



September 28, 2011

FOUNDATION INVESTIGATION AND DESIGN REPORT

**REPLACEMENT OF WICKLOW RIVER BRIDGE NORTH
HIGHWAY 7037, SITE NO. 39E-174
TOWNSHIP OF LAMARCHE, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5139-06-00, AGREEMENT NO. 5008-E-0037**

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REPORT





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PART A

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the replacement of the Wicklow River North Bridge (Site No. 39E-174), located on Highway 7037 (south of Cochrane) in the Township of Lamarche.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) dated November 17, 2008. Golder's proposal P81-1685, dated December 2008, for foundation engineering services associated with the replacement bridge is contained in Sections 5.8 and 6.8 of LEA's Technical Proposal that forms part of the Consultant's Agreement Number 5008-E-0037 for this project. Subsequent to the award of the engineering services contract, the Preliminary and Detail Design investigation phases were combined to Detail Design level only and the originally planned detour was eliminated from the project scope. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated September 16, 2009. The General Arrangement drawing for the replacement bridge was provided to Golder by LEA on April 11, 2011.

Subsurface information for the existing bridge is contained in the following Geocon Ltd. report available on GEOCREs (Geocon 1959):

- "Soil Conditions and Foundations Report for the Proposed Bridge, Highway 11 Detour, Cochrane, Ontario" and "Boring Plan and Soil Stratigraphy, Drawing 59-F-228C" dated April 30, 1959, GEOCREs NO. 42A-015A&B.

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement location by borehole drilling, in situ testing and laboratory testing on selected samples. The location of the investigated area is shown on Drawing 1.

2.0 SITE DESCRIPTION

The site is situated in the Township of Lamarche on Highway 7037 crossing the Wicklow River, approximately 1.6 km east of the junction with Highway 11. The surrounding stable land is generally flat, sloping steeply near and towards the river. The area is occupied mainly by residential developments, with grass and tree covered terrain beyond the various property limits. The river banks adjacent to the existing bridge area are vegetated with landscaped grass and small shrubs. The river flows in a northeast to southwest direction and is about 14 m wide at the existing bridge location.

The existing structure consists of a 58 m long by 4.9 m wide single-lane five-span Bailey bridge constructed in 1961. The existing structure is founded on timber piles. Based on available GEOCREs report (Geocon 1959), the piles were to be driven to depths between 9 m and 15 m below the adjacent ground surface. The actual length of the piles is unknown. The existing ground surface at the north and south project limits is at about Elevation 263.9 m and 262.7 m, respectively. The existing ground surface slopes down towards the north and south "abutments" at Elevation 258.2 m and 258.0 m, respectively, and then steeply to the river. The existing embankment front slopes are formed at approximately 3.6 horizontal to 1 vertical (3.6H:1V) on the north side of the river and 2.6H:1V on the south side of the river. The existing embankment appears to be constructed of earth fill, with side slopes ranging between about 1.6H:1V to 2.0H:1V at the northeast and southwest limits, respectively, where the embankment is skewed closer to the river.



The water level in the river was measured between Elevation 250.2 m and 252.1 m during the field investigation (i.e. April 8 to 15, 2011), and changed rapidly in the span of hours due to the Spring runoff (see Section 4.2.11 for details). The high water level is reported to be Elevation 251.8 m. The existing highway embankment grade is approximately 6 m above the surrounding ground surface adjacent to the river.

3.0 INVESTIGATION PROCEDURES

The fieldwork at the bridge site was carried out between April 8 and 15, 2011, during which time a total of six (6) boreholes (WN-1 to WN-6) were advanced: four boreholes (WN-1 to WN-4) at the proposed bridge abutments and approaches; and two boreholes (WN-5 and WN-6) near the toe of the existing northeast and southwest side slopes. The locations of and ground surface elevations at the boreholes are shown on Drawing 1.

Boreholes WN-1 to WN-4 were drilled using a CME 55 track-mounted drill rig and Boreholes WN-5 and WN-6 were drilled using a tripod mounted portable drill rig, both supplied and operated by George Downing Estate Drilling Ltd. (Downing) of Grenville-Sur-La-Rouge, Quebec. The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers, NW casing with wash boring and NQ size core barrels. In general, soil samples were obtained at intervals of depth of about 0.75 m to 3.0 m, using a 50 mm outer diameter (O.D.) split-spoon sampler operated by automatic hammers on the drill rigs and by manual hammers with the portable equipment, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Selected samples of the cohesive soils were obtained using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using MTO Standard 'N' size vanes. Two boreholes were partially advanced by coring through the very dense strata when split-spoon sampling indicated such conditions. All boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The boreholes for the bridge approaches, WN-1 and WN-4, were advanced to a depth of 15.8 m below ground surface. The boreholes for bridge abutments, WN-2 and WN-3, were advanced to refusal at depths of 46.1 m and 37.0 m below ground surface, respectively. The boreholes advanced on the existing side slopes near the toe of the existing embankments, WN-5 and WN-6, were advanced to depths of 17.1 m and 14.3 m below ground surface, respectively.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendix A. Piezometers were installed in Boreholes WN-1 and WN-4 to allow monitoring of the stabilized groundwater level at these locations. The piezometers consist of 19 mm O.D. rigid PVC tubing with a 3.0 m long slotted screen sealed within the silty clay deposit. Flush mounted caps were used at ground surface. Details of the piezometer installations and water level readings are presented on the Record of Borehole sheets in Appendix A. The piezometers will be decommissioned at a later date.

Flowing artesian groundwater conditions were first observed within the silty sand till deposit in Borehole WN-2 and near the bottom of the silty clay to clay deposit in Borehole WN-3. Details of the sealing of the artesian boreholes are given in Section 4.2.11.



Traffic protection was implemented for the boreholes drilled within the roadway in accordance with the Traffic Protection Plan for this project and MTO Book 7 “Temporary Conditions Manual of the Ontario Traffic Manual” (2001).

The fieldwork was supervised throughout by a member of our technical staff who located the boreholes, arranged for the clearance of underground services at the borehole locations, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil and cobbles/boulders 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 samples. One-dimensional consolidation (oedometer) tests were carried out on two Shelby tube samples of the cohesive soil.

The locations of the boreholes were laid out in the field by Golder relative to the existing bridge features. Golder surveyed the geodetic ground surface elevation of the boreholes once completed, referencing an existing benchmark located approximately 14 m north of the roadway centreline and 86 m east of the east limit of the existing bridge. The northing and easting coordinates were determined by plotting the boreholes relative to the existing bridge features shown on the General Arrangement drawing. The MTM NAD 83 northing and easting coordinates, ground surface elevations referenced to Geodetic datum and borehole depth for each borehole are presented on the Record of Borehole sheets in Appendix A and are summarised below.

Borehole	Borehole Location		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
WN-1	5428936.6	304214.6	258.3	15.8
WN-2	5428914.5	304207.7	257.6	46.1
WN-3	5428854.7	304217.2	257.9	37.0
WN-4	5428835.0	304209.3	259.2	15.8
WN-5	5428914.4	304225.1	252.3	17.1
WN-6	5428860.8	304206.5	254.5	14.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on terrain mapping by the Ontario Geological Survey¹, the subsurface soils in the vicinity of the site consist of glaciolacustrine plain deposits comprised of silts and clays bordering areas of organic terrain.

Based on bedrock mapping by the Ministry of Northern Development and Mines², this site is located in the Superior Province. The bedrock of this domain consists of mafic to ultramafic metavolcanic rocks bordering with areas of massive granodiorite to granite from the neoproterozoic to mesoproterozoic era.

¹ Ministry of Natural Resources, Northern Ontario Engineering Geology Terrain Study, Ontario Geological Survey Map 5026

² Ministry of Northern Development and Mines, Bedrock Geology of Ontario, East-Central Sheet, Map 2543.



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 presented on the Record of Borehole sheets in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes is shown in profile and cross-section on Drawings 1 and 2.

The existing ground surface encountered at the north and south approaches along Highway 7037 (WN-1 and WN-4) is at Elevation 258.3 m and 259.2 m, respectively. The existing ground surface encountered along the side slopes near the proposed abutments (WN-2 and WN-3) is at Elevation 257.6 m and 257.9 m, respectively. The existing ground surface near the toe of the existing northeast slope (WN-5) and about mid-way down the southwest slope (WN-6) is at Elevation 252.3 m and 254.5 m, respectively.

In general, the subsoils consist of fill and alluvium underlain by a deposit of stiff to firm silty clay to clay, the lower portion of which is varved. Cohesionless deposits of silt, silty sand till, sand and gravel and silt till deposits underlie the silty clay to clay deposit at depth. These cohesionless deposits are underlain by cobbles and boulders. Refusal to split-spoon advancement was encountered at drilled depths of 46.1 m and 37.0 m below ground surface at Boreholes WN-2 and WN-3, respectively. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt

A 50 mm and 25 mm thick layer of asphalt was encountered from ground surface in Boreholes WN-1 and WN-4, respectively.

4.2.2 Topsoil

A 50 mm to 75 mm thick layer of topsoil was encountered from ground surface in Boreholes WN-2, WN-3, WN-5 and WN-6.

4.2.3 Fill

Boreholes WN-1 and WN-4 were advanced within the shoulders or driving lane of the existing highway and Boreholes WN-2, WN-3 and WN-6 were drilled within the existing embankments. These boreholes encountered roadway/embankment fill consisting of granular fill and/or silty clay fill underlying the asphalt or topsoil. The fill material extends to depths between 0.8 m and 2.3 m below the existing ground surface, between Elevation 257.7 m and 253.7 m.

Granular Fill

Granular fill consisting of frozen to damp, brown gravelly sand, sand and silt, sand and gravel, or sand was encountered in Boreholes WN-1, WN-3, WN-4 and WN-6. The fill in Boreholes WN-2 and WN-6 is slightly organic.



The granular fill is between 0.1 m and 1.4 m thick and was encountered between Elevation 259.1 m and 254.4 m.

One SPT 'N'-value measured in the frozen granular fill is 72 blows per 0.3 m of penetration. One SPT 'N'-value measured in the non-frozen granular fill is 5 blows per 0.3 m of penetration indicating a loose relative density.

Grain size distribution tests were carried out on two samples of the granular fill and the results are shown on Figure B-1.

The natural water content measured on two samples of the granular fill is about 7 percent (gravelly sand fill) and about 22 percent (sand and silt fill).

Silty Clay Fill

A 0.7 m to 2.2 m thick layer of frozen to moist, brown to grey silty clay fill was encountered below the topsoil in Borehole WN-2 and below the granular fill in Boreholes WN-1, WN-3 and WN-4. The silty clay fill contains some to with sand, trace to some gravel and, in Borehole WN-1, is slightly organic. The surface of the silty clay fill was encountered between Elevation 259.0 m and 256.8 m.

The SPT 'N'-values measured in the frozen silty clay fill are between 24 and 27 blows per 0.3 m of penetration. The SPT 'N'-values measured in the non-frozen silty clay fill are between 7 and 10 blows per 0.3 m of penetration, suggesting a firm to stiff relative density.

Grain size distribution tests were carried out on two samples of the silty clay fill and the results are presented on Figure B-2. An Atterberg limits test was carried out on one sample of the silty clay fill and the test result is presented on Figure B-3. The liquid limit is 39 percent, the plastic limit is 20 percent and the plasticity index is 19 percent. These results indicate this sample of the fill is classified as silty clay of intermediate plasticity.

The natural water content measured on two samples of the silty clay fill is about 22 percent.

4.2.4 Clayey Silt to Silty Clay (Alluvium)

A deposit of wet, brown to black to grey, clayey silt to silty clay alluvium was encountered underlying the fill material in Boreholes WN-1, WN-2 and WN-6 and below the topsoil in Borehole WN-5. The alluvium contains trace to with sand and is slightly organic. The surface of the alluvium deposit was encountered between Elevation 256.0 m and 252.2 m and the thickness of the deposit ranges from 1.4 m to 4.8 m.

The SPT 'N'-values measured within the alluvium range from 2 blows to 13 blows per 0.3 m of penetration, suggesting a soft to stiff consistency.

Grain size distribution tests were carried out on three samples of the alluvium and the results are presented on Figure B-4. Atterberg limits tests were carried out on three samples of the alluvium deposit and the results are presented on Figure B-5. The liquid limits range from 32 percent to 45 percent, the plastic limits range from 19 percent to 20 percent and the plasticity indices range from 14 percent to 25 percent. The results indicate the alluvium deposit is classified as clayey silt of low plasticity to silty clay of intermediate plasticity.

The natural moisture content measured on seven samples of the alluvium range from about 25 percent to 60 percent.

The organic content measured on three samples of the alluvium is between 4 percent and 5 percent.



4.2.5 Silty Clay to Clay

A deposit of moist to wet, brown to grey, silty clay to clay containing trace to some sand was encountered below the alluvium in Boreholes WN-1, WN-2, WN-5 and WN-6 and below the fill in Boreholes WN-3 and WN-4. The surface of the silty clay deposit was encountered between Elevation 257.7 m and 250.2 m and the thickness of the deposit ranges between 12.2 m and 18.3 m where the deposit was fully penetrated. Boreholes WN-1, WN-4 and WN-6 were terminated within this deposit. The upper 0.8 m to 2.3 m of the silty clay to clay deposit encountered in Boreholes WN-1, WN-3 and WN-4 is considered to be the desiccated/weathered crust. The portion of the deposit between about Elevation 248 m and Elevation 241 m contains clay and silty clay/clayey silt varves. Towards the bottom of the deposit at about Elevation 238 m, a higher concentration of silt layers was encountered.

The SPT 'N'-values measured in the silty clay crust range from 5 blows to 10 blows per 0.3 m of penetration suggesting a stiff consistency. The SPT 'N'-values below the desiccated crust range from 0 blows (i.e. weight of hammer) to 6 blows per 0.3 m. In situ field vane testing carried out in the main portion of the deposit below the stiff upper crust measured undrained shear strengths ranging between 34 kPa and 57 kPa. The sensitivity of the silty clay to clay deposit is calculated to range between 3 and 12. The in situ vane test results, together with the SPT 'N'-values, indicate that the silty clay to clay deposit generally has a firm consistency becoming stiff towards the bottom of the deposit.

Grain size distribution tests were carried out on six samples of this deposit and the results are presented on Figure B-6. Atterberg limits tests were carried out on twenty-one samples of this deposit and the test results are presented on Figure B-7. The liquid limits range from 28 percent to 68 percent, the plastic limits range from about 16 percent to 27 percent and the plasticity indices range from about 11 percent to 41 percent.

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

The natural moisture content measured on several samples of the silty clay to clay deposit ranges from 22 percent to 59 percent.

Two laboratory consolidation (oedometer) tests were carried out on specimens of the silty clay to clay obtained from Boreholes WN-2 (north abutment) and WN-3 (south abutment) and the test results are shown on Figures B-8 and B-9, respectively. The preconsolidation stresses were estimated from the Void Ratio versus logarithmic Pressure plots using the Casagrande method as well as from the Total Work versus Pressure plots. The unit weight of the samples from Boreholes WN-2 and WN-3 is 17.1 kN/m^3 and 18.4 kN/m^3 , respectively, and the measured specific gravity is 2.7. 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)
WN-2/10	243.6	165	170	5	1.0	1.4	0.05	0.66	1.2×10^{-3}
WN-3/10	248.4	137	180	43	1.3	1.0	0.04	0.40	1.4×10^{-3}

Note: *For approximate stress range between the effective overburden stress and the final stress due to a 2.0 m and 1.6 m embankment grade raise at the north and south abutments that is $140 \text{ kPa} \leq \sigma_v' \leq 190 \text{ kPa}$.

where: σ_{vo}' effective overburden stress in kPa
 σ_p' preconsolidation stress 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

4.2.6 Silt

A deposit of moist to wet, grey silt containing trace to some clay was encountered below the silty clay to clay deposit in Boreholes WN-2, WN-3 and WN-5. The surface of the silt deposit was encountered between Elevation 238.6 m and 237.5 m and the thickness of the deposit ranges from 3.4 m to 6.1 m. Borehole WN-5 was terminated within this deposit.

The SPT 'N'-values measured in the silt deposit range from 6 blows to 12 blows per 0.3 m of penetration. The portable tripod drill rig could not advance Borehole WN-5 below Elevation 238 m. A Dynamic Cone Penetration Test (DCPT) was initially driven from the bottom of the borehole between Elevation 238 m and 235.8 m, followed by a split-spoon at the bottom of the cone hole, yielding an SPT 'N'-value of 18 blows per 0.3 m of penetration. The SPT 'N'-values indicate that the silt deposit is compact to dense in relative density.

Grain size distribution tests were carried out on three samples of the silt deposit and the results are shown on Figure B-10. An Atterberg limits test was carried out on one sample of this deposit and the test result is presented on Figure B-11. The liquid limit is 25 percent, the plastic limit is about 20 percent and the plasticity index is about 5 percent, indicating that the deposit may be classified as silt of slight plasticity.

The natural moisture content measured on five samples of the silt ranges from 24 percent to 29 percent.

4.2.7 Silty Sand and Gravel (Till)

A deposit of moist to wet, grey silty sand and gravel till containing trace to some clay was encountered below the silt deposit in Boreholes WN-2 and WN-3. The surface of the till deposit was encountered at Elevation 232.1 m and 232.0 m and the deposit is 11.8 m and 9.5 m thick at the respective boreholes.

The SPT 'N'-values measured in the silty sand and gravel till range from 10 blows to 180 blows per 0.3 m of penetration indicating a compact to dense relative density. SPT 'N'-values of 43 blows and 133 blows per 0.15 m of penetration were recorded within this deposit and the split-spoon sampler was noted to be bouncing.

Seven samples of the till deposit were obtained using an NQ sized core barrel at depths where split-spoon samples could not be taken and coring was required to advance the borehole. In Borehole WN-2, the recovered soil core samples contained a 0.3 m diameter boulder between Elevation 223.9 m and 223.6 m and below Elevation 223.6 m contained gravel and cobble fragments. In Borehole WN-3, the recovered soil core sample also contained 0.15 m and 0.18 m diameter (cobble-size) rock.



Grain size distribution tests were carried out on four samples of the silty sand and gravel till and the results are shown on Figure B-12. Due to the nature of sampling method (i.e. NW casing with wash boring or NQ coring) within this deposit, it is suspected that some fines (i.e. silt and/or clays) were “washed” from the samples. In addition, due to the relatively large size of the coarse fractions (i.e. gravel), the grain size distributions may be skewed. Based on the laboratory test results and our visual examination, this deposit generally consists of silty sand and gravel till containing trace to some clay.

The natural moisture content measured on six samples of the silty sand and gravel till ranges between 6 percent and 26 percent and is typically lower with depth.

4.2.8 Silt (Till)

A deposit of moist to wet, grey silt till containing trace clay, sand and gravel was encountered underlying the silty sand and gravel till deposit in Borehole WN-2 and underlying the silty sand till deposit in Borehole WN-3. The surface of the silt till deposit was encountered at Elevation 220.3 m and 222.5 m and the thickness of the deposit is 4.5 m thick in Borehole WN-2 where it was fully penetrated and is 1.6 m thick in Borehole WN-3, which terminated within this deposit.

Soil/rock coring using an NQ sized core barrel was used to advance through the upper 1.5 m of the silt till deposit in Borehole WN-2. The SPT ‘N’-values measured within the silt till range from 127 blows to 207 blows per 0.15 m of penetration, indicating a very dense relative density.

Grain size distribution tests were carried out on two samples of the silt till deposit and the results are shown on Figure B-13.

The natural moisture content measured on two samples of the silt till is 13 percent and 18 percent.

4.2.9 Cobbles and Boulders

A deposit of cobbles and boulder within a silt, sand and gravel matrix was encountered underlying the silt till deposit in Borehole WN-2. The surface of the cobbles and boulders deposit was encountered at Elevation 215.8 m and the deposit was penetrated for 4.3 m to the borehole termination depth.

Soil/rock coring using an NQ sized core barrel was used to advance the borehole through the cobbles and boulders to a depth of 46.1 m below ground surface (i.e. to Elevation 211.5 m). The recovered core samples contained soil and granite rock core of cobble and boulder sizes between 0.15 m and 0.30 m. A split-spoon sample was taken at a depth of 46.1 m and the split-spoon was noted to be bouncing during the 100 blows driven for a penetration of 0.02 m.

4.2.10 Groundwater

Groundwater levels were measured in the open boreholes during and upon completion of drilling. Piezometers were installed in WN-1 and WN-4 and sealed within the silty clay deposit to monitor the groundwater levels over time. The measured groundwater levels in the open boreholes and piezometers are presented below.



Borehole	Installation	Time and/or Date	Groundwater Depth (m)	Groundwater Elevation (m)
WN-1	Open Borehole	Dry to bottom of borehole at 15.8 m depth (Elev. 242.5 m) upon completion of drilling	--	--
	Piezometer	April 21, 2011	12.8	245.5
		April 28, 2011	11.5	246.8
		July 3, 2011	3.10	255.2
WN-2	Open Borehole	April 9, 2011	2.1 m above ground surface (9.5 m above river level at time of drilling)	259.7
WN-3	Open borehole	April 13, 2011	2.4 m above ground surface (8.2 m above river level at time of drilling)	260.3
WN-4	Open Borehole	Dry to bottom of borehole at 15.8 m depth (Elev. 243.3 m) upon completion of drilling	--	--
	Piezometer	April 21, 2011	5.9	253.2
		April 28, 2011	7.8	251.4
		July 3, 2011	2.3	256.9
WN-5	Open Borehole	Upon completion of drilling (April 14, 2011)	0.3	252.0
WN-6	Open Borehole	Upon completion of drilling (April 15, 2011)	4.3	250.2

Groundwater levels encountered in the boreholes during and shortly after drilling may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling.

The water level in Wicklow River was measured between Elevation 250.2 m and 252.1 m during the field investigation from April 8 to 15, 2011 and at Elevation 250.2 m on July 3, 2011. The high water level is reported to be Elevation 251.8 m. It should be noted that the river water level fluctuated rapidly and significantly during the course of the drilling at this site, up to 1.9 m in a matter of hours, attributed to Spring runoff.

Groundwater and river water levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

Artesian groundwater conditions were observed when advancing Borehole WN-2 through the silty sand till deposit in and near the bottom of the silty clay to clay deposit in Borehole WN-3.

Boreholes WN-2 and WN-3 were sealed full column with cement grout, consistent with Ontario Regulation 903 - Wells (as amended) using a mix design appropriate for the level of artesian head. On April 21 and during return visits to the site on April 28 and July 3, 2011 to obtain water level readings in the piezometers, it was confirmed visually that Boreholes WN-2 and WN-3 did not show artesian flow groundwater conditions.



5.0 CLOSURE

The field drilling program was supervised by Mr. Indulis Dumpis. This report was prepared by Mr. David Muldowney, P.Eng., and the technical aspects were reviewed by Ms. Sarah E.M. Coyne, P.Eng., Associate. A quality control review of the report was provided by Mr. Jorge M.A. Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project.

Report Signature Page

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PART B

FOUNDATION DESIGN REPORT
REPLACEMENT OF WICKLOW RIVER BRIDGE NORTH
HIGHWAY 7037, SITE NO. 39E-174
TOWNSHIP OF LAMARCHE, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5139-06-00, AGREEMENT NO. 5008-E-0037



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed new Highway 7037 bridge and associated retaining/wing wall structures over the Wicklow River North. 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. 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

We understand that the existing Wicklow River Bridge North was constructed in 1961 and consists of a 58 m long by 4.9 m wide, five-span structure supported by timber piles. Based on the GEOCRE report (Geocon 1959), the existing timber piles were presumably driven to depths between 9 m and 15 m below the adjacent ground surface to avoid penetrating into the artesian groundwater bearing stratum. Based on these assumed pile embedment lengths, the timber piles likely terminate in the silty clay to clay deposit.

We understand that the proposed replacement bridge will be a 52 m long by 6 m wide, single-span, integral abutment structure with 6 m long approach slabs constructed along the existing alignment. The proposed grade of the replacement bridge is between Elevation 251.5 m and 251.8 m at the north and south abutments, respectively. Given that the abutments have been moved closer to the river, the grade raise is approximately 2.0 m and 1.6 m above the existing ground surface at the north and south abutments, respectively. Beyond the abutments, the grade raise is about 0.3 m and 0.5 m, at the north and south abutments, respectively. The new bridge will be about 1 m wider than the existing structure, requiring minimal widening on each side of the existing embankment. We understand that the roadway will be closed and traffic will be detoured along Highway 11 during construction of the replacement bridge.

The subsurface conditions in the vicinity of the proposed bridge generally consist of fill and/or alluvium underlain by an up to 18 m thick deposit of stiff to firm silty clay to clay, where fully penetrated, of which the upper 0.8 m to 2.3 m is considered to be the desiccated/weathered crust in some boreholes. Cohesionless deposits of silt, silty sand till, sand and gravel, silt till and cobbles and boulders were encountered underlying the cohesive strata. Artesian conditions were noted after penetrating the silty sand till deposit in Borehole WN-2 (north abutment) and near the bottom of the silty clay to clay deposit in Borehole WN-3 (south abutment). Upon completion of drilling, the water levels within these boreholes were recorded at approximately 2.1 m and 2.4 m above the existing ground surface (Elevation 259.7 m and 260.3 m), respectively, which corresponds to approximately 9.5 m and 8.2 m above the river water level at the time of drilling.

Boreholes WN-2 and WN-3 were terminated 46.1 m and 37.0 m below ground surface (Elevation 211.5 m and 220.9 m), respectively. These boreholes were terminated within very dense silt till and or cobbles and boulders.



6.2 Foundations

Given the extent of fill material and alluvium at the abutments at both bridge sites, shallow foundations are not recommended due to the low geotechnical axial resistance that would be available from these deposits to support the abutments. Spread footings on a granular pad are also not recommended due to the potential for settlement of the subsoils and embankment instability as a result of footing pressure and increased soil weight.

Caisson foundations are not considered feasible due to the artesian groundwater conditions, and the presence of cobbles and possible boulders (as inferred from difficult drilling advance) in the lower cohesionless deposits.

Deep foundations comprised of driven steel H-piles are considered to be the preferred alternative from a foundations geotechnical axial resistance perspective and suitability for integral abutment design. Since bedrock was not encountered to 46 m and 37 m depths investigated at the proposed abutments, the foundation design should be based on the use of friction piles. Steel tube piles are not considered appropriate for this site as these piles are displacement piles which potentially could create a large void along the length of the piles leading to artesian groundwater flow along the pile.

Table 1 summarizes the advantages, disadvantages, relative costs and risks/consequences of the foundation alternatives for the replacement structure. Design recommendations for the recommended option are given in the sections below.

6.3 Deep Foundations

We recommend that the replacement bridge be supported on steel H-piles driven through the cohesive silty clay to clay deposits and penetrating approximately 3 m into the underlying 100-blow (or equivalent) cohesionless sand and gravel or silt till deposits. This will require the piles to penetrate the cohesionless deposit under artesian groundwater pressure encountered near the base of the silty clay to clay deposit. Friction piles terminating sufficiently above the artesian groundwater deposit within the silty clay to clay deposit will not achieve a sufficient geotechnical axial resistance to support the structure.

6.3.1 Driven Piles Tip Elevation

The estimated pile lengths given below are based on the underside of pile cap elevations shown on the General Arrangement drawing. The tip elevations correspond to the estimated termination depth of the friction piles, approximately 3 m into the lower sand and gravel and silt till deposits.

