provided with at least 150 mm of OPSS 1010 Granular A material for bedding purposes, which will form part of the recommended granular pad.

Backfill and cover for concrete culverts should be completed in accordance with OPSD 803.010. Backfill to both box culvert and open footing culvert walls should consist of granular fill meeting the requirements of OPSS 1010 Granular 'A' or Granular 'B' Type II, but with less than 5 percent passing the No. 200 sieve. The backfill should be placed and compacted in accordance with SP105S10. The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 500 mm. The culvert extension should be designed for the full overburden pressure and live load, assuming an embankment fill unit weight of 22 kN/m³ for Granular 'A', and 21 kN/m³ for Granular 'B' Type II or granular fill above and/or surrounding the culvert.

If the flow velocities are sufficiently high, provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) in the culvert extension area. The requirements for and design of erosion protection measures for the inlet of the culvert extension should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert extension outlet should be consistent with the standard presented in OPSD 810.010 Rip-Rap Treatment Type A.

6.8.4 Settlement and Culvert Connection Requirements

As discussed in Section 6.7.3.5, post-construction settlement of the native silty clay to clay will occur under the culvert extension and the magnitude will be variable along the length of the extension, being highest where the new fill is thickest, i.e. at the connection point. The grade of the culvert extension is such that downward gravity flow will still occur even when settlement is complete.

Since the culvert will be constructed before the embankment (and before preloading takes place), we recommend that the structural designer determine, based on this predicted magnitude of settlement and the actual change in embankment geometry and loading, that an articulation connection is required between the existing concrete open footing culvert and the culvert extension/replacement to maintain long-term serviceability of the culvert. It is estimated that approximately 100 mm of settlement will occur after culvert construction at the connection point, decreasing towards the end of the culvert extension.



6.9 Subgrade Preparation and Embankment Construction

Prior to embankment construction, all topsoil/vegetation/organic soils must be removed below the footprint of the proposed embankments. Although the existing fill is not considered to be an appropriate subgrade due to the cohesive nature of the fill, given the proximity of the existing bridge and the potential requirement for shoring, it is not practical to remove all existing fill. In general, the existing fill may be left in place but should be removed to at least 0.5 m below the new pile cap level at the abutment locations, where encountered at this level. All softened/loosened fill or native material should be stripped from below the new approach embankments, prior to placement of new fill.

Granular fill material specifications and placement should be carried out in accordance with the requirements as outlined in Special Provision SP206S03. We understand that LEA is proposing to construct the approach embankments with earth fill and, as such, all granular fill should be placed in regular lifts with loose thickness not exceeding 300 mm and compacted to at least 95 percent of the standard Proctor maximum dry density. Side slopes for granular fill embankments should be no steeper than 2H:1V and in accordance with Section 6.7.4 as appropriate to mitigate stability. The final lift of fill prior to placement of the roadway 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.

In order to minimize differential settlement between the existing embankment slopes and the newly placed embankment fill, as at the west approach, the new fill should be keyed into the existing embankment side slope per the requirements of OPSD 208.010.

The abutment front slopes and side slopes adjacent to the river require erosion protection. 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 rip rap (300 mm diameter), rock protection or concrete slope paving. The potential for scour below the pile caps should be taken into account in the design of the bridge foundations.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding should be carried out as soon as possible after construction where granular fill 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 to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil and seeding.

6.10 Design and Construction Considerations

6.10.1 Excavations

Excavations for construction of the pile caps and the abutments will extend to 0.5 m below the base of the proposed pile caps. The elevation of the base of the excavations, the depth below the adjacent existing roadway surface or ground surface and the depth below the measured water level are presented in Table 6.

The elevation of the underside of the pile cap is about 5 m above the underside of the existing spread footings at the west and east abutments. At both abutments, the excavations can be carried out in open cut except where temporary roadway protection will be required between the existing embankment and the excavation. This is most critical at the east abutment where the existing and new abutments are located almost side by side. The new west abutment is located about 20 m west of the existing west abutment.

At the piers, excavations may be able to be carried out in open cut. Depending on the proximity to the river and the Contractor's construction techniques, temporary shoring may be required.



Open cut slopes within the fill materials should be maintained at no steeper than 2H:1V through the material above and below the water level.

Conventional excavation equipment should be suitable for excavation through the on-site soils; however, the Contractor shall be made aware of the potential for obstructions as discussed in Section 6.10.4.

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 existing fill materials and the native soils should be classified as Type 3 soil, according to the OHSa.

6.10.2 Temporary Shoring

Temporary excavation support systems should be designed and constructed in accordance with Special Provision SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19.

Temporary roadway protection will be required between the existing bridge and the excavation for the new abutment pile caps and for pile installation and abutment construction. As discussed above and in Table 6, shoring is anticipated to be about 7.1 m and 7.5 m high at the west and east abutments, respectively.

At the piers, if there is insufficient room to construct the pile caps in an open cut excavation, then consideration should be given to the use of temporary shoring such as a sheet-pile cut-off wall or other cofferdam type construction.

The Contractor should be notified that the existing east abutment/approach was rehabilitated in 2006 using EPS fill. The Contractor should ensure that temporary works will not impact or be impacted by the presence of this fill. In this regard, an Operational Constraint (OC), should be included in the report; an example is included in Appendix C.

6.10.3 Groundwater and Surface Water Control

Although perched water was not encountered within the fill during our investigation, previous boreholes (Shaheen & Peaker, 2006) indicated the presence of perched water within the embankment fill. The Contractor should therefore anticipate perched water within the embankment fill during excavation work for the abutments.

The excavation for the piers will be located adjacent to Frederick House River. In order to construct the pile caps, groundwater inflow should be expected and controlled dewatering within an open cut excavation or temporary shoring will be required. Shoring, if required, should be advanced to an appropriate depth to control groundwater inflow from the river. The Contractor should be alerted that excavations for the pile caps and sub-excavation will be advanced through cohesive soils, and the base of the excavations will be in cohesionless soils. The cohesionless soils are expected to be unstable below the groundwater level at this site. This Contractor should be alerted to this in an OC, an example of which is included in Appendix C.

The Contractor is responsible to ensure that appropriate construction procedures and equipment are used for construction. Surface water should be directed away from the excavation at all times.

6.10.4 Obstructions

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



Cobbles and boulders were noted to be contained within the sand and gravel deposit overlying the bedrock at the east abutment and east pier locations. Further, a layer of cobbles and boulders was encountered directly overlying the bedrock at the west abutment. The existing fill material may also contain cobbles and boulders. In one borehole, an obstruction was encountered within the upper 3 m of embankment fill, thought to be associated with an existing pipe culvert.

This Contractor should be alerted to the presence of cobbles/boulders and other obstructions (including existing and/or older bridge foundation elements) in an OC, an example of which is included in Appendix C.

6.10.5 West Embankment Monitoring

Monitoring of settlement and lateral movement of the new and existing west approach embankments and the existing bridge itself should be carried out during construction of the new west approach embankment and during the preloading phase. This can be accomplished using a series of surface points established along the top of the embankment and bridge structure. The instruments should be installed as soon as possible after subgrade preparation and prior to embankment filling to be able to record as much of the settlement occurring under the new embankment as possible. Settlement rods (plates) under the embankment and settlement pins on the existing bridge should be installed prior to the start of construction. In addition to settlement monitoring points, vibrating wire piezometers and standpipe piezometers should also be installed 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.

The instrument locations, types and details should be included in a NSSP, to be developed once the final design is confirmed and under separate cover. The final documents related to monitoring should be prepared on the basis of concurrence with MTO with the monitoring program and included in the Contract Documents and Contract Administration assignment, as appropriate.

6.10.6 Existing Structure Monitoring

Given the age and condition of the existing structure abutments, the close proximity of the existing and proposed abutments and the requirement for the existing structure to remain in operation during construction of the new structure, it is recommended that the abutments of the existing structure be monitored for settlement and lateral movement while in operation during the new construction, especially during installation of temporary roadway protection, excavation for the new abutments and during pile driving (during seating of the pile on bedrock and/or advance through cobbles and boulders). This could be carried out using survey points (lateral and vertical deformation) and/or settlement points. We understand that LEA has an NSSP for this purpose and it should be included in the Contract Documents.

7.0 CLOSURE

This report was prepared by Ms. Kerry Salvatori Lee, P.Eng., and Ms. Sarah E. M. Coyne, P.Eng., an Associate with Golder Sudbury. The technical aspects were reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact, who also conducted a quality control review of the report.



Report Signature Page

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KSL/SEMC/FJH/lb

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FOUNDATION REPORT - FREDERICK HOUSE BRIDGE REPLACEMENT HIGHWAY 11, GWP 5541-05-00

Table 1: Evaluation of Foundation Alternatives – West Abutment and Pier

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles Driven to Bedrock	1	<ul style="list-style-type: none"> ■ Straightforward construction ■ Pile cap would be located above founding level of existing abutments 	<ul style="list-style-type: none"> ■ Possibility of piles “hanging up” on cobbles and boulders deposit ■ Temporary roadway protection required 	<ul style="list-style-type: none"> ■ Lower relative costs compared with caisson option 	<ul style="list-style-type: none"> ■ Risk of piles “hanging up” on cobbles and boulders.
Caissons Socketted into Bedrock	2	<ul style="list-style-type: none"> ■ Reduced number of deep elements compared to steel H-piles 	<ul style="list-style-type: none"> ■ Temporary liners would be required for groundwater control and support through overburden above and below water level ■ Difficulties may be encountered advancing caissons through cobbles and boulders ■ Concrete for caissons would have to be placed by tremie methods below the water level ■ May be difficult socketting caissons into metasediment bedrock ■ Temporary roadway protection required 	<ul style="list-style-type: none"> ■ Cost many times higher than for piles 	<ul style="list-style-type: none"> ■ Risk of difficulties encountering cobbles and boulders above the bedrock surface. ■ Risk of not achieving seal bedrock socket due to silt seams within bedrock
Spread Footings on Native Deposits	NF		<ul style="list-style-type: none"> ■ Native subgrade too deep to be practical ■ Extensive roadway protection required ■ Low axial resistance expected on this material 	<ul style="list-style-type: none"> ■ Major cost for roadway protection and for excavation 	<ul style="list-style-type: none"> ■ Risk of impacting existing bridge during construction



FOUNDATION REPORT - FREDERICK HOUSE BRIDGE REPLACEMENT HIGHWAY 11, GWP 5541-05-00

Table 2: Evaluation of Foundation Alternatives – East Abutment and Pier

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-Piles Driven to Refusal within the Sand and Gravel Deposit	1	<ul style="list-style-type: none"> ■ Straightforward construction ■ Shorter pile lengths ■ Eliminates possibility for pile (designed to seat in rock) “hanging up” on a boulder 	<ul style="list-style-type: none"> ■ May not achieve required capacity at the specified elevation and deeper/additional piles may be required 	<ul style="list-style-type: none"> ■ Lower cost than piles to bedrock 	<ul style="list-style-type: none"> ■ Risk of piles not achieving design capacity
Steel H-Piles Driven to Bedrock	2	<ul style="list-style-type: none"> ■ Straightforward construction 	<ul style="list-style-type: none"> ■ Possibility of piles “hanging up” on cobbles and boulders deposit before reaching the bedrock surface ■ Cofferdam construction may be required for pile cap construction adjacent to river 	<ul style="list-style-type: none"> ■ Lower relative costs compared with caisson option 	<ul style="list-style-type: none"> ■ Risk of piles “hanging up” on cobbles and boulders.
Caissons Socketted into Bedrock	3	<ul style="list-style-type: none"> ■ Reduced number of deep elements compared to steel H-piles ■ Possible elimination of pile cap 	<ul style="list-style-type: none"> ■ Temporary liners would be required for groundwater control and support through overburden ■ Difficulties may be encountered advancing caissons through cobbles and boulders ■ Concrete for caissons would have to be placed by tremie methods below the water level ■ May be difficult socketting caissons into metasediment bedrock ■ Cofferdam construction may be required for pile cap construction adjacent to river 	<ul style="list-style-type: none"> ■ Cost many times higher than for piles 	<ul style="list-style-type: none"> ■ Risk of difficulties encountering cobbles and boulders above the bedrock surface ■ Risk of not achieving seal bedrock socket due to silt seams within bedrock
Shallow Spread Footings on Native Deposits	NF	<ul style="list-style-type: none"> ■ Straightforward construction 	<ul style="list-style-type: none"> ■ Low geotechnical resistance available ■ Differential settlement between piers and abutments on piles anticipated 	<ul style="list-style-type: none"> ■ Lower relative cost to piles 	<ul style="list-style-type: none"> ■ Risk of differential settlement between abutment and pier foundation elements



**FOUNDATION REPORT - FREDERICK HOUSE BRIDGE REPLACEMENT
HIGHWAY 11, GWP 5541-05-00**

Table 3: Parameters for Horizontal Subgrade Reaction

Soil Unit	Location	Elevation (m)	n_h (kPa/m)	s_u (kPa)
Silty Clay to Silt (Fill)	West Abutment East Pier East Abutment	Above 243.4 Above 244.9 Above 251.0	2,200	--
Soft to Stiff Silty Clay to Clay/Alluvium	West Abutment West Pier East Pier East Abutment	243.4 – 242.6 Above 241.2 244.9 – 242.0 251.0 – 246.5	--	30
Very Loose to Dense Silt to Sand and Silt	West Abutment West Pier East Abutment	242.6 – 232.7 241.2 – 231.8 246.5 – 241.6	1,300	--
Compact to Very Dense Sand and Gravel (containing Cobbles and Boulders)	West Abutment West Pier East Pier East Abutment	232.7 – 229.3 231.8 – 231.4 242.0 – 230.7 241.6 – 229.5	4,400	--



FOUNDATION REPORT - FREDERICK HOUSE BRIDGE REPLACEMENT HIGHWAY 11, GWP 5541-05-00

Table 4: Results of Settlement Analyses

Deposit	West Approach STA 14+280		West Approach STA 14+300		West Abutment STA 14+338		East Approach
	North Side	South Side	North Side	South Side	North Side	South Side	
New Granular B Fill	25 (rapid)	10 (rapid)	25 (rapid)	10 (rapid)	25 (rapid)	10 (rapid)	10 (rapid)
Existing Granular Fill	n/a	n/a	n/a	n/a	n/a	n/a	10 (rapid)
Existing Cohesive Fill	<25	<25	<25	<25	100 (rapid)	75 (rapid)	25 (rapid)
Silty Clay to Clay – Primary Consolidation	275 (time-dependent)	25 (rapid)	275 (time-dependent)	25 (rapid)	60 (rapid)	50 (rapid)	50 (rapid)
Silty Clay to Clay – Secondary Consolidation (up to 10 years)	25 (long-term)	n/a	25 (long-term)	n/a	n/a	n/a	n/a
Cohesionless Deposits	225 (rapid)	30 (rapid)	225 (rapid)	30 (rapid)	325 (rapid)	90 (rapid)	50 (rapid)
Total Settlement	575 (time-dependent)	90 (rapid)	575 (time-dependent)	90 (rapid)	510 (rapid)	225 (rapid)	145 (rapid)
Total Post-Embankment Construction Settlement (without mitigation)	300 (mitigation required)	<25	300 (mitigation required)	<25	<25	<25	<25
Total Post-Embankment Construction Settlement (with EPS but without preloading)	125 (Figure 13)	n/a	150 (Figure 14)	n/a	n/a	n/a	n/a
Total Post-Construction Settlement (after preloading for 4 months and installation of EPS)	<25 (Figure 13)	n/a	<25 (Figure 14)	n/a	n/a	n/a	n/a
Total Differential Post- Construction Settlement (N-S) without preloading	275		275		<25		<25

Notes: 1. Values in table are estimated maximum settlement values.

2. Values in table for preloading assume embankment will be constructed to final grade before the wait period.



FOUNDATION REPORT - FREDERICK HOUSE BRIDGE REPLACEMENT HIGHWAY 11, GWP 5541-05-00

Table 5a: Evaluation of Stability/Settlement Mitigation Alternatives – West Approach

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Slope Flattening/ Berms	1	<ul style="list-style-type: none"> Would improve stability Straightforward construction 	<ul style="list-style-type: none"> May not have enough property to construct required berms 	<ul style="list-style-type: none"> Cost of additional fill for stabilizing berms is minor 	<ul style="list-style-type: none"> Risk of not having required property
Lightweight Fill (EPS)	1	<ul style="list-style-type: none"> Reduces load on compressible soils thereby improving stability and reducing total, post-construction and differential settlement 	<ul style="list-style-type: none"> Stabilizing toe berms still required but may be reduced to within property limits High EPS fill material cost 	<ul style="list-style-type: none"> Relative cost of EPS is up to an order of magnitude higher than for other materials 	<ul style="list-style-type: none"> Reduce risk of stability and/or settlement issues
Preloading	1	<ul style="list-style-type: none"> If preloading conducted prior to pile construction, downdrag loads can be reduced Would reduce the magnitude of post-construction settlement 	<ul style="list-style-type: none"> Time for preloading is between about 4 to 8 months after embankment construction to final grade to achieve 90% consolidation 	<ul style="list-style-type: none"> Generally low cost alternative Additional costs required for instrumentation and monitoring of embankment 	<ul style="list-style-type: none"> Primary and part of secondary (creep) settlement of foundation soils will occur Risk of impacting construction schedule
Grade Lowering	2	<ul style="list-style-type: none"> Determined by highway designers Reduces load on compressible soils thereby improving stability and reducing total, post-construction and differential settlement 	<ul style="list-style-type: none"> May not be feasible depending on overall site survey Would likely still have the settlement and stability issues noted in this report but to a lesser extent 	<ul style="list-style-type: none"> Cost of additional fill for stabilizing berms Additional costs required for instrumentation and monitoring of embankment 	<ul style="list-style-type: none"> Reduce risk of stability and/or settlement issues
Sub-Excavation of cohesive fill and native silty clay to clay	NF	<ul style="list-style-type: none"> Eliminate stability and settlement issue 	<ul style="list-style-type: none"> Extensive shoring and excavation required; too deep to be practical at this site 	<ul style="list-style-type: none"> Major costs for shoring 	<ul style="list-style-type: none"> Risk associated with shoring
Wick Drains/ Ground Improvement	NF	<ul style="list-style-type: none"> May reduce preload time and/or reduce magnitude of post-construction settlement 	<ul style="list-style-type: none"> Complex and costly installation Pre-drilling required for wick drains May not significantly reduce settlement or preload period 	<ul style="list-style-type: none"> Additional cost for specific ground improvement technique 	

NF indicates that alternative has been considered but is not feasible.



**FOUNDATION REPORT - FREDERICK HOUSE BRIDGE REPLACEMENT
HIGHWAY 11, GWP 5541-05-00**

Table 5b: Evaluation of Stability/Settlement Mitigation Alternatives – East Approach

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Slope Flattening/ Berms	1	<ul style="list-style-type: none">■ Would improve stability■ Straightforward construction■ Minimal additional fill required		<ul style="list-style-type: none">■ Cost of additional fill is minimal	<ul style="list-style-type: none">■ Low risk



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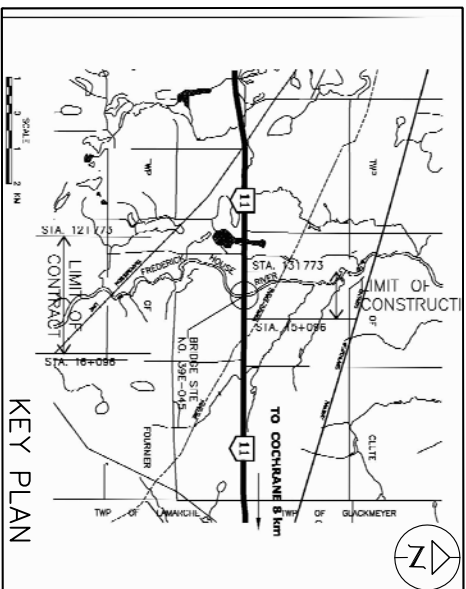
Table 6: Foundation Excavation Details

Location (Relevant Borehole)	Approximate Underside of Pile Cap Elevation	Elevation of Base of Excavation*	Approximate Existing Ground Surface	Existing Highway Grade	Depth Below Existing Highway Grade	Depth Below/Above Existing Ground Surface	Groundwater/ River Elevation	Depth Below Groundwater/ River Elevation	Comments
West Abutment (FR-2)	251.8	251.3	249(N) 256(S)	258.1	7.1	2.0 above (N) 5.0 below (S)	244.1	Above groundwater level, may be perched water within fill	Fill required to underside of pile cap level at the north edge Temporary roadway protection required
West Pier (FR-3)	242	241.5	245(N) 248(S)	n/a	n/a	4.2 below (N) 7.2 below (S)	240.7 (BH) 242.3 (River)	1.5 m below river level	Possible cut-off wall required adjacent to river
East Pier (FR-4A)	240	240	247(N) 244(S)	n/a	n/a	7.0 below (N) 4.0 below (S)	241.9 (BH) 242.3 (River)	2.3 m below river level	Possible cut-off wall/cofferdam required adjacent to river
	recommend 242	241.5				5.5 below (N) 2.5 below (S)		0.8 m below river level	
East Abutment (FR-5)	251.0	250.5	256	257.5	7.5	6.0 below	247.4	Above groundwater level, may be perched water within fill	Temporary roadway protection required












*Assumes removal of existing fill up to 0.5 m below the pile cap level at the abutments and removal of all fill at the piers.



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LEGEND

	Borehole – Current Investigation (Golder 2009)		
	Borehole – Approximate Location (Shaheen & Pecker 2006)		
	Borehole and DCPT – Approximate Location (Shaheen & Pecker 2006)		
	Seal		
	Piezometer		
	Standard Penetration Test Value		
	Blows/0.3 m unless otherwise stated (Std. Pen. Test, 4/75/blow)		
	Rock Quality Designation (RQD)		
	WL upon completion of drilling		
	WL in piezometer, measured on July 31, 2009		
	Refusal		
No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
FR- 1	249.8	5435620.5	294567.2
FR- 2	247.2	5435620.8	294587.5
FR- 3	244.9	5435622.4	294624.8
FR- 4	242.9	5435608.2	294700.4
FR- 4A	249.0	5435615.4	294706.7
FR- 5	255.0	5435611.9	294739.3
FR- 6	257.5	5435607.6	294757.8
FR- 7	252.0	5435627.1	294533.0
FR- 8	258.5	5435599.3	294545.4
FR- 9	245.2	5435641.9	294587.4
FR- 10	246.9	5435641.5	294567.5

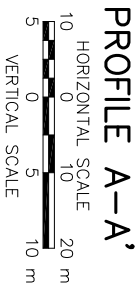
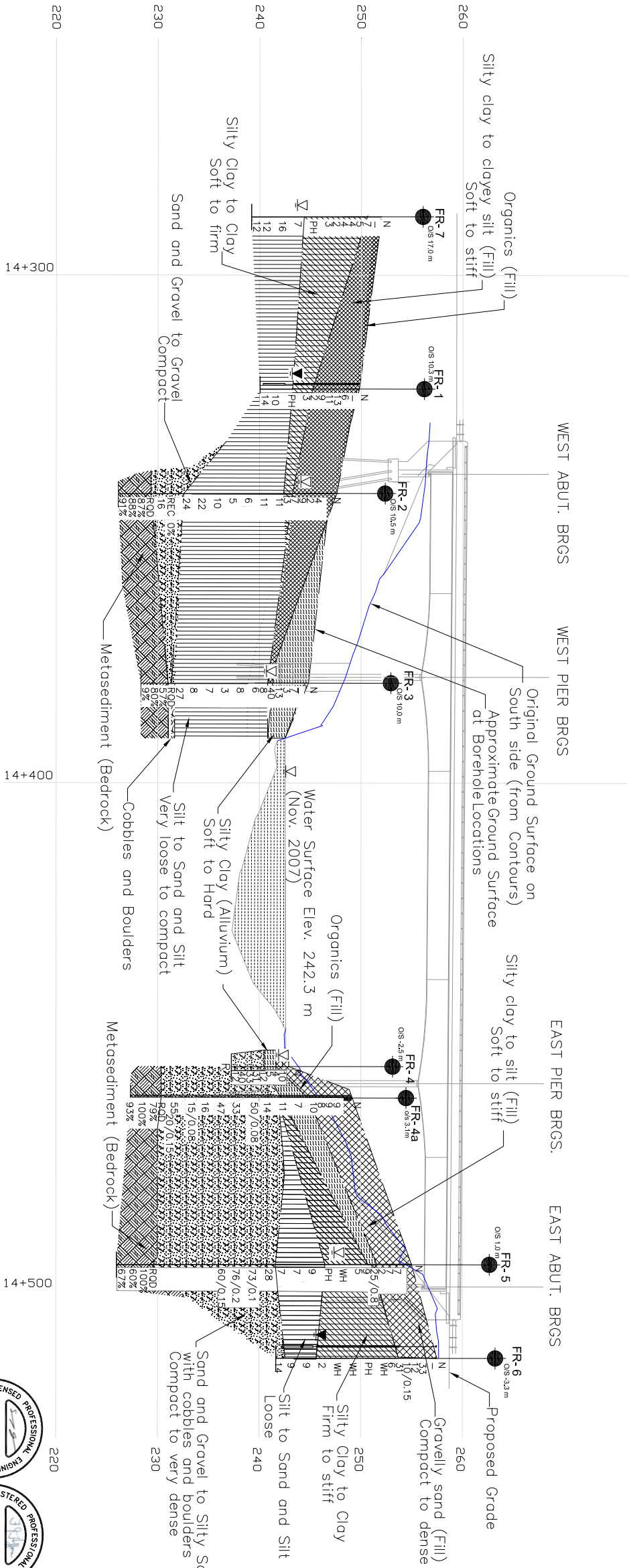
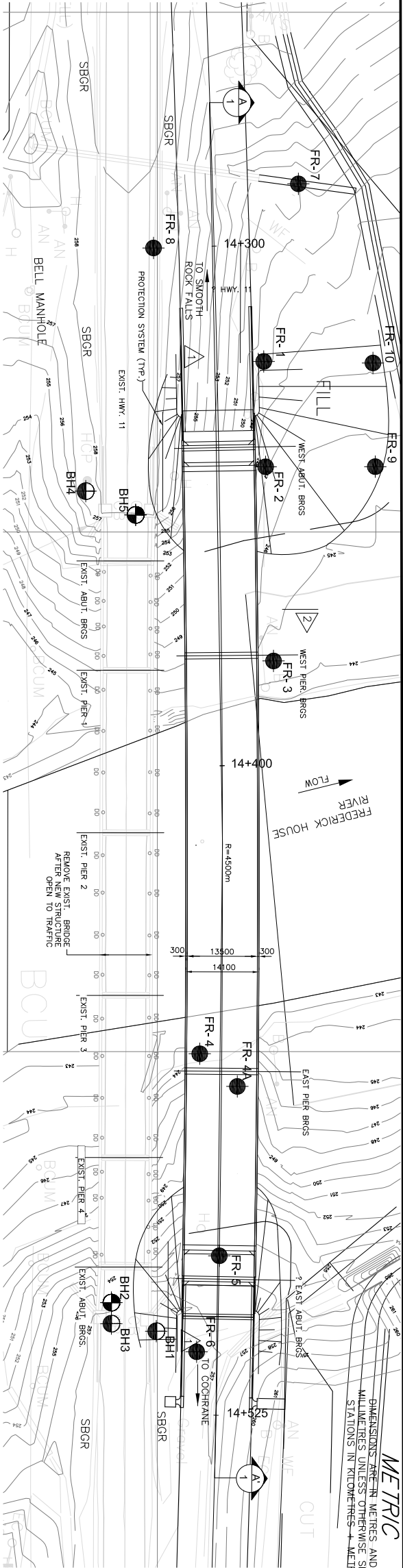
NOTES

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 2.101 of OPS General Conditions.

NO.	DATE	BY	REVISION	DIST.
Geocres No. 42H-37				
HWY-11	PROJECT NO. 07-1191-0007			
SUBWD LG	CHKD. DATE: Nov 2009			SITE: 39E-045
DRAWN: MM	CHKD. SEWC	APPD. FJH		DWG. 1



REFERENCES

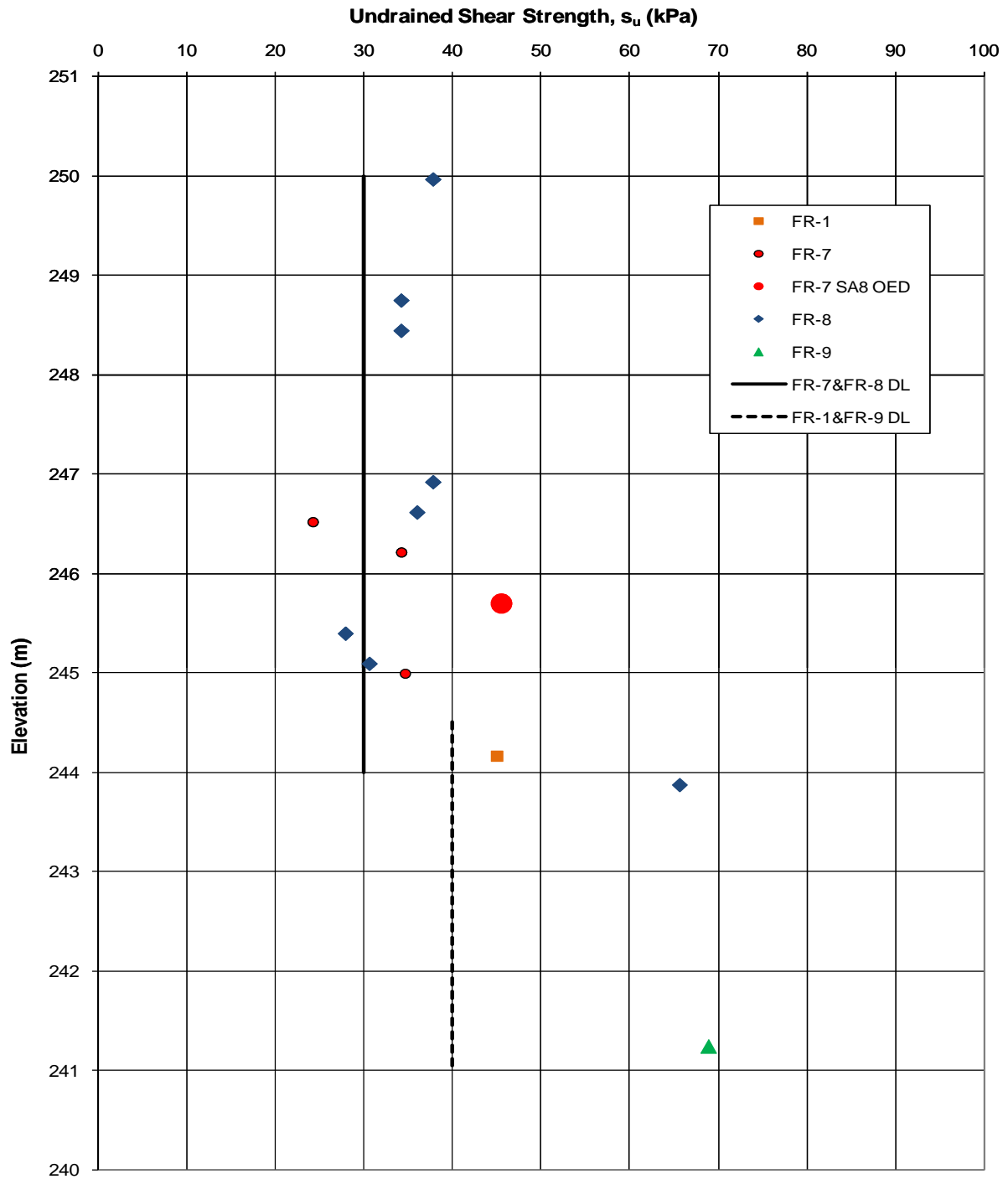
Base plan provided in digital format by LEA Group Ltd., drawing file no. FRED-GA_JUNE23_09.dwg (received June 23, 2009) and key plan, drawing file no. 8461 FH KEYPLAN.dwg (received June 19, 2009).

Previous boreholes from " Foundation Investigation Report Highway 11, Frederickhouse River Bridge Structure Rehabilitation, WP. 647-90-01, Geocres No. 42H-32, by Shoheen & Pecker Limited, dated Feb. 1, 2006".



UNDRAINED SHEAR STRENGTH VS. ELEVATION West Approach

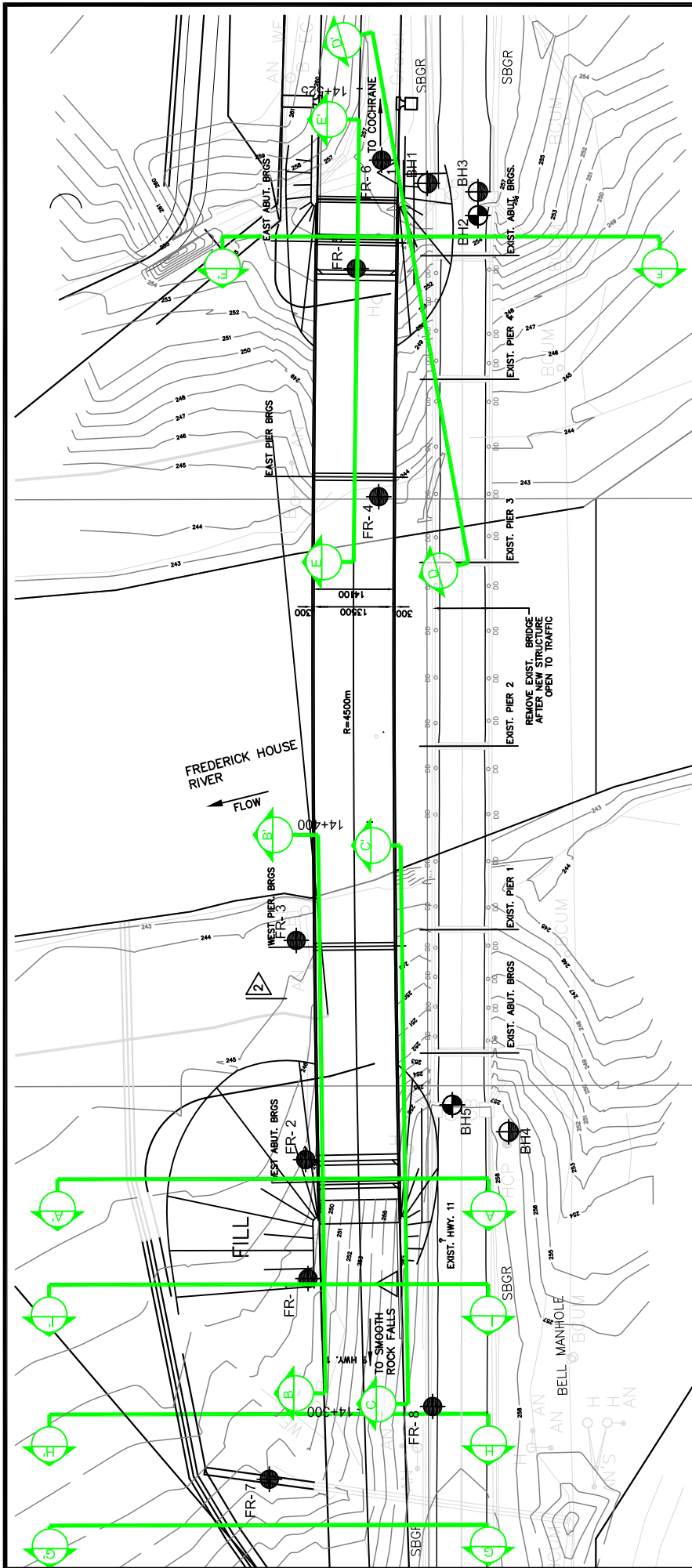
FIGURE 1



Date: January 2010
Project: 07-1191-0007-FH

Golder Associates

Drawn: TR
Checked: SEMC



LEGEND

- Borehole - Current Investigation (Golder 2009)
- Borehole - Approximate Location (Shaheen & Peaker 2006)
- Borehole and DCPT - Approximate Location (Shaheen & Peaker 2006)

NOTES

1. Sketch shown for illustration purposes. Section lines are approximate only
2. Refer to Section 6.8. of accompanying Foundation Design Report.

REFERENCE

Base plan provided in digital format by LEA Group Ltd., drawing file no. FRED-GA_JUNE23_09.dwg (received June 23, 2009) and key plan, drawing file no. 8461 FH KEYPLAN.dwg (received June 19, 2009).

Previous boreholes from " Foundation Investigation Report, Highway 11, Frederickhouse River Bridge Structure Rehabilitation, W.P. 647-90-01, Geocres No. 42H-32, by Shaheen & Peaker Limited, dated Feb. 1, 2006".

TITLE

PROJECT	No07-1191-0007		
FILE	No0711910007FIG2.DWG		
REV. 0	SCALE AS SHOWN		
DESIGN			
CADD	MM	JAN 2010	
CHECK	SEMC	JAN 2010	
REVIEW	FJH	JAN 2010	

FIGURE 2

CRITICAL STABILITY SECTIONS

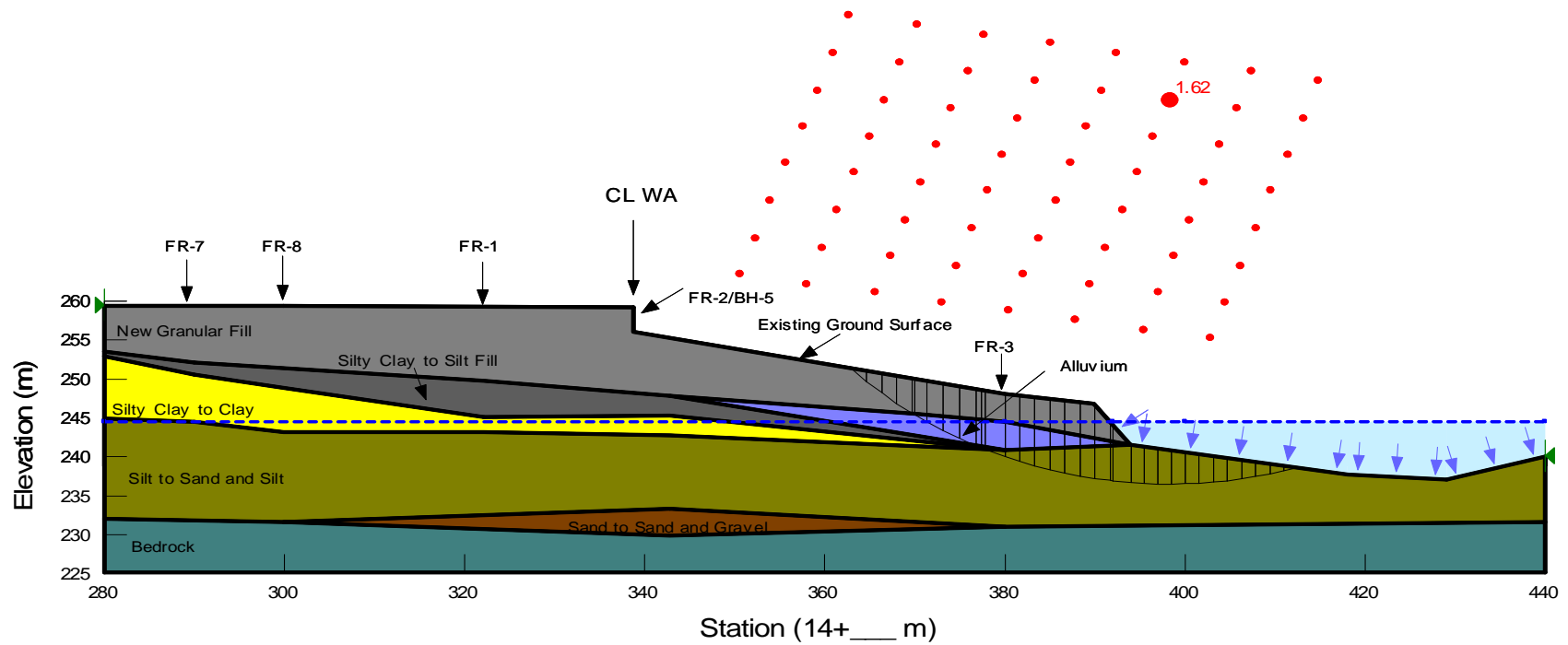
PROJECT

FREDERICK HOUSE RIVER BRIDGE
GWP 5403-05-00



STABILITY ANALYSIS
West Abutment Front Slope, South Side (Section C-C)

FIGURE 3



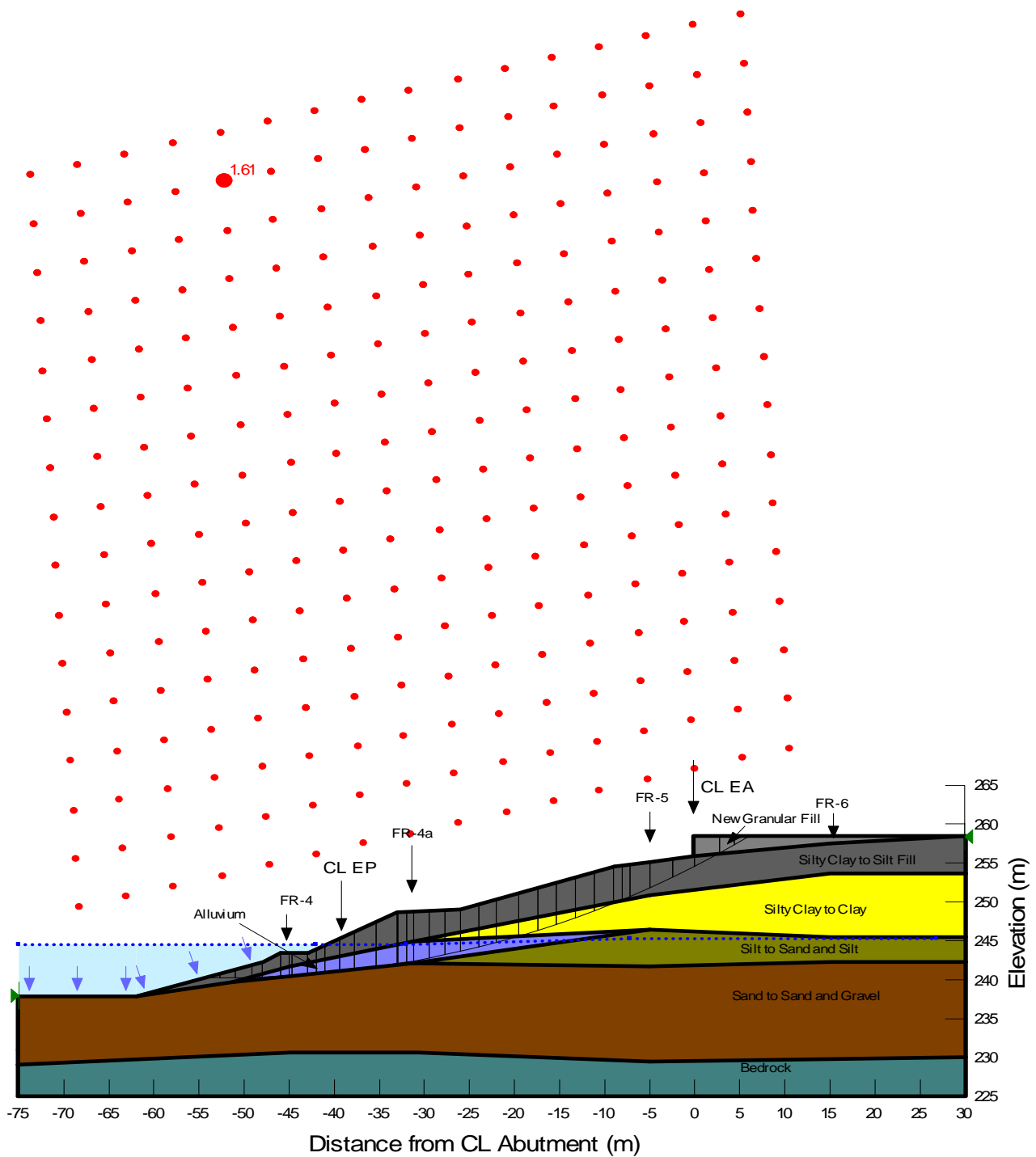
Date: January 2010
Project: 07-1191-0007-FH

Golder Associates

Drawn: TR
Checked: SEMC

STABILITY ANALYSIS East Abutment Front Slope (Section E-E)

FIGURE 4



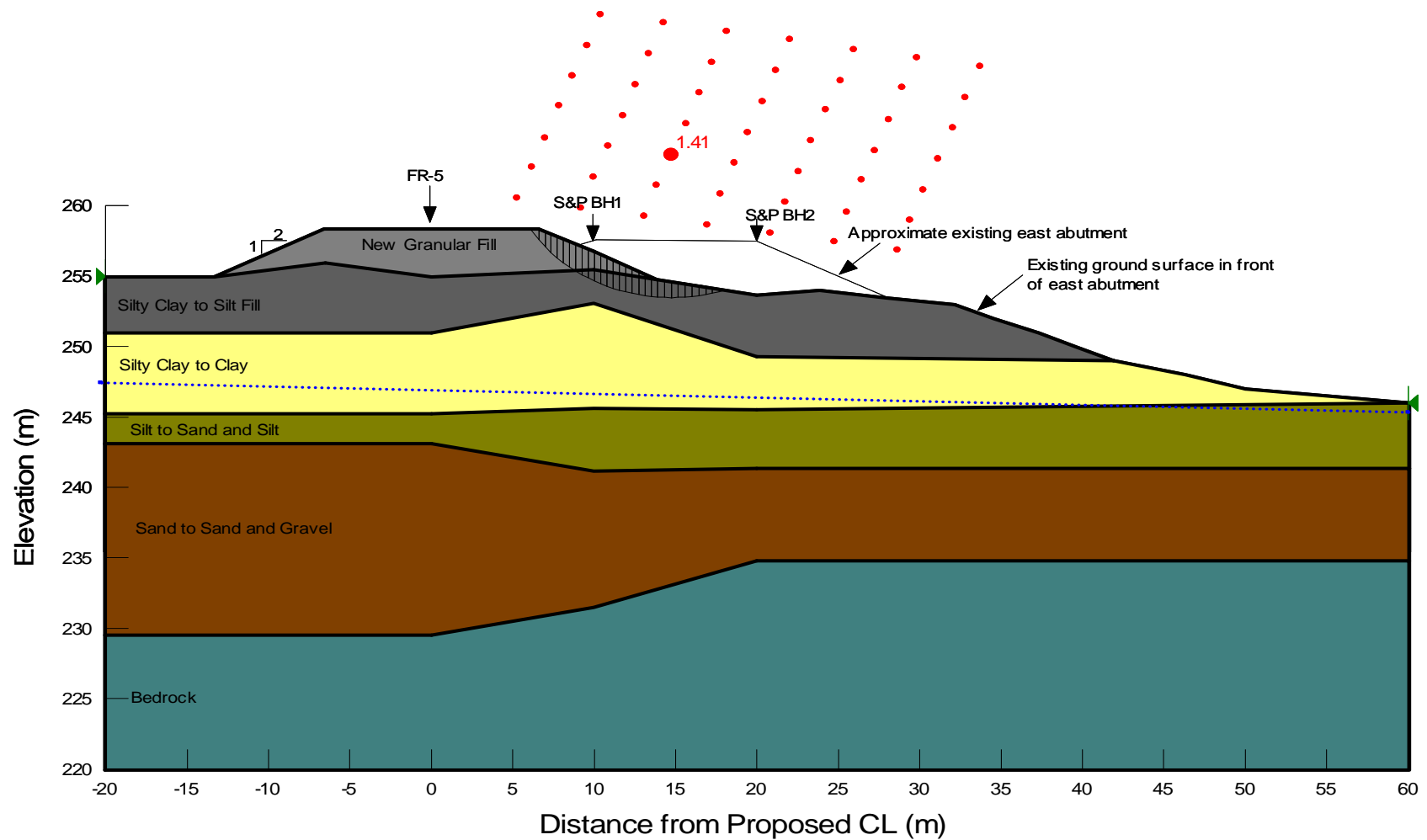
Date: January 2010
Project: 07-1191-0007-FH