Abutment	Borehole Number	Proposed Underside of Pile Cap	Recommended Tip Elevation	Estimated Design Pile Length
North	WN-2	251.9 m	223.5 m	28.4 m
South	WN-3	252.3 m	222.5 m	29.8 m



6.3.2 Geotechnical Axial Resistance

For friction piles, the geotechnical axial resistance is a function of the shaft resistance and the toe resistance. The factored geotechnical axial resistance at Ultimate Limit States (ULS) is a combination of the shaft and toe resistance and utilizing a factor of 0.5 on the calculated ultimate resistance in accordance with the CHBDC (2006) and current MTO Foundations practice. The axial resistance at Serviceability Limit States (SLS) (for 25 mm of settlement) assumes that the pile will settle approximately 10 mm to 15 mm to mobilize shaft friction. The ULS and SLS values for two different pile types driven to the elevations given above are presented below. Generally, HP310X110 piles are used; however, we understand that a heavier pile section, HP310X132, is being considered for structural reasons.

Pile Section	Factored Geotechnical Axial Resistance at ULS	Geotechnical Axial Resistance at SLS (for 25 mm settlement)
HP310X110	1,600	1,400
HP310X132	1,800	1,600

The length of the replacement bridge is shorter than the existing bridge and, as such, the proposed abutments will be closer to the river than the existing embankment geometry. Since the boreholes could not be advanced through the existing bridge deck and the ground topography limited access along the existing embankment side slopes, the boreholes were located approximately 3 m to 4 m behind the proposed abutments. Therefore, the depths to each soil layer at the location of the new bridge abutments may differ from the depths at the borehole locations given above. Based on the General Arrangement drawing showing the location of the proposed abutments and site observations of the relative location of the abutments to the existing timber pile bents, it is not expected that the steel piles for the new bridge abutment would intercept the existing timber piles.

Because the piles will be driven into/through the artesian groundwater bearing deposit, a filter sand blanket (see Section 6.9.4) should be constructed immediately below the pile cap to dissipate artesian groundwater and filter soil fines that may be carried upwards to the surface of the native soils. A contingency for grouting of any space/voids created along the piles should be included in the contract documents. Four Non Standard Special Provision (NSSP) items detailing the grouting method should be included in the Contract Documents; an example is provided in Appendix C. In addition, flange reinforcement or driving shoes/rock points should not be used as discussed in Section 6.3.4.

If corrugated steel pipes (CSPs) are to be installed as part of the integral abutment design (through which the H-piles will be driven), which we understand is not the case for this site, the CSPs should be backfilled with a loose, fine to medium sand. An NSSP detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is provided in Appendix C.

6.3.3 Downdrag

The difference between the unit weights of the existing and new fill combined with the up to 2.0 m grade raise will induce settlement in the stiff to firm silty clay to clay deposits. Downdrag loads (negative skin friction) will be induced on the friction piles as a result of the addition of approach embankment fill after pile installation



is complete as well as from the resulting settlement of the cohesive soil relative to the piles. Downdrag loads will need to be taken into account for design of the piles supporting the abutments unless mitigation of settlement is carried out prior to pile installation.

The structural design of the abutment piles should be based on an estimated unfactored downdrag load of 250 kN acting on the piles (HP310X110 or HP310X132). 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 (2006) for ULS conditions.

6.3.4 Pile Driving Notes and Set Criteria

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations). The piles should not be fitted with rock points, driving shoes or flange plates (reinforced tips) in order to minimize the width of the gap that may be created as the piles are driven through the silty clay to clay stratum. This will increase the likelihood of the silty clay to clay to “self-seal” against the pile and hence reduce the potential for the creation of a pathway for artesian groundwater along the pile. Further, given that the piles are long and require splicing, the “butt welds” from OPSD 300.150 (Steel H-Pile Splice) should be used to splice the pile sections to minimize the pathway as opposed to the splice plates. The heavier pile section, HP310X132, is recommended to reduce the potential for damage to the pile during driving and penetration through the cobbles and boulders within the cohesionless deposits to the tip elevation.

For friction piles, the pile capacity must be verified in the field by the use of the Hiley Formula (Standard Structural Drawing SS 103-11) during the final stages of driving for the ultimate capacity at the elevations provided above. The ultimate geotechnical axial resistance predicted from the Hiley Formula should then be multiplied by a geotechnical resistance factor equal to 0.5 in accordance with Table 6.1 in the CHBDC (2006) to verify the factored ULS design value.

The pile driving note to be added to the drawings for this project is Note 2 in Clause 3.3.3 of the Structural Manual (MTO, 2008).

For HP310X110 piles, the note should read:

- Piles to be driven in accordance with Standard Structural Drawing SS 103-11 using an ultimate geotechnical resistance of 3,200 kN per pile but must be driven below EL 226.5 m (North Abutment) and EL 225.5 m (South Abutment).

For HP310X132 piles, the note should read:

- Piles to be driven in accordance with Standard Structural Drawing SS 103-11 using an ultimate geotechnical resistance of 3,600 kN per pile but must be driven below EL 226.5 m (North Abutment) and EL 225.5 m (South Abutment).

Assessment of ultimate pile resistance by the Hiley Formula should commence once the pile reaches a depth of not more than 3.0 m above the design pile tip elevation given in Section 6.3 and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley Formula is not achieved within the 3.0 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the



Hiley Formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48-hour wait period, the Contract Administrator should be notified and the Contractor must obtain authorization from the Contract Administrator prior to driving the pile below the design pile tip elevation.

A NSSP, which indicates that “butt” welds are to be used for splicing the piles and outlines the above pile capacity verification procedure, should be included in the Contract and an example is included in Appendix C.

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 are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

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

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

For cohesionless soils:

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

and for cohesive soils:

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

It is understood that an integral abutment foundation design is being considered; however, we understand that CSP liners are not required at this site. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-piles will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

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



**FOUNDATION REPORT, REPLACEMENT OF WICKLOW RIVER BRIDGE NORTH
HIGHWAY 7037, SITE NO. 39E-174 GWP 5139-06-00**

Foundation Element (Relevant Borehole)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
North Abutment (WN-2)	Loose Sand (Filter Blanket)	251.9 to 251.4	1,300	-
	Stiff Silty Clay (Alluvium)	251.4 to 250.5	-	50
	Firm to Stiff Silty Clay to Clay	250.5 to 248.0	-	35 (see Figure 1*)
		248.0 to 241.0	-	32 (see Figure 1*)
		241.0 to 237.5	-	32 increasing linearly to 65 (see Figure 1*)
	Loose to Compact Silt	237.5 to 232.1	2,800	-
	Compact to Very Dense Silty Sand Till (one instance of a cobble recorded)	232.1 to 226.4	11,000	-
	Very Dense Sand and Gravel (several instances of cobbles/boulders recorded)	226.4 to 223.5	4,400	-
South Abutment (WN-3)	Loose Sand (Filter Blanket)	252.3 to 251.8	1,300	-
	Firm to Stiff Silty Clay	251.8 to 248.0	-	35 (see Figure 1*)
	Firm to Stiff Silty Clay	248.0 to 241.0		32 (see Figure 1*)
		241.0 to 238.1		32 increasing linearly to 65 (see Figure 1*)
	Loose to Compact Silt	238.1 to 232.0	2,800	-
	Compact to Very Dense Silty Sand Till (one instance of a cobble recorded)	232.0 to 226.0	4,400	-
		226.0 to 222.5	11,000	

*For design line under existing embankment.

For a single HP310X110 or HP310X132 extending to the design tip elevations provided above, the estimated factored lateral resistances at ULS and at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using Broms' (1964) method as outlined in the CFEM (2006) and the commercially available program LPILE Plus (Version 5.0), produced by Ensoft Inc.

Pile Size	Lateral Resistance (kN)	
	ULS (Factored)	SLS (10 mm of deflection)
HP310X110	120	40
HP310X132	135	45



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

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

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

Reduction for group effects is negligible when the centre to centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.

6.3.6 Frost Protection

At this site, the pile caps should be provided with a minimum of 2.5 m of conventional soil cover for frost protection (as per OPSD 3090.100 Foundation, Frost Penetration Depths for Northern Ontario). Alternatively, rigid polystyrene insulation could be used to reduce the required thickness of soil cover. As a guideline for design, it is generally adopted by the MTO that a thickness of 25 mm of rigid polystyrene insulation should be assumed to be equivalent to about 300 mm of conventional soil cover. The insulation, if used, should be placed vertically along the face of the foundation (to the base of the pile cap) and extend horizontally for a distance of 2.5 m beyond the face. A minimum of 1 m of soil cover should be placed over the rigid insulation.

6.4 Seismic Considerations

6.4.1 Site Coefficient

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



6.4.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the CHBDC. According to Table A3.1.1 of the CHBDC and Table C4.2 of the Commentary, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio (A) for the Cochrane area is 0.05. The reference Peak Ground Acceleration (PGA) is 0.018 g based on the NRC website; however, the more conservative CHBDC value has been used in the assessment. Based on experience, for the subsurface conditions at this site, a 50 percent amplification of the ground motion may occur (i.e. Site Coefficient, $S=1.5$ for Soil Profile III from Table 4.4 of CHBDC), resulting in an increase in the Peak Ground Acceleration (PGA) from 0.05 g to 0.075 g at the ground surface.

We understand, based on Section 4.4.4 of the CHBDC, that this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Zone Performance 1.

6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 6.4.2, seismic (earthquake) loading need not be analyzed for this structure.

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

- Select free-draining granular fill meeting the specifications of SP 110S13 Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls Abutment, Backfill) and OPSD 3121.150 (Walls Retaining, Backfill).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specifications as outlined in the Northern Region Directive (2002) for backfill to structures adjacent to rock fill embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls Abutment, Backfill Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.



- For restrained structures, the granular fill should be placed in a zone with width equal to at least 2.5 m behind the back of the walls (in accordance with Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5H:1V extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b), Case II, of the Commentary to the CHBDC).
- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

	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	0.35

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

	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

- Where EPS is installed behind the abutment or retaining wall, the pressure acting over the thickness of the EPS may be calculated using the following values:
 - EPS unit weight: 0.5 kN/m³
 - Coefficients of static lateral earth pressure:
 - Active, K_a 0.11
 - At rest, K_o 0.11

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

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 Approach Embankment Design

The Wicklow River Bridge North replacement structure will be located along the same alignment as the existing structure. The existing ground surface slopes down towards the north and south abutments of the existing bridge at Elevation 258.2 m and 258.0 m, respectively. The replacement bridge will be about 1 m wider than the existing bridge and the highway grades will be raised by about 0.3 m and 0.5 m at the north and south approaches, respectively. At the location of the proposed north and south abutments, which will be about 2 m to 4 m closer to the river than the current north and south ends of the existing bridge, the existing sloping ground surface is at approximately 256.5 m and 257.2 m, respectively. Therefore, the proposed highway grade will be raised 2.0 m and 1.6 m above the existing ground surface at the new north and south abutments, respectively. The proposed grade is approximately 6 m above the surrounding ground surface adjacent to the river.

The existing embankments are formed at about 3.1H:1V and 2.5H:1V on the north and south front slopes to the river, respectively. The existing embankments are formed at about 1.6H:1V and 2H:1V at the northeast and southwest side slopes. The northwest and southeast side slopes are formed at a slope angle flatter than 2H:1V. The proposed front slopes will remain at approximately the existing slope angle and the side slopes will be formed at 2H:1V. To achieve this 2H:1V slope angle at the northeast side slope and avoid filling into the river or onto adjacent property, we understand that a 5.3 m long retaining wall is proposed (refer to Section 6.7).

The subsurface conditions in the vicinity of the proposed bridge generally consist of fill and/or silty clay to clayey silt alluvium underlain by a deposit of brown to grey silty clay to clay. Underlying the cohesive deposit are cohesionless deposits of silt, silty sand till, sand and gravel, silt till and cobbles and boulders. Details of the soil and groundwater conditions are given in Section 4.2. The design lines for the magnitude of undrained shear strength, pre-consolidation pressure and index properties (water content and Atterberg limits) versus elevation, for the assessment of stability and settlement of the north and south approach embankments are summarized on Figures 1 to 3.

The following sections present the design assumptions and methodology, parameter selection and results of stability and settlement analysis for the new approach embankments, including recommendations for stability and settlement mitigation measures, as required.

6.6.1 Design Assumptions

For the bridge approach embankments within 20 m of the abutments, the analyses assume that the existing fill and alluvium will only be removed to the underside of the proposed filter blanket recommended under the pile cap (i.e. to Elevation 251.4 m and Elevation 251.8 m at the north and south abutments, respectively) and within the frost taper zone prior to construction of the widened/raised embankments. This means that some fill and/or alluvium will be left in place and this has been considered in the analysis.

The low river water level used for design is Elevation 250.2 m and the high river water level used for design is Elevation 252.1 m, corresponding to the river water levels measured during the subsurface investigation. Groundwater levels beyond the river have been assumed to rise approximately 1 m higher further behind the abutments within the approaches.



Granular 'B' Type II fill has been assumed for the construction of the approach embankments since this material will be used to backfill the abutments. Rock fill is not being considered as there is not a local source available and due to the limited quantity required. Granular fill embankments are assumed to have side slopes at 2H:1V. The proposed front slopes will be formed at approximately 3.1H:1V and 2.5H:1V at the north and south sides of the river, respectively, consistent with the existing slope angle.

6.6.2 Stability

Analyses were performed on the critical sections of the proposed approach embankments for conditions during and after construction to assess the stability and liquefaction potential for the proposed embankment height, existing geometry and soil stratigraphy. The critical embankment sections at this site are the front slopes (towards the river) and the northeast and southwest side slopes. The geometry of the proposed approach embankments, existing ground surface and existing river bed included in the analyses are based on the information from the General Arrangement drawing provided by LEA, which we understand is based on recent survey data.

6.6.2.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.17), 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 (FoS). The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions at the end of construction. This FoS 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 FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design. In addition, effective stress (drained) analyses were conducted to assess long-term conditions applying a FoS of 1.3.

6.6.2.2 Parameter Selection

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

For the cohesive fill and native soils, 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 'N'-value results and other laboratory test data. Where appropriate, Bjerrum's correction factor (1973) as a function of the plasticity index of the soil (Figure 3) was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests (Figure 1).



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

The effective stress parameters for the cohesive strata (effective friction angle and cohesion) for evaluating long-term drained conditions were estimated using empirical correlations with plasticity index (PI) proposed by Mitchell (1993), Ladd et al. (1977) and Kulhawy and Mayne (1990).

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed approach areas. The parameters used in the analysis for the silty clay to clay deposits at the toe of the slopes and under the existing bridge embankments are shown graphically on Figure 1. The slope stability analyses model geometry and stratigraphy are shown on Figures 4 to 7 for the critical sections identified above.

Soil Type	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Angle of Internal Friction
New Granular 'B' Type II Fill	21	--	35°
EPS Fill	0.5	--	15°
Existing Gravelly Sand to Sand and Silt Fill	20	--	30°
Existing Silty Clay Fill	18	50 kPa	30°
Silty Clay to Clayey Silt Alluvium	17	50 kPa	30°
Silty Clay (Crust)	18	50 kPa	30°
Silty Clay to Clay (transition zone)	18	50 kPa decreasing to 35 kPa	30°
Silty Clay to Clay (under existing embankment)	18	See Figure 1*	27°
Silty Clay to Clay (at toe of side slope)	18	See Figure 1*	27°
Silt	19	--	27°

*For design lines under embankment or at toe of slope, as appropriate for the model.

6.6.2.3 Results of Analysis

The results of the stability analyses indicate that all critical slopes have a FoS of less than the target 1.3 for the undrained case. The minimum FoS is based on a deep-seated, global trial failure surface that would impact the operation of the roadway. Therefore, measures to mitigate stability will be required to achieve a FoS of greater than 1.3 as discussed in Section 6.6.5.



6.6.3 Liquefaction Potential and Seismic Analysis

As noted in Section 6.4.2, this site is located in Seismic Zone 1 with a $PGA < 0.075$. 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.

6.6.4 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. Time-dependent consolidation settlement of the silty clay to clay deposit below approximately Elevation 251 m is expected. Settlement of the existing fill, cohesionless deposits and upper stiff alluvium and desiccated silty clay crust is expected to be elastic, occurring during or shortly after construction. In addition, settlement of the embankment fill will also occur.

The following sections summarize the methodology, criteria, simplified stratigraphy, unit weights and deformation parameters employed for the different soils types in the approach areas. The maximum estimated settlement of the foundation soils in these areas (due to the loading imposed by the new approach embankment fills) and a discussion on the rate of settlement is presented below.

6.6.4.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using the commercially available program Settle3D (Version 2.003) produced by Rocscience Inc. as well as hand/spreadsheet calculations. The rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory. The model geometry and stratigraphy at the abutments are shown on Figures 4 and 5, as used for the stability analyses for the north and south front slopes, respectively.

6.6.4.2 Settlement Criteria

Based on MTO's "Embankment Settlement Criteria for Design" Final Draft dated March 2, 2010, the following post-construction settlement and differential settlement criteria are considered acceptable to occur within 20 years post-paving for the bridge approach embankments and the new Highway 69 embankments at this site.

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

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



6.6.4.3 Parameter Selection

The immediate compression of the existing fill, native cohesionless deposits and stiff silty clay to clay alluvium and the upper stiff desiccated silty clay to clay crust, was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the firm to stiff silty clay to clay deposit was assessed using the results of the in situ field vane tests and the laboratory consolidation tests 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 Koppula (1986).

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, the comparison to the previous data (Geocon 1959) 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' = pre-consolidation stress (kPa) (see Figure 2)

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 (i.e. ranging from 0.88 to 1 for this site as Plasticity Indices range from about 11 percent to about 41 percent as shown on Figure 3)

It is known that some secondary consolidation settlement occurs following the completion of primary settlement. This secondary settlement, or creep settlement, occurs over the long term (i.e. decades) for the normally consolidated clays at this site. The magnitude of secondary (creep) settlement (Mesri 1973 as quoted in Holtz and Kovacs 1981) was estimated using the following:

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

Based on Mesri (1973), the following empirical correlation was utilized to estimate $C_{\alpha\epsilon}$ from water content:

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

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



The following simplified stratigraphy, unit weights and deformation parameters have been employed in the settlement analysis of the proposed approach embankments. The thickness of the materials varies across the site and the maximum thicknesses have been used in the settlement estimates. Given the relatively low embankment heights, the full thickness of the silty clay to clay deposit (up to about 14 m) will not be impacted by the fill loading and the estimated effective compressible thickness has been used in the analyses as noted below and has been confirmed by computer analysis. Further, the lower cohesionless soils are not anticipated to settle under the imposed loading due to the great depth below ground surface that these strata are present. The thicknesses given below assumed that only the existing fill and alluvium have been removed in the abutment areas, filter blanket zones and frost taper zones (as per OPSD 3101.150 – Walls, Abutment, Backfill Minimum Granular Requirement) and therefore some of these materials may be left in place.

Deposit	Maximum Thickness at North or South Abutment (m)	Maximum Thickness at North or South Approach 20 m Behind Abutment (m)	Unit Weight (kN/m³)	Deformation Properties
New Granular 'B' Type II Fill*	N = 7.1 (2.9)* S = 7.0 (2.4)*	N = 1.0 (~0)* S = 1.0 (~0)*	21	See Section 6.6.4.4
Existing Gravelly Sand to Sand and Silt Fill	N = n/a S = n/a	N = 0.5 S = n/a	20	E' = 10 MPa
Existing Silty Clay Fill	N = n/a S = n/a	N = 0.8 S = 0.7	18	E' = 15 MPa
Silty Clay Alluvium (containing organics)	N = 0.9 S = n/a	N = 1.5 S = n/a	17	E' = 15 MPa
Silty Clay (crust)	N = n/a S = n/a	N = 2.2 S = 0.8	18	E' = 10 MPa
Silty Clay to Clay (under existing embankment)	N = 13.9 S = 13.7 (impacted thickness is estimated to be between about 5 m and 7 m)**	N = >10.2 S = >13.5 (not expected to be impacted by low fill height)	18	(see below)

* This value represents the total thickness of new fill. The effective fill thickness (in brackets) considers the total loading from the existing fill and the native soils that are to be removed and replaced by new fill.

** The maximum thickness of main cohesive deposit that will be influenced by embankment loading (approximately two times the effective height of the embankment below the base of the fill) is estimated to be between 5 m and 7 m of the total thickness of the stratum.

n/a Indicates deposit not encountered below the new fill.

The following consolidation parameters were estimated for the silty clay to clay deposit based on the results of laboratory consolidation tests performed on specimens of the silty clay to clay obtained from Boreholes WN-2 and WN-3 and compared with values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described above and comparison to the previous site data (Geocon 1959) as well as review of the data contained in Quigley and Ogunbadejo (1972). The plot of preconsolidation pressure and in situ vertical effective stress versus elevation is provided on Figure 2 and considers the varved nature of the deposit.



Approximate Elevation (m)	Silty Clay to Clay Stratum	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_r	C_c
251.0 – 248.0	Upper	(see Figure 2)		1.8 – 1.0	0.65 – 1.1	0.2 – 0.5	0.02 – 0.05
248.0 – 241.0	Varved Zone	(see Figure 2)		1.0	1.3	0.6	0.06
241.0 – 238.0	Lower	n/a*		n/a*	n/a*	n/a*	n/a*

*Not impacted by loading.

As discussed in Section 4.2.5, an approximately 7 m thick portion of the silty clay to clay deposit is varved, between about Elevation 248 and 241 m, as evidenced both visually in the samples and confirmed in the plot of water content and Atterberg limits versus elevation on Figure 3. In the upper portion of the deposit, between about Elevation 251 m and 248 m, the coefficient of consolidation, c_v (n/c), equal to $3.0 \times 10^{-3} \text{ cm}^2/\text{s}$ is considered appropriate for the normally consolidated range and a c_v (o/c) of $1.7 \times 10^{-2} \text{ cm}^2/\text{s}$ is considered appropriate for the recompression range.

A review of the laboratory consolidation test data and case studies of embankment settlement on varved clays in Northern Ontario (including the New Liskeard area) in literature including by Stermac et al. (1967), Lo and Stermac (1965) and Milligan et al. (1962), an average value of c_v (n/c), equal to $1.4 \times 10^{-3} \text{ cm}^2/\text{s}$ is considered appropriate for the normally consolidated varved clay stratum at this site. Further, based on a back-analysis of the embankment settlement monitoring data in the above noted literature, the c_v (n/c) value should be applied to, and the time-rate settlement analysis should consider, the full varved clay deposit thickness which is impacted by the new fill loading. In the recompression range, a value of c_v (o/c) of $2.3 \times 10^{-2} \text{ cm}^2/\text{s}$ is considered appropriate.

Since the water content determinations in the laboratory are typically carried out on combined samples of both the clay and clayey silt laminae, the actual water content (and therefore the value of $C_{a(\epsilon)}$) in the clay laminae will be higher than in the clayey silt laminae. The secondary compression index, $C_{a(\epsilon)}$, was estimated directly from the results of the laboratory consolidation tests (using the dial reading versus log-time plots) resulting in a more representative value of $C_{a(\epsilon)}$ of about 1.3 percent. The creep settlement calculation was carried out by considering only the summed thickness of the clay laminae portion of the varved clay/clayey silt deposit, which is estimated to be about 90 percent of the total thickness of the deposit.

6.6.4.4 Results of Analysis

A summary of the results of the settlement analysis at the critical sections (i.e. at the abutments and the approaches 20 m behind the abutments) is presented below. Settlement of the new Granular B Type II fill above the water level is typically less than 25 mm if placed and compacted properly (see Section 6.8). Figure 8 shows the estimated consolidation settlement versus time for the clay deposit.



Critical Section	Relevant Borehole	Estimated Immediate Settlement (mm)			Consolidation Settlement (mm)		Total Settlement	Post-Construction Settlement
		New Granular Fill*	Existing Fill/ Alluvium	Silty Clay to Clay	Silty Clay to Clay			
					Primary	Creep***		
North Approach	WN-1	<25 mm	<25 mm	<25 mm	0	0	<25 mm	<25 mm
North Abutment	WN-2		<25 mm	<25 mm	120	80	225 mm	200 mm
South Abutment	WN-3		<25 mm	<25 mm	115	50	190 mm	165 mm
South Approach	WN-4		<25 mm	<25 mm	0	0	<25 mm	<25 mm

* Granular 'B' Type II.

** Assumes that post-construction settlement begins when the embankment has reached its final height, in this case, when the abutment is completely backfilled.

*** Creep magnitude from completion of about 90 percent of primary consolidation to the 20-year design life.

Based on the c_v values given in Section 6.6.4.3, it is estimated that about 90 percent of the primary consolidation settlement will be completed in about 4 years and 7 years at the north and south abutments, respectively. The magnitude of secondary (creep) settlement is expected to be about 115 mm per log cycle. Therefore, less than one log cycle of creep will occur from completion of about 90% of primary consolidation to the design life of the approach embankment (i.e. 20 years) as indicated above.

Since the post-construction settlement at the abutments is greater than the 25 mm allowable criteria (see Section 6.6.4.2), settlement mitigation will be required. Given the potential for differential settlement along the proposed northeast retaining wall, located between about 7 m and 12 m behind the proposed north abutment, settlement mitigation will also be required in this area.

6.6.5 Mitigation of Stability Issues and/or Time-Dependent Settlements

In order to achieve a FoS equal to or greater than 1.3 for both the front slopes and side slopes and to minimize post-construction settlement, the alternatives presented below can be considered. The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table 2. The preferred alternative from a foundations perspective to provide the best technical solution in terms of the long-term operational performance of the roadway is the use of lightweight fill comprised of expanded polystyrene (EPS) fill within a portion of the approach embankments behind the abutments to reduce the loading on the underlying firm silty clay to clay deposit. Other alternatives such as changing the bridge span lengths and/or highway geometry, preloading/staged construction, sub-excavation, slope flattening/toe berms and wick drains have also been considered, but do not appear to be of advantage over the use of EPS.



6.6.5.1 *Lightweight Fill*

We recommend that EPS fill be used to reduce the embankment fill load on the subsoils thereby improving stability, reducing post-construction settlement and eliminating downdrag loads on the piles. Typically, EPS is considered to be too expensive to consider as a solution, however, given the small volume of material to be used at this site, it is not precluded as an option based on cost alone.

The results of the stability analysis using EPS in the zone behind both abutments and achieving a FoS of greater than 1.3 are shown on Figures 4 to 7 for the north and south front slopes and northeast and southwest side slopes (including the northeast retaining wall). Although the results show that 3 m maximum thickness of EPS is required, this value is being governed by the southwest side slope. To maintain equilibrium of loading behind both abutments, the 3 m thickness of EPS on the south approach has been mirrored on the north side; however, the results show that a thickness of 2 m of EPS would be required to achieve the minimum FoS of 1.3 at the northeast side slope and retaining wall.

The EPS should be 3.0 m thick from the abutment to 12 m behind the abutments (and at the end of the retaining wall on the northeast side), stepping up (decreasing in thickness) in 0.5 m increments from 12 m behind the abutments at approximately 5H:1V (or approximately 0.5 m by 2.5 m long steps), for a taper distance of about 12.5 m. This will result in the EPS extending a total distance of about 24.5 m behind the abutments. Figure 9 shows the recommended configuration of the EPS behind the abutments in the approach areas in both plan and cross-sections. EPS is typically provided in blocks 0.5 m thick to accommodate the steps. Appropriate staggered layout, ties and spacers should be used to ensure the EPS block mass acts as a single unit. The EPS should be placed on a granular pad consisting of a 300 mm thick layer of Granular 'B' Type II and 100 mm thick levelling layer of mortar sand or Granular 'A'. The finished top of the EPS blocks should be covered with a 6 mil (0.15 mm) thick polyethylene sheet and provided with a minimum 1 m of conventional granular cover (pavement structure) and include a 125 mm thick concrete slab, for ballast and for protection against differential icing at the roadway structure. It is estimated that a volume of EPS of about 900 m³ will be required to satisfy the recommendations for this site.

Buoyancy of the EPS is not a consideration in this case since the base of the EPS blocks will be well above the highest measured water level.