Golder Associates

Drawn: TR
Checked: SEMC

STABILITY ANALYSIS
East Abutment Cross-Section (Section F-F)

FIGURE 5



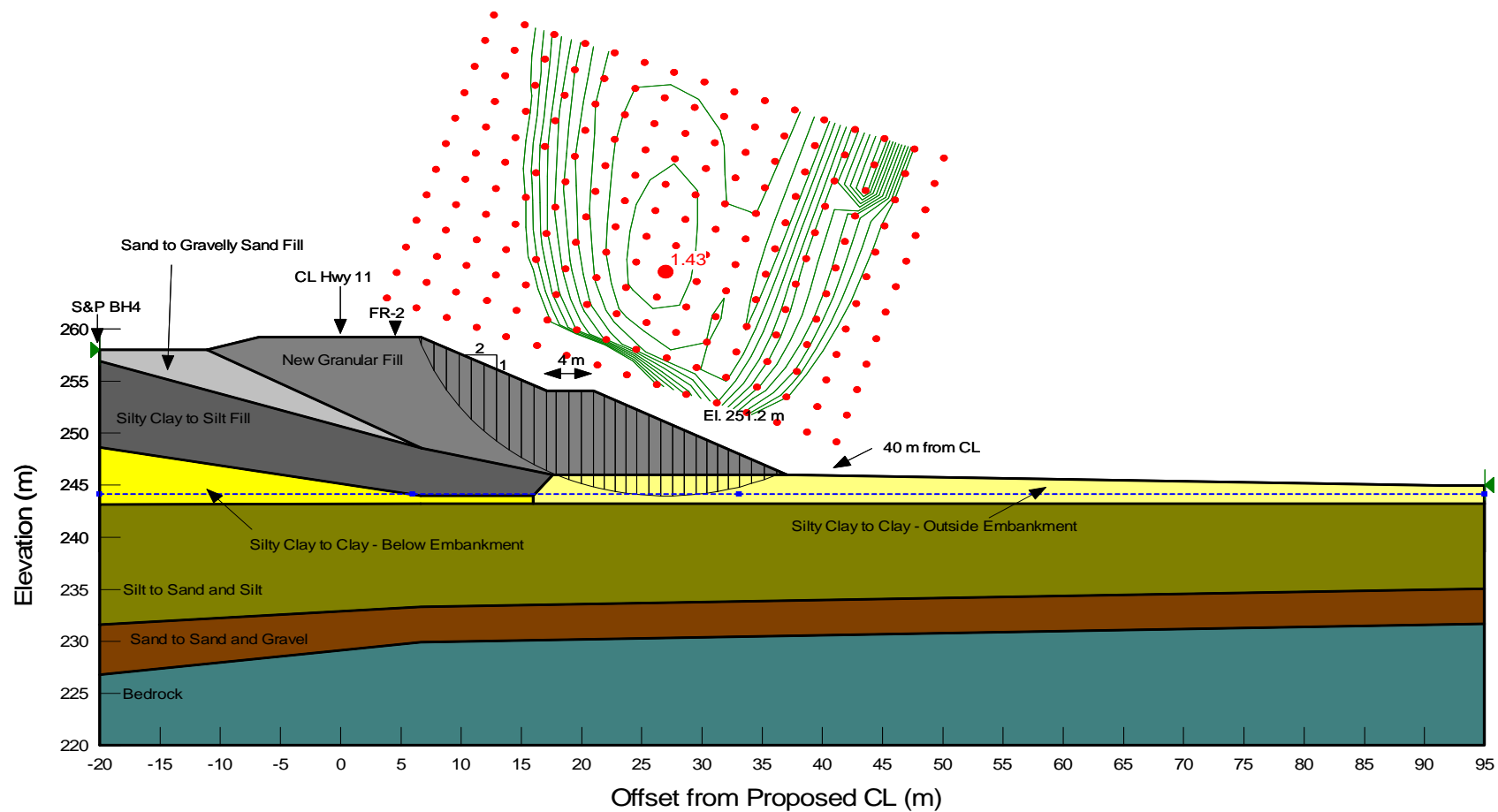
Date: January 2010
Project: 07-1191-0007-FH

Golder Associates

Drawn: TR
Checked: SEMC

West Abutment Cross-Section with Berms (Section A-A)

FIGURE 6



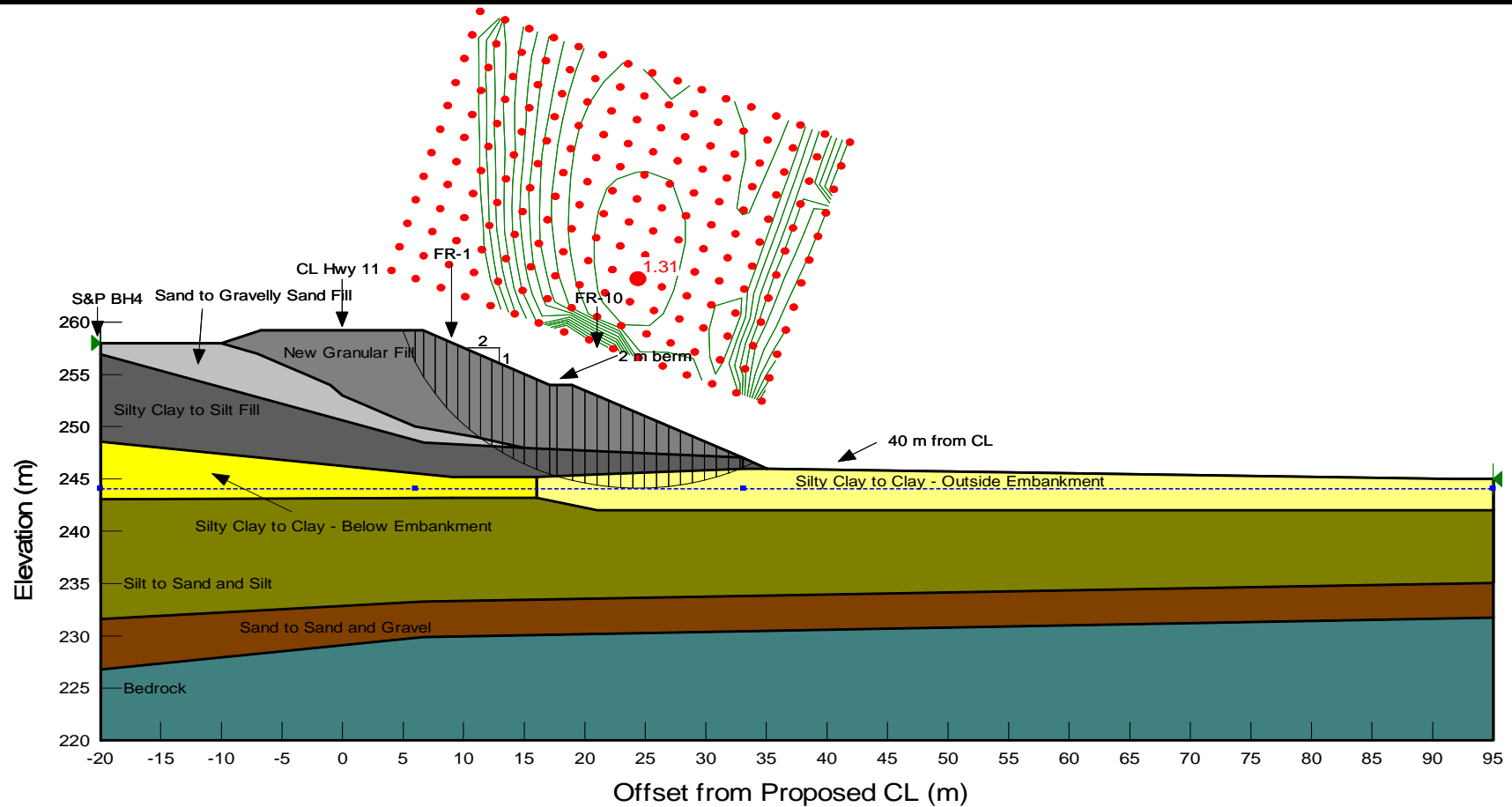
Date: January 2010
Project: 07-1191-0007-FH

Golder Associates

Drawn: TR
Checked: SEMC

STABILITY ANALYSIS
Cross-Section with Berms (Section I-I STA 14+320)

FIGURE 7



Date: January 2010
Project: 07-1191-0007-FH

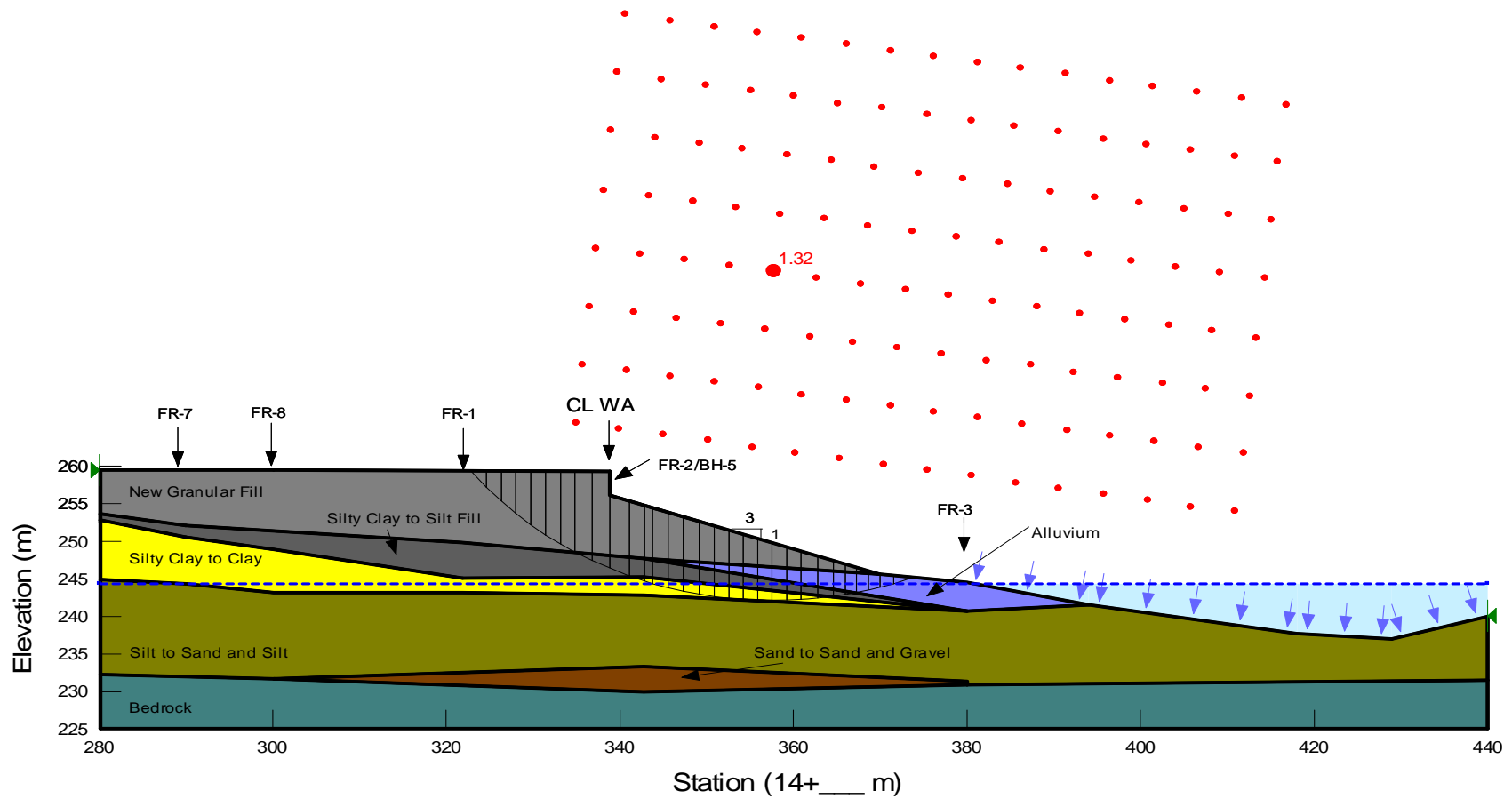
Golder Associates

Drawn: TR
Checked: SEMC

STABILITY ANALYSIS

West Abument Front Slope, North Side with Slope Flattening (Section B-B)

FIGURE 8



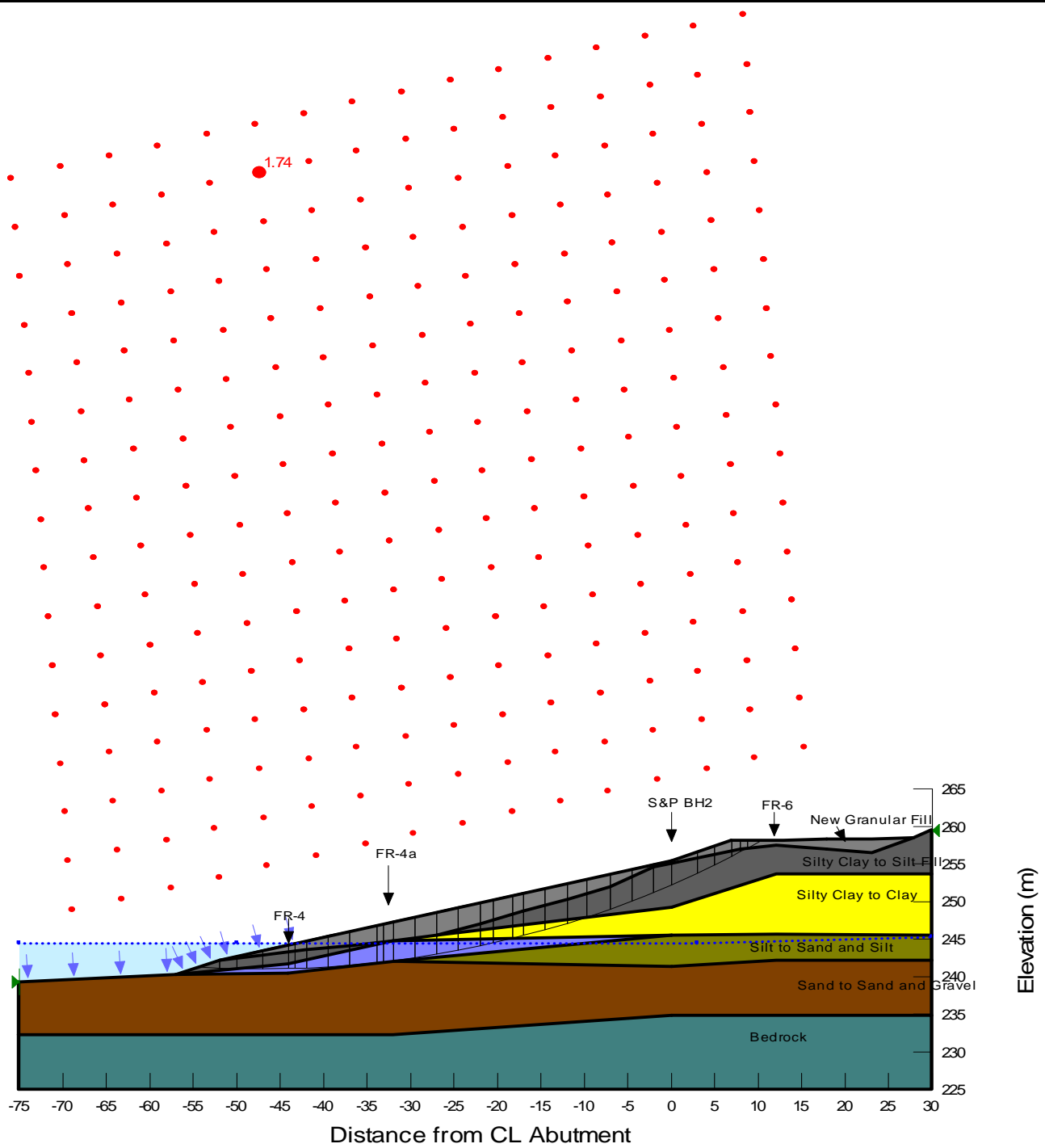
Date: January 2010
Project: 07-1191-0007-FH

Golder Associates

Drawn: TR
Checked: SEMC

STABILITY ANALYSIS
East Abutment Front Slope with Slope Flattening
(Section D-D)

FIGURE 9



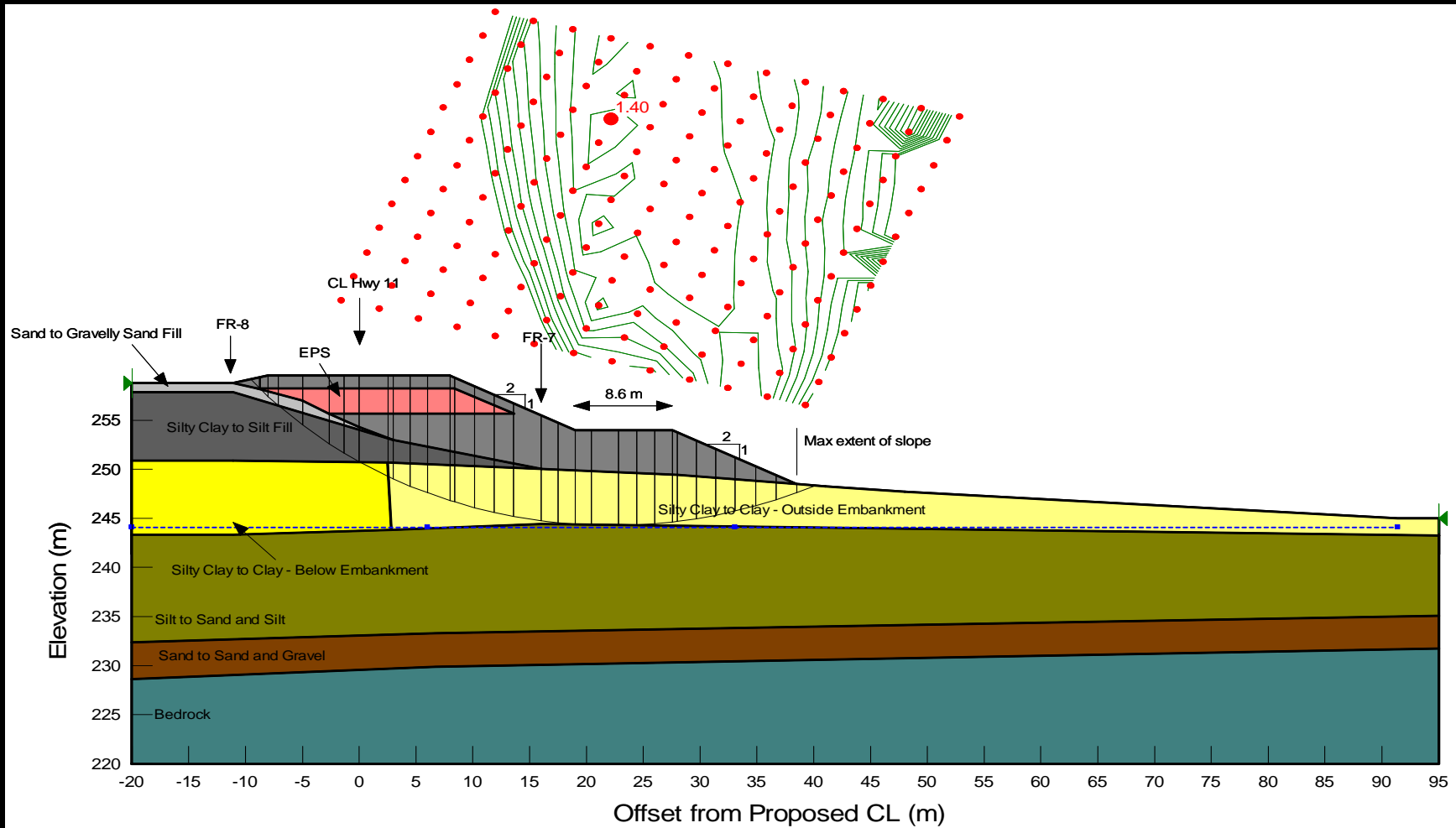
Date: January 2010
 Project: 07-1191-0007-FH

Golder Associates

Drawn: TR
 Checked: SEMC

STABILITY ANALYSIS
STA 14+300 Cross Section with Berms and 2.6 m EPS (Section H-H)

FIGURE 10



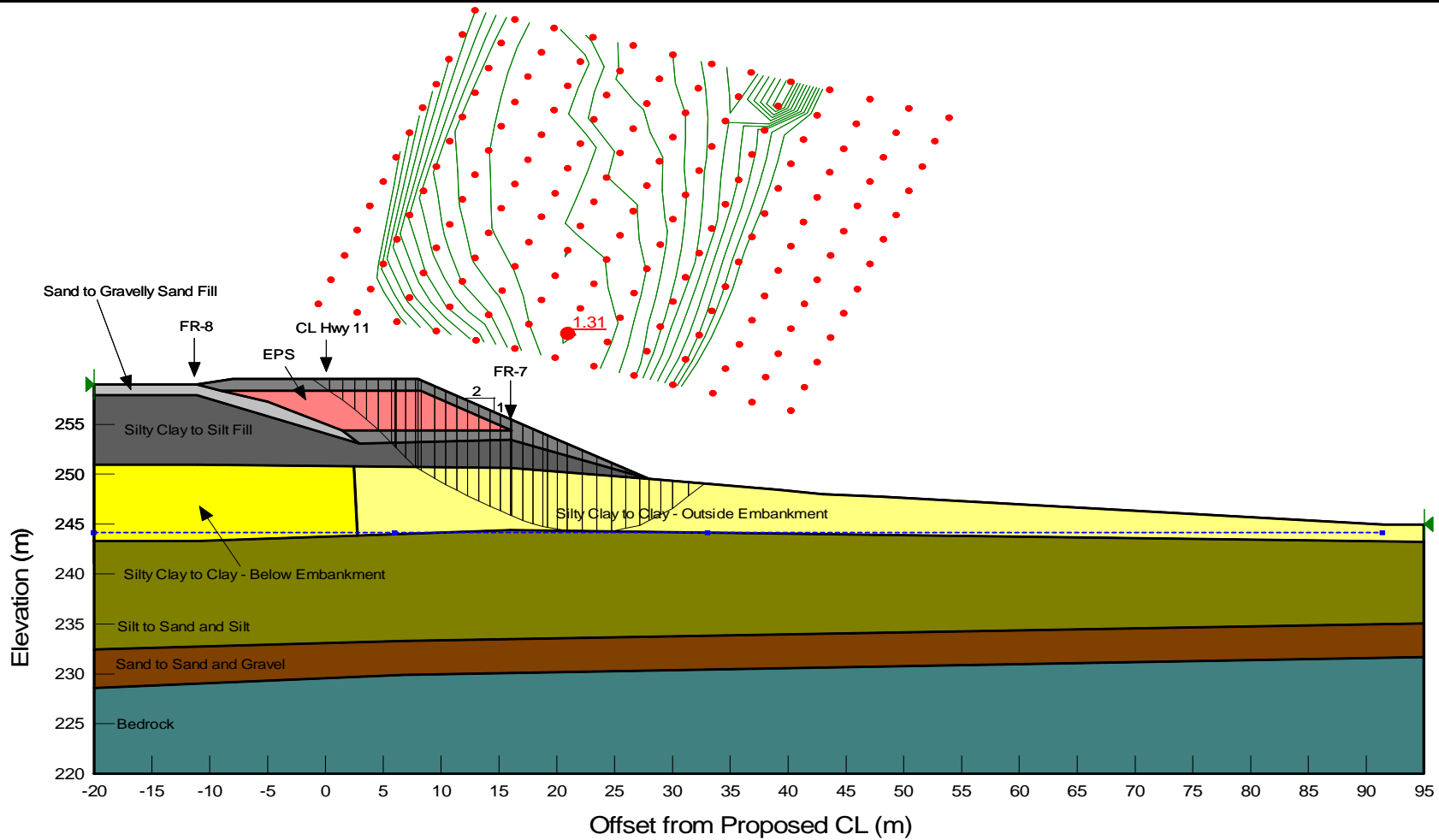
Date: January 2010
Project: 07-1191-0007-FH

Golder Associates

Drawn: TR
Checked: SEMC

STABILITY ANALYSIS
STA 14+280 Cross Section with Berms and 3.9 m EPS (Section G-G)

FIGURE 11



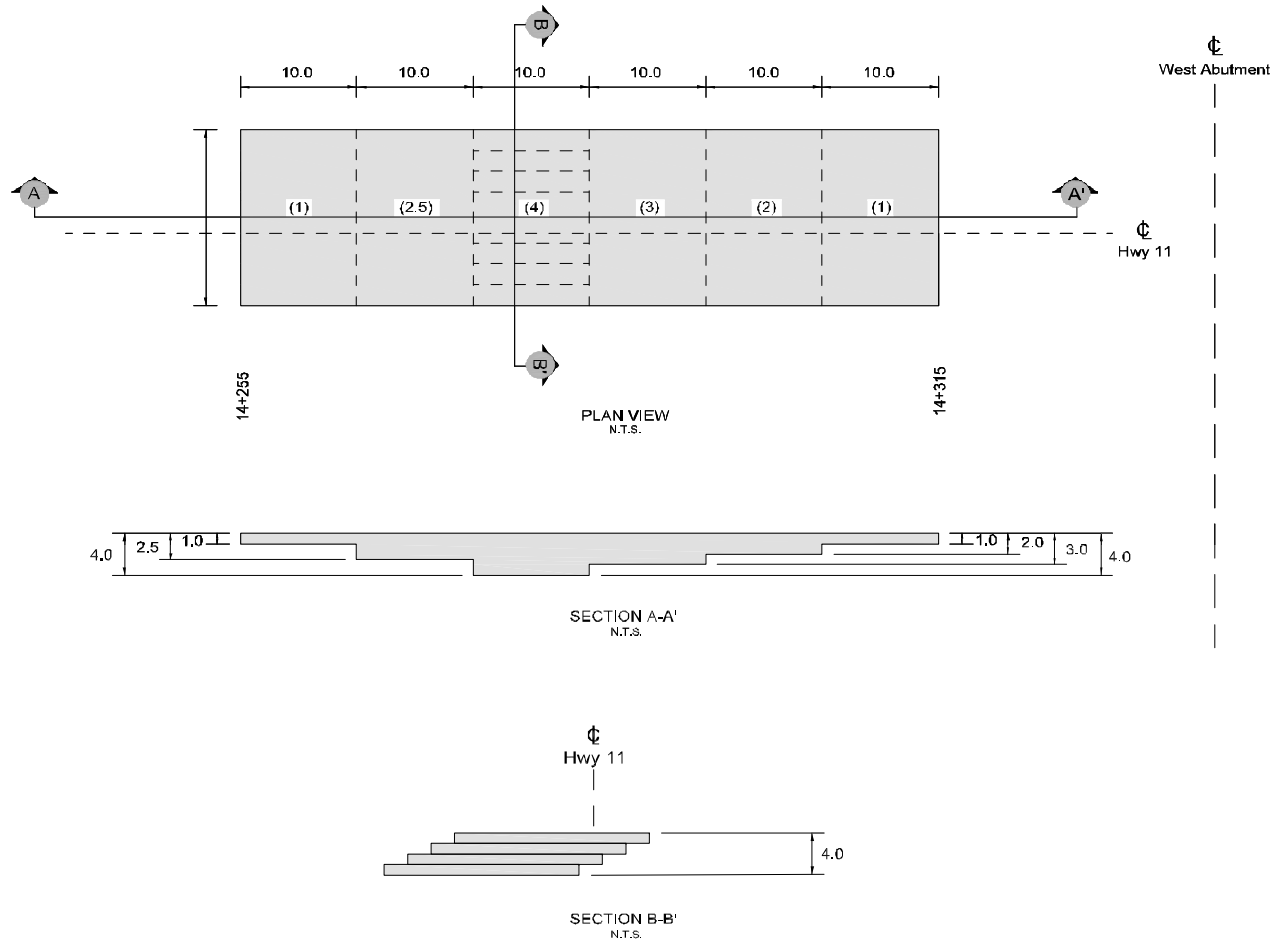
Date: January 2010
Project: 07-1191-0007-FH

Golder Associates

Drawn: TR
Checked: SEMC

EPS DETAILS
West Approach STA 14+255 to STA 14+315

FIGURE 12



Notes:

1. All units are in metres.
2. '(3)' indicates EPS thickness in metres.
3. Stepping as required below embankment slopes not shown.

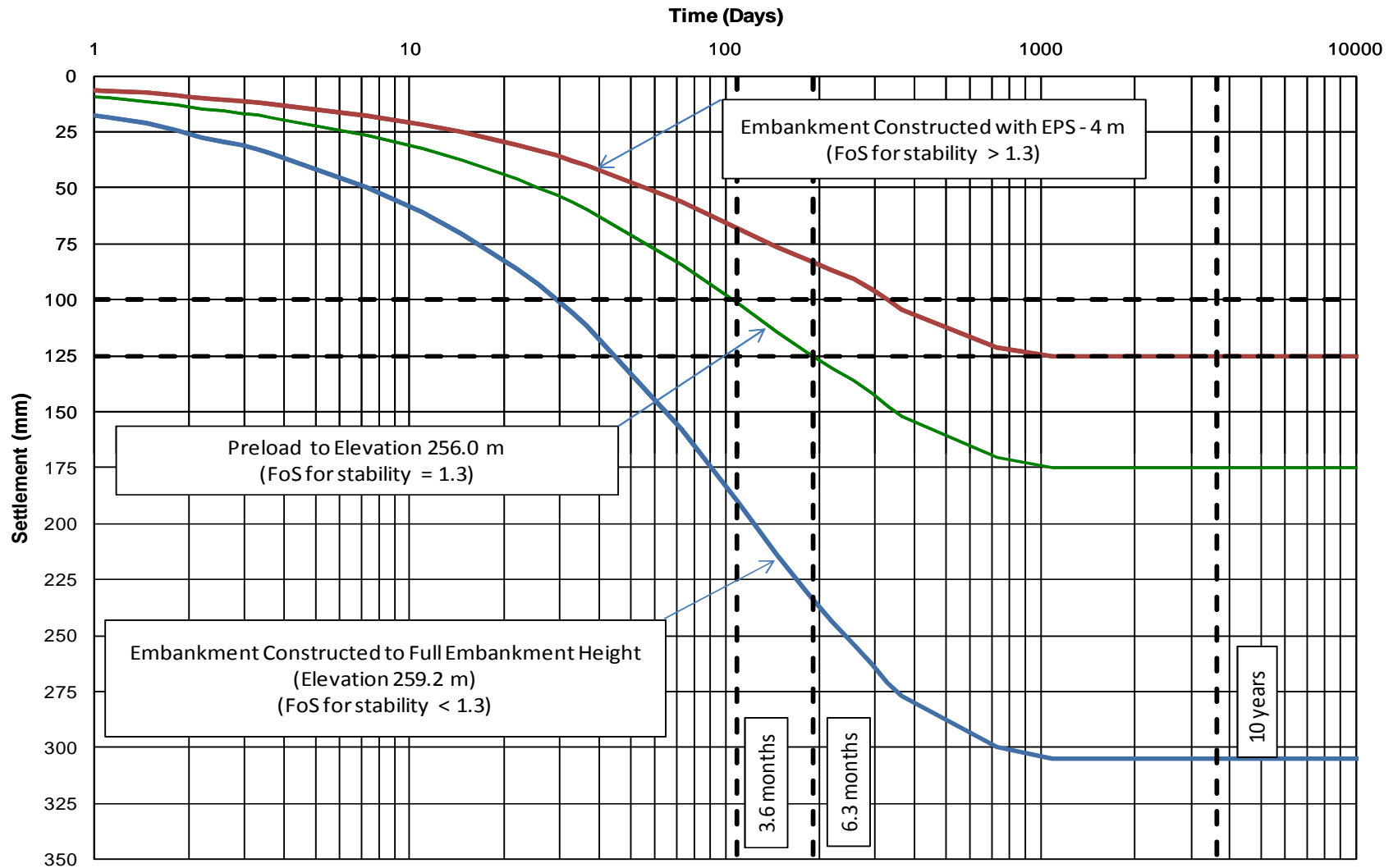
Date: January 2010
 Project: 07-1191-0007-FH

Golder Associates

Drawn: AW
 Checked: SEMC

SETTLEMENT ANALYSIS West Approach, North Side STA 14+280

FIGURE 13



Date: January 2010
Project: 07-1191-0007-FH

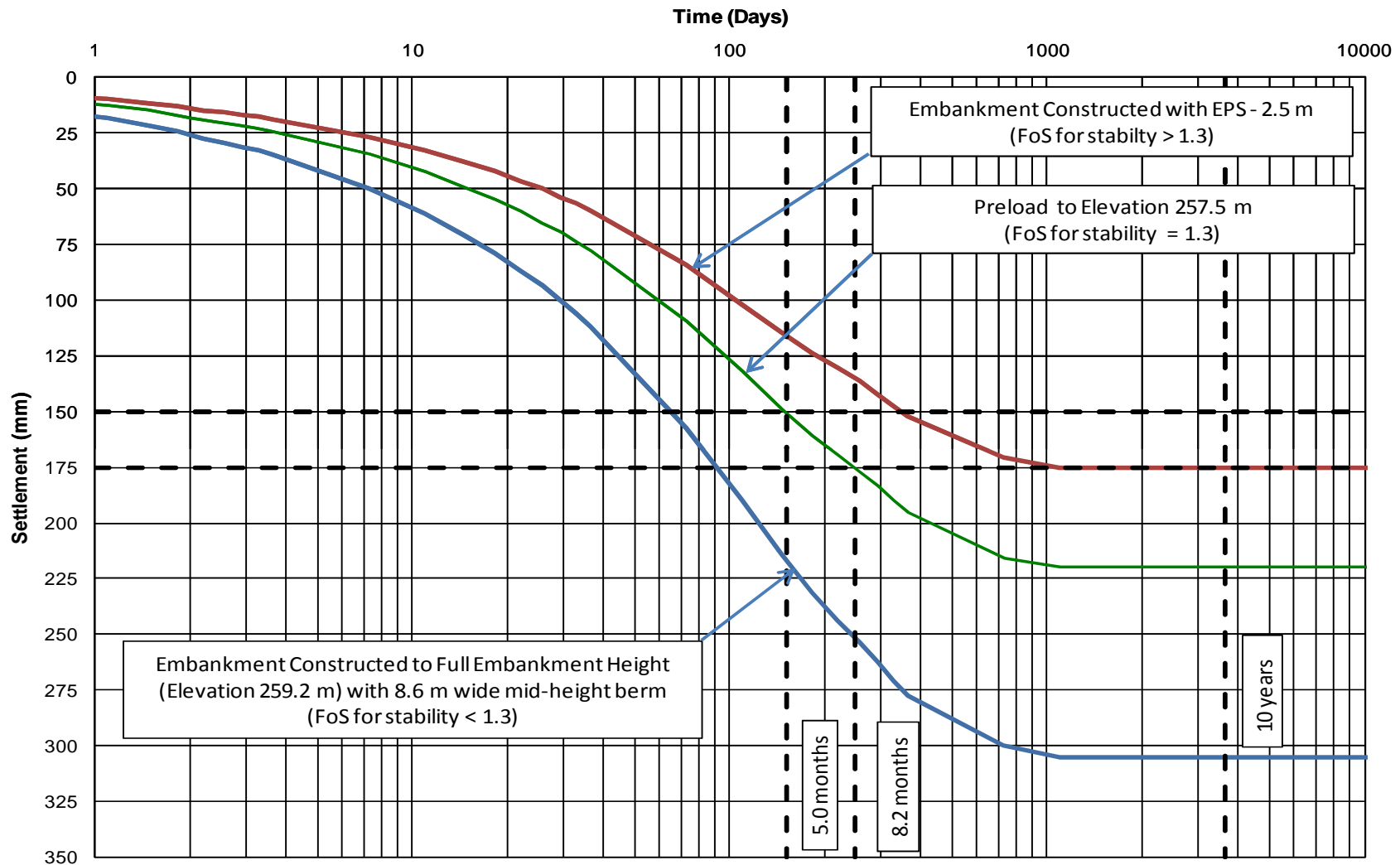
Golder Associates

Drawn: SEMC
Checked: FJH

SETTLEMENT ANALYSIS

West Approach, North Side STA 14+300

FIGURE 14



Date: January 2010
Project: 07-1191-0007-FH

Golder Associates

Drawn: SEMC
Checked: FJH



APPENDIX A

RECORD OF BOREHOLE AND DRILLHOLE SHEETS

LIST OF SYMBOLS

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

I. GENERAL

π	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
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	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
γ'	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 ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x 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
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	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.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezcone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² 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 (Relative Density)	N 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 ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
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)
γ	unit weight

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

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	P - Polished
FO - Foliation / Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane / Zone	R - Ridged / Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To	⊥ - Perpendicular To

PROJECT 07-1191-0007			RECORD OF BOREHOLE No FR- 1			1 OF 1 METRIC															
W.P. 5541-05-01			LOCATION N 5435620.5 ; E 294567.2			ORIGINATED BY TDM															
DIST HWY 11			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY MM															
DATUM Geodetic			DATE April 23, 2009			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 _p — W — W _L			γ			GR SA SI CL		
249.8	GROUND SURFACE							20 40 60 80 100													
0.0	Organics (FILL) Brown Wet		1	AS	-																
0.2	Silty clay, trace sand, trace gravel (FILL) Soft to stiff Brown Moist to wet		2	SS	6		249														
			3	SS	13		248														
			4	SS	11		247														
			5	SS	9		246														
			6	SS	2		245														
245.2	SILTY CLAY, varved Soft to firm Grey Wet		7	SS	3		244														
	Dark grey clay laminae 5 mm to 20 mm thick. Light grey clayey silt laminae 25 mm to 50 mm thick.		8	TO	PH		243														
243.2	SILT, some sand, trace to some clay Compact Grey Wet		9	SS	10		242														
6.6			10	SS	14		241														
240.0	End of Borehole																				
9.8	Note: 1. Water level at a depth of 4.4 m below ground surface (Elev. 245.4 m) upon completion of drilling. 2. Water level measured in piezometer at a depth of 5.6 m below ground surface (Elev. 244.2 m) on April 28, 2009 and at 6.9 m depth (Elev. 242.9 m) on July 31, 2009.																				

PROJECT <u>07-1191-0007</u>				RECORD OF BOREHOLE No FR-2				1 OF 2 METRIC						
W.P. <u>5541-05-01</u>		LOCATION <u>N 5435620.8 ; E 294587.5</u>				ORIGINATED BY <u>TDM</u>								
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>								
DATUM <u>Geodetic</u>		DATE <u>April 22 to 24, 2009</u>				CHECKED BY <u>AB</u>								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
247.2	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100					
0.0	Organics (FILL)		1	AS	-		247							
0.2	Clayey silt to silty clay, trace sand, trace gravel (FILL)		2	SS	4		246							
	Soft to stiff Brown Moist		3	SS	2		245							
			4	SS	9		244							
			5	SS	7		243							
243.4	SILTY CLAY		6	SS	3	▽	243							
3.8	Soft Brown Wet		7	SS	11		242							
242.6	SILT to Sandy SILT, trace to some clay		8	SS	11		241							
4.6	Loose to compact Grey Wet		9	SS	6		240							
	Difficulty advancing augers at 7.5 m depth. Switch to NW Casing.		10	SS	5		239							
			11	SS	10		238							
			12	SS	22		237							
			13	SS	24		236							
232.7							235							
14.5							234							
							233							

Continued Next Page

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

MIS-MTO.001 FREDERICK_HOUSE_0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

PROJECT <u>07-1191-0007</u>				RECORD OF BOREHOLE No FR- 2				2 OF 2 METRIC									
W.P. <u>5541-05-01</u>		LOCATION <u>N 5435620.8 ;E 294587.5</u>				ORIGINATED BY <u>TDM</u>											
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>											
DATUM <u>Geodetic</u>		DATE <u>April 22 to 24, 2009</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
229.3	SAND and GRAVEL, trace silt Compact Grey Wet Difficult casing advance at 14.5 m depth.		14	RC	-		232										
							231										
			15	SS	16		230										
17.9	METASEDIMENT (BEDROCK) Bedrock cored from 17.9 m to 21.1 m depth. For coring details refer to Record of Drillhole FR- 2.		1	RC	REC 100%		229										RQD = 87%
			2	RC	REC 100%		228										RQD = 88%
226.1			3	RC	REC 100%		227										RQD = 91%
21.1	End of Borehole Note: 1. Water level at a depth of 3.7 m below ground surface (Elev. 243.5 m) upon completion of drilling.																

PROJECT: 07-1191-0007

RECORD OF DRILLHOLE: FR- 2

SHEET 1 OF 1

LOCATION: N 5435620.8 ;E 294587.5

DRILLING DATE: April 22 to 24, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 45

DRILLING CONTRACTOR: Downing Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	PENETRATION RATE min/m	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION
DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	PENETRATION RATE min/m	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION
18	04/24/2009 NO Coring	Refer to Previous Page METASEDIMENT, containing occasional silt seams Fine grained Slightly weathered Very strong Grey		229.30 17.90	1								
19					2								
20					3								
21		End of Drillhole		226.10 21.10									
22													
23													
24													
25													
26													
27													

DEPTH SCALE

1 : 50




LOGGED: TDM

CHECKED: AB

MIS-RCK 004 FREDERICK HOUSE_0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

PROJECT 07-1191-0007		RECORD OF BOREHOLE No FR- 3		1 OF 2 METRIC	
W.P. 5541-05-01		LOCATION N 5435622.4 ; E 294624.8		ORIGINATED BY TDM	
DIST HWY 11		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY MM	
DATUM Geodetic		DATE April 22, 2009		CHECKED BY AB	
SOIL PROFILE			SAMPLES		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES
244.9	GROUND SURFACE				
0.0	Organics (FILL)		1	AS	-
0.2	Brown Wet Silty clay, some sand and gravel, trace organics (ALLUVIUM) Soft to hard Brown Wet		2	SS	7
			3	SS	3
			4	SS	13
			5	SS	40
241.2					
3.7	SILT to SAND and SILT, trace to some clay, trace gravel Very loose to compact Grey Wet Sample 7: One large gravel piece was retained on 26.5 mm sieve.		6	SS	8
			7	SS	6
			8	SS	8
			9	SS	3
			10	SS	7
			11	SS	8
			12	SS	27
231.8	COBBLES and BOULDERS		13	RC	-
231.4	METASEDIMENT (BEDROCK) Bedrock cored from 13.5 m to 16.5 m depth. For coring details refer to Record of Drillhole FR- 3.		1	RC	REC 90%
13.5			2	RC	REC 100%
DYNAMIC CONE PENETRATION RESISTANCE PLOT			SHEAR STRENGTH kPa		
20 40 60 80 100			○ UNCONFINED + FIELD VANE		
● QUICK TRIAXIAL × REMOULDED			20 40 60 80 100		
PLASTIC LIMIT Wp			NATURAL MOISTURE CONTENT W		
LIQUID LIMIT Wl			WATER CONTENT (%)		
10 20 30			UNIT WEIGHT γ		
kN/m³			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
GR SA SI CL			0 46 38 16		
			0 12 79 9		
			32 54 12 2		
			0 35 62 3		
			RQD = 57%		
			RQD = 80%		

<div style="display: flex; justify-content: space-between;"> PROJECT <u>07-1191-0007</u> RECORD OF BOREHOLE No FR- 3 2 OF 2 METRIC </div>																		
W.P. <u>5541-05-01</u>		LOCATION <u>N 5435622.4 ;E 294624.8</u>				ORIGINATED BY <u>TDM</u>												
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>MM</u>												
DATUM <u>Geodetic</u>		DATE <u>April 22, 2009</u>				CHECKED BY <u>AB</u>												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)					
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 </div>					<div style="display: flex; justify-content: space-between;"> 10 20 30 </div>						
	METASEDIMENT (BEDROCK) Bedrock cored from 13.5 m to 16.5 m depth.		2	RC														
	For coring details refer to Record of Drillhole FR- 3.		3	RC	REC 65%													RQD = 80%
228.4						229												
16.5	End of Borehole Note: 1. Water level at a depth of 3.8 m below ground surface (Elev. 241.1 m) upon completion of drilling.																	

PROJECT: 07-1191-0007

RECORD OF DRILLHOLE: FR-3

SHEET 1 OF 1

LOCATION: N 5435622.4 ;E 294624.8

DRILLING DATE: April 22, 2009

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 45

DRILLING CONTRACTOR: Downing Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE mm/min	FLUSH	RECOVERY TOTAL CORE % SOLID CORE % R.Q.D. %	FRACT. INDEX PER 0.3 m	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diameter Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION						
																				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.

		Refer to Previous Page		231.40															
14	04/22/2009 NQ Coring	METASEDIMENT, containing occasional silt seams Fine grained Slightly weathered Very strong Grey <																	

DEPTH SCALE



1 : 50



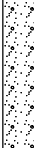


LOGGED: TDM

CHECKED: AB

MIS-RCK 004 FREDERICK HOUSE_0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

PROJECT <u>07-1191-0007</u>		RECORD OF BOREHOLE No FR- 4				1 OF 1 METRIC												
W.P. <u>5541-05-01</u>		LOCATION <u>N 5435608.2 ; E 294700.4</u>				ORIGINATED BY <u>MR</u>												
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>Portable Equipment, NW and BW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>												
DATUM <u>Geodetic</u>		DATE <u>April 21, 2009</u>				CHECKED BY <u>AB</u>												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
242.9	GROUND SURFACE							20	40	60	80	100						
0.0	Organics, some sand (FILL) Stiff Brown Wet		1	SS	10		242											
242.3			2	SS	4			241										
0.6	Clayey silt to silt, trace sand (FILL) Firm Grey Wet		3	SS	5													
241.7			4	SS	12				240									
1.2	Clayey silt to silt, trace sand, organic seams (ALLUVIUM) Firm to stiff Grey Wet	5	SS	31														
240.5		6	SS	33	239													
2.4	SAND to Silty SAND, some gravel, some clay, containing cobbles Compact to dense Grey Wet	7	SS	40														
		8	SS	12						238								
237.3																		
5.6	End of Borehole Casing Refusal Notes: 1. Could not advance casing with portable equipment below 5.6 m depth. 2. No recovery in sample 4. 3. Small trickle of water coming over top of casing at 3.2 m depth. 4. Water level at a depth of 1.0 m below ground surface (Elev. 241.9 m) upon completion of drilling.																	