A NSSP should be included in the Contract for Rigid Expanded Polystyrene Embankment Fill and an example is included in Appendix C.

6.6.5.2 *Bridge/Highway Geometry*

Consideration could be given to moving the abutments further back from the river to reduce the post-construction settlement at the abutments and to improve stability. Further, lowering the highway grade at the bridge could also be considered. In either case, some post-construction settlement would still occur due to the settlement resulting from the difference in density between the existing soils and new fill and any additional regrading fill that may be used under the pavement structure. In general, stability would be improved for the front slopes, however, the northeast side slope, including retaining wall, would still require stability mitigation. Further, lowering the grade would result in a lower underside of pile cap excavation, which could extend below the water level and result in the need for temporary shoring/dewatering, whereas the current bridge arrangement will require a dry, open cut excavation for pile cap construction.



6.6.5.3 *Preloading/Staged Construction*

It is estimated that 90 percent of primary consolidation settlement would be completed in about 1 year for the north abutment and 1.7 years for the south abutment (for the presently proposed embankment configuration [i.e. 2.0 m and 1.6 m grade raise]). The total post-construction settlement would be about 60 mm (including remaining primary and creep settlement) and additional mitigation measures would be required to reduce this value below the settlement criteria.

However, given that the embankments are not stable at the full embankment loading, a staged construction approach to preloading would be required. Given the sequencing of construction and time required to construct the preload embankment in stages and the uncertainty associated with obtaining the necessary strength gain in the underlying soils to maintain stability during each stage of embankment construction, preloading/staged construction is not recommended. Further, an embankment instrumentation and monitoring program would be required.

6.6.5.4 *Slope Flattening/Toe Berms*

The use of toe berms or flatter slopes would improve overall stability at the site. However, due to the proximity of the river and other property restrictions, and given that this alternative would not mitigate post-construction settlement or downdrag loading on piles (unless combined with preloading/staged construction methods), this option is not recommended.

6.6.5.5 *Sub-Excavation of Silty Clay to Clay Deposit*

Due to the depth of the silty clay to clay deposit (approximate 11 m below the underside of pile cap) and the potential for encountering artesian groundwater conditions, sub-excavation of the firm, compressible soils and subsequent backfilling of the excavated area with Granular 'B' Type II material (or rock fill) is not considered practical for the limited area over which such remedial works would be required.

6.6.5.6 *Wick Drains*

Wick drains would decrease the time required for primary settlement. However, since this option would have to be combined with the preloading/staged construction, this option is not recommended.

6.7 *Retaining Wall*

We understand that a retaining wall is required to accommodate a 2H:1V side slope at the northeast corner of the replacement bridge without filling into the river which flows at a skew to this slope. The current embankment slope angle is approximately 1.6H:1V and therefore the grade on the outside of the wall will be approximately 1.3 m lower than the proposed highway grade. From the drawings provided by LEA, the proposed retaining wall will extend 5.3 m beyond the end of the 6.8 m long northeast abutment wing wall. To accommodate the required 2.5 m thickness of soil cover for frost protection on the outside of the wall, the proposed retaining wall is to be founded at about Elevation 254.2 m, resulting in a 4.6 m high retaining wall (i.e. 1.3 m of exposed wall). The subsoils at the founding level consist of stiff silty clay desiccated crust or firm to stiff silty clay alluvium (i.e. slightly organic). This foundation level is approximately 3 m above the surface of the firm silty clay and 6 m above the surface of the firm, varved silty clay to clay.



The proposed retaining wall could consist of a concrete gravity wall or a Retained Soil System (RSS) wall. We understand that an RSS wall is not desirable from a highway design perspective due to the difficulty associated with installing guide rail as well as the need for a proprietary design. Table 3 summarizes the advantages, disadvantages, relative costs and risks/consequences of the different wall types at this site from a foundations perspective. Design recommendations for the two options are given in the sections below.

The results of the stability analysis for this portion of the embankment side slope including the retaining wall, as presented in Section 6.6.2.3, indicate that the overall global stability has a FoS less than 1.3. Further, the results of the settlement analysis given in Section 6.6.4.4 indicate that post-construction settlement would be differential along the wall and greater than what could be functionally tolerated by a retaining wall. Therefore, mitigation of stability and settlement using 3 m of EPS behind the abutment (and thus behind the retaining wall) is required prior to constructing the wall as described in Section 6.6.5.1.

6.7.1 Concrete Gravity Retaining Wall

If no-foundation site constraints preclude the use of an RSS wall, then a concrete gravity wall would be appropriate for this site. The wall could be supported by spread footings, or deep foundations if the bearing resistance is not sufficient to allow a spread footing option. Stability and settlement mitigation works must be implemented prior to construction of the wall as discussed above.

For a 3 m wide strip footing founded on the firm to stiff silty clay crust or alluvium at Elevation 254.2 m, the resistances are governed by the shear strength of the underlying silty clay to clay deposit and a factored geotechnical resistances at ULS of 150 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 125 kPa may be used for design. If a granular fill pad constructed of compacted Granular 'B' Type II extending to approximate Elevation 252.2 m (i.e. at least 2 m thick) is used to support the strip footing, values of 225 kPa at ULS and 125 kPa at SLS may be used for design. .

Resistance to lateral forces/sliding resistance between the base of the poured concrete (i.e. cast-in-place) footing and the compacted granular pad should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$ may be taken as 0.50 between the base of the cast-in-place concrete footings and the silty clay to clay, constructed in-the-dry. For pre-cast concrete footings, $\tan \delta$ should be taken as 0.40. These values represent unfactored values.

6.7.2 Retained Soil System (RSS) Walls

If non-foundation site constraints allow, an RSS wall could be considered at this site. An RSS wall consists generally of granular fill placed and compacted in layers, and reinforced with fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the vertical face of the retained soil structure and to prevent loss of fill material. A typical RSS wall has the front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. In this case, the full frost depth is not required and the founding elevation of the footing could be raised. However, since EPS is required for mitigation of stability, it may be difficult to incorporate EPS fill into an RSS wall design.



The final grading design should be checked to provide approximately 0.3 m of embedment for the front facing footing of the RSS wall. A minimum 150 mm thick granular fill levelling pad comprised of Granular 'B' Type II will be required under the footing and the RSS mass. This granular fill levelling pad should be constructed over the native soils at the founding level and should extend a minimum of 1 m beyond the edges of the footing and soil mass. Granular 'B' Type II is recommended for the levelling pad since it is already being used to backfill the abutment.

The RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which has been taken as 0.8 times the height of the wall (assumed to be a maximum of 4.6 m high) for an approximate reinforced width of 3.7 m. A factored geotechnical axial resistance at ULS of 150 kPa and a geotechnical resistance at SLS (for 25 mm of settlement) of 100 kPa may be used for assessment of the reinforced mass founded on the properly prepared subgrade and levelling pad. This assumes that the backfill is fully comprised of Granular 'B' Type II and no EPS.

The resistance to lateral forces / sliding resistance between the compacted fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \delta$, between the compacted granular fills of the RSS wall and the properly prepared subgrade may be taken as 0.50. This represents an unfactored value.

The internal stability of the wall should be checked by the RSS wall supplier/designer.

6.8 Subgrade Preparation and Embankment Construction

For the bridge approach embankments within 20 m of the abutments, removal of the topsoil and fill and partial removal of the slightly organic silty clay to clay alluvium to a total depth of about 1 m is required below the reconstructed embankment footprint prior to placement of new fill. All softened/loosened soils should also be stripped from below the approach embankments, prior to placement of new fill. The backfill in the frost taper zone should be constructed in accordance with OPSD 3101.150 (Walls, Abutment, Backfill Minimum Granular Requirement).

Granular fill materials and placement should be in accordance with the requirements as outlined in SP 206S03 (Earth Excavation, Grading). All granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 percent of the standard Proctor maximum dry density. Prior to placement of the pavement granular subbase and base courses, the final lift of embankment fill should be compacted to not less than 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. The new fill should be keyed into the existing embankment side slopes per the requirements of OPSD 208.010 (Benching of Earth Slopes).

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



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

6.9 Design and Construction Considerations

6.9.1 Excavations

The excavation for pile cap and filter blanket construction will extend to Elevation 251.4 m and 251.8 m at the north and south abutments, respectively, resulting in excavations up to 6.2 m below the existing ground surface at the abutments. Excavation for the northeast retaining wall footing will extend to Elevation 254.2 m, that is up to 4.1 m below ground surface. Since the water level was encountered between Elevation 250.2 and 252.1 m, it is possible that, depending on the time of year, the abutment excavations could extend up to 0.7 m below the ground/river water level.

As it is proposed that the road will be closed during construction, it will be possible to carry out the excavations in open cut. Open cut slopes within the fill materials, silty clay to clay alluvium and stiff silty clay to clay should be maintained at no steeper than 2H:1V above the water level and below the water level, if encountered.

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. □

6.9.2 Groundwater and Surface Water Control

Although perched water was not encountered within the fill during the investigation, it is possible that water is perched within the fill materials, and the Contractor should anticipate perched water within the fill sub-excavation for the bridge approach embankments. Surface water should be directed away from the excavation at all times.

The excavation for the abutments will be located adjacent to the Wicklow River. As discussed in Section 6.9.1, the excavations for the pile caps and filter blanket may be up to 0.7 m below the high river water level as measured during the time of the subsurface investigation. If the open cut excavations extend below the water level, temporary shoring with dewatering may be required. Temporary shoring and dewatering could be in the form of a sheet-pile cut off wall, a cofferdam or temporary pumping from properly filtered sumps below the base of the excavation for localized groundwater control.

6.9.3 Temporary Shoring

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



6.9.4 Filter Blanket

Given the depths to the artesian groundwater bearing deposits as noted during the subsurface investigation (refer to Record of Borehole sheets in Appendix A) and the recommended pile tip elevations (given in Section 6.3.1), we recommend that a drainage/filter blanket consisting of a minimum 0.5 m thick layer of concrete fine aggregate (OPSS 1002, Aggregates, Concrete) be placed below the underside of the pile caps encasing all the piles. The base of the filter blanket, 0.5 m below the underside of pile cap, will extend to Elevation 251.4 m and 251.8 m at the north and south abutments, respectively. The concrete fine aggregate layer should extend a minimum of 0.5 m horizontally beyond each of the pile caps. Further, the excavation at the front of the abutment (towards the river) should be backfilled with free draining material extending at least 0.5 m horizontally from the front face of the abutment.

6.9.5 Obstructions

As part of the design and construction of the new abutment foundations, careful consideration should be given to the location of the existing piles relative to temporary shoring (if required) and replacement bridge piles. Specifically, the designer should check that the new piles (batter and orientation) and temporary shoring do not interfere with the existing piles. This should be checked to the full extent of the pile/shoring length.

The existing timber piles extend to depths between 9 m and 15 m below the ground surface, which is above the elevation where artesian pressures were encountered during the field investigation. However, if the timber piles are removed, there would be only about half the thickness of the silty clay stratum resisting the artesian pressure at the base of the extraction hole and, therefore, there would be a risk of creating a pathway for artesian groundwater and for potential ground loss. Further, backfilling the pile holes, where removed within the river channel, may not be feasible or practical below the water level. We recommend that the existing timber piles be left in place and “cut off” at the river bed level and not be pulled out.

7.0 CLOSURE

This report was prepared by Mr. David Muldowney, P.Eng., and the technical aspects were reviewed by Ms. Sarah E.M. Coyne, P.Eng., Associate. A quality control review of the report was provided by Mr. Jorge M.A. Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project.



Report Signature Page

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DAM/SEMC/JMAC/lb

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 - ASTM D1587 Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
 - ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil
- Commercial Software
- LPILE PLUS (Version 5.0) by Ensoft Inc.
 - GeoStudio (Version 7.17) by Geo-Slope International Ltd.
 - Settle 3D (Version 2.003) by Rocscience Inc.



Ministry of Transportation Ontario Special Provisions

- SP 110S13 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
- SP 206S03 Earth Excavation, Grading
- SP 511S01 Rip Rap; Rock Protection, Gravel Sheeting

Ontario Provincial Standard Drawings

- OPSD 208.010 Benching of Earth Slopes
- OPSD 3000.150 Foundation Piles, Steel H-Pile Splice
- OPSD 3090.100 Foundation, Frost Penetration Depths for Northern Ontario
- OPSD 3101.150 Walls Abutment, Backfill Minimum Granular Requirement
- OPSD 3101.200 Walls Abutment, Backfill Rock
- OPSD 3121.150 Walls Retaining, Backfill Minimum Granular Requirement

Ontario Provincial Standard Specifications

- OPSS 501 Construction Specification for Compacting
- OPSS 511 Construction Specification for Rip Rap, Rock Protection, and Granular Sheeting
- OPSS 539 Construction Specification for Temporary Protection Systems
- OPSS 802 Construction Specification for Topsoil
- OPSS 804 Construction Specification for Seed and Cover
- OPSS 903 Construction Specification for Deep Foundations
- OPSS 1002 Material Specification for Aggregates – Concrete
- OPSS 1004 Material Specification for Aggregate - Miscellaneous

Ontario Provincial Standard Structural Drawings

- SS 103-11 Pile Driving Control, 2002

Ontario Water Resources Act

- Ontario Regulation 468/10 Amendment to Ontario Regulation 903
- Ontario Regulation 903/90 Wells



**FOUNDATION REPORT, REPLACEMENT OF WICKLOW RIVER BRIDGE NORTH
HIGHWAY 7037, SITE NO. 39E-174 GWP 5139-06-00**

Table 1: Evaluation of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven Steel H-Piles	1	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Higher axial resistance compared to spread footings. ■ Fewer problems associated with artesian groundwater compared to caissons. ■ Suitable for integral abutment design. 	<ul style="list-style-type: none"> ■ Risk of problems associated with penetrating the artesian deposit, such as pile settlement or ground loss. ■ Requires excavation for pile cap construction. 	<ul style="list-style-type: none"> ■ Relative costs lower than caissons but higher than spread footings. 	<ul style="list-style-type: none"> ■ Low risk of not achieving design resistance at design pile tip elevation. ■ Low risk of problems associated with artesian groundwater during pile driving, but grouting along the piles could be necessary.
Caissons	2	<ul style="list-style-type: none"> ■ Higher axial resistances compared to steel H-piles. ■ Pile cap can be constructed at the bridge deck level, therefore excavation is not required. 	<ul style="list-style-type: none"> ■ Higher risk of problems associated with penetrating the artesian deposits and cobbles and boulders compared to piles. ■ Not suitable for integral abutment design. 	<ul style="list-style-type: none"> ■ Relative costs much higher than steel H-piles. 	<ul style="list-style-type: none"> ■ High risk of not reaching the required termination depth due to the presence of cobbles and boulders. ■ High risk of construction problems associated with artesian groundwater during caisson installation.
Shallow Spread Footings on granular pad	3	<ul style="list-style-type: none"> ■ Conventional construction. ■ Removes potential complications associated with artesian groundwater conditions. 	<ul style="list-style-type: none"> ■ Construction of granular pad will cause consolidation settlement of the underlying native soils. ■ Front slope will likely not be stable with extra load from footings. ■ Dewatered excavation (cofferdam) required adjacent to the river to allow for construction of a granular pad in-the-dry. ■ Not suitable for integral abutment design. 	<ul style="list-style-type: none"> ■ Typically lower cost than deep foundations. ■ Cost of extra fill for granular pad. 	<ul style="list-style-type: none"> ■ Post-construction settlement of subsoils will occur and will require mitigation prior to footing construction. ■ Some risk of settlement of the granular pad if sub-aqueous filling is carried out (without compaction).



**FOUNDATION REPORT, REPLACEMENT OF WICKLOW RIVER BRIDGE NORTH
HIGHWAY 7037, SITE NO. 39E-174 GWP 5139-06-00**

Table 2: Evaluation of Stability Mitigation Alternatives – Side Slopes

Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Lightweight Fill (EPS)	1	<ul style="list-style-type: none"> Reduces load on compressible subsoils thereby improving stability and reducing post-construction settlement of foundation subsoils, including under the retaining wall. Straightforward installation of EPS blocks within embankment fill. Will be placed above the high water level and therefore buoyancy is not a concern. No time delay in construction. Eliminates downdrag loading on piles. 	<ul style="list-style-type: none"> High cost of EPS material. Requires concrete slab and minimum of 1 m of conventional soil cover to mitigate potential for differential icing. 	<ul style="list-style-type: none"> EPS cost is up to an order of magnitude higher than other fill materials. 	
Change in Bridge Geometry <ul style="list-style-type: none"> Lower grade Move abutments back to location of existing abutment or beyond 	2	<ul style="list-style-type: none"> Improve front slope stability. Lowering grade would reduce consolidation settlement of subsoils. 	<ul style="list-style-type: none"> Consolidation settlement of subsoils would still occur due to difference between existing and new fill, however, would be less than for abutments in currently proposed location. If grade is lowered, resulting lower pile caps would require excavation below the water level and subsequent shoring/dewatering for construction in-the-dry. 	<ul style="list-style-type: none"> Possible additional costs for shoring/cofferdam. Longer bridge span would be more expensive 	<ul style="list-style-type: none"> Mitigation of settlement may still be required. Longer bridge could require central pier.
Preloading/Staged Construction	NF	<ul style="list-style-type: none"> Reduces post-construction settlement. 	<ul style="list-style-type: none"> Staged construction will be required due to stability. Time delay in schedule to allow for preloading (or staged preloading). Logistically difficult to construct staged preload in limited area beside river. Instrumentation and monitoring program would be required. 	<ul style="list-style-type: none"> Increased overall costs due to extra time required in schedule. Cost of instrumentation and monitoring program. 	<ul style="list-style-type: none"> Moderate risk that not enough strength gain will occur in underlying cohesive soils to allow the next stage of embankment load to be placed.



**FOUNDATION REPORT, REPLACEMENT OF WICKLOW RIVER BRIDGE NORTH
HIGHWAY 7037, SITE NO. 39E-174 GWP 5139-06-00**

Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Flatten Slopes/Toe Berms	NF	<ul style="list-style-type: none">■ Lowers the driving force thereby improving stability.	<ul style="list-style-type: none">■ Would require retaining wall at toe of slope to avoid filling into the river, or require wider right-of-way.■ Consolidation settlement of subsoils will occur.	<ul style="list-style-type: none">■ Cost of permanent retaining walls at toe of slopes.	<ul style="list-style-type: none">■ Post-construction settlement of subsoils will occur.
Sub-excavation of Silty Clay to Clay Deposit	NF	<ul style="list-style-type: none">■ Reduces post-construction settlement.■ Improves stability only if removed at and beyond the toe of slope.	<ul style="list-style-type: none">■ Requires significant excavation below the water level including beyond the toe of slope (and into river).■ Shoring may be required.	<ul style="list-style-type: none">■ Cost of replacement backfill.■ Cost of excavation/shoring.	<ul style="list-style-type: none">■ High risk of post-construction settlement of fill material placed below the water level.■ Moderate risk of instability.
Wick Drains	NF	<ul style="list-style-type: none">■ Reduces time for primary consolidation to occur.	<ul style="list-style-type: none">■ Increases magnitude of creep settlement.■ Must be used in conjunction with preloading/ staged construction due to stability concerns.	<ul style="list-style-type: none">■ Additional cost of foundation investigation and design, and instrumentation and monitoring program.	<ul style="list-style-type: none">■ Creep settlement will occur.

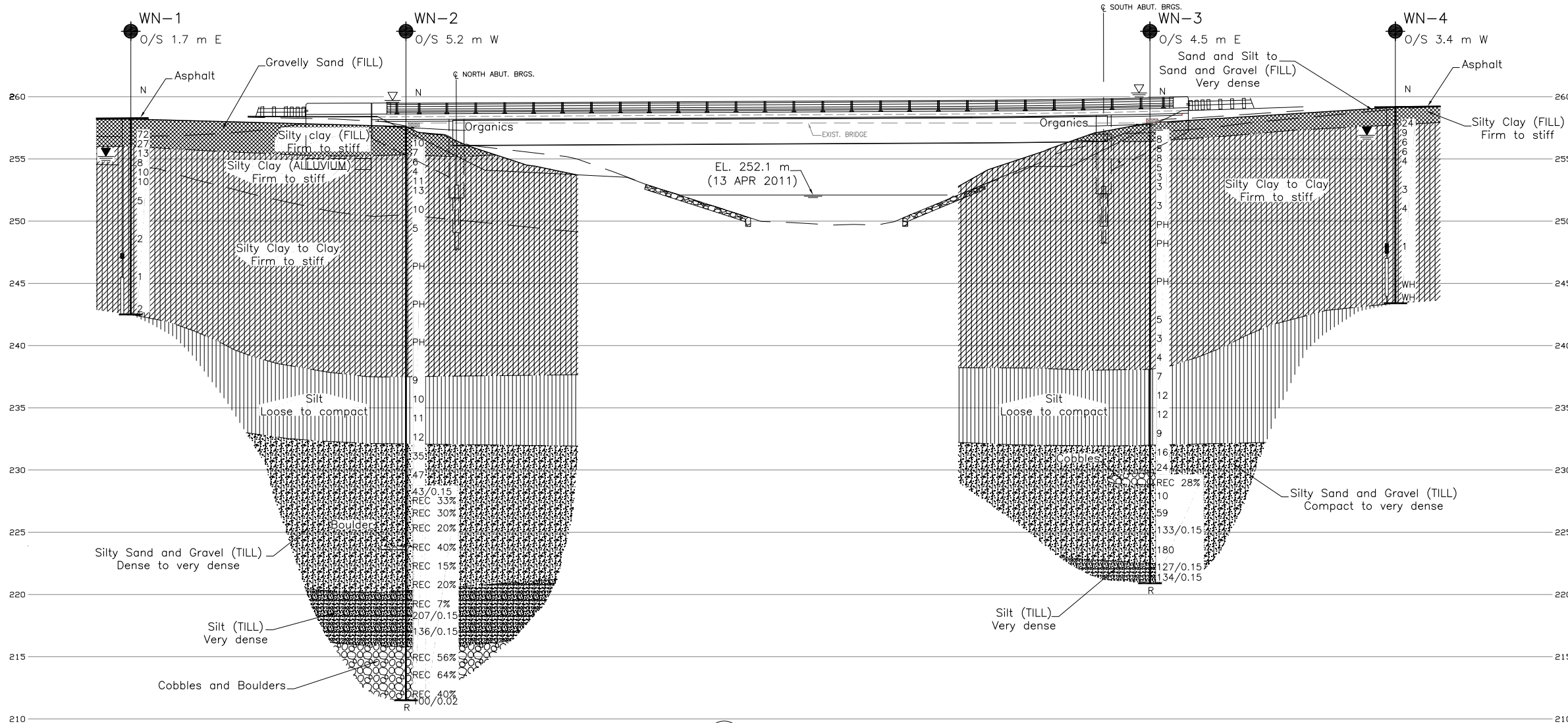
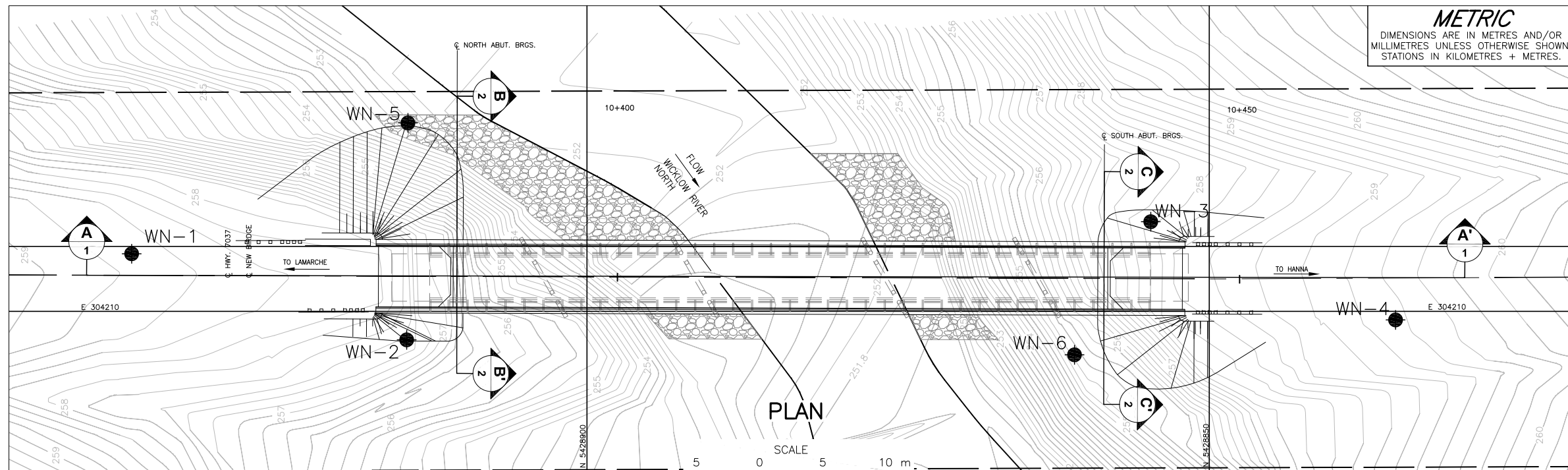
NF: Considered not technically, cost or schedule feasible to mitigate stability and/or settlement.



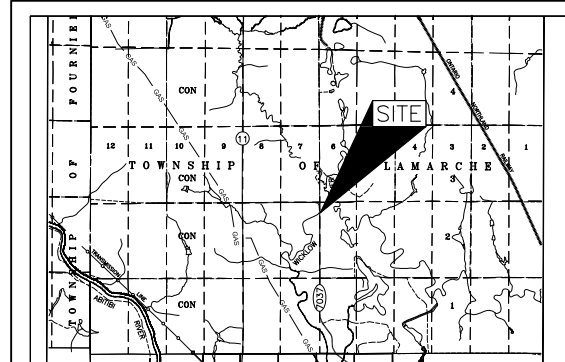
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HIGHWAY 7037, SITE NO. 39E-174 GWP 5139-06-00**

Table 3: Evaluation of Retaining Wall Alternatives

Wall Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Concrete Gravity Wall	1	<ul style="list-style-type: none">■ Straightforward construction.■ Can be used with EPS fill.■ Likely faster construction than RSS wall system.	<ul style="list-style-type: none">■ Lower footing elevation compared to RSS wall requires additional excavation depth.	<ul style="list-style-type: none">■ Higher cost of materials compared to RSS wall.	<ul style="list-style-type: none">■ Settlement and stability must be mitigated prior to wall construction.
Retained Soil System (RSS) Wall	2	<ul style="list-style-type: none">■ Straightforward construction.■ Higher footing elevation and less excavation required compared to concrete wall.■ Can better accommodate some settlement or differential settlement.	<ul style="list-style-type: none">■ Difficult to incorporate EPS fill into wall backfill.■ Proprietary wall system■ Likely require importing granular fill for wall mass rather than using earth fill.	<ul style="list-style-type: none">■ Typically lower cost than concrete wall.	<ul style="list-style-type: none">■ Settlement and stability must be mitigated prior to wall construction.

CONT No.
GWP No. 5139-06-00WICKLOW RIVER BRIDGE NORTH
HIGHWAY 7037
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on JULY 03, 2011
- WL upon completion of drilling
- Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
WN-1	258.3	5428936.6	304214.6
WN-2	257.6	5428914.5	304207.7
WN-3	257.9	5428854.7	304217.2
WN-4	259.2	5428835.0	304209.3
WN-5	252.3	5428914.4	304225.1
WN-6	254.5	5428860.8	304206.5

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 2.01 of OPS General Conditions.

REFERENCE

Base plan provided in digital format by LEA, drawing file no. 8743-WN-S01.dwg, dated MAY 2011, received JUNE 16, 2011, x8743-WN-Contour.dwg, received APR 27, 2011, x8743-WN Base.dwg, received MAY 20, 2011.