PROJECT <u>07-1191-0007</u>			RECORD OF BOREHOLE No FR- 4a			1 OF 2 METRIC								
W.P. <u>5541-05-01</u>			LOCATION <u>N 5435615.4 ; E 294706.7</u>			ORIGINATED BY <u>ID</u>								
DIST <u> </u> HWY <u>11</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers/ NW Casing Wash Boring</u>			COMPILED BY <u>DA</u>								
DATUM <u>Geodetic</u>			DATE <u>July 29 and 30, 2009</u>			CHECKED BY <u>AB</u>								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
249.0 0.0	GROUND SURFACE Silty clay to clay, trace organics (FILL) Loose Brown Moist		1	SS	9		248							
			2	SS	9		247							
			3	SS	8		246							
			4	SS	10		245							
244.9 4.1	Silty clay with organics, fine sand seams (ALLUVIUM) Firm to stiff Grey to black Wet		5	SS	7		244							
			6	SS	11		243							
			7	SS	14		242							
242.0 7.0	SAND and GRAVEL, containing cobbles Compact to very dense Grey Wet		8	SS	50/0.08		241							
	Hard augering and spoon bouncing at 8.4 m depth. Switched to NW casing.						240							
							239							
							238							
							237							
		9	SS	33		236								
		10	SS	47		235								
		11	SS	16										

Continued Next Page

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

MIS-MTO-001 FREDERICK_HOUSE_0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

PROJECT <u>07-1191-0007</u>		RECORD OF BOREHOLE No FR- 4a				2 OF 2 METRIC														
W.P. <u>5541-05-01</u>		LOCATION <u>N 5435615.4 ;E 294706.7</u>				ORIGINATED BY <u>ID</u>														
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers/ NW Casing Wash Boring</u>				COMPILED BY <u>DA</u>														
DATUM <u>Geodetic</u>		DATE <u>July 29 and 30, 2009</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					WATER CONTENT (%)			γ				
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 10 20 30			kN/m ³	GR SA SI CL			
--- CONTINUED FROM PREVIOUS PAGE ---																				
230.7	SAND and GRAVEL, containing cobbles Compact to very dense Grey Wet		12	SS	15/0.08		233													
								232												
			13	SS	55															
			14	SS	20/0.15															
230.7	METASEDIMENT (BEDROCK)						231													
18.3	Bedrock cored from 18.3 to 21.7 m depth. For coring details refer to Record of Drillhole FR-4a		1	RC	REC 100%		230										RQD = 79%			
			2	RC	REC 100%		229										RQD = 100%			
			3	RC	REC 100%		228										RQD = 93%			
227.3	End of Borehole																			
21.7	Notes: 1. Sand and gravel and cobbles (up to 150 mm dimension) recovered from core runs 1 to 4. 2. Water level at a depth of 10.7 m (Elev. 238.3 m) and rising upon completion of drilling.																			

MIS-MTO 001 FREDERICK_HOUSE_0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Downing Drilling Ltd.

CHECKED: AB

MIS-RCK 004 FREDERICK HOUSE 0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

RECORD OF BOREHOLE No FR- 5

1 OF 2 **METRIC**

PROJECT 07-1191-0007

W.P. 5541-05-01

LOCATION N 5435611.9 ; E 294739.3

ORIGINATED BY EHS

DIST HWY 11

BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring

COMPILED BY MM

DATUM Geodetic

DATE April 19 to 21, 2009

CHECKED BY AB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										W _P W W _L		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										WATER CONTENT (%)		
255.0	GROUND SURFACE							20	40	60	80	100								
0.0	Clayey silt to silty clay, containing gravelly sand layers (FILL) Soft to stiff Grey to brown Moist		1	AS	-															
			2	SS	7		254													
			3	SS	7		253													
			4	SS	2		252													
	Gravelly sand layers 0.2 m thick at 3.2 m and 3.8 m depth.		5	SS	15															
251.0							251													
4.0	SILTY CLAY, varved Firm to stiff Grey to brown Moist to wet		6	SS	9															
			7	SS	5		250													
			8	SS	WH		249													
							248													
	Dark grey clay laminae 5 mm to 20 mm thick. Light grey silt laminae 10 mm to 50 mm thick.		9	TO	PH		247													
246.5																				
8.5	SILT to Silty SAND, some gravel, trace to some clay Loose to compact Grey Wet		10	SS	9		246													
							245													
			11	SS	7		244													
							243													
			12	SS	7		242													
241.6	Switched to NW Casing at 13.4 m depth.																			
13.4	SAND and GRAVEL, trace to some silt, containing cobbles and boulders Very dense Grey Wet Contains silty sand above 14.1 m depth		13	SS	28		241													

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

MIS-MTO 001 FREDERICK_HOUSE_0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

PROJECT 07-1191-0007		RECORD OF BOREHOLE No FR- 5				2 OF 2		METRIC										
W.P. 5541-05-01		LOCATION N 5435611.9 ; E 294739.3				ORIGINATED BY EHS												
DIST HWY 11		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring				COMPILED BY MM												
DATUM Geodetic		DATE April 19 to 21, 2009				CHECKED BY AB												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100	20 40 60 80 100	10 20 30								
--- CONTINUED FROM PREVIOUS PAGE ---																		
	SAND and GRAVEL, trace to some silt, containing cobbles and boulders Very dense Grey Wet		14	SS	23/0.08		239											
			15	SS	76/0.23		238											
							237											
			16	SS	60/0.15		236											
							235											
							234											
							233											
							232											
							231											
							230											
229.5																		
25.5	METASEDIMENT (BEDROCK) Bedrock cored from 25.5 m to 29.0 m depth. For coring details refer to Record of Drillhole FR- 5.		1	RC	REC 100%		229									RQD = 100%		
			2	RC	REC 73%		228									RQD = 60%		
			3	RC	REC 84%		227									RQD = 67%		
226.0																		
29.0	End of Borehole Note: 1. Water level at a depth of 7.6 m below ground surface (Elev. 247.4 m) upon completion of drilling.						226											

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

RECORD OF BOREHOLE No FR- 6

1 OF 2 **METRIC**

PROJECT 07-1191-0007

W.P. 5541-05-01

LOCATION N 5435607.6 ; E 294757.8

ORIGINATED BY EHS

DIST HWY 11

BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring

COMPILED BY MM

DATUM Geodetic

DATE April 18, 2009



CHECKED BY AB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT		LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
257.5 0.0	GROUND SURFACE Gravelly sand, containing clayey silt to silty clay layers (FILL) Compact to dense Brown Moist		1	AS	-		257							
			2	SS	33		256							
			3	SS	12		255							
	Obstruction encountered at 2.3 m depth.		4	SS	10/0.15		254							
			5	SS	31		253							
253.7 3.8	SILTY CLAY to CLAY, varved Firm to stiff Grey Moist to wet		6	SS	6		252							
			7	SS	WH		251							
	Dark grey clay laminae 5 mm to 10 mm thick. Light grey silty clay laminae 25 mm thick.		8	TO	PH		250							
			9	SS	WH		249							
			10	SS	WH		248							
			11	SS	2		247							
245.6 11.9	SILT, some clay, trace sand Loose Grey Wet		12	SS	9		246							
			13	SS	9		245							
							244							
							243							

Continued Next Page

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

MIS-MTO 001 FREDERICK_HOUSE_0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

PROJECT <u>07-1191-0007</u>				RECORD OF BOREHOLE No FR- 6				2 OF 2 METRIC										
W.P. <u>5541-05-01</u>		LOCATION <u>N 5435607.6 ;E 294757.8</u>				ORIGINATED BY <u>EHS</u>												
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MM</u>												
DATUM <u>Geodetic</u>		DATE <u>April 18, 2009</u>				CHECKED BY <u>AB</u>												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)					
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>						
242.3						242												
15.2	SAND and GRAVEL		14	SS	14													
241.7	Compact Grey Wet																	
15.8	End of Borehole																	
Notes: 1. Obstruction encountered at 2.3 m depth. Moved borehole 2.5 m N. 2. Water level measured in piezometer at a depth of 11.4 m below ground surface (Elev. 246.1 m) on April 19, 2009. 3. Water level measured in piezometer at a depth of 13.9 m below ground surface (Elev. 243.6 m) on April 28, 2009 and at a depth of 11.9 m depth (Elev. 245.6 m) on July 31, 2009.																		

PROJECT <u>07-1191-0007</u>			RECORD OF BOREHOLE No FR- 7			1 OF 1 METRIC																							
W.P. <u>5541-05-01</u>			LOCATION <u>N 5435627.1 ; E 294533.0</u>			ORIGINATED BY <u>MR</u>																							
DIST <u> </u> HWY <u>11</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>			COMPILED BY <u>MM</u>																							
DATUM <u>Geodetic</u>			DATE <u>April 27, 2009</u>			CHECKED BY <u>AB</u>																							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			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	W _p	W	W _L	20	40	60	80	100	10	20	30	γ	GR	SA	SI	CL
252.0	0.0	GROUND SURFACE																											
		Organics, some sand (FILL) Brown Moist		1	AS	-																							
251.4	0.6	Silty clay, trace sand, trace organics (FILL) Firm Brown		2	SS	7		251																					
250.6	1.4	SILTY CLAY to CLAY, varved Soft to firm Grey Moist to wet		3	SS	5		250																					
				4	SS	4		249																					
				5	SS	4		248																					
				6	SS	2		247																					
				7	SS	3		246																					
		Less than 5 mm thick silt to clayey silt varves.		8	TO	PH		245																17.9					
244.4	7.6	SILT, some sand, trace to some clay Loose to compact Grey Wet		9	SS	7		244																					
				10	SS	16		243																					
				11	SS	12		242																					
				12	SS	12		241																					
239.2	12.8	End of Borehole						240																					
		Note: 1. Water level at a depth of 8.3 m below ground surface (Elev. 243.7 m) upon completion of drilling.																											

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

MIS-MTO 001 FREDERICK_HOUSE_0711910007_METRIC.GPJ GAL-MASS.GDT 20/1/10



PROJECT 07-1191-0007		RECORD OF BOREHOLE No FR- 8		1 OF 2 METRIC	
W.P. 5541-05-01		LOCATION N 5435599.3 ; E 294545.4		ORIGINATED BY MR	
DIST HWY 11		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY MM	
DATUM Geodetic		DATE April 28, 2009		CHECKED BY AB	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	*N VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100	W _p W W _L					
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							
258.5	GROUND SURFACE													
0.0	Sand and gravel (FILL) Brown Moist		1	AS	-									
257.9														
0.6	Clayey silt, trace sand (FILL) Firm to very stiff Grey to brown Moist		2	SS	10									
			3	SS	19									
			4	SS	12									
			5	SS	10									
			6	SS	12									
			7	SS	8									
		8	SS	16										
250.9														
7.6	SILTY CLAY to CLAY, varved Firm to stiff Grey Wet		9	SS	4									
			10	TO	PH									
			11	SS	WH									
			12	SS	WH									
		13	SS	1										






Continued Next Page

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

MIS-MTO 001 FREDERICK_HOUSE_0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

PROJECT <u>07-1191-0007</u>			RECORD OF BOREHOLE No FR- 8			2 OF 2 METRIC													
W.P. <u>5541-05-01</u>			LOCATION <u>N 5435599.3 ;E 294545.4</u>			ORIGINATED BY <u>MR</u>													
DIST <u> </u> HWY <u>11</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>			COMPILED BY <u>MM</u>													
DATUM <u>Geodetic</u>			DATE <u>April 28, 2009</u>			CHECKED BY <u>AB</u>													
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)						
							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 10 20 30							
--- CONTINUED FROM PREVIOUS PAGE ---																			
243.3	15.2		14	SS	6		243												0 10 77 13
							242												
			15	SS	12		241												
							240												
239.6			16	SS	26														
18.9	End of Borehole Note: 1. Water level at a depth of 15.9 m below ground surface (Elev. 242.6 m) upon completion of drilling.																		

PROJECT <u>07-1191-0007</u>		RECORD OF BOREHOLE No FR- 9		1 OF 1 METRIC	
W.P. <u>5541-05-01</u>		LOCATION <u>N 5435641.9 ; E 294587.4</u>		ORIGINATED BY <u>ID</u>	
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>DA</u>	
DATUM <u>Geodetic</u>		DATE <u>July 30, 2009</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × REMOULDED</div></div>					<div><div>102030</div><div>W_p W W_L</div></div>				
								<div><div>20406080100</div><div></div><div></div></div>					<div><div>102030</div><div></div><div></div></div>				
245.2	GROUND SURFACE																
0.0	TOPSOIL		1	SS	6		245										
244.7																	
0.5	SAND, trace gravel Loose Brown Wet		2	SS	7		244										
243.4																	
1.8	SILTY CLAY to CLAYEY SILT, varved Soft to stiff Grey Wet		3	SS	8		243										
			4	SS	6												
			5	SS	3		242										
241.1																	
4.1	SILT to Sandy SILT Loose Grey Wet		6	SS	8		241										
							240										
			7	SS	9		239										
238.5																	
6.7	End of Borehole Notes: 1. Harder augering below 4.1 m depth. 2. Water level at a depth of 1.5 m (Elev. 243.7 m) upon completion of drilling																

MIS-MTO 001 FREDERICK_HOUSE_0711910007_METRIC.GPJ GAL-MISS.GDT 20/1/10

PROJECT <u>07-1191-0007</u>		RECORD OF BOREHOLE No FR- 10		1 OF 1 METRIC	
W.P. <u>5541-05-01</u>		LOCATION <u>N 5435641.5 ;E 294567.5</u>		ORIGINATED BY <u>ID</u>	
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>DA</u>	
DATUM <u>Geodetic</u>		DATE <u>July 31, 2009</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								<div><div><div></div><div></div><div></div><div></div><div></div></div><div>20406080100</div></div>					W _p	W	W _L		
246.9	GROUND SURFACE																
0.0	TOPSOIL																
0.2	Silty SAND with clay pockets Loose Brown Wet		1	SS	4												
245.4																	
1.5	SILTY CLAY to CLAYEY SILT, trace sand, trace gravel Stiff Grey Wet		2	SS	11												
			3	SS	14												
			4	SS	15												
242.0			5	SS	10												
4.9	SILT, trace sand Compact Grey Wet																
			6	SS	19												
240.2																	
6.7	End of Borehole Note: 1. Water level at a depth of 4.4 m (Elev. 242.5 m) and rising, upon completion of drilling.																



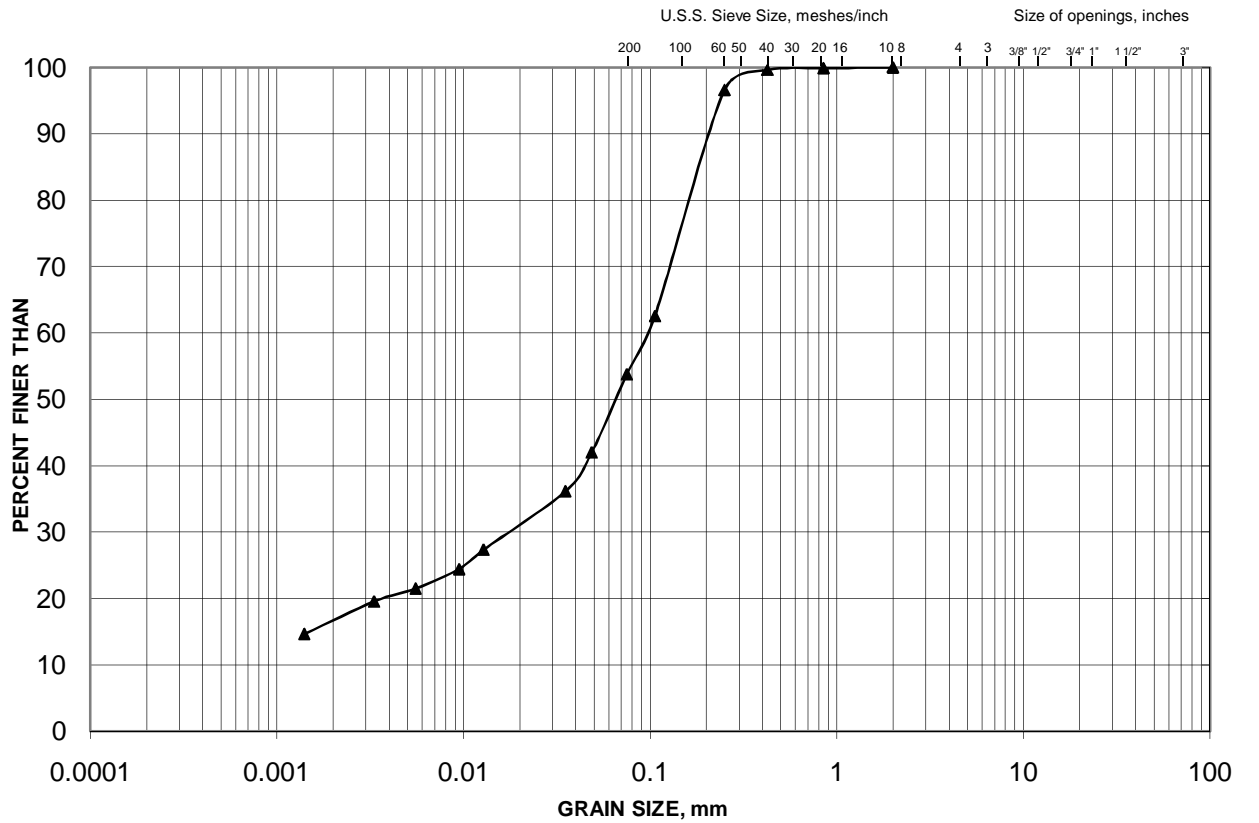
APPENDIX B

LABORATORY TEST RESULTS

GRAIN SIZE DISTRIBUTION

Alluvium

**FIGURE
B-1**



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

LEGEND

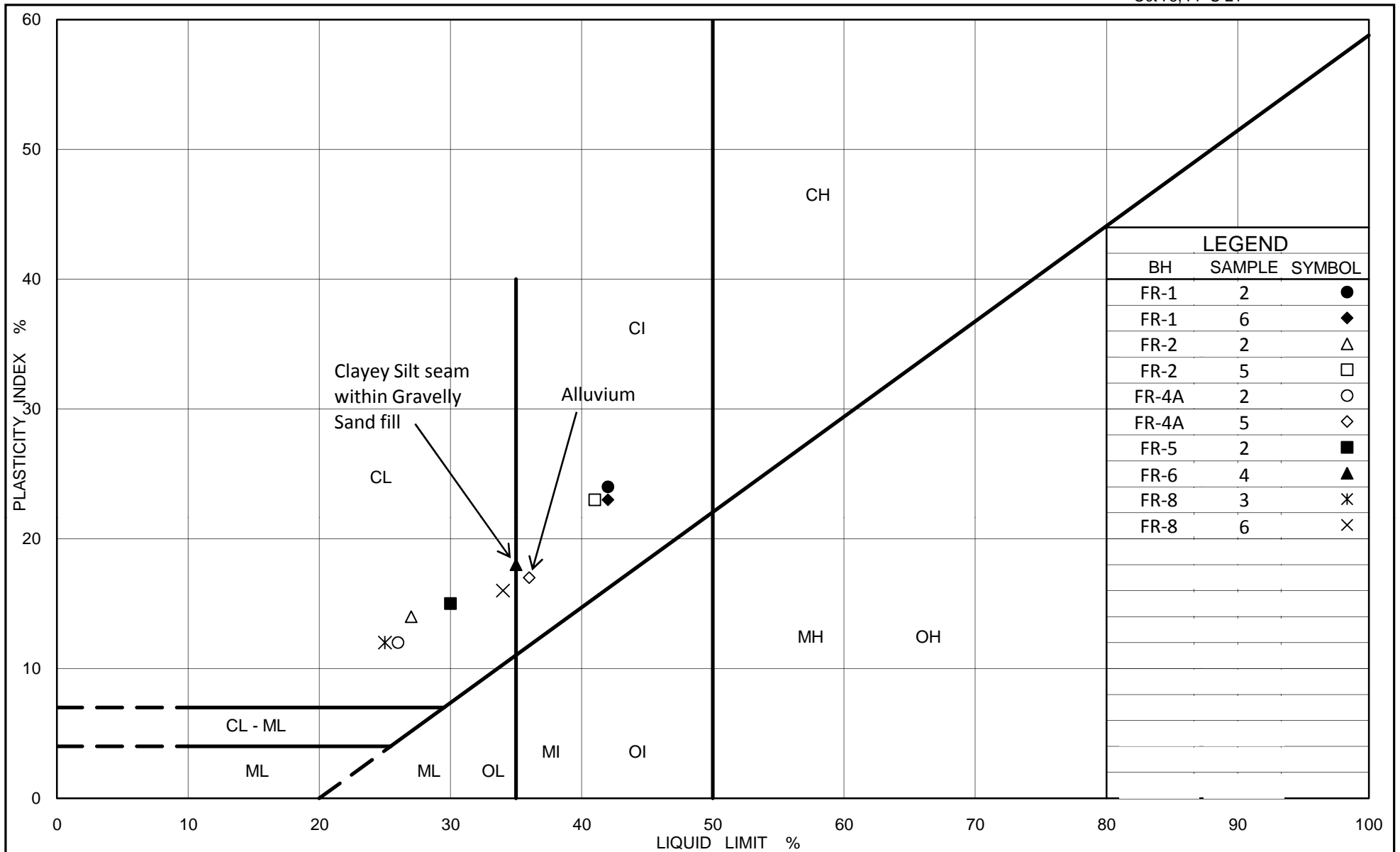
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
▲	FR-3	2	243.4