NO.	DATE	BY	REVISION
1	2011	JUL	1
Geocres No. 42A-88			
HWY. 7037		PROJECT NO. 09-1191-0022	DIST.
SUBM'D. DAM	CHKD. SEMC	DATE: SEPT 2011	SITE: 39E-174
DRAWN: JJJ	CHKD.	APPD. JMAC	DWG. 1

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MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5139-06-00



WICKLOW RIVER BRIDGE NORTH
HIGHWAY 7037

SHEET

SOIL STRATA



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- R Refusal

BOREHOLE CO-ORDINATES

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WN-5	252.3	5428914.4	304225.1
WN-6	254.5	5428860.8	304206.5

NOTES

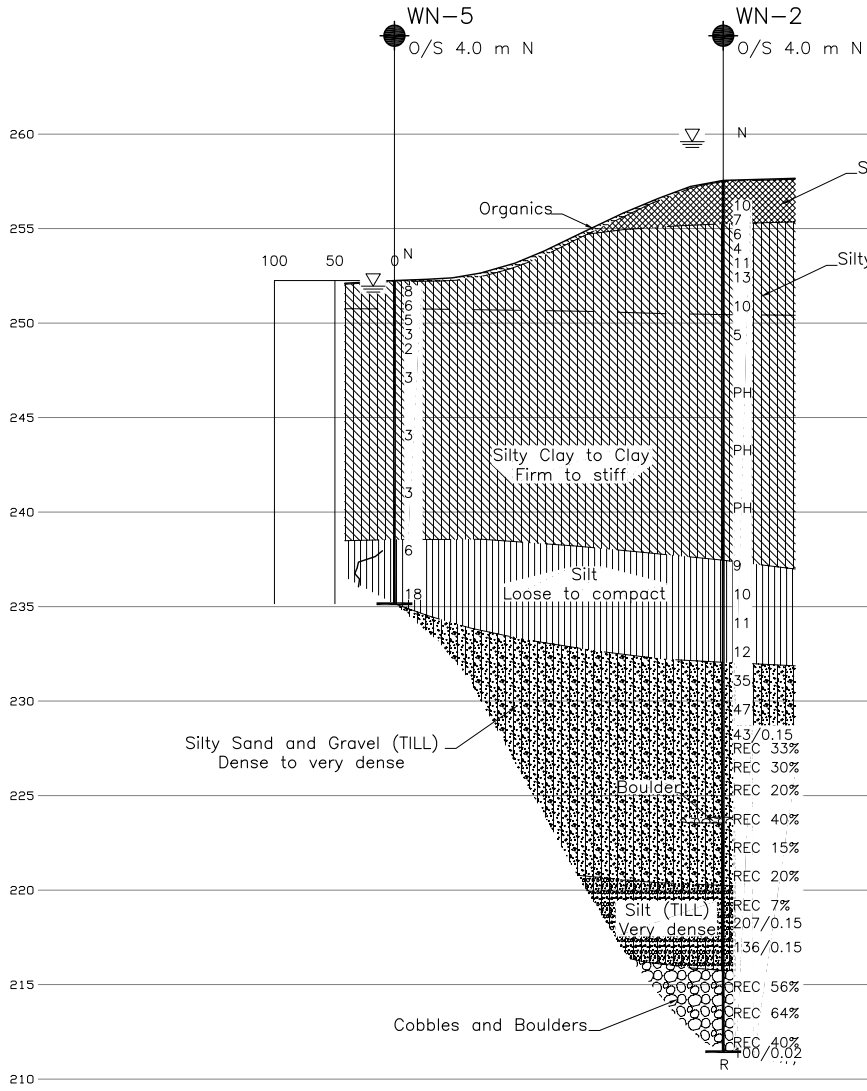
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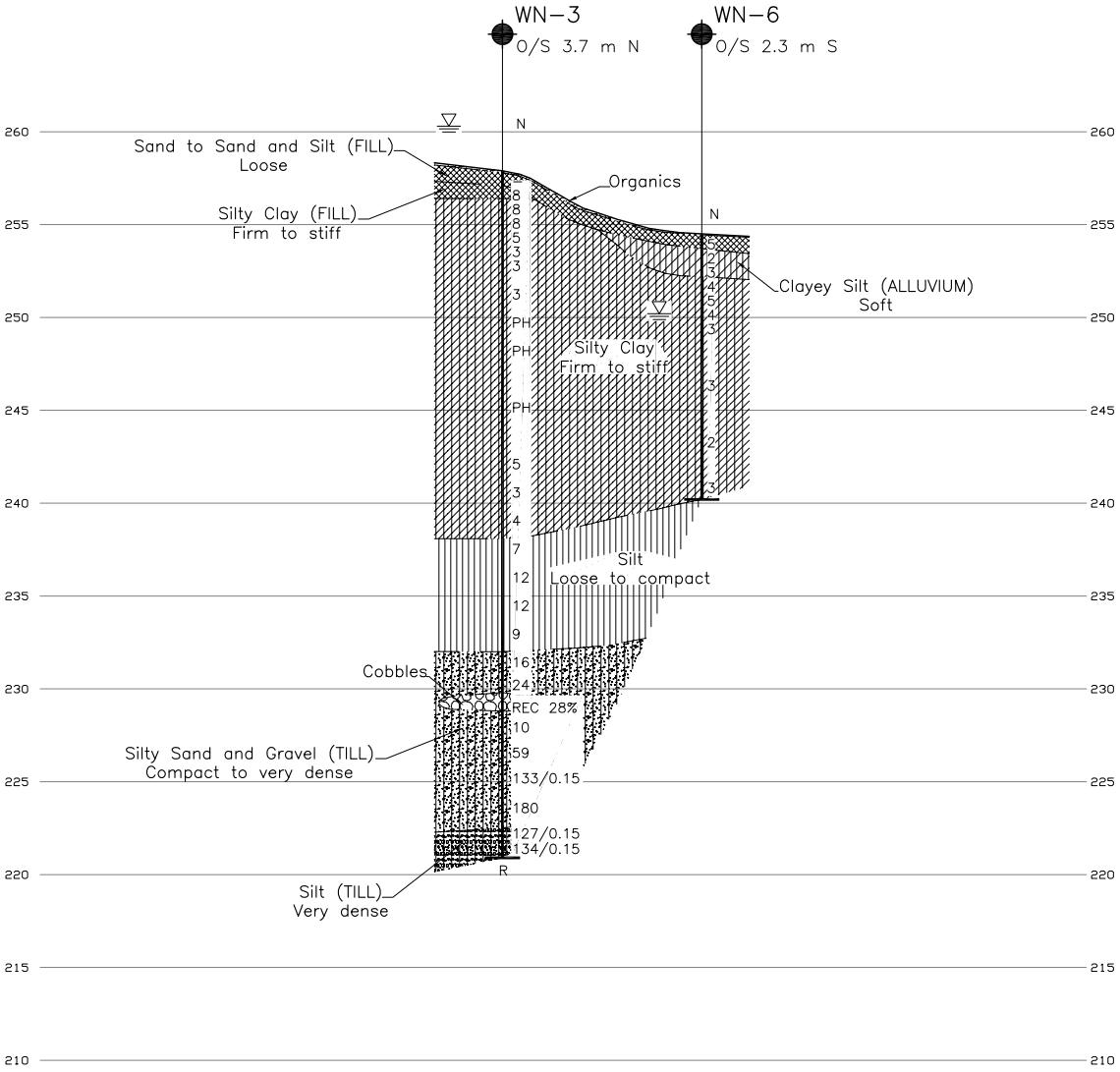
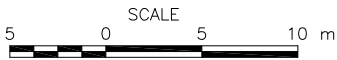
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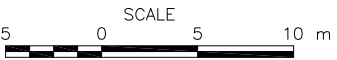
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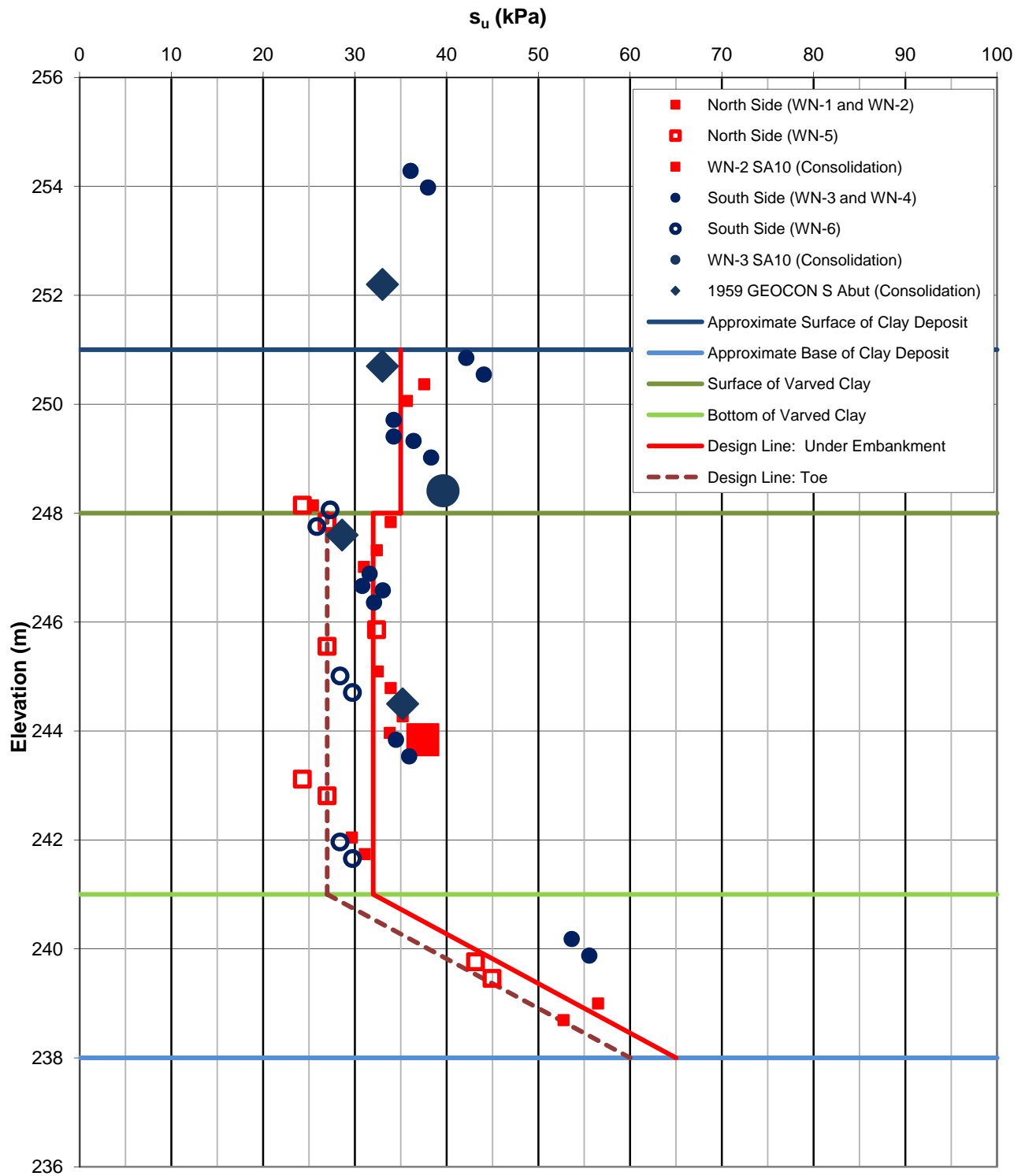
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HIGHWAY 7037



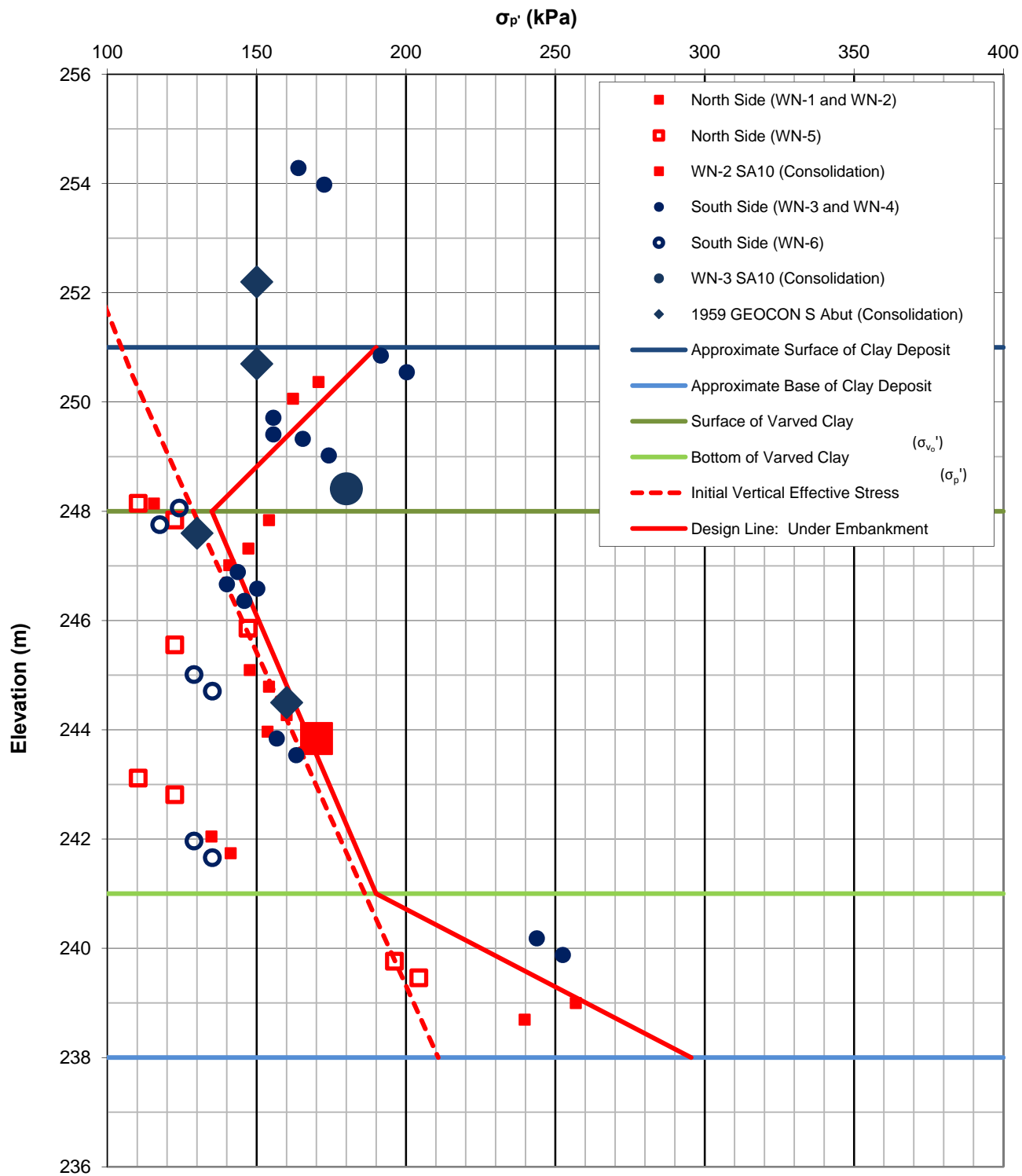
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HIGHWAY 7037



NO.	DATE	BY	REVISION
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HWY. 7037	PROJECT NO. 09-1191-0022	DIST.	
SUBM'D. DAM	CHKD. SEMC	DATE: SEPT 2011	SITE: 39E-174
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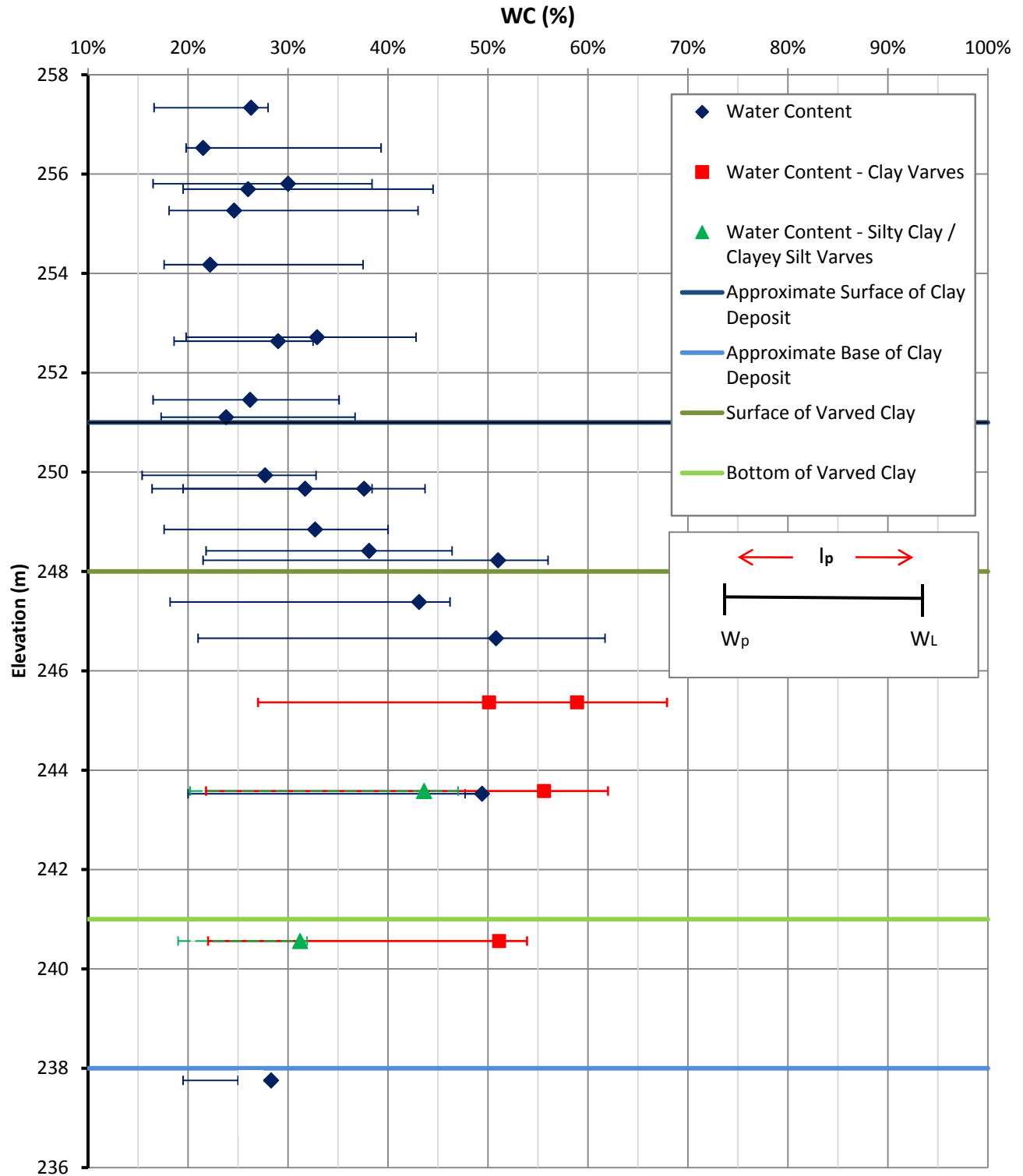
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	CADD	--			
	CHECK	SEMC	SEPT 2011		
	REVIEW	JMAC	SEPT 2011	Figure 1	




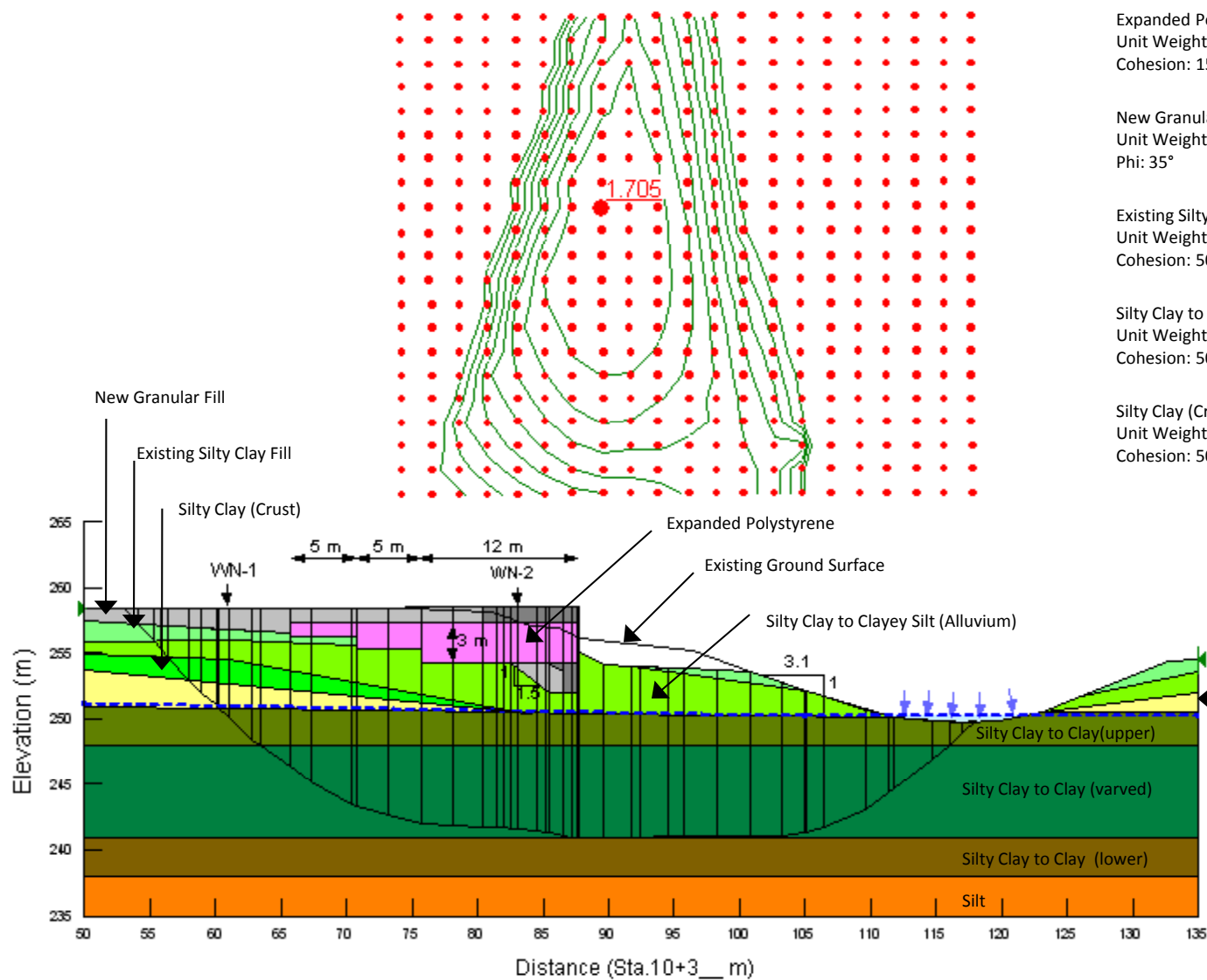
PROJECT				WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037			
TITLE				PRE-CONSOLIDATION PRESSURE VERSUS ELEVATION			
PROJECT No. 09-1191-0022				FILE No. ----			
DESIGN	DAM	SEPT 2011		SCALE	AS SHOWN	REV.	
CADD	--						
CHECK	SEMC	SEPT 2011					
REVIEW	JMAC	SEPT 2011					



Figure 2



PROJECT		WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037			
TITLE		WATER CONTENT AND ATTERBERG LIMITS VERSUS ELEVATION			
	PROJECT No.	09-1191-0022		FILE No. ----	
	DESIGN	DAM	SEPT 2011	SCALE	AS SHOWN REV.
	CADD	--			
	CHECK	SEMC	SEPT 2011		
	REVIEW	JMAC	SEPT 2011	Figure 3	



Expanded Polystyrene
Unit Weight: 0.5kN/m³
Cohesion: 15 kPa

New Granular Fill
Unit Weight: 21 kN/m³
Phi: 35°

Existing Silty Clay Fill
Unit Weight: 18 kN/m³
Cohesion: 50 kPa

Silty Clay to Clayey Silt (Alluvium)
Unit Weight: 17 kN/m³
Cohesion: 50 kPa

Silty Clay (Crust)
Unit Weight: 18 kN/m³
Cohesion: 50 kPa

Silty Clay to Clay (transition zone)
Unit Weight: 18 kN/m³
Cohesion: 50 kPa to 35 kPa

Silty Clay to Clay (upper)
Unit Weight: 18 kN/m³
Cohesion: 35 kPa

Silty Clay to Clay (varved)
Unit Weight: 18 kN/m³
Cohesion: 32 kPa

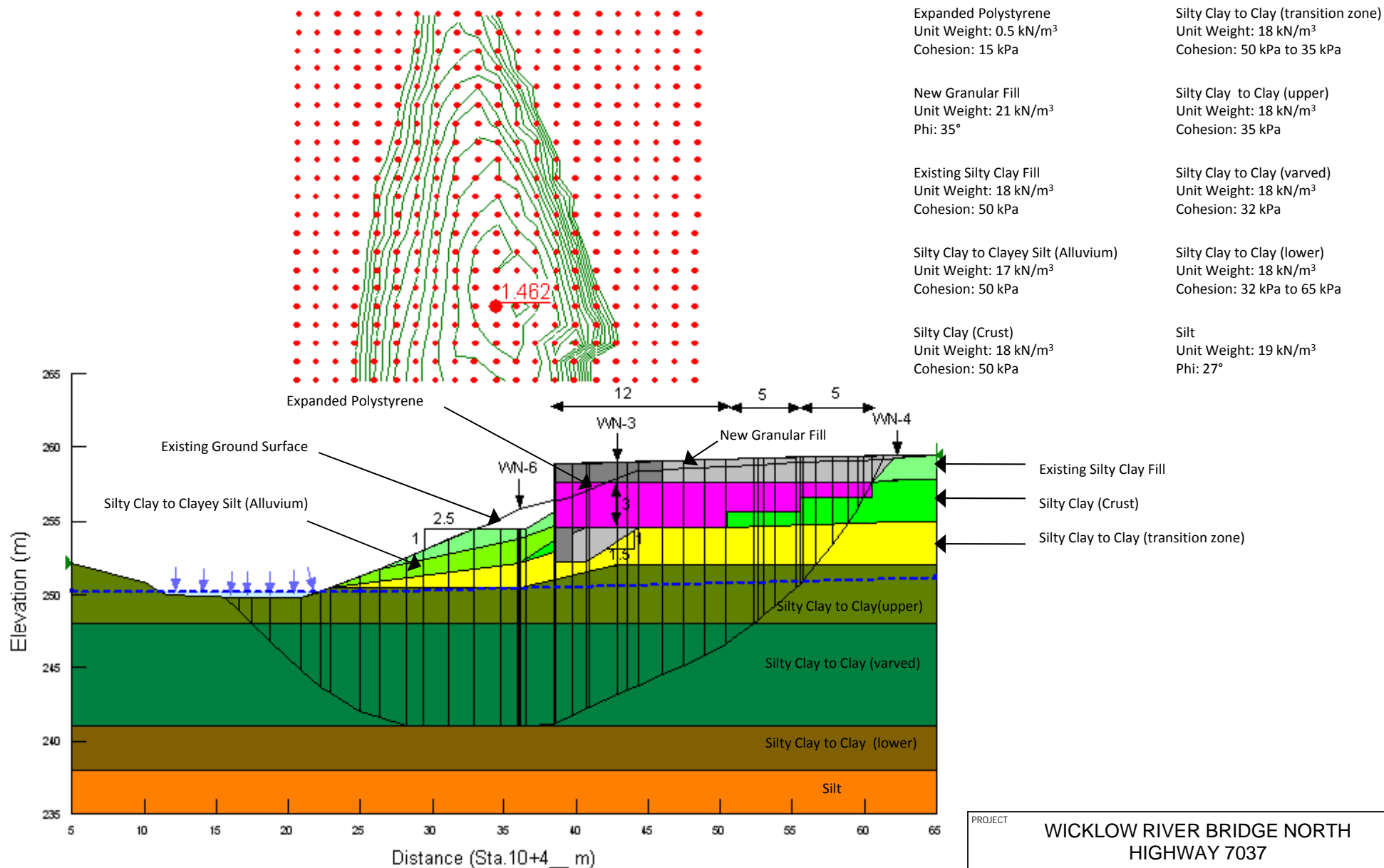
Silty Clay to Clay (lower)
Unit Weight: 18 kN/m³
Cohesion: 32 kPa to 65 kPa