Project Number: 07-1191-0007-FR

Checked By: SEMC

Golder Associates

Date: January 2010



Ministry of Transportation

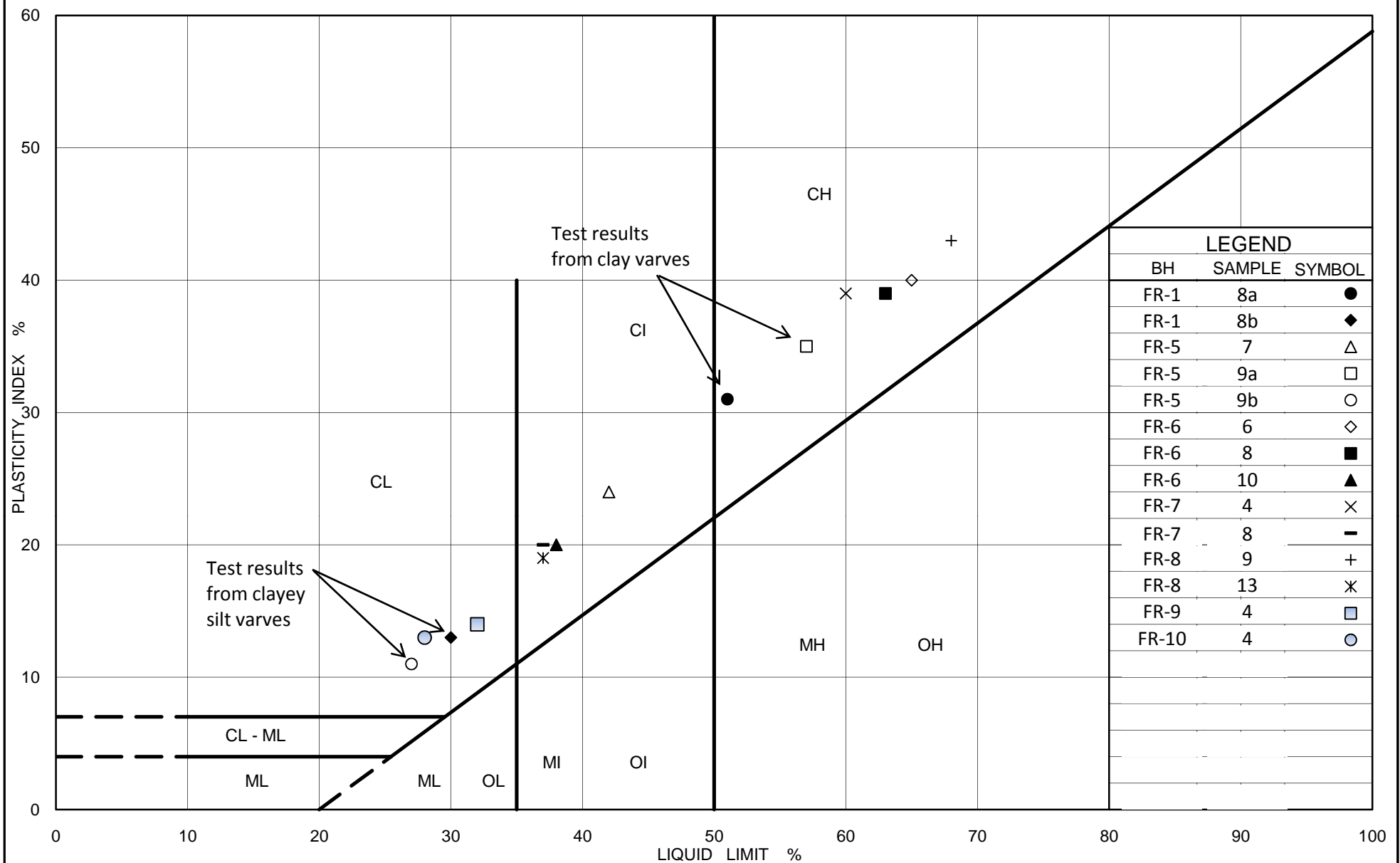
Ontario

PLASTICITY CHART Clayey Silt to Silty Clay (Fill/Alluvium)

FIG No. B-2

Project No. 07-1191-0007-FR

Checked By: SEMC



Ministry of Transportation

Ontario

PLASTICITY CHART

Silty Clay to Clay

FIG No. B-3

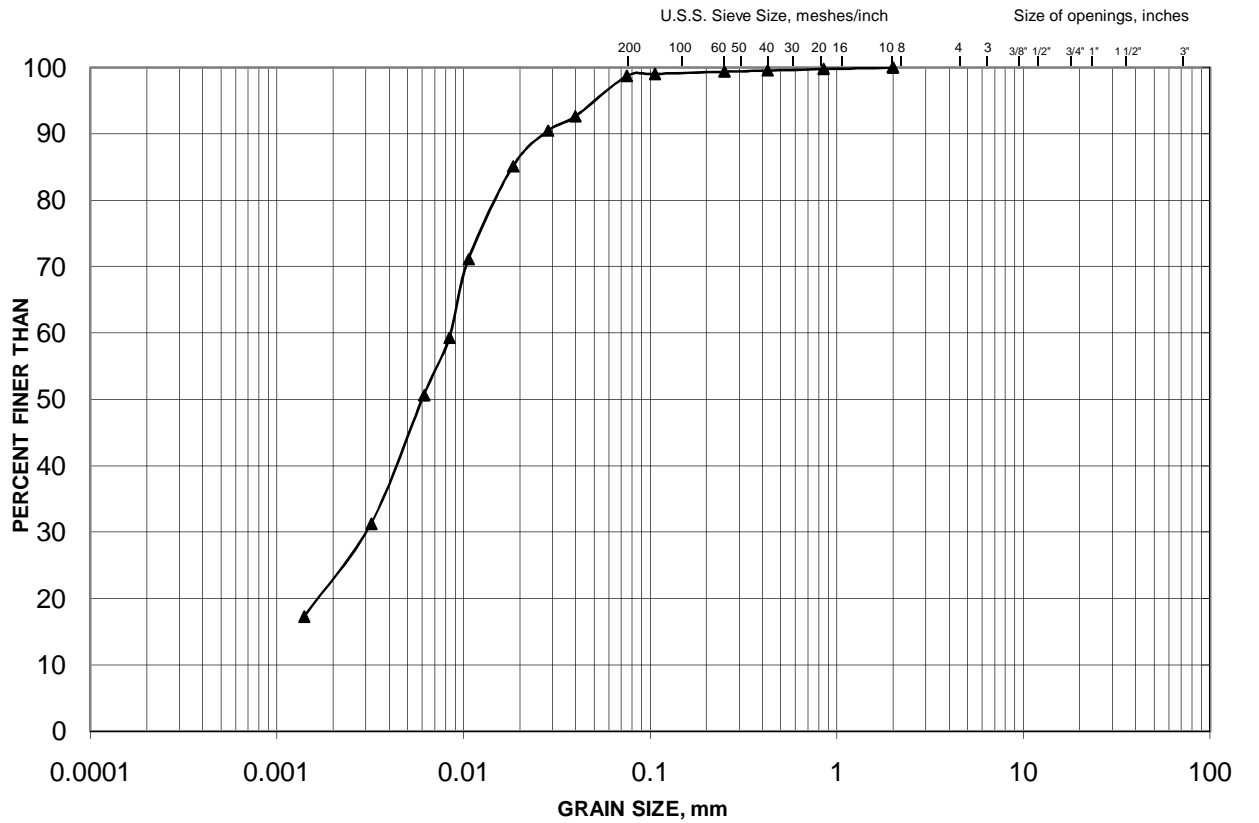
Project No. 07-1191-0007-FR

Checked By: SEMC

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE
B-4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
▲	FR-1	8	243.5

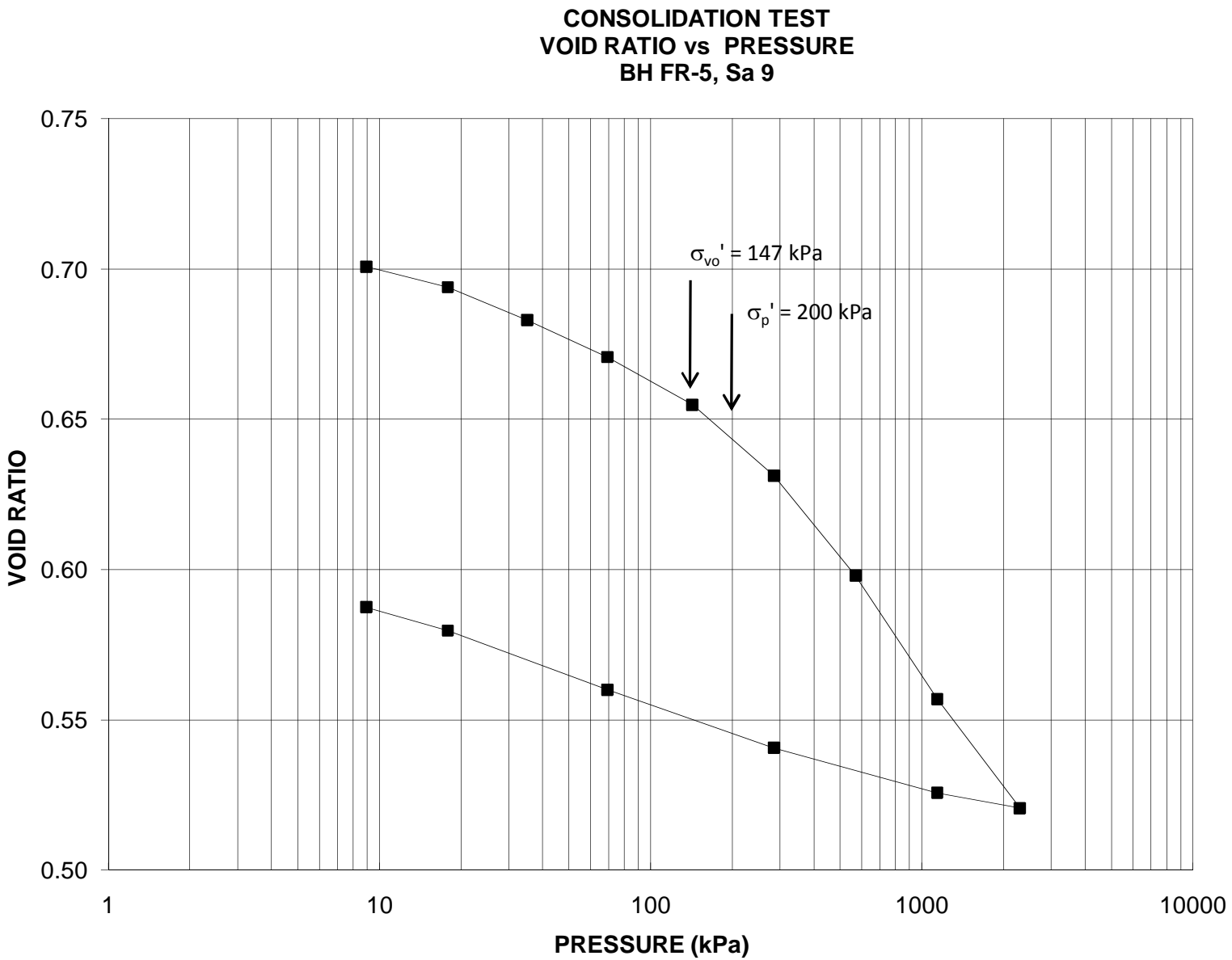
Project Number: 07-1191-0007-FR

Checked By: SEMC

Golder Associates

Date: January 2010

OEDOMETER CONSOLIDATION SUMMARY						FIGURE B-5 Page 1 of 4		
SAMPLE IDENTIFICATION								
Project Number		07-1191-0007-FR		Borehole, Sample		FR-5, 9		
				Sample Depth, (m)		7.9		
TEST CONDITIONS								
Test Type		Standard		Load Duration, hr		24		
Oedometer Number		1						
Date Started		April 29/09						
Date Completed		May 14/09						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		2.538		Unit Weight, kN/m ³		20.0		
Sample Diameter, cm		6.342		Dry Unit Weight, kN/m ³		15.5		
Area, cm ²		31.59		Specific Gravity, assumed		2.7		
Volume, cm ³		80.17		Solids Height, cm		1.486		
Water Content, %		28.9		Volume of Solids, cm ³		46.96		
Wet Mass, g		163.48		Volume of Voids, cm ³		33.22		
Dry Mass, g		126.78		Degree of Saturation, %		110.5		
TEST COMPUTATIONS								
Pressure	Primary Consolidation	Corr. Height	Void	Average Height	t ₅₀	cv.	m _v	k
kPa	mm	cm	Ratio	cm	s	cm ² /s	m ² /MN	cm/s
0.0	0	2.538	0.707	2.538				
8.9	0.10	2.528	0.701	2.533	180	0.00699	0.437	2.992E-07
17.9	0.10	2.518	0.694	2.523	400	0.00312	0.452	1.382E-07
35.1	0.16	2.502	0.683	2.510	300	0.00412	0.376	1.519E-07
69.2	0.18	2.483	0.671	2.493	400	0.00304	0.213	6.374E-08
142.6	0.24	2.460	0.655	2.472	250	0.00479	0.129	6.078E-08
284.9	0.35	2.425	0.631	2.442	230	0.00508	0.100	4.986E-08
570.5	0.50	2.375	0.598	2.400	250	0.00452	0.071	3.167E-08
1139.6	0.61	2.314	0.557	2.345	140	0.00770	0.045	3.407E-08
2300.0	0.54	2.260	0.521	2.287	80	0.01282	0.020	2.529E-08
1139.6	-0.08	2.268	0.526	2.264				
284.9	-0.22	2.290	0.541	2.279				
69.2	-0.29	2.319	0.560	2.305				
17.9	-0.29	2.348	0.580	2.334				
8.9	-0.12	2.360	0.587	2.354				
Notes: k calculated using cv based on t ₅₀ values.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		2.360		Unit Weight, kN/m ³		20.6		
Sample Diameter, cm		6.342		Dry Unit Weight, kN/m ³		16.7		
Area, cm ²		31.59		Specific Gravity, assumed		2.7		
Volume, cm ³		74.54		Solids Height, cm		1.486		
Water Content, %		23.8		Volume of Solids, cm ³		46.96		
Wet Mass, g		156.90		Volume of Voids, cm ³		27.59		
Dry Mass, g		126.78		Degree of Saturation, %		110.7		
Prepared By: SL			Golder Associates			Checked By: AB		

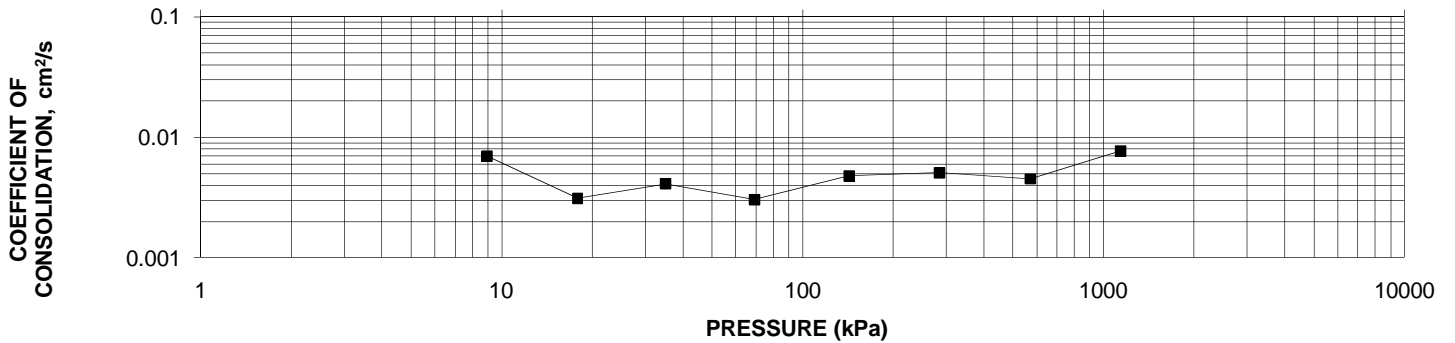


OEDOMETER CONSOLIDATION SUMMARY

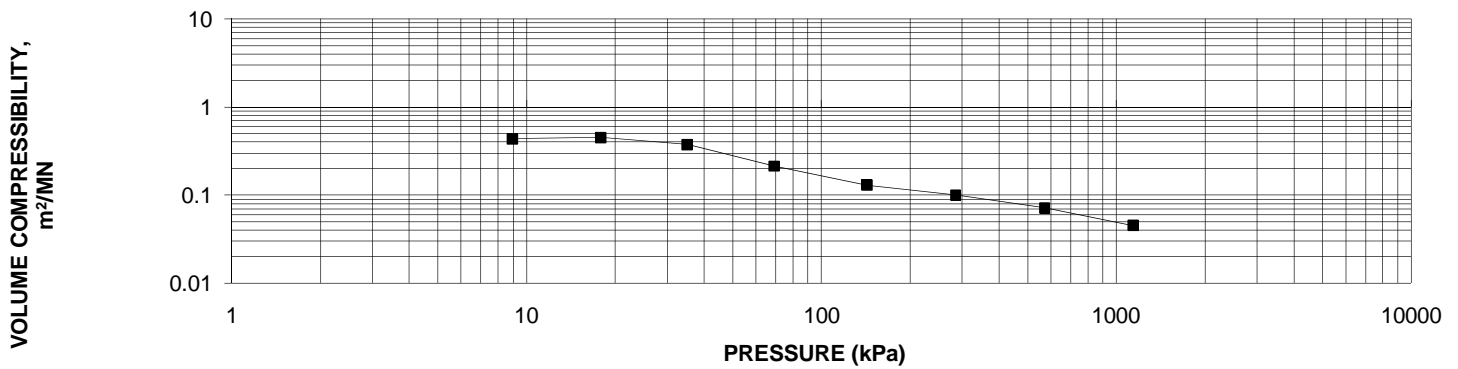
FIGURE B-5

Page 3 of 4

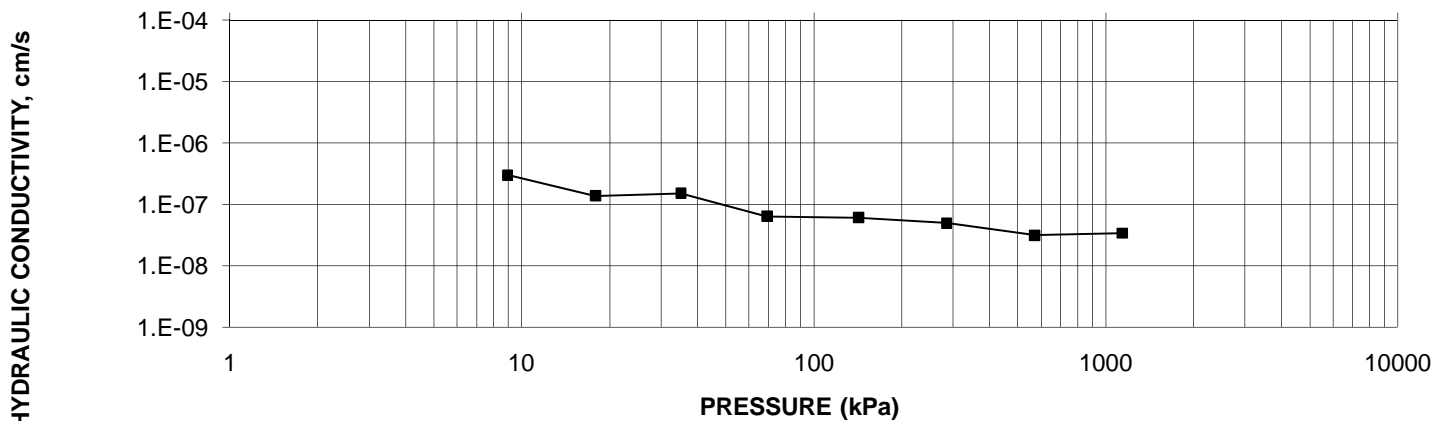
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH FR-5, Sa 9

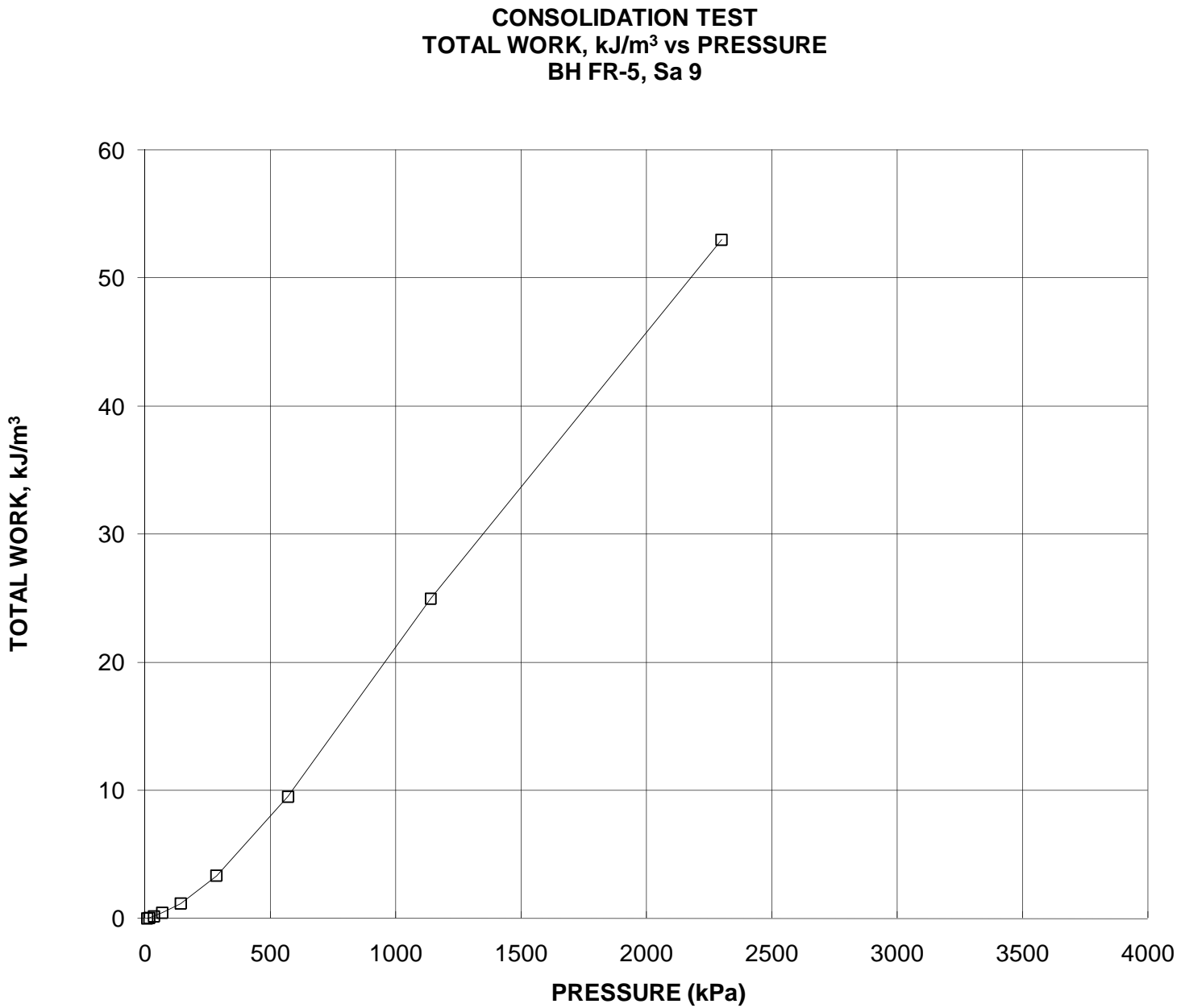


CONSOLIDATION TEST
MV m²/MN vs PRESSURE (kPa)
BH FR-5, Sa 9

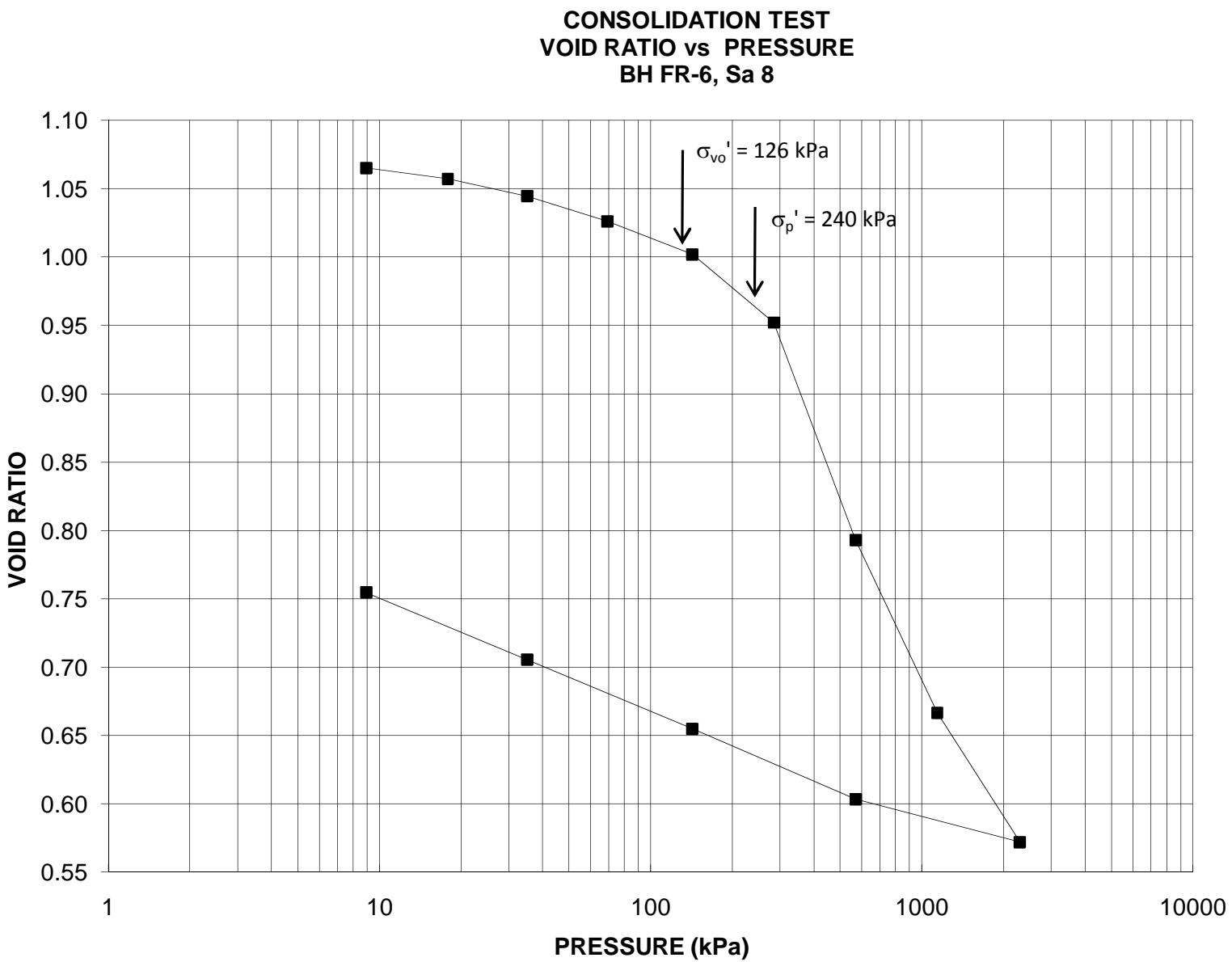


CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH FR-5, Sa 9





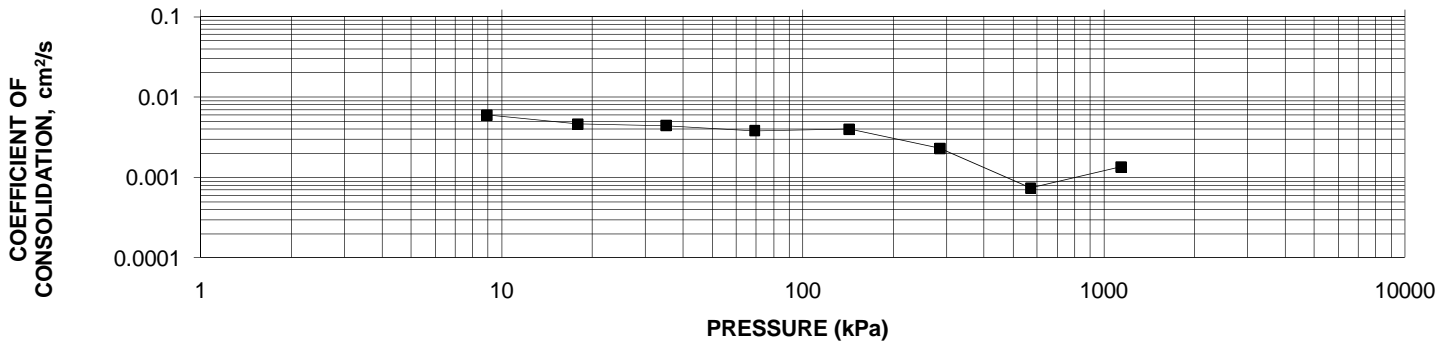
OEDOMETER CONSOLIDATION SUMMARY						FIGURE B-6 Page 1 of 4		
SAMPLE IDENTIFICATION								
Project Number		07-1191-0007-FR		Borehole, Sample		FR-6, 8		
				Sample Depth, (m)		6.4		
TEST CONDITIONS								
Test Type		Standard		Load Duration, hr		24		
Oedometer Number		1						
Date Started		June 1/09						
Date Completed		June 15/09						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		2.538		Unit Weight, kN/m ³		17.6		
Sample Diameter, cm		6.342		Dry Unit Weight, kN/m ³		12.8		
Area, cm ²		31.59		Specific Gravity, assumed		2.7		
Volume, cm ³		80.17		Solids Height, cm		1.226		
Water Content, %		37.3		Volume of Solids, cm ³		38.72		
Wet Mass, g		143.54		Volume of Voids, cm ³		41.46		
Dry Mass, g		104.54		Degree of Saturation, %		94.1		
TEST COMPUTATIONS								
Pressure	Primary Consolidation	Corr. Height	Void Ratio	Average Height	t ₅₀	cv.	m _v	k
kPa	mm	cm	Ratio	cm	s	cm ² /s	m ² /MN	cm/s
0.0	0	2.538	1.071	2.538				
8.9	0.07	2.531	1.065	2.535	210	0.00600	0.300	1.764E-07
17.9	0.10	2.522	1.057	2.526	270	0.00463	0.429	1.950E-07
35.1	0.16	2.506	1.045	2.514	280	0.00442	0.360	1.560E-07
69.2	0.23	2.483	1.026	2.495	320	0.00381	0.263	9.851E-08
142.6	0.30	2.454	1.002	2.469	300	0.00398	0.163	6.379E-08
284.9	0.61	2.393	0.952	2.423	500	0.00230	0.175	3.945E-08
570.5	1.95	2.198	0.793	2.295	1400	0.00074	0.285	2.065E-08
1139.6	1.55	2.043	0.667	2.120	650	0.00136	0.124	1.648E-08
2300.0	1.16	1.927	0.572	1.985	400	0.00193	0.049	9.266E-09
570.5	-0.39	1.965	0.603	1.946				
142.6	-0.63	2.028	0.655	1.997				
35.1	-0.62	2.090	0.705	2.059				
8.9	-0.60	2.151	0.755	2.120				
Notes:								
k calculated using cv based on t ₅₀ values.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		2.151		Unit Weight, kN/m ³		18.6		
Sample Diameter, cm		6.342		Dry Unit Weight, kN/m ³		15.1		
Area, cm ²		31.59		Specific Gravity, assumed		2.7		
Volume, cm ³		67.94		Solids Height, cm		1.226		
Water Content, %		23.3		Volume of Solids, cm ³		38.72		
Wet Mass, g		128.90		Volume of Voids, cm ³		29.22		
Dry Mass, g		104.54		Degree of Saturation, %		83.4		
Prepared By: SL			Golder Associates			Checked By: AB		



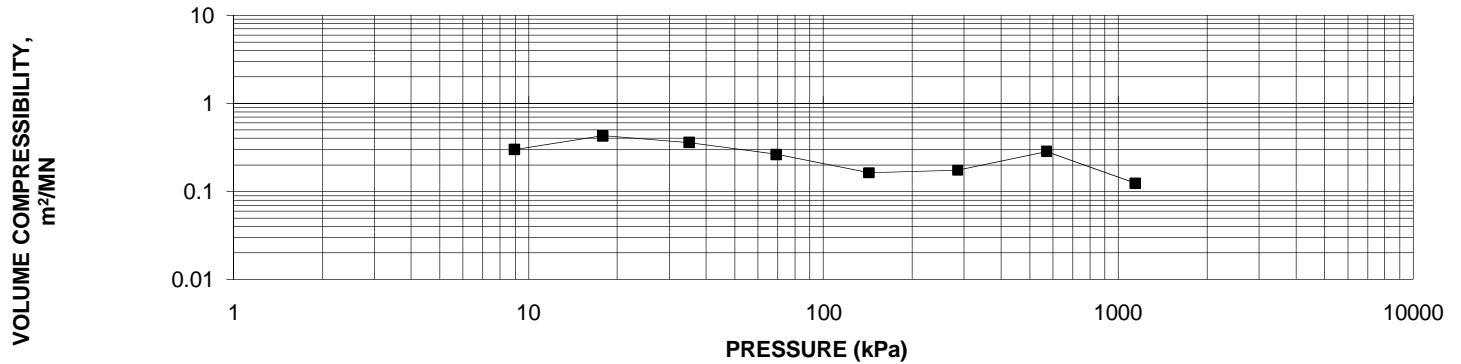
OEDOMETER CONSOLIDATION SUMMARY

FIGURE B-6
Page 3 of 4

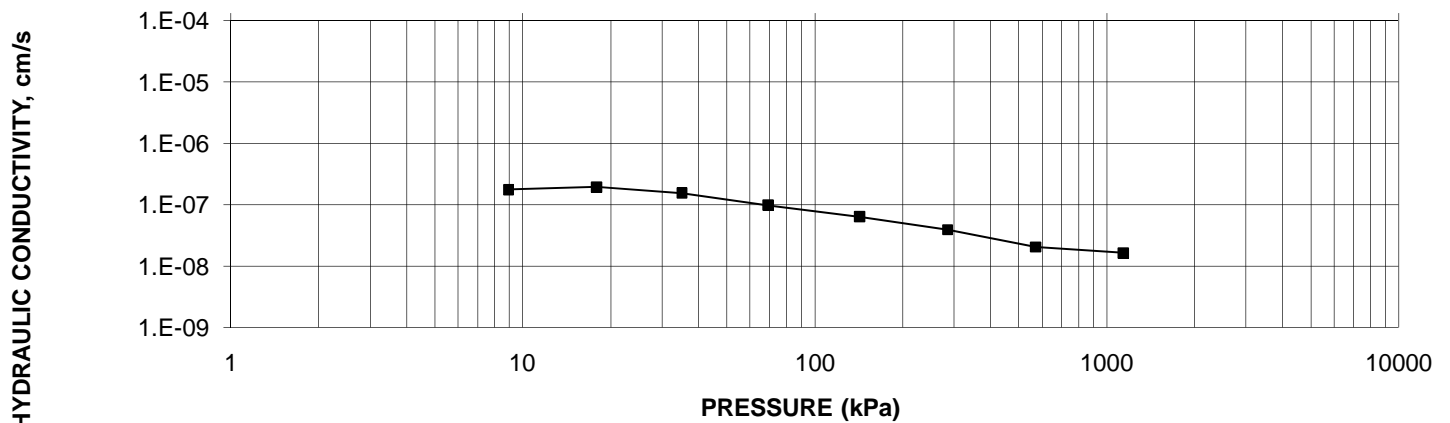
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH FR-6, Sa 8

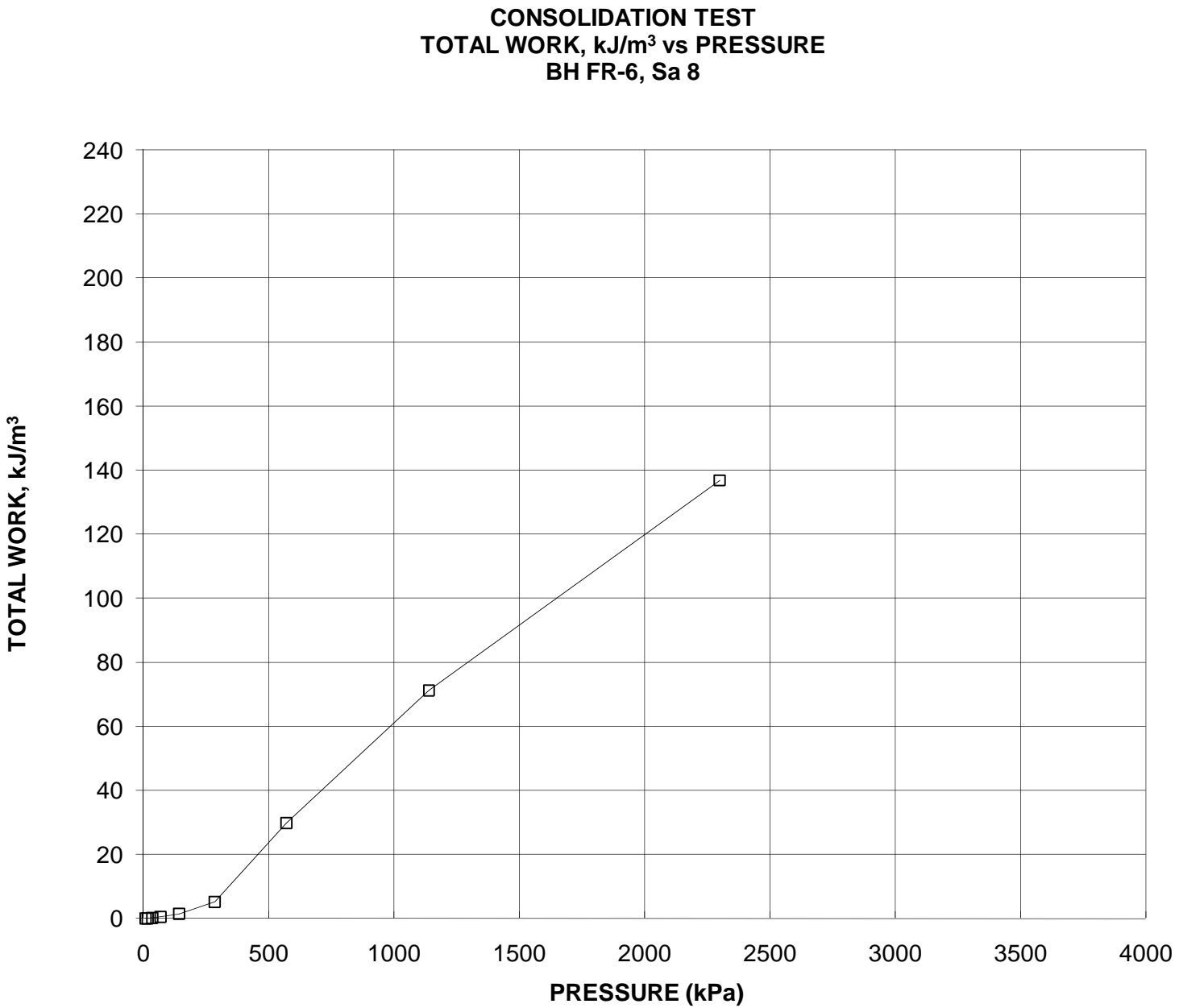


CONSOLIDATION TEST
MV m²/MN vs PRESSURE (kPa)
BH FR-6, Sa 8

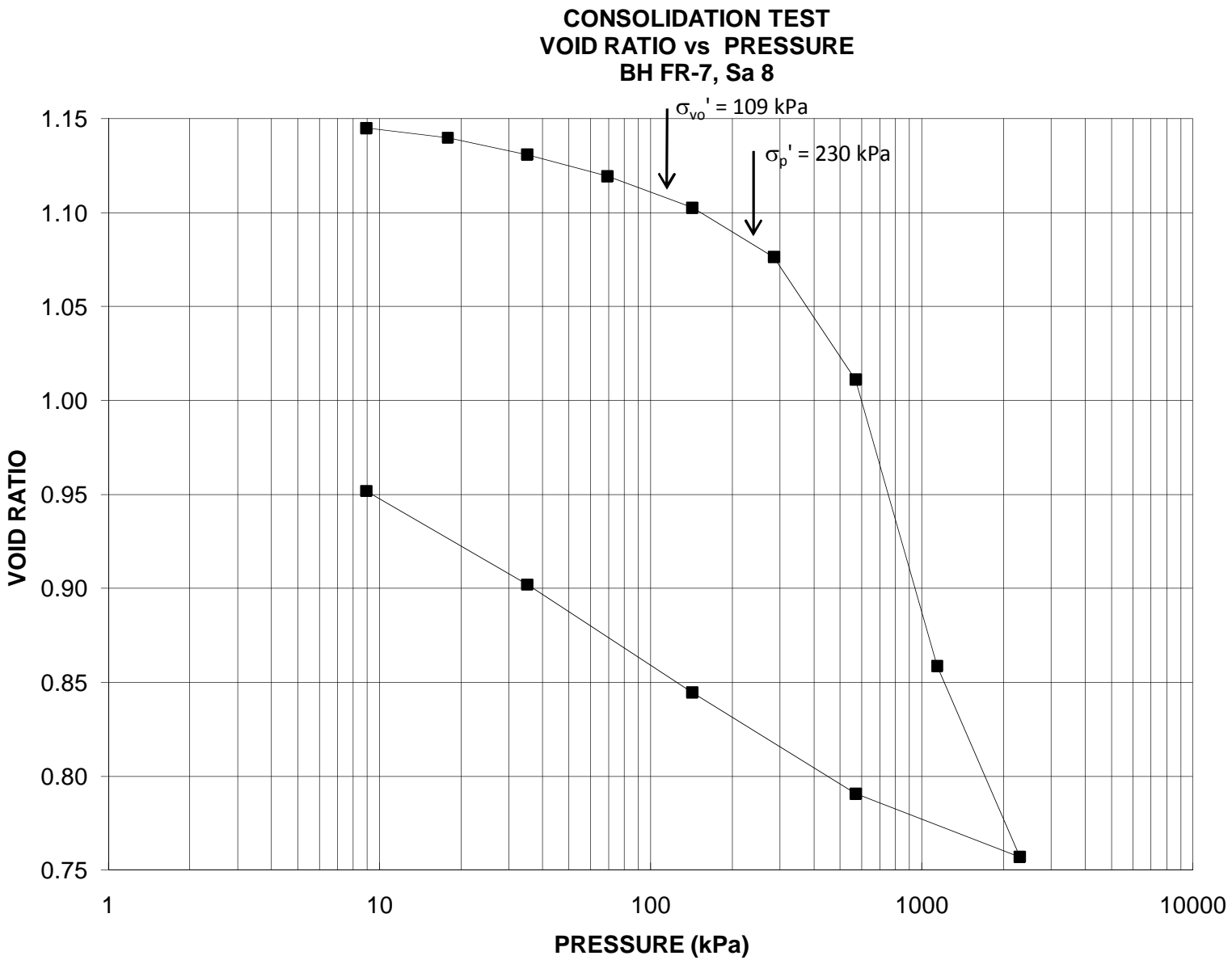


CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH FR-6, Sa 8





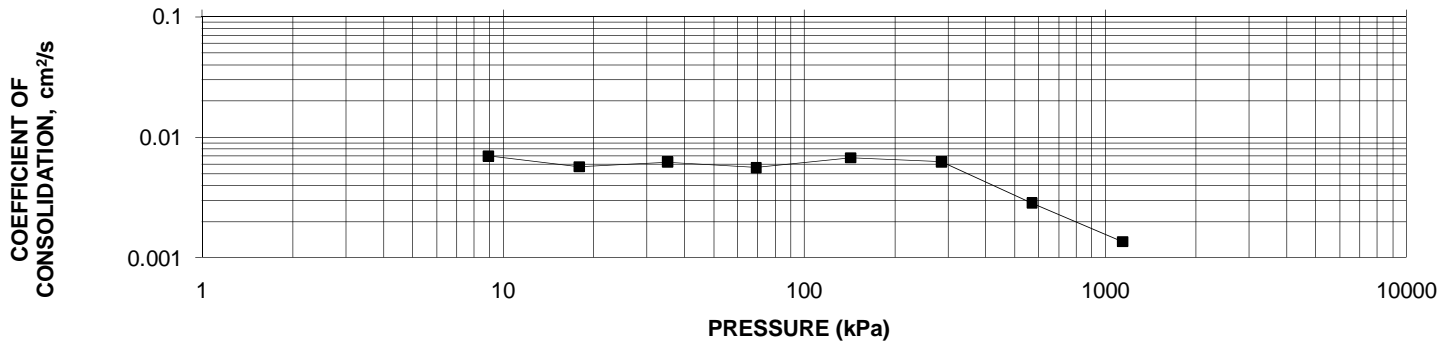
OEDOMETER CONSOLIDATION SUMMARY						FIGURE B-7 Page 1 of 4		
SAMPLE IDENTIFICATION								
Project Number		07-1191-0007-FR		Borehole, Sample		FR-7, 8		
				Sample Depth, (m)		6.3		
TEST CONDITIONS								
Test Type		Standard		Load Duration, hr		24		
Oedometer Number		1						
Date Started		May 14/09						
Date Completed		May 28/09						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		2.538		Unit Weight, kN/m ³		17.9		
Sample Diameter, cm		6.342		Dry Unit Weight, kN/m ³		12.3		
Area, cm ²		31.59		Specific Gravity, assumed		2.7		
Volume, cm ³		80.17		Solids Height, cm		1.181		
Water Content, %		45.4		Volume of Solids, cm ³		37.29		
Wet Mass, g		146.43		Volume of Voids, cm ³		42.88		
Dry Mass, g		100.69		Degree of Saturation, %		106.7		
TEST COMPUTATIONS								
Pressure	Primary Consolidation	Corr. Height	Void	Average Height	t ₅₀	cv.	m _v	k
kPa	mm	cm	Ratio	cm	s	cm ² /s	m ² /MN	cm/s
0.0	0.00	2.538	1.150	2.538				
8.9	0.06	2.532	1.145	2.535	180	0.00700	0.256	1.756E-07
17.9	0.06	2.526	1.140	2.529	220	0.00570	0.265	1.483E-07
35.1	0.11	2.516	1.131	2.521	200	0.00623	0.244	1.490E-07
69.2	0.14	2.502	1.119	2.509	220	0.00561	0.159	8.725E-08
142.6	0.20	2.482	1.103	2.492	180	0.00676	0.108	7.147E-08
284.9	0.31	2.451	1.076	2.467	190	0.00628	0.088	5.404E-08
570.5	0.77	2.374	1.011	2.413	400	0.00285	0.110	3.078E-08
1139.6	1.80	2.194	0.859	2.284	750	0.00136	0.133	1.782E-08
2300.0	1.20	2.074	0.757	2.134	320	0.00279	0.047	1.290E-08
570.5	-0.40	2.114	0.791	2.094				
142.6	-0.64	2.178	0.845	2.146				
35.1	-0.68	2.245	0.902	2.212				
8.9	-0.59	2.304	0.952	2.275				
Notes: k calculated using cv based on t ₅₀ values.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		2.304		Unit Weight, kN/m ³		18.7		
Sample Diameter, cm		6.342		Dry Unit Weight, kN/m ³		13.6		
Area, cm ²		31.59		Specific Gravity, assumed		2.7		
Volume, cm ³		72.79		Solids Height, cm		1.181		
Water Content, %		37.5		Volume of Solids, cm ³		37.29		
Wet Mass, g		138.41		Volume of Voids, cm ³		35.50		
Dry Mass, g		100.69		Degree of Saturation, %		106.3		
Prepared By: SL			Golder Associates			Checked By: AB		