Silt
Unit Weight: 19 kN/m³
Phi: 27°

PROJECT		WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037			
TITLE		STABILITY ANALYSIS NORTH FRONT SLOPE MITIGATED			
PROJECT No. 09-1191-0022		FILE No. ----			
DESIGN	DAM	SEPT 2011	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	SEMC	SEPT 2011			
REVIEW	JMAC	SEPT 2011			



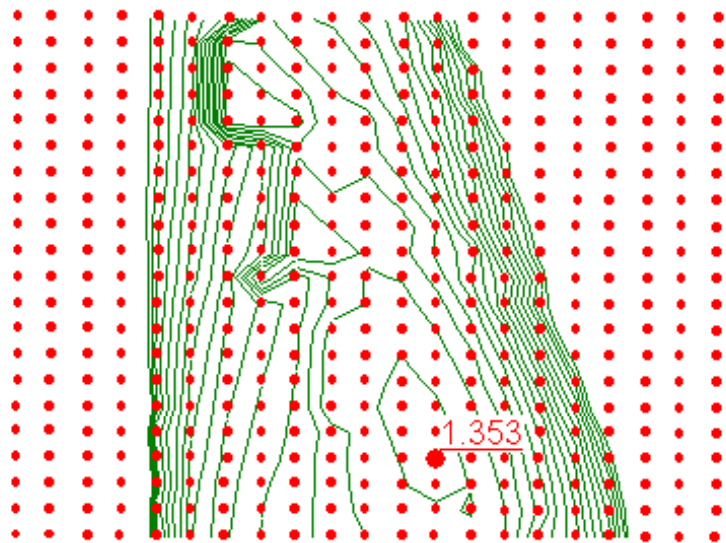
Figure 4



PROJECT				WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037			
TITLE				STABILITY ANALYSIS SOUTH FRONT SLOPE MITIGATED			
PROJECT No. 09-1191-0022				FILE No. ----			
DESIGN	DAM	SEPT 2011	SCALE	AS SHOWN	REV.		
CADD	--						
CHECK	SEMC	SEPT 2011					
REVIEW	JMAC	SEPT 2011					



Figure 5



Expanded Polystyrene
Unit Weight: 0.5kN/m³
Cohesion: 15 kPa

Silty Clay (Crust)
Unit Weight: 18 kN/m³
Cohesion: 50 kPa

New Granular Fill
Unit Weight: 21 kN/m³
Phi: 35°

Silty Clay to Clay (transition zone)
Unit Weight: 18 kN/m³
Cohesion: 50 kPa to 35 kPa

Existing Granular Fill
Unit Weight: 20kN/m³
Phi: 30°

Silty Clay to Clay (upper)
Unit Weight: 18 kN/m³
Cohesion: 35 kPa

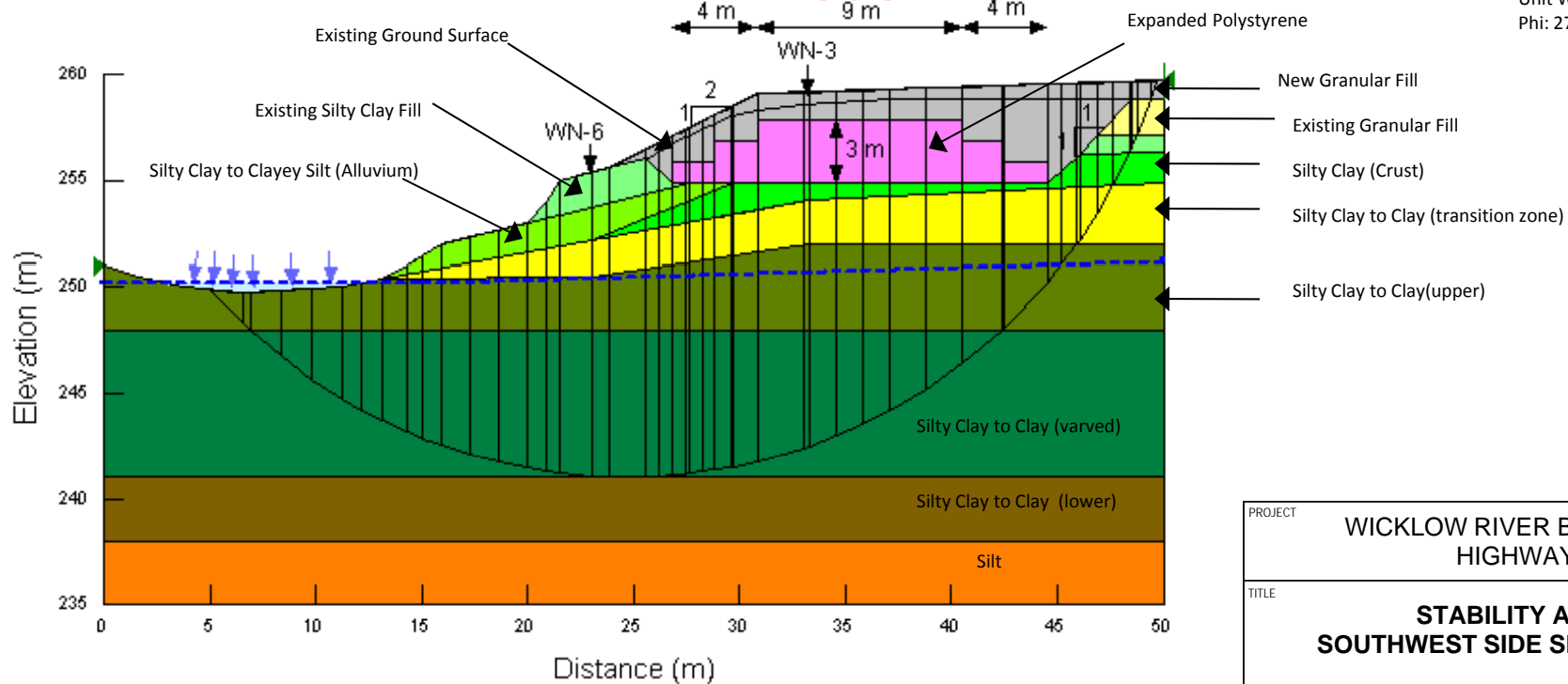
Existing Silty Clay Fill
Unit Weight: 18 kN/m³
Cohesion: 50 kPa

Silty Clay to Clay (varved)
Unit Weight: 18 kN/m³
Cohesion: 32 kPa

Silty Clay to Clayey Silt (Alluvium)
Unit Weight: 17 kN/m³
Cohesion: 50 kPa

Silty Clay to Clay (lower)
Unit Weight: 18 kN/m³
Cohesion: 32 kPa to 65 kPa

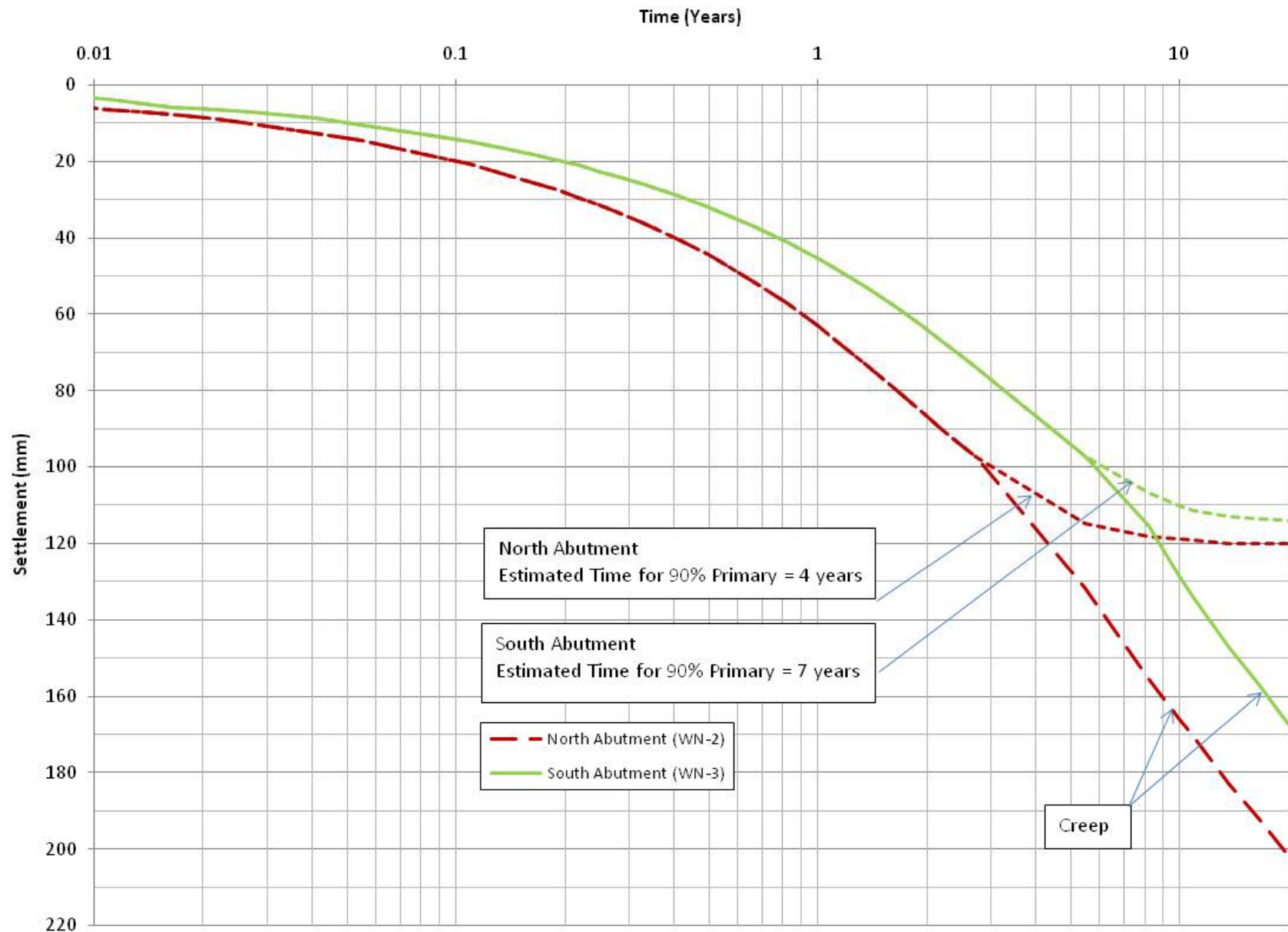
Silt
Unit Weight: 19 kN/m³
Phi: 27°



PROJECT				WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037			
TITLE				STABILITY ANALYSIS SOUTHWEST SIDE SLOPE MITIGATED			
PROJECT No. 09-1191-0022				FILE No. ----			
DESIGN	DAM	SEPT 2011	SCALE	AS SHOWN	REV.		
CADD	--						
CHECK	SEMC	SEPT 2011					
REVIEW	JMAC	SEPT 2011					



Figure 7




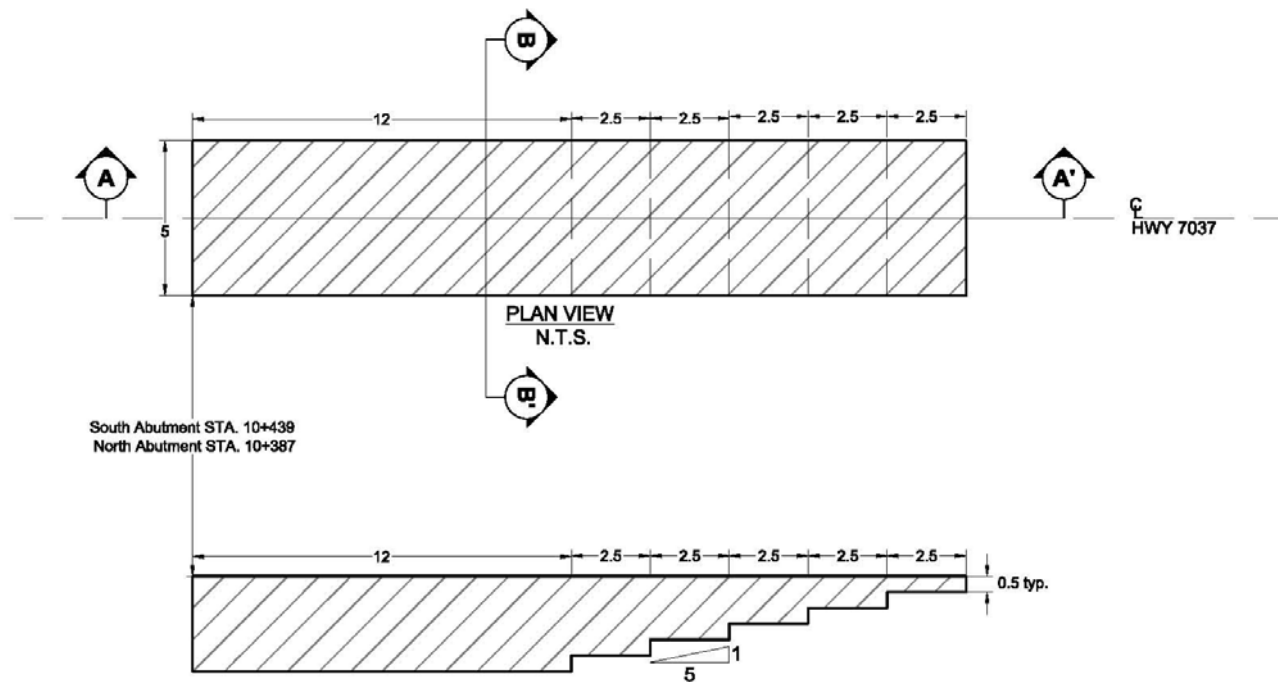
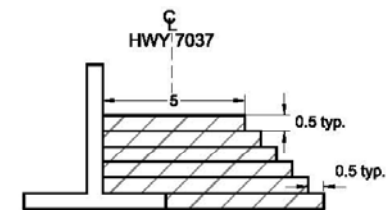
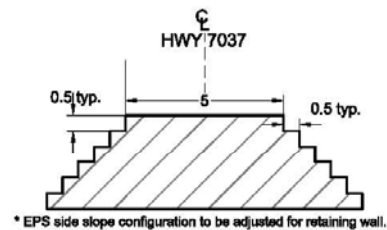
PROJECT	WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037			
TITLE	ESTIMATED CONSOLIDATION SETTLEMENT VERSUS LOG TIME			
	PROJECT No.	09-1191-0022	FILE No.	----
	DESIGN	DAM	SEPT 2011	SCALE AS SHOWN
	CADD	--		REV.
	CHECK	SEMC	SEPT 2011	
	REVIEW	JMAC	SEPT 2011	

Figure 8



NOTES:

1. All units are in metres.
2. Refer to accompanying Foundation Design Report, Section 6.6.5.1
3. Bedding of 300 mm at GBT2 and 100 mm mortar sand, levelling pad shall be placed below the EPS.
4. A minimum 300 mm thick layer of Granular "B" Type I should be placed above the side slopes of the EPS.
5. All EPS, including side slopes, should be protected with a minimum 1.2 m of soil cover and 6 mm thick polyethylene sheeting.
6. A 125 mm thick concrete slab is required on the top of the highest level of EPS blocks.



PROJECT		WICKLOW RIVER NORTH BRIDGE HIGHWAY 7037			
TITLE		RECOMMENDED EPS CONFIGURATION			
		PROJECT No. 09-1191-0022	FILE No. ----		
DESIGN	DAM	SEPT 2011	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	SEMC	SEPT 2011			
REVIEW	JMAC	SEPT 2011			



Figure 9



APPENDIX A

Record of Boreholes



LIST OF SYMBOLS

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

1. 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
FoS	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 multiplied by acceleration due to gravity).

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
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

Piezo-Cone Penetration Test (CPT)

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test 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.

V. MINOR SOIL CONSTITUENTS

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

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

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

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

CSUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

[illegible]

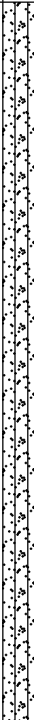

Continued Next Page

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

PROJECT		09-1191-0022		RECORD OF BOREHOLE No WN-2		2 OF 4 METRIC									
W.P.		5139-06-00		LOCATION N 5428914.5; E 304207.7		ORIGINATED BY ID									
DIST		Cochrane HWY 7037		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring		COMPILED BY JJL									
DATUM		Geodetic		DATE April 8 to 10, 2011		CHECKED BY SC									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			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	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L	WATER CONTENT (%)	γ kN/m³	GR	SA	SI	CL
--- CONTINUED FROM PREVIOUS PAGE ---															
237.5	SILTY CLAY to CLAY, trace sand Firm to stiff Grey Wet Switched to NW Casing at 15.2 m depth. Containing silt layers below approximately Elev. 241 m.	[Pattern]	11	TO	PH		242	4 +			18.1				
20.1	SILT, trace to some clay Loose to compact Grey Wet Containing some sand below 22.4 m depth (Elev. 235.2 m).	[Pattern]	12	SS	9		241								
			13	SS	10		240								
			14	SS	11		239	5 +							
			15	SS	12		238								
232.1	Silty SAND and Gravel, trace to some clay (TILL) Dense to very dense Grey Wet Artesian conditions (water flowing out of casing) below 25.9 m depth (Elev. 231.7 m).	[Pattern]	16	SS	35		237								
25.5			17	SS	47		236								
			18	SS	43/0.15		235								
			19	RC	REC 33%		234								
	Spoon bouncing at 29.2 m depth, switched to NQ Coring at 29.2 m depth. Recovered silty sand, trace gravel (TILL).						233								
							232								
							231								
							230								
							229								
							228								

Continued Next Page

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

PROJECT		09-1191-0022		RECORD OF BOREHOLE No WN-2		3 OF 4 METRIC											
W.P.		5139-06-00		LOCATION		N 5428914.5; E 304207.7											
DIST		Cochrane HWY 7037		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring											
DATUM		Geodetic		DATE		April 8 to 10, 2011											
				ORIGINATED BY		ID											
				COMPILED BY		JLL											
				CHECKED BY		SC											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60	W _p W W _L	γ	GR SA SI CL					
--- CONTINUED FROM PREVIOUS PAGE ---																	
220.3 37.3	Silty SAND and Gravel, trace to some clay (TILL) Dense to very dense Grey Wet Artesian conditions (water flowing out of casing) below 25.9 m depth (Elev. 231.7 m). Boulder cored between 33.7 m and 34.0 m depth (Elev. 223.9 -223.6 m) Gravel and cobble fragments recovered from core sample numbers 23 and 24. Pockets of clayey silt, trace sand, trace gravel at 36.6 m depth.		20	RC	REC 30%		227										
			21	RC	REC 20%		226										
			22	RC	REC 40%		225										
			23	RC	REC 15%		224										
			24	RC	REC 20%		223										
			25	RC	REC 7%		222										
215.8 41.8	SILT, trace clay, sand and gravel (TILL) Very dense Grey Wet Switched to NW casing at 38.8 m depth. Recovered pink / grey granite rock core at: Depth (m) Thickness (mm) 41.8 305 42.3 150 42.6 150 42.9 150 43.2 150 43.3 230 43.8 305 44.5 280		26	SS	207/0.15		221										
			27	SS	136/0.15		220										
			28	RC	REC 56%		219										
29	RC	REC 64%		218													
213				217													
				216													
				215													
				214													
				213													

Continued Next Page

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

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

PROJECT <u>09-1191-0022</u>		RECORD OF BOREHOLE No WN-2				4 OF 4 METRIC	
W.P. <u>5139-06-00</u>		LOCATION <u>N 5428914.5; E 304207.7</u>				ORIGINATED BY <u>ID</u>	
DIST <u>Cochrane</u> HWY <u>7037</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring</u>				COMPILED BY <u>JJL</u>	
DATUM <u>Geodetic</u>		DATE <u>April 8 to 10, 2011</u>				CHECKED BY <u>SC</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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W		
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100					
211.5		30	RC	REC 40%		212										
46.1	END OF BOREHOLE SPOON REFUSAL Note: 1. Water level at 2.1 m above ground surface (Elev. 259.7 m) prior to coring through cobbles and boulders deposit.	31	CS	100/0.0%												

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

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

PROJECT		09-1191-0022		RECORD OF BOREHOLE No WN-3		2 OF 3		METRIC				
W.P.		5139-06-00		LOCATION		N 5428854.7; E 304217.2		ORIGINATED BY ID				
DIST		Cochrane HWY 7037		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring		COMPILED BY JJL				
DATUM		Geodetic		DATE		April 13 and 14, 2011		CHECKED BY SC				
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
238.1	SILT CLAY, trace sand Firm to stiff Grey Wet Switched to NW Casing at 15.2 m depth. Artesian conditions (water flowing out of casing) first encountered below 16.8 m depth (Elev. 241.1 m) Containing silt layers below approximately Elev. 241 m.		12	SS	5		242					
							241					
							240					
			13	SS	3							
			14	SS	4							
							239					
19.8	SILT, trace to some clay and / or clayey layers, trace sand Loose to compact Grey Wet Artesian groundwater level measurement taken within silt deposit at 22.9 m depth (Elev. 235.0 m)		15	SS	7		238					
							237					
			16	SS	12		236					
							235					
			17	SS	12		234					
							233					
			18	SS	9							
232.0	Silty SAND and Gravel, trace to some clay (TILL) Compact to very dense Grey Wet Casing refusal at 27.9 m depth. Switched to NQ coring between 27.9 m and 29.4 m depth. Recovered a 0.15 m length grey granite cobble at 28.1 m depth and a 0.18 m length white quartz cobble at 28.9 m depth..		19	SS	16		232					
25.9							231					
			20	SS	24		230					
			-	RC	REC 28%		229					
			21	SS	10		228					

Continued Next Page

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

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

PROJECT 09-1191-0022				RECORD OF BOREHOLE No WN-4				1 OF 2 METRIC					
W.P. 5139-06-00				LOCATION N 5428835.0; E 304209.3				ORIGINATED BY ID					
DIST Cochrane HWY 7037				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY JLL					
DATUM Geodetic				DATE April 15, 2011				CHECKED BY SC					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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		ELEVATION SCALE	SHEAR STRENGTH kPa					
259.2	GROUND SURFACE						20 40 60 80 100						
0.0	ASPHALT (Surface Treatment)						20 40 60 80 100						
0.2	Sand and gravel, trace silt (FILL) Frozen Brown		1	SS	24		20 40 60 80 100						
	Silty clay, some sand (FILL) Very stiff (Frozen) Brown												
257.7	CLAYEY SILT, trace sand Stiff Brown Moist		2	SS	9		20 40 60 80 100						
256.9	SILTY CLAY to CLAY, trace to some sand Firm Grey Wet		3	SS	6		20 40 60 80 100						
2.3			4	SS	6		20 40 60 80 100						
			5	SS	4		20 40 60 80 100						
			6	SS	3		20 40 60 80 100						
			7	SS	4		20 40 60 80 100						
			8	SS	1		20 40 60 80 100						
			9	SS	WH		20 40 60 80 100						

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

Continued Next Page

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

PROJECT		RECORD OF BOREHOLE				2 OF 2		METRIC									
09-1191-0022		No WN-4															
W.P. 5139-06-00		LOCATION N 5428835.0; E 304209.3				ORIGINATED BY ID											
DIST Cochrane HWY 7037		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY JJL											
DATUM Geodetic		DATE April 15, 2011				CHECKED BY SC											
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 (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60					
243.4			10	SS	WH		244										
15.8	END OF BOREHOLE Note: 1. Borehole dry upon completion of drilling. 2. Water level in piezometer at a depth of 5.9 m (Elev. 253.2 m), 7.8 m (Elev. 251.4 m), and 2.3 m (Elev. 256.9 m) on April 21, 2011, April 28, 2011 and July 3, 2011 respectively.																

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

[illegible]

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

Continued Next Page

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

PROJECT		RECORD OF BOREHOLE				No WN-5		2 OF 2		METRIC								
W.P. 09-1191-0022		LOCATION				N 5428914.4; E 304225.1		ORIGINATED BY ID										
DIST Cochrane HWY 7037		BOREHOLE TYPE				Portable Equipment, NW Casing, Wash Boring		COMPILED BY JJL										
DATUM Geodetic		DATE				April 12 to 14, 2011		CHECKED BY SC										
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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60 WATER CONTENT (%)						
235.2	SILT, some clay and / or clay layers Loose to compact Grey Wet Split Spoon advanced from bottom of DCPT hole at 16.5 m depth for 2.2 m.		10	SS	18		237											
17.1	END OF BOREHOLE Note: 1. Tripod could not advance borehole beyond 14.3 m depth (Elev. 240.2 m). 2. Water level at a depth of 0.3 m below ground surface (Elev. 252.0 m) upon completion of drilling.						236										0 0 86 14	

[illegible]

PROJECT <u>09-1191-0022</u>		RECORD OF BOREHOLE No WN-6				2 OF 2 METRIC	
W.P. <u>5139-06-00</u>		LOCATION <u>N 5428860.8; E 304206.5</u>				ORIGINATED BY <u>ID</u>	
DIST <u>Cochrane</u> HWY <u>7037</u>		BOREHOLE TYPE <u>Portable Equipment, NW Casing, Wash Boring</u>				COMPILED BY <u>JJL</u>	
DATUM <u>Geodetic</u>		DATE <u>April 14 and 15, 2011</u>				CHECKED BY <u>SC</u>	

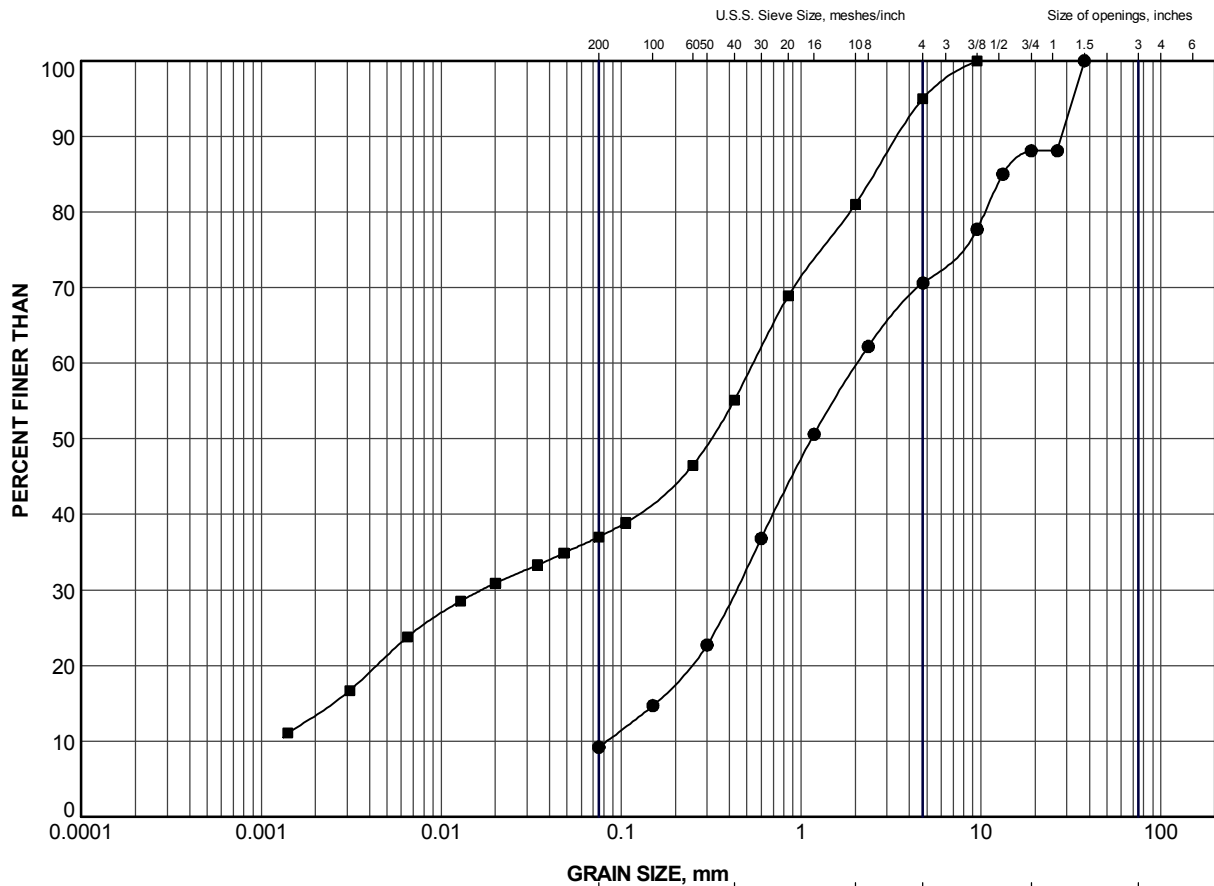
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			W _L							
	--- 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 </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>																	
	END OF BOREHOLE Note: 1. Tripod could not advance borehole beyond 14.3 m depth (Elev. 240.2 m). 2. Water level at a depth of 4.3 m below ground surface (Elev. 250.2 m) upon completion of drilling.																							

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:



APPENDIX B


Laboratory Test Results

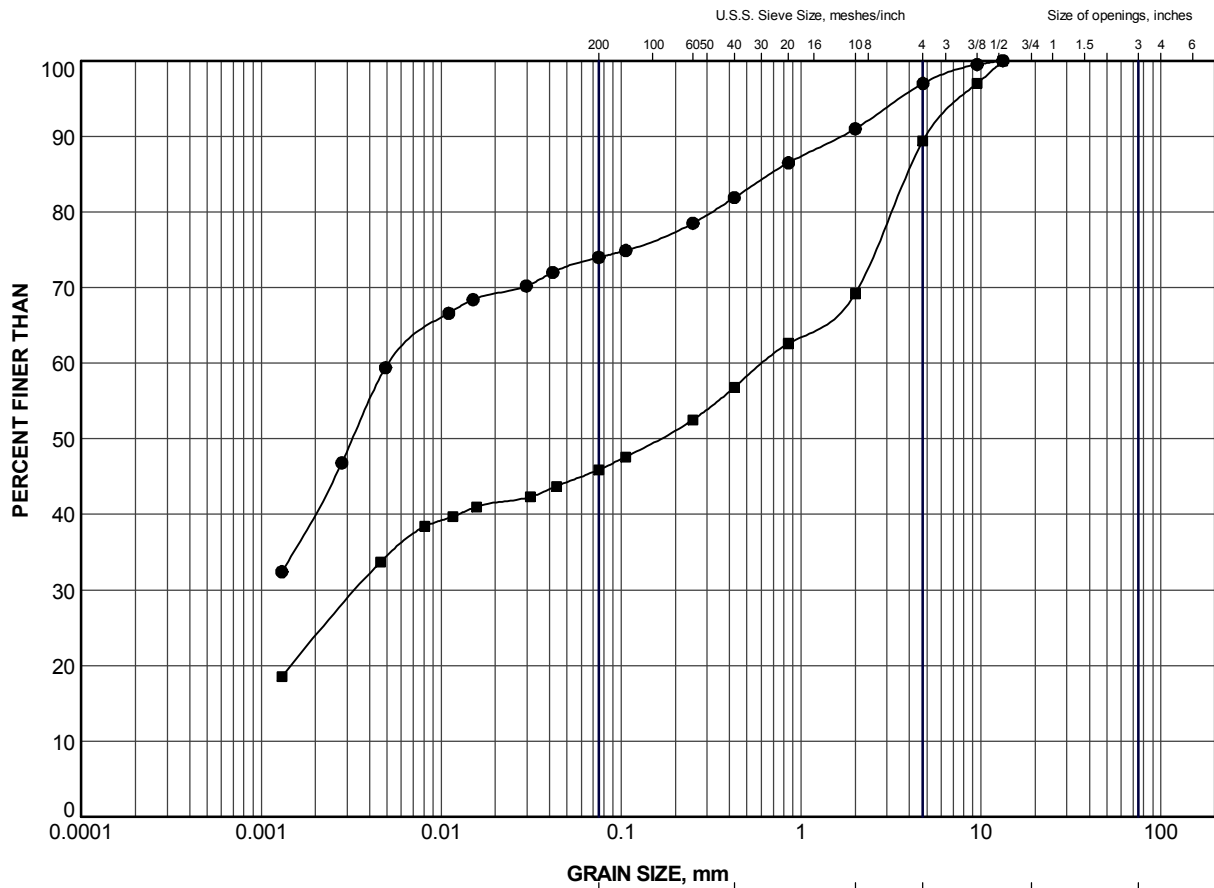


CLAY AND SILT	SAND SIZE, mm			GRAVEL SIZE, mm		Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WN-1	1	257.2
■	WN-3	1	257.6


PROJECT						WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037					
TITLE						GRAIN SIZE DISTRIBUTION GRAVELLY SAND TO SILTY SAND (FILL)					
PROJECT No.			09-1191-0022			FILE No.			09-1191-0022.GPJ		
DRAWN		J.J.L.		Sep 2011		SCALE		N/A		REV.	
CHECK		DAM		Sep 2011							
APPR				Sep 2011							
 Golder Associates SUDBURY, ONTARIO						FIGURE B-1					

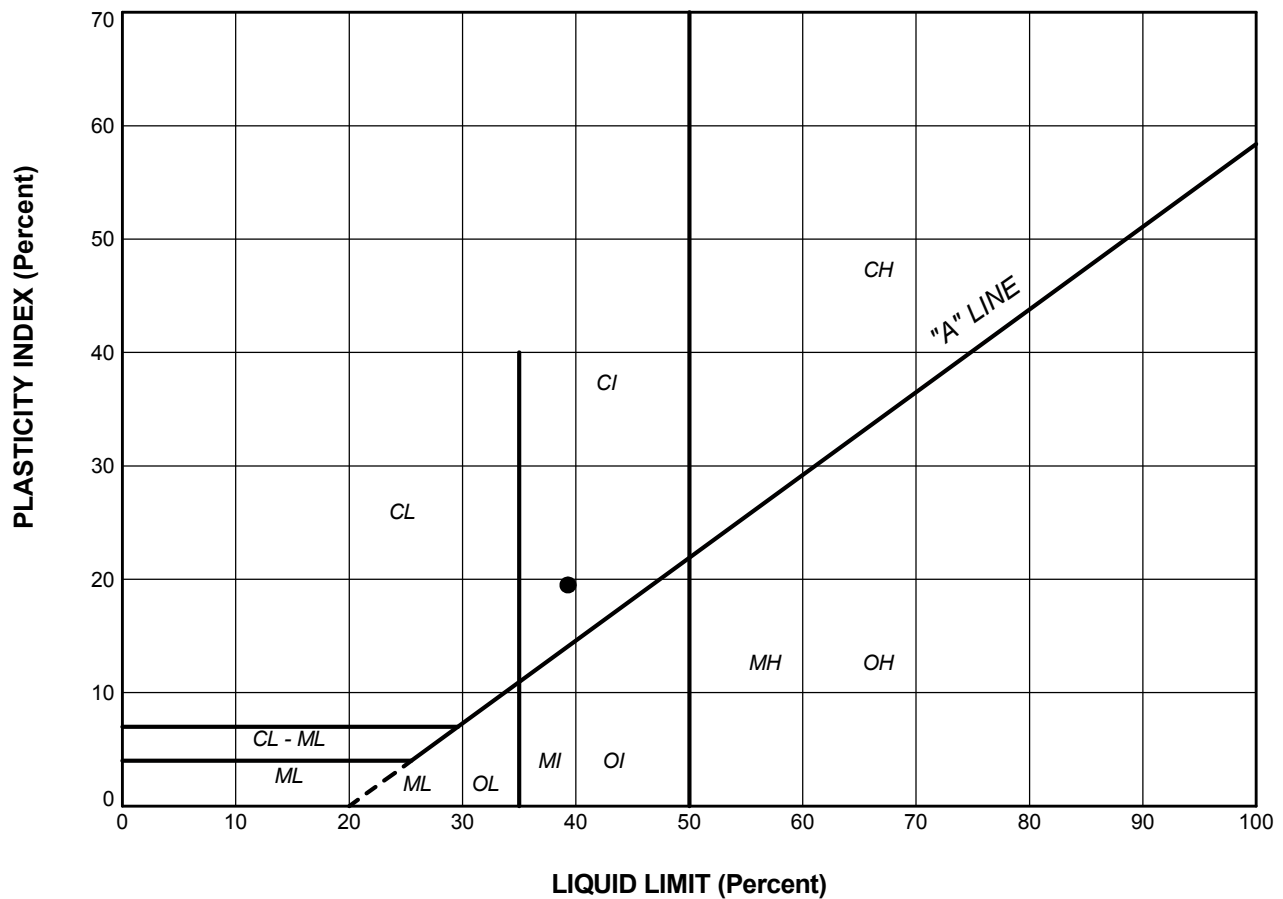


GRAVEL SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WN-2	1	256.5
■	WN-3	2	256.8

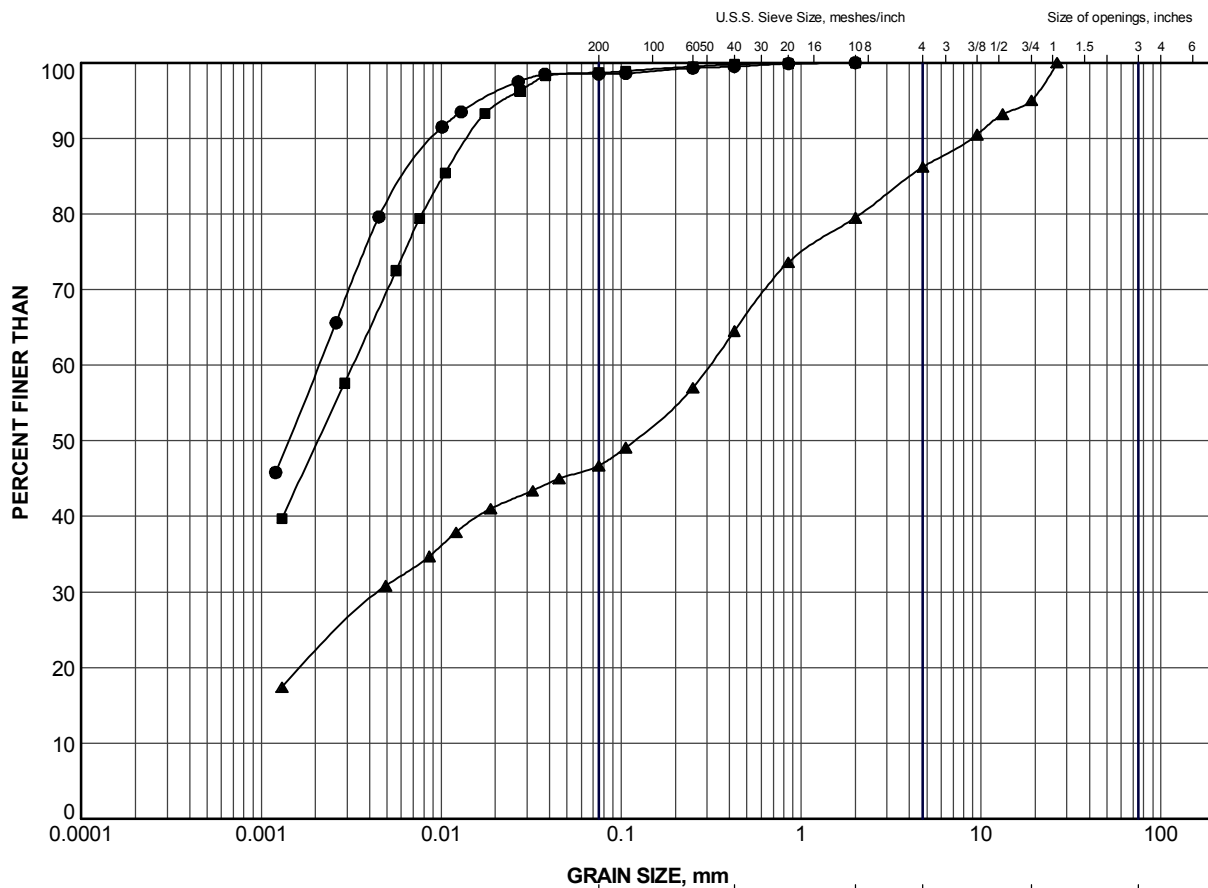
PROJECT						WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037					
TITLE						GRAIN SIZE DISTRIBUTION SILTY CLAY (FILL)					
PROJECT No.			09-1191-0022			FILE No.			09-1191-0022.GPJ		
DRAWN		J.J.L.		Sep 2011		SCALE		N/A		REV.	
CHECK		DAM		Sep 2011							
APPR				Sep 2011							
 Golder Associates SUDBURY, ONTARIO						FIGURE B-2					



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WN-2	1	39.3	19.8	19.5


PROJECT				
WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037				
TITLE				
PLASTICITY CHART SILTY CLAY (FILL)				
PROJECT No.		09-1191-0022		FILE No.
DRAWN		JJL	Sep 2011	SCALE
CHECK		DAM	Sep 2011	REV.
APPR			Sep 2011	
 Golder Associates SUDBURY, ONTARIO		FIGURE B-3		

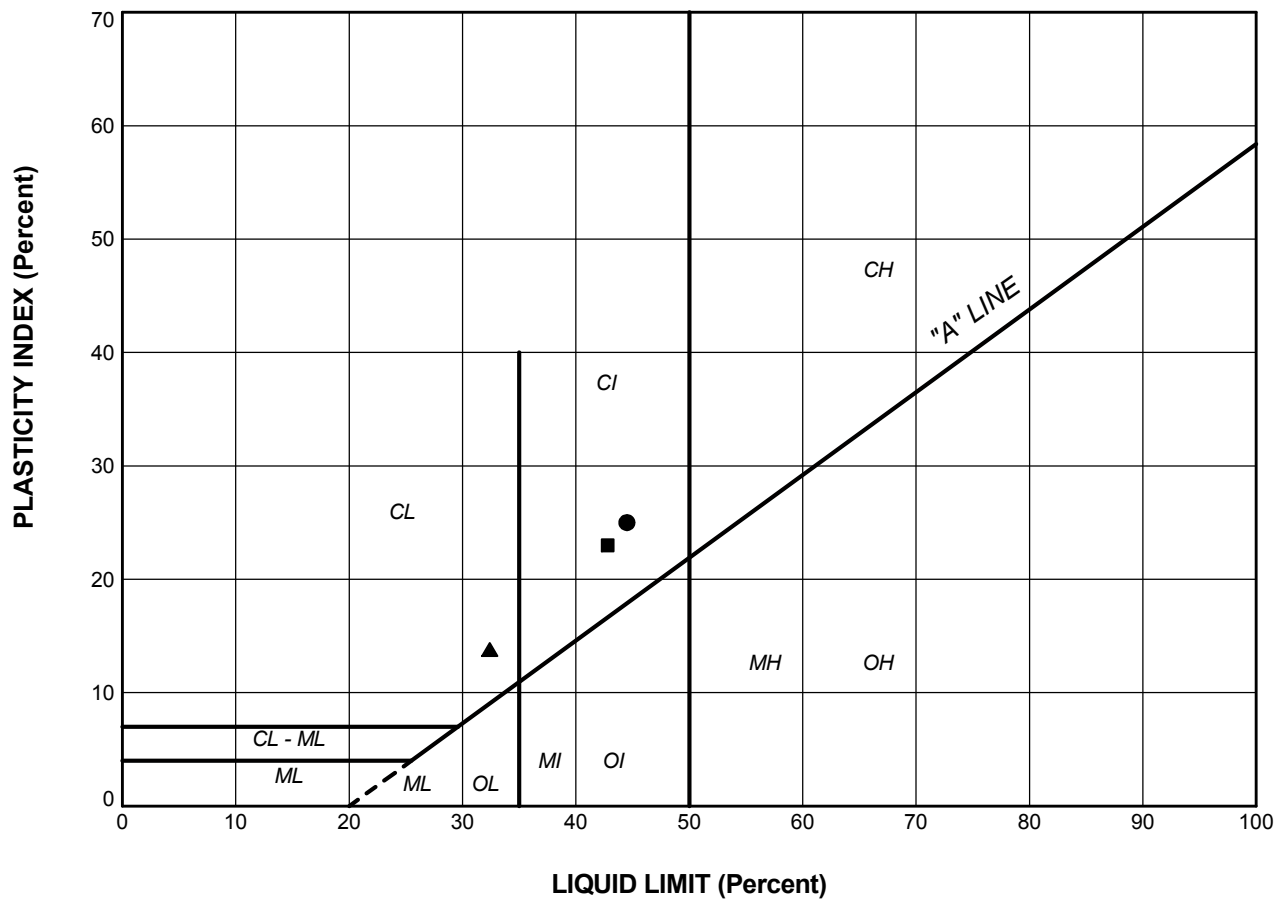


CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WN-1	3	255.7
■	WN-2	6	252.8
▲	WN-6	3	252.7

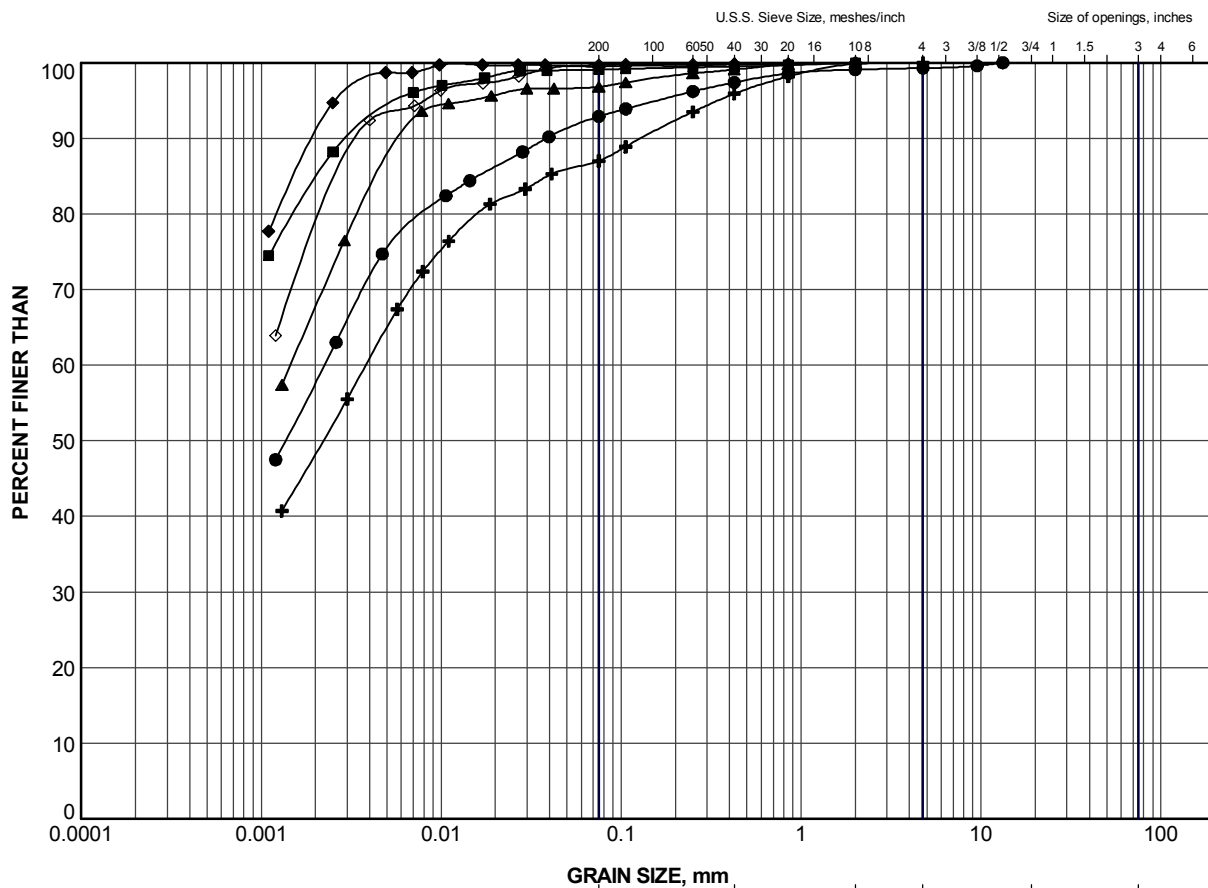
PROJECT					
WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY (ALLUVIUM)					
PROJECT No.		09-1191-0022		FILE No. 09-1191-0022.GPJ	
DRAWN	JJL	Sep 2011	SCALE	N/A	REV.
CHECK	DAM	Sep 2011			
APPR		Sep 2011			
 Golder Associates SUDBURY, ONTARIO			FIGURE B-4		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WN-1	3	44.5	19.5	25.0
■	WN-2	6	42.8	19.8	23.0
▲	WN-6	3	32.4	18.6	13.8

PROJECT					
WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037					
TITLE					
PLASTICITY CHART CLAYEY SILT TO SILTY CLAY (ALLUVIUM)					
PROJECT No.		09-1191-0022		FILE No.	
DRAWN		JJL		Sep 2011	
CHECK		DAM		Sep 2011	
APPR				Sep 2011	
 Golder Associates SUDBURY, ONTARIO				SCALE N/A REV.	
				FIGURE B-5	



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WN-1	8	248.9
■	WN-2	9	246.6
▲	WN-3	10	248.5
+	WN-4	6	252.8
◆	WN-5	6	247.4
◇	WN-6	9	243.5

PROJECT

WICKLOW RIVER BRIDGE NORTH
HIGHWAY 7037

TITLE

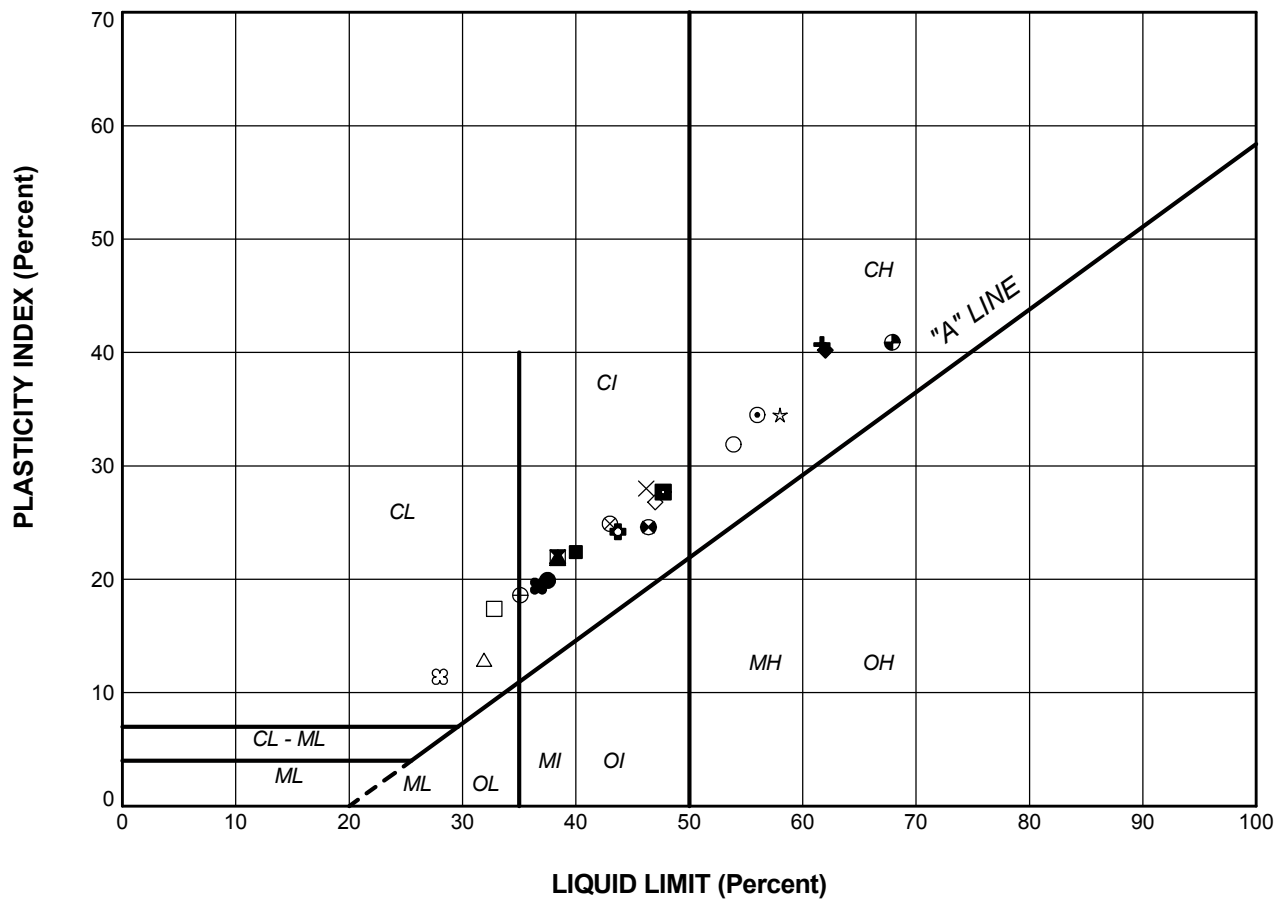
GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY



Golder Associates
SUDBURY, ONTARIO

PROJECT No.	09-1191-0022	FILE No.	09-1191-0022.GPJ
DRAWN	JJL	Sep 2011	SCALE N/A
CHECK	DAM	Sep 2011	REV.
APPR		Sep 2011	

FIGURE B-6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WN-1	5	37.5	17.6	19.9
■	WN-1	8	40.0	17.6	22.4
▲	WN-2	8	38.4	16.4	22.0
+	WN-2	9	61.7	21.0	40.7
◆	WN-2	10	62.0	21.8	40.2
◇	WN-2	10	47.0	20.2	26.8
○	WN-2	11	53.9	22.0	31.9
△	WN-2	11	31.9	19.0	12.9
⊗	WN-3	4	43.0	18.1	24.9
⊕	WN-3	8	35.1	16.5	18.6
□	WN-3	9	32.8	15.4	17.4
⊙	WN-3	10	46.4	21.8	24.6
●	WN-3	11	67.9	27.0	40.9
☆	WN-3	11	58.0	23.5	34.5
⊗	WN-4	2	28.0	16.6	11.4
⊕	WN-4	4	38.4	16.5	21.9
⊙	WN-4	8	56.0	21.5	34.5
⊕	WN-5	4	43.7	19.5	24.2
×	WN-5	6	46.2	18.2	28.0
■	WN-6	5	36.7	17.3	19.4
■	WN-6	9	47.7	20.0	27.7