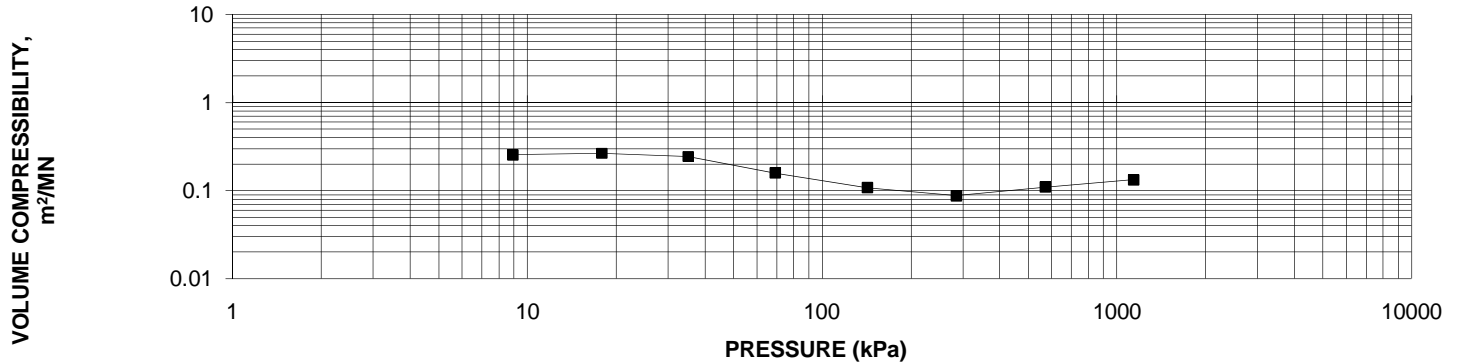
OEDOMETER CONSOLIDATION SUMMARY

FIGURE B-7
Page 3 of 4

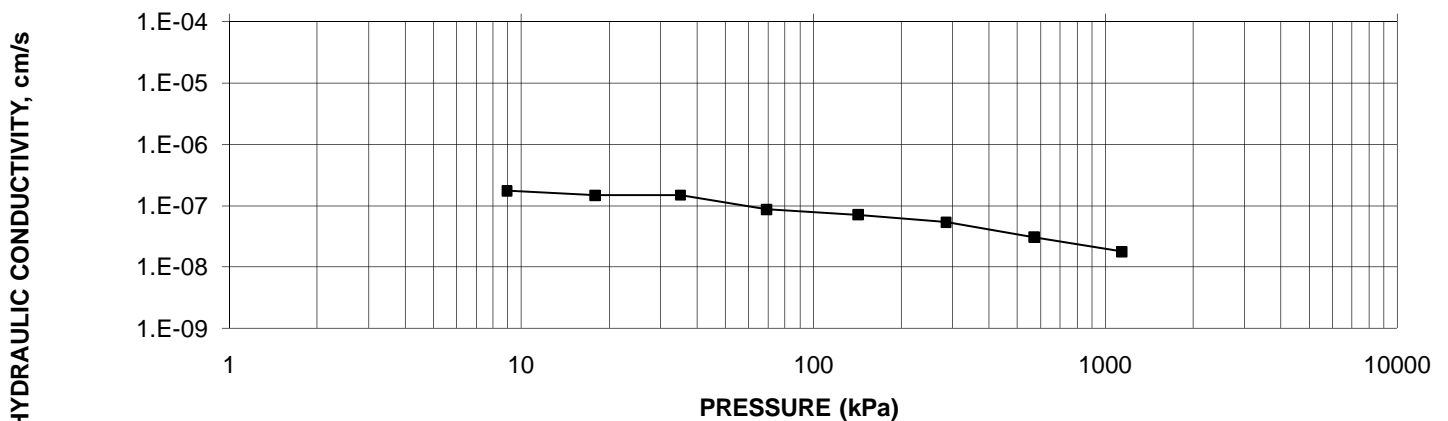
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH FR-7, Sa 8

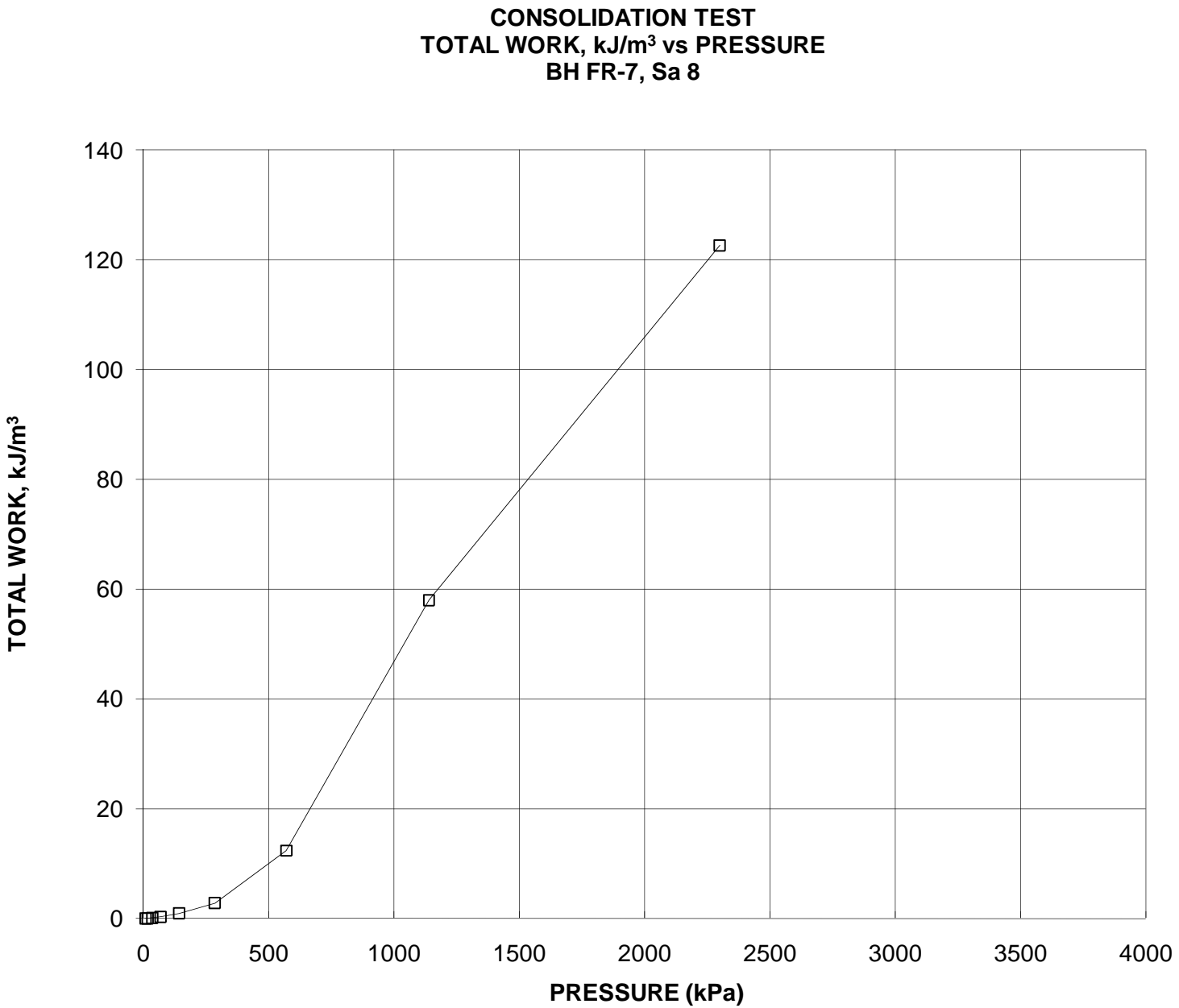


CONSOLIDATION TEST
MV m²/MN vs PRESSURE (kPa)
BH FR-7, Sa 8



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH FR-7, Sa 8

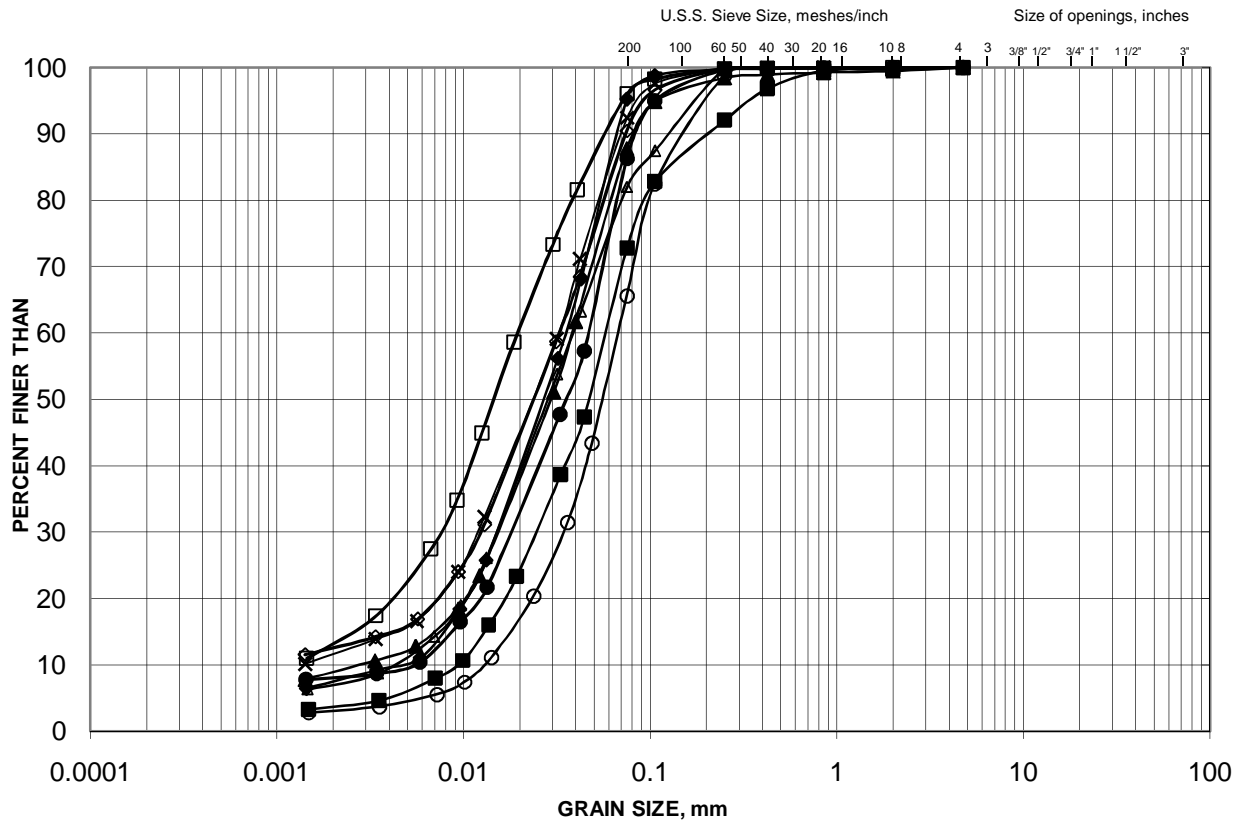




GRAIN SIZE DISTRIBUTION

Silt to Sand and Silt

FIGURE
B-8a



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
△	FR-1	9	241.9
◆	FR-2	7	242.9
■	FR-2	11	236.8
▲	FR-3	6	240.4
○	FR-3	10	235.1
×	FR-5	10	245.6
□	FR-6	12	245.0
●	FR-7	10	242.6
◇	FR-8	15	241.4

Project Number: 07-1191-0007-FR

Checked By: SEMC

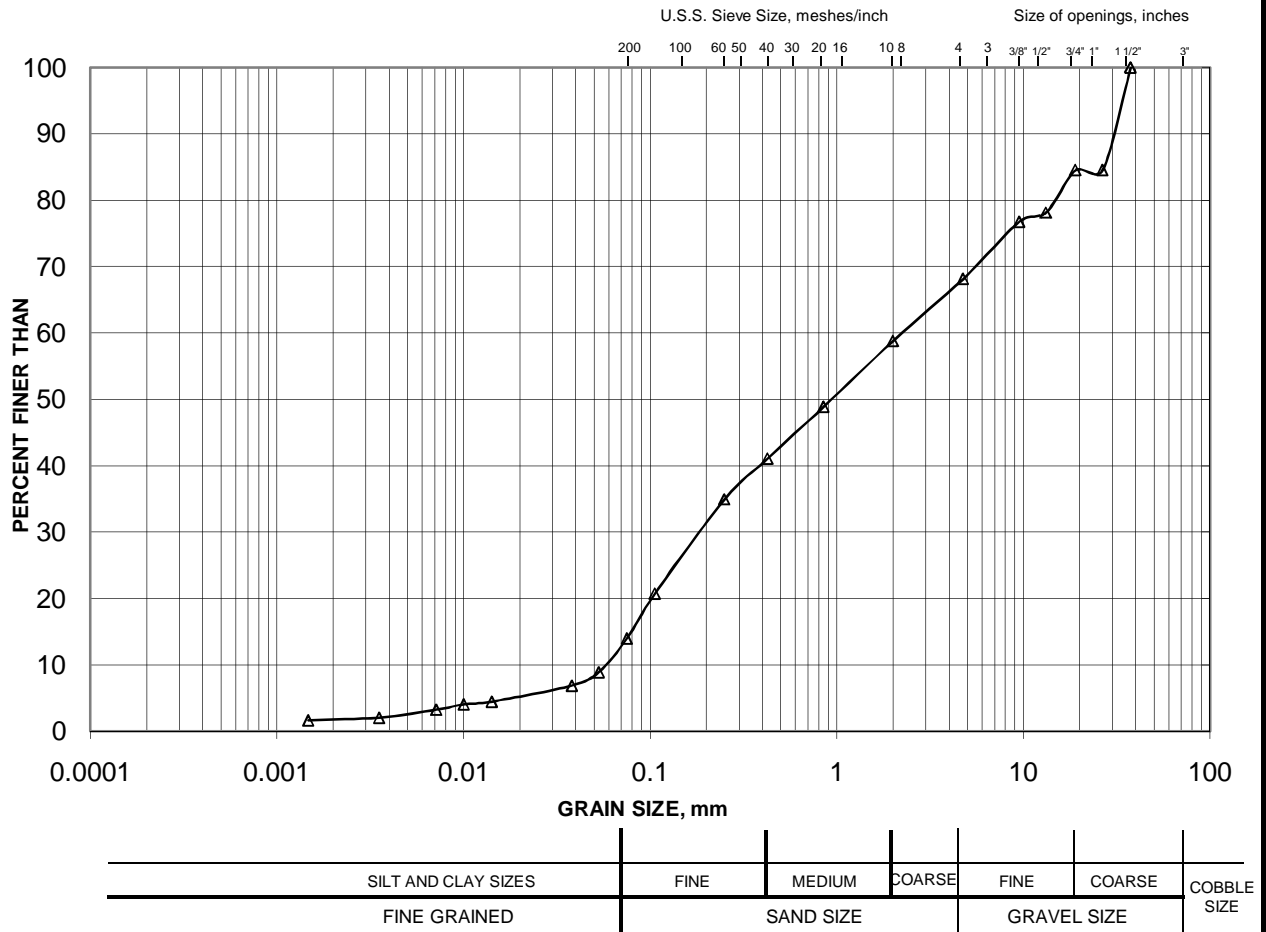
Golder Associates

Date: January 2010

GRAIN SIZE DISTRIBUTION

Sand and Gravel Interlayers

FIGURE
B-8b



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—▲—	FR-3	7	239.6

Project Number: 07-1191-0007-FR

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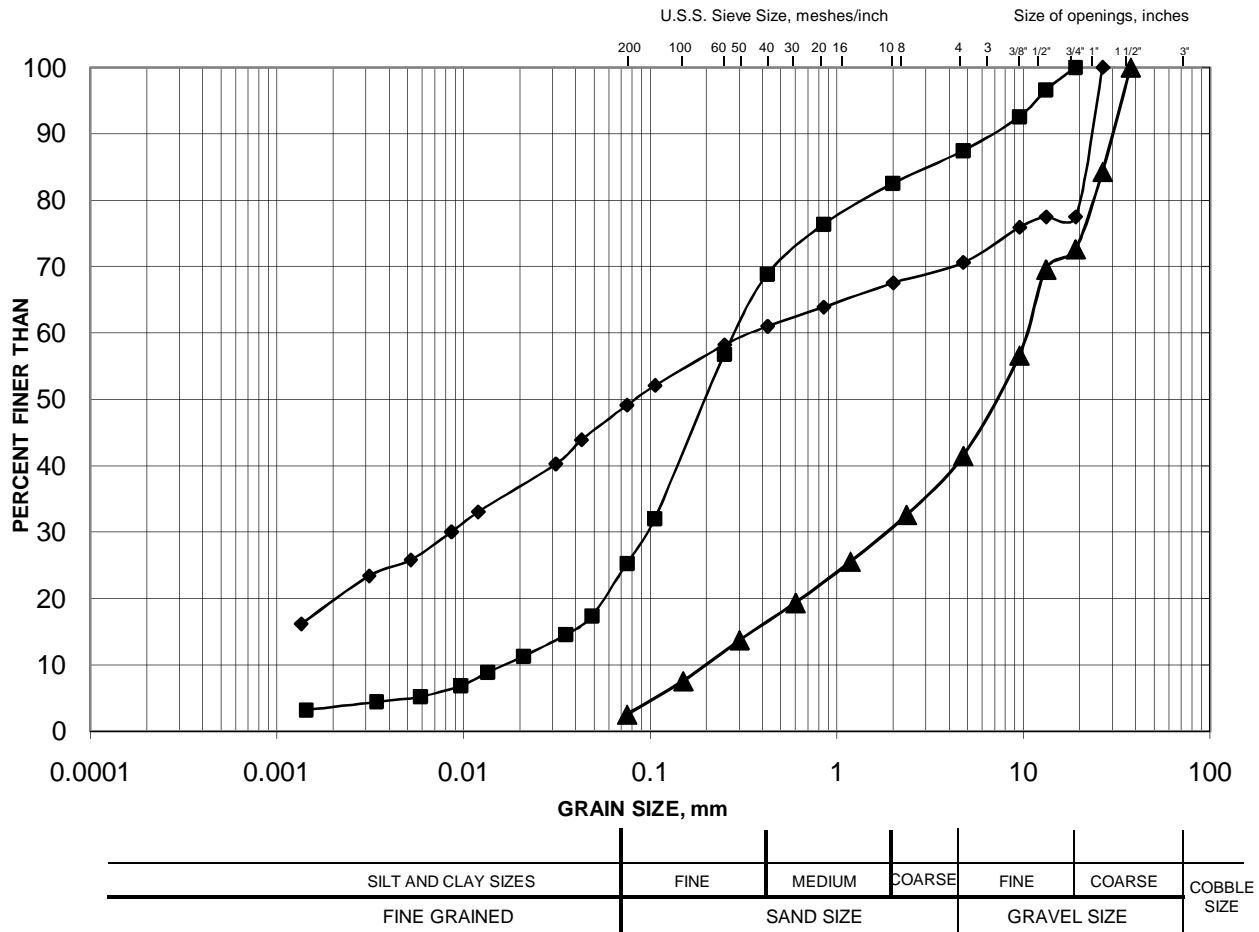
Golder Associates

Date: January 2010

GRAIN SIZE DISTRIBUTION

Sand to Sand and Gravel

**FIGURE
B-9**



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◆	FR-4	5	240.0
■	FR-4A	10	236.5
▲	FR-5	13	241.3

Project Number: 07-1191-0007-FR

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Date: January 2010

TABLE B-1
UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS
HIGHWAY 11, FREDERICK HOUSE RIVER BRIDGE
GWP 5541-05-00, SITE 39E-045

Borehole Number	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
FR-2	19.1	228.7	Metasediment	47.4	142
FR-3	14.4	230.5	Metasediment	47.7	175
FR-4A	19.5	229.5	Metasediment	47.4	99
FR-5	26.3	228.7	Metasediment	47.6	89

Compiled by: EC
Reviewed By: SEMC



APPENDIX C

NON-STANDARD SPECIAL PROVISIONS AND OPERATIONAL CONSTRAINTS

RIGID EXPANDED POLYSTYRENE EMBANKMENT – Item No.

Special Provision

REQUIREMENTS FOR EXPANDED POLYSTYRENE EMBANKMENT FILL

1.0 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 a 300 mm thick layer of Granular 'B' Type II, mortar sand, polyethylene sheeting and concrete top pad as shown on the Contract Drawings.

2.0 REFERENCES

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

2.1 National Standard of Canada

CAN/CGSB – 51.20 M87

2.2 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

2.3 OPSS – Ontario Provincial Standard Specifications

OPSS 212 Borrow

OPSS 501 Compaction

OPSS 517 Dewatering

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

OPSS 1605 Expanded Extruded Polystyrene Pavement Insulation

OPSS 1860 Geotextiles

3.0 SUBSURFACE CONDITIONS

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

4.0 DEFINITIONS

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

Rigid Expanded Polystyrene: Moulded 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: Quality Verification Engineer means 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.

5.0 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.

6.0 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 the 300 mm thick Granular B Type II drainage layer and the 100 mm thick mortar sand levelling pad.
- 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 125 mm thick reinforced 30 MPa concrete top pad.
- 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 Levelling Pad

The levelling 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, telephone 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 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 ² .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (mm)	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 1,200 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 ± 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 mm 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 with ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

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 Pad

The concrete top pad shall consist of 125 mm of reinforced 30 MPa concrete.

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 recommendations.

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. Any softened, loosened or deleterious materials at the foundation/base elevation shall be subexcavated and replaced with OPSS 1010 Granular "B" Type II material.

9.2 Levelling Pad

Place, level and compact a layer of mortar sand material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The levelling 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 levelling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

- (1) The individually marked blocks shall be placed on the prepared levelling 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.
- (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 levelling 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.

10.0 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 requirements.

11.0 QUALITY ASSURANCE

11.1 General

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

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.

12.0 MEASUREMENT FOR PAYMENT

12.1 Actual Measurement

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

13.0 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.

Operational Constraint – Obstructions/Ground Control

Pile cap construction below the groundwater and/or river water levels must be carried out in-the-dry. The excavations shall be kept stable during the work.

The Contactor shall be alerted that the cohesionless soils at the site are water-bearing and susceptible to soil cave-in, sloughing and boiling. The contractor is responsible to ensure that appropriate construction procedures and equipment are used for construction.

The Contractor shall be alerted that the fill and slope materials may contain cobbles and/or boulders and other obstructions.

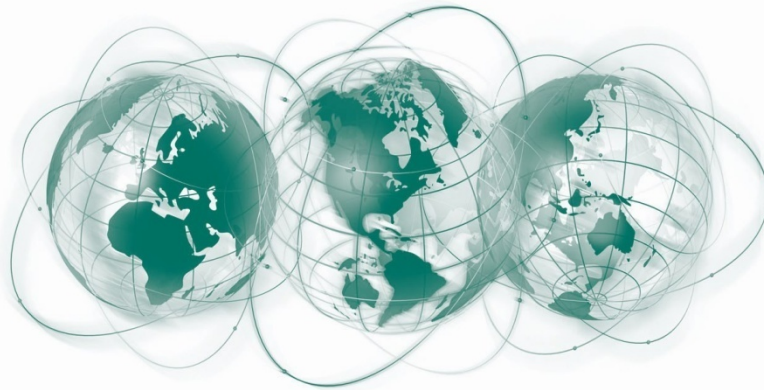
The Contactor shall be alerted that timber cribbing and/or old shoring elements may be present at the site.

The Contractor shall be alerted that the existing east bridge abutment was remediated using EPS and care should be taken to locate and avoid this material during shoring installation, where appropriate.

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