PROJECT					
WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037					
TITLE					
PLASTICITY CHART CLAYEY SILT TO CLAY					
PROJECT No.		09-1191-0022		FILE No.	
DRAWN		J.J.L.		Sep 2011	
CHECK		DAM		Sep 2011	
APPR				Sep 2011	
SCALE		N/A		REV.	
Golder Associates SUDBURY, ONTARIO		FIGURE B-7			

CONSOLIDATION TEST SUMMARY**FIGURE B-8****Pg. 1 of 4****SAMPLE IDENTIFICATION**

Project Number: 09-1191-0022

Sample Number: 10

Borehole Number: WN-2

Sample Depth, m: 13.7

TEST CONDITIONS

Test Type Standard

Load Duration, hr 24

Oedometer Number 1

Date Started June 18/11

Date Completed June 30/11

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.550	Unit Weight, kN/m ³	17.12
Sample Diameter, cm	6.330	Dry Unit Weight, kN/m ³	11.02
Area, cm ²	31.47	Specific Gravity, assumed	2.70
Volume, cm ³	80.25	Solids Height, cm	1.061
Water Content, %	55.36	Volume of Solids, cm ³	33.39
Wet Mass, g	140.06	Volume of Voids, cm ³	46.86
Dry Mass, g	90.15	Degree of Saturation, %	106.5

TEST COMPUTATIONS

Pressure kPa	Primary Consolidation	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s	Total Work kJ/m ³
0	0	2.550	1.403	2.550					
9	0.02	2.548	1.401	2.549	60	0.0230	1.05E-04	2.37E-07	0.004
18	0.04	2.544	1.397	2.546	580	0.0024	1.70E-04	3.94E-08	0.025
35	0.08	2.535	1.390	2.540	820	0.0017	1.90E-04	3.11E-08	0.111
69	0.14	2.521	1.376	2.528	1160	0.0012	1.65E-04	1.89E-08	0.404
107	0.11	2.510	1.366	2.516	1058	0.0013	1.14E-04	1.41E-08	0.788
143	0.09	2.501	1.357	2.506	1109	0.0012	1.06E-04	1.25E-08	1.245
285	0.68	2.433	1.293	2.467	2018	0.0006	1.88E-04	1.18E-08	7.063
571	2.10	2.223	1.095	2.328	6615	0.0002	2.88E-04	4.90E-09	44.003
1140	1.32	2.091	0.971	2.157	2940	0.0003	9.10E-05	2.99E-09	94.799
2279	0.96	1.995	0.880	2.043	1500	0.0006	3.31E-05	1.91E-09	173.279
1140	-0.16	2.011	0.896	2.003					
285	-0.58	2.069	0.950	2.040					
69	-0.72	2.141	1.018	2.105					
9	-1.138	2.254	1.125	2.197					

Note:

k calculated using α based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.254	Unit Weight, kN/m ³	17.25
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m ³	12.46
Area, cm ²	31.47	Specific Gravity, assumed	2.70
Volume, cm ³	70.94	Solids Height, cm	1.061
Water Content, %	38.45	Volume of Solids, cm ³	33.39
Wet Mass, g	124.81	Volume of Voids, cm ³	37.55
Dry Mass, g	90.15		

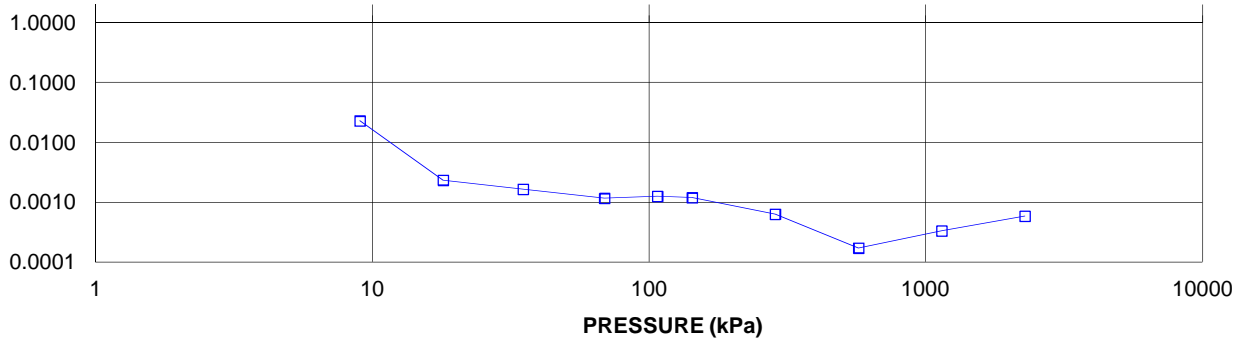
CONSOLIDATION TEST SUMMARY

FIGURE B-8

Pg. 2 of 4

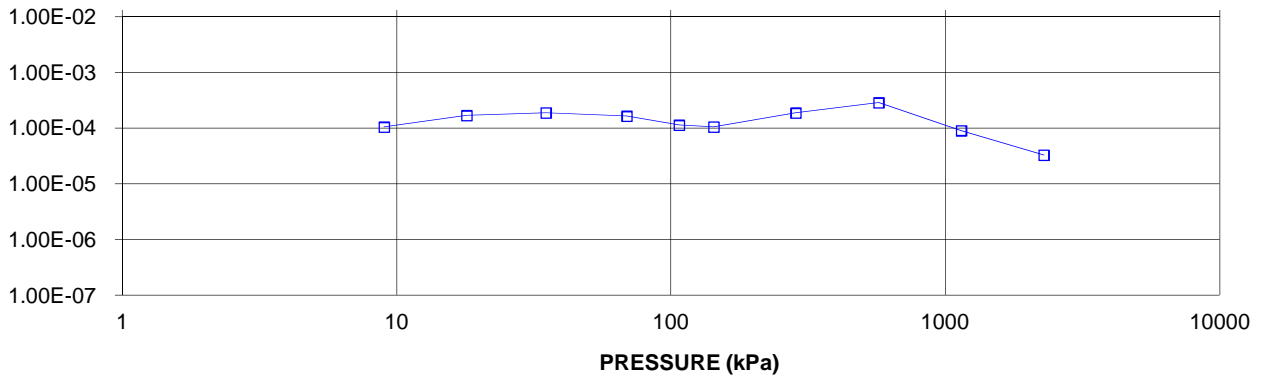
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH WN-2 SA 10



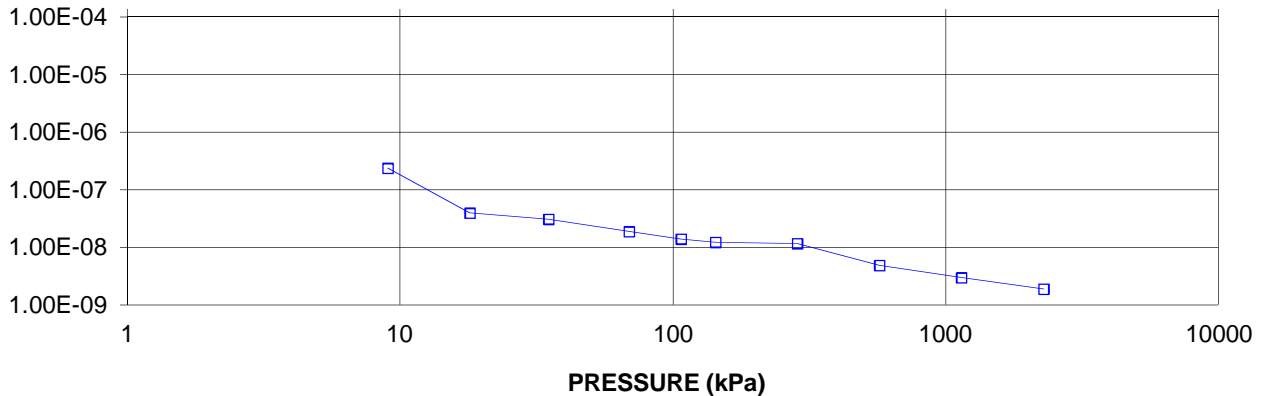
VOLUME COMPRESSIBILITY, m²/kN

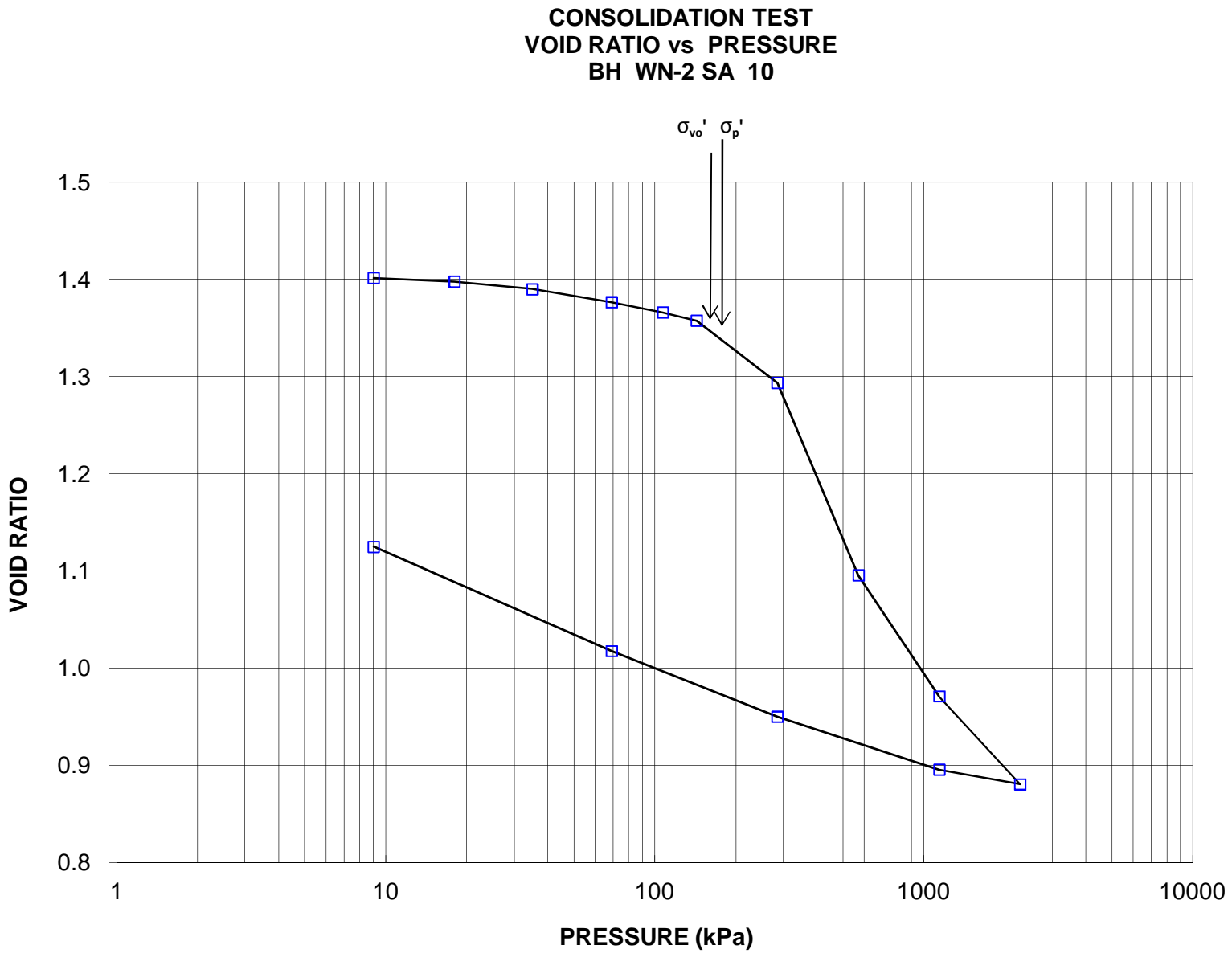
CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH WN-2 SA 10

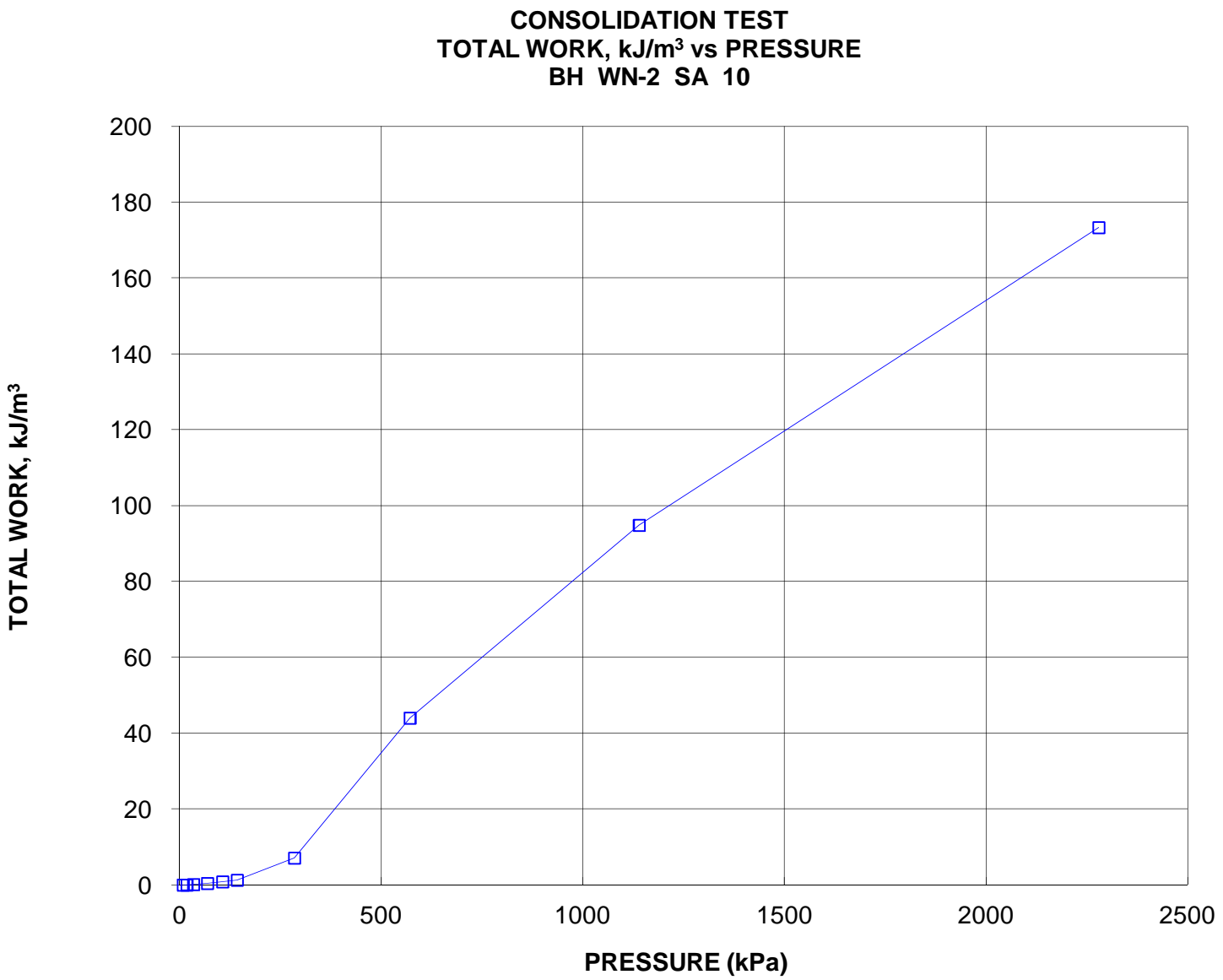


HYDRAULIC CONDUCTIVITY,
cm/s

CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH WN-2 SA 10







CONSOLIDATION TEST SUMMARY**FIGURE B-9**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number: 09-1191-0022

Sample Number: 10

Borehole Number: WN-3

Sample Depth, m: 9.5

TEST CONDITIONS

Test Type Standard

Load Duration, hr 24

Oedometer Number 1

Date Started May 11/11

Date Completed May 25/11

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.550	Unit Weight, kN/m ³	18.41
Sample Diameter, cm	6.330	Dry Unit Weight, kN/m ³	13.28
Area, cm ²	31.47	Specific Gravity, measured	2.71
Volume, cm ³	80.25	Solids Height, cm	1.276
Water Content, %	38.64	Volume of Solids, cm ³	40.17
Wet Mass, g	150.63	Volume of Voids, cm ³	40.08
Dry Mass, g	108.65	Degree of Saturation, %	104.7

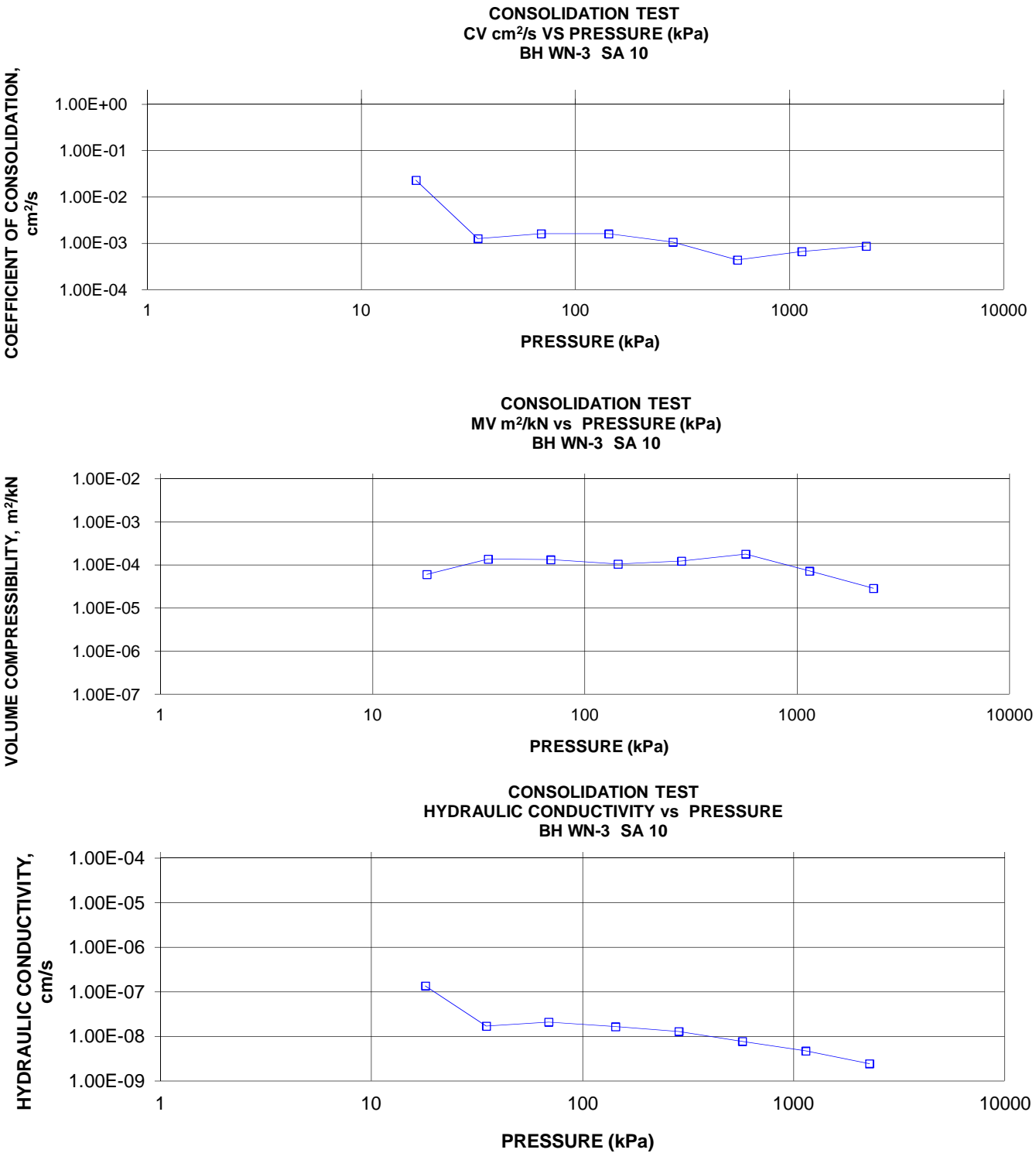
TEST COMPUTATIONS

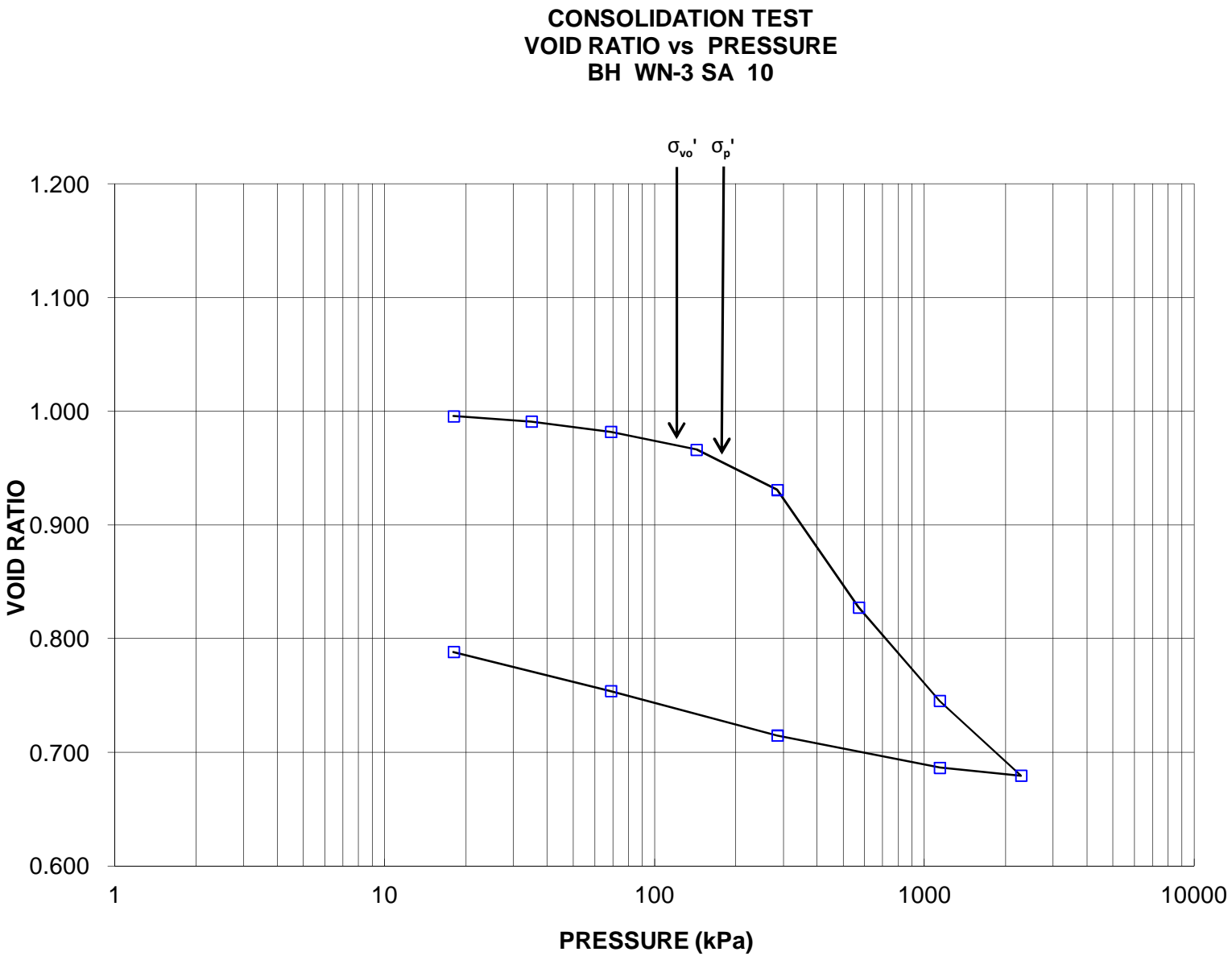
Pressure kPa	Primary Consolidation	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s	Total Work kJ/m ³
0	0	2.550	0.998	2.550					
18	0.03	2.547	0.996	2.549	60	0.0230	6.10E-05	1.37E-07	0.010
35	0.06	2.541	0.991	2.544	1080	0.0013	1.38E-04	1.72E-08	0.072
69	0.12	2.530	0.982	2.535	840	0.0016	1.34E-04	2.13E-08	0.310
143	0.20	2.510	0.966	2.520	840	0.0016	1.06E-04	1.66E-08	1.148
285	0.45	2.465	0.931	2.487	1220	0.0011	1.24E-04	1.31E-08	4.985
571	1.32	2.333	0.828	2.399	2770	0.0004	1.81E-04	7.81E-09	27.908
1140	1.05	2.228	0.745	2.280	1650	0.0007	7.24E-05	4.74E-09	66.418
2279	0.84	2.144	0.679	2.186	1160	0.0009	2.89E-05	2.47E-09	130.881
1140	-0.09	2.153	0.687	2.148					
285	-0.36	2.189	0.715	2.171					
69	-0.50	2.239	0.754	2.214					
18	-0.44	2.283	0.788	2.261					

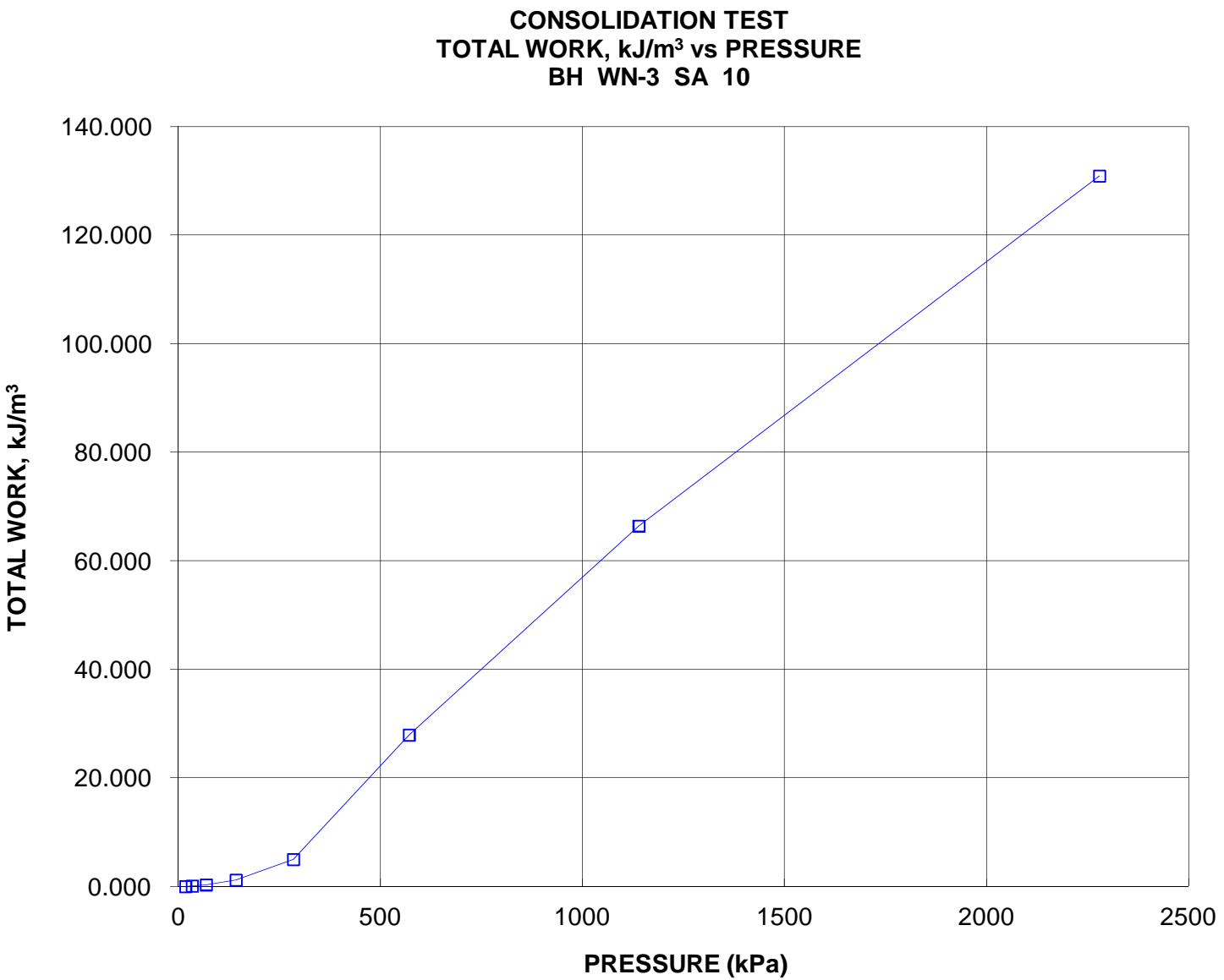
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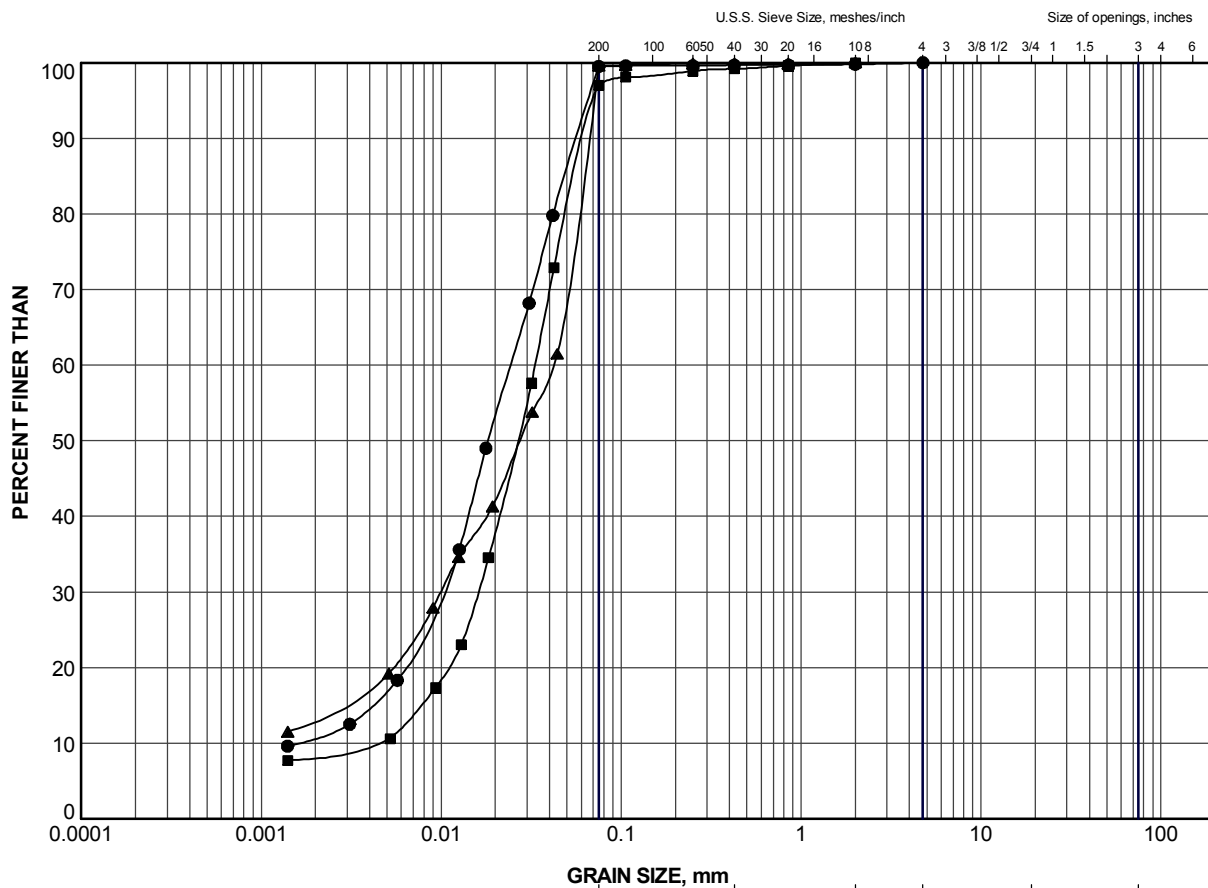
k calculated using α based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.283	Unit Weight, kN/m ³	18.94
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m ³	14.83
Area, cm ²	31.47	Specific Gravity, measured	2.71
Volume, cm ³	71.83	Solids Height, cm	1.276
Water Content, %	27.71	Volume of Solids, cm ³	40.17
Wet Mass, g	138.76	Volume of Voids, cm ³	31.67
Dry Mass, g	108.65		








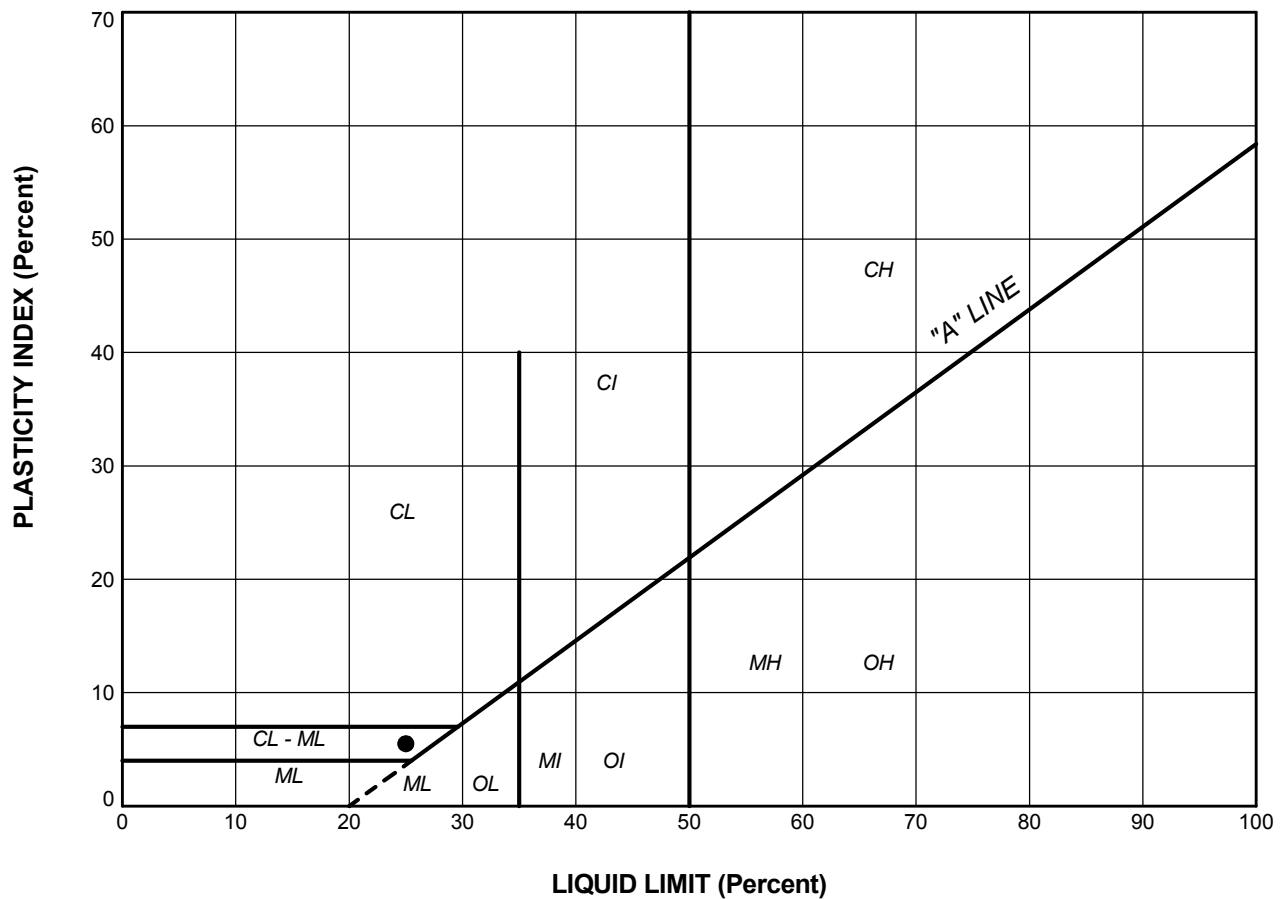


CLAY AND SILT		GRAVEL SIZE, mm					Cobble Size
		fine	medium	coarse	fine	coarse	
		SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WN-2	13	236.0
■	WN-3	18	233.2
▲	WN-5	10	235.5

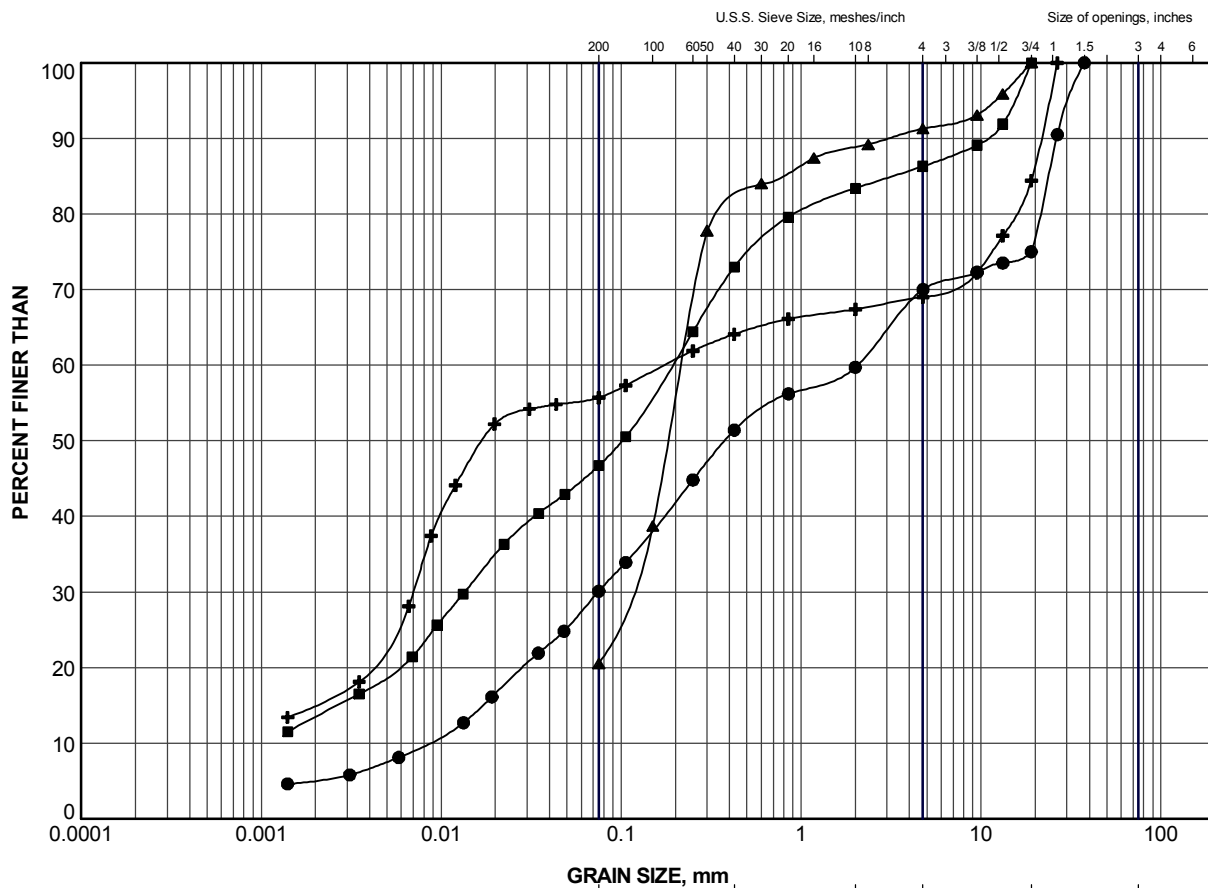
PROJECT						WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037					
TITLE						GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.			09-1191-0022			FILE No.			09-1191-0022.GPJ		
DRAWN	JJL	Sep 2011	CHECK	DAM	Sep 2011	SCALE	N/A	REV.			
APPR		Sep 2011				FIGURE B-10					
 Golder Associates SUDBURY, ONTARIO											



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WN-3	15	25.0	19.5	5.5

PROJECT					
WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037					
TITLE					
PLASTICITY CHART SILT					
PROJECT No. 09-1191-0022			FILE No. 09-1191-0022.GPJ		
DRAWN	JJL	Sep 2011	SCALE	N/A	REV.
CHECK	DAM	Sep 2011	FIGURE B-11		
APPR		Sep 2011			
 Golder Associates SUDBURY, ONTARIO					



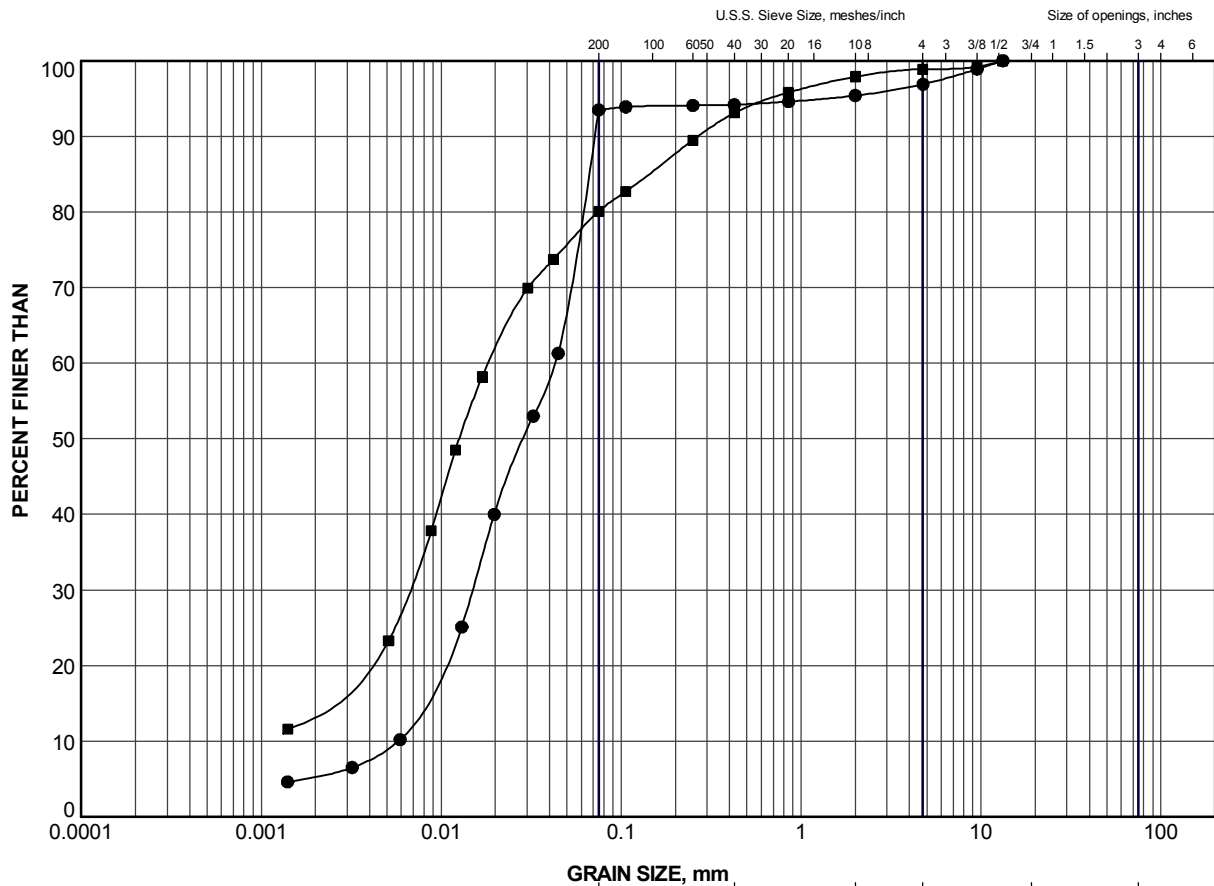
CLAY AND SILT		GRAVEL SIZE, mm				Cobble Size	
		fine	medium	coarse	fine		coarse
		SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WN-2	17	229.9
■	WN-2	22	224.1
▲	WN-3	21	228.2
+	WN-3	23	225.4

PROJECT						WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037					
TITLE						GRAIN SIZE DISTRIBUTION SILTY SAND AND GRAVEL (TILL)					
PROJECT No.			09-1191-0022			FILE No.			09-1191-0022.GPJ		
DRAWN	JJL	Sep 2011	CHECK	DAM	Sep 2011	SCALE	N/A	REV.			
APPR		Sep 2011				FIGURE B-12					






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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WN-2	26	218.6
■	WN-3	25	222.5

PROJECT					WICKLOW RIVER BRIDGE NORTH HIGHWAY 7037				
TITLE					GRAIN SIZE DISTRIBUTION SILT (TILL)				
PROJECT No.		09-1191-0022		FILE No.		09-1191-0022.GPJ			
DRAWN	JJL	Sep 2011	SCALE	N/A	REV.				
CHECK	DAM	Sep 2011							
APPR		Sep 2011							
 Golder Associates SUDBURY, ONTARIO			FIGURE B-13						



APPENDIX C

Non-Standard Special Provisions

CONTROL OF ARTESIAN WATER SEEPAGE –
MOBILIZATION/DEMOBILIZATION - Item No.

Special Provision

Scope

Work under this item shall include mobilization and demobilization of the grouting plant to/from the sites for controlling artesian water seepage during pile driving at the abutments. The grout plant and drilling equipment shall be mobilized once prior to commencing pile driving and demobilize from site upon completing the pile driving. The work shall also include moving the plant from one abutment location to the other.

Payment

Payment at the contract price shall be for all labour, equipment, and materials necessary to mobilize and demobilize the drilling and grouting plant to/from the Wicklow River Bridge North site and relocating the equipment from one abutment to the other during the pile driving operation.

CONTROL OF ARTESIAN WATER SEEPAGE – STANDBY - Item No.

Special Provision

Scope

Work under this item shall include standby costs for all labour, equipment, and material for drilling and grouting plant after mobilization during the pile driving at the abutments.

Measurement and Payment

Measurement for payment shall be paid at an hourly rate for the actual standby time incurred during the installation of the piles at each abutment.

Payment at the contract price shall be for all labour, equipment, and materials necessary for the grouting plant and drilling operation to remain on standby during installation of the piles at the abutments.

CONTROL OF ARTESIAN WATER SEEPAGE – DRILLING AND GROUTING -
Item No.

Special Provision

Scope

Work under the above item shall be carried out at the abutment piles. Work shall be performed by an experienced contractor with adequate specialized experience in the control of artesian water seepage.

The Contractor shall provide all labour, materials, tools, equipment and perform all work necessary for augering/drilling with casing and chemical grouting to control artesian water seepage at the direction of the Contract Administrator as follows:

1. The Contractor shall auger/drill cased chemical grouting holes immediately adjacent to the HP piles, install a Multi Port Sleeve Pipe system (MPSP) in each drilled hole and remove the casing. The tip elevation of the grouting hole shall be at EL. 243 m. Under no circumstances shall the tip elevation of the grouting hole reach the silt layer underlying the silty clay to clay stratum.
2. The MPSP system shall be plastic or steel with a minimum 50 mm o.d. Each length of pipe shall be screwed and socketted to fit together flush. The pipe shall have holes drilled and covered with minimum 4 mm thick rubber sleeves along its length at 500 mm spacing.
3. A polypropylene non-woven geotextile fabric bag shall be placed over every second set of sleeved holes at one metre spacing and fastened by pipe clamps above and below the sleeve location. Each polypropylene non-woven geotextile fabric bag shall have a minimum volume of 0.03 m³ (1 ft³).
4. A cementitious grout to inflate the bags shall be pumped into the bags utilizing an inflatable double packer system which is inserted into the sleeved pipe to isolate each grout bag location. The maximum pressure to be used to inflate the MPSP system grout bags should not exceed 1500 kPa.
5. The artesian water seepage at each grouting zone between the grout bags shall be verified by water pressure testing. A fluorescent dye may be introduced into the test water to visually verify the zone connection to the artesian water seepage to the surface.
6. Each grouting zone found to connect to the artesian water seepage shall be pressure grouted with a single component water reactive polyurethane chemical grout.
7. If after completion of the grouting on one side of the H-Pile, the artesian water seepage is not completely sealed, the same procedure shall be repeated on the other side of the H-Pile.

8. When the artesian water seepage in one location is satisfactorily sealed, the Contractor shall repeat the same procedure at other locations where artesian water seepage was observed.

Measurement for Payment

Measurement for payment shall be for installing sufficient MRSP for sealing the artesian water seepage at each pile location, as measured on site by the Contract Administrator. For bidding purposes, the Contractor shall assume that control of artesian water seepage will be required at five (5) pile locations at each site. The estimated tender quantity may be increased or decreased as required.

Basis of Payment

Payment shall be at the unit price bid for successfully sealing the artesian water seepage at each pile location and shall be payment in full for all labour, equipment and materials necessary to complete the work.

CONTROL OF ARTESIAN WATER SEEPAGE – GROUT - Item No.

Special Provision

Scope

Work under the above tender item shall include the supply of grout for the control of Artesian Water Seepage. The grout to be used shall be either MME-UNIVERSAL, distributed by MME Multiurethanes, Mississauga, Ontario or Hydro-Active Cut, manufactured by DeNeef Const. Chemical, distributed by Janac Sales, Toronto, Ontario.

Method of Operation

The requirements for the usage of the grout shall conform to Control of Artesian Water Seepage – Drilling and Grouting.

Measurement for Payment

Measurement is cubic metre (m³) of grout installed to seal the artesian water seepage up through the pile locations.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material required to supply the grout.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Non-Standard Special Provision

Scope

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

SUBMISSION AND DESIGN REQUIREMENTS

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

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

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

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

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

MATERIAL

Corrugated Steel Pipe

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

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

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

Sand Fill

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

Table 1 – Sand Fill Gradation Requirements

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

CONSTRUCTION

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

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Install piles by driving to the design tip elevation or bedrock if end-bearing piles are selected.
4. Place loose sand into the CSP.
5. Remove temporary spacers.

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

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

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

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

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

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

Basis of Payment

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

H-PILES - HP310x132 - Item No.

Special Provision

903.07.02 Driven Piles

903.07.02.01 Pile Driving Requirements and Restrictions

Section 903.07.02.01 of OPSS 903 is amended by the addition of the following:

The Contractor shall commence assessment of the pile capacity by the Hiley Formula (Standard Drawing SS-103-11) once the pile reaches a depth of 3.0 m above the design pile tip elevation shown in the Contract Drawings and assess the ultimate axial resistance of the pile using the Hiley Formula at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley Formula is not achieved within the 3.0 m interval down to the design pile tip elevation the Contractor shall stop pile driving and notify the Contract Administrator. At this depth the pile should be allowed to rest for 48 hours, and the Hiley Formula shall then be applied immediately upon re-striking of the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator shall be notified and authorization given prior to driving the pile below the design pile tip elevation.

903.07.02.03.03 H-Piles, Tube Piles and Sheet Piles

Section 903.07.02.03.03 of OPSS 903 is amended by the addition of the following:

HP 310x132 pile splices shall be butt welded to the details and provisions of OPSD 3000.150.

RIGID EXPANDED POLYSTYRENE EMBANKMENT FILL - Item No.

Special Provision

1. SCOPE

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

As part of the work under this item, the Contractor shall supply and place 300 mm Granular B, Type I side cushion material, at least 300 mm of Granular B, Type II bedding and a 100 mm mortar sand levelling pad, as well as polyethylene sheeting, geotextile underlay, and reinforced concrete top slab above the EPS, at the abutments as shown on the Contract Drawings.

2. REFERENCES

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

National Standards of Canada

CAN/CGSB - 51.20 M87

ASTM

ASTM D1621	Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM C203	Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation
ASTM C177	Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus
ASTM D2842	Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Test Method for Measuring the Minimum Oxygen Content
ASTM D2126	Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

OPSS - Ontario Provincial Standard Specification

OPSS 212	Construction Specification for Borrow
OPSS 501	Construction Specification for Compacting
OPSS 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavations
OPSS PROV 904	Construction Specification for Concrete Structures

OPSS 1004	Material Specification for Aggregates, Miscellaneous
OPSS 1010	Material Specification for Aggregates, Base, Subbase, Select Subgrade and Backfill Material
OPSS 1605	Material Specification for Extruded Expanded Polystyrene Pavement Insulation
OPSS 1860	Material Specification for Geotextiles

3. SUBSURFACE CONDITIONS

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

4. DEFINITIONS

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

Rigid Expanded Polystyrene

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

Rigid Extruded Expanded Polystyrene

Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot

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

Quality Verification Engineer

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

5. QUALIFICATION

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

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

6. SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 Delivery, Storage, Handling and Protection

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

6.3 Construction

The contractor shall submit full details of the following:

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

The levelling pad shall consist of 100 mm thick layer of mortar sand with gradation and physical requirements as specified in OPSS 1004. Bedding below the levelling pad shall consist of 300 mm of Granular B, Type II, as indicated in the drawings. The Granular B, Type I side cushion material should conform to OPSS 1010.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
 - 1) Geometry
 - 2) Nominal Density
 - 3) Compressive Strength
 - 4) Flexural Strength
 - 5) Thermal Resistance
 - 6) Dimensional Stability
 - 7) Flammability
 - 8) Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.

8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

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

Table 1 – Material Properties

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

7.2.2.1 Geometry

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

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

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

7.2.2.2 Compressive Strength

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

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, method 1, Procedure B.2.7.4.

7.2.2.4 Dimensional Stability

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

7.2.2.5 Thermal Resistance

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

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

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

7.2.2.6 Flammability

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

7.2.2.7 Water Absorption

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

7.2.2.8 Chemical Resistance

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

7.2.2.9 Biological Resistance

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

7.2.2.10 Environmental

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

7.3 Polyethylene Sheeting

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

7.4 Concrete Top Slab

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

8.0 DELIVERY, STORAGE AND HANDLING

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

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

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

9.0 CONSTRUCTION

9.1 Foundation Preparation

Foundation preparation of the base upon which the polystyrene blocks are to be placed shall be carried out to the design elevations shown on the drawings as earth excavation or embankment construction. Any softened, loosened or deleterious materials at the base of the excavation or embankment level shall be sub-excavated and replaced with Granular 'A' or Granular 'B' Type II material in accordance with OPSS 1010.

9.2 Levelling Pad

Place, level and compact a 100 mm thick layer of OPSS 1004 mortar sand material on geotextile underlay in accordance with OPSS 501 to within ± 30 mm of the design elevation. Bedding below the mortar sand shall consist of 300 mm of Granular B, Type II on geotextile underlay. 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. The levelling pad must be placed in-the-dry.

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 except at the vertical construction joints.
- (3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- (4) Sloping end adjustments at the abutments shall be accomplished by 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.

9.4 Concrete Top Slab

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

9.5 Backfill

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

10. EQUIPMENT

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

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

11. QUALITY ASSURANCE

11.1 General

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

11.2 Sampling Frequency

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

11.3 Acceptance/Rejection

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

12. MEASUREMENT FOR PAYMENT

12.1 Actual Measurement

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

13. PAYMENT

13.1 Basis of Payment

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

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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