

**REPORT ON
GEOTECHNICAL INVESTIGATION
PROPOSED WATERMAIN CROSSING UNDER HIGHWAY 403
SOUTH OF DUNDAS STREET, OAKVILLE, ONTARIO
REGIONAL MUNICIPALITY OF HALTON**

**Prepared for:
DOWNUNDER GEOTECHNICAL LIMITED**

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1. INTRODUCTION

SPL Consultants Limited (SPL) was retained by Downunder Geotechnical Limited (Downunder) to undertake a geotechnical investigation for the proposed watermain (WM) crossing under Highway 403 in the Town of Oakville, within the Regional Municipality of Halton, Ontario.

The proposed 400 mm diameter HDPE watermain in this section is to be approximately 136 m in length (shaft to shaft) and is proposed to be constructed as a trenchless installation within about 110 m of MTO Right of Way (ROW). The proposed invert of the watermain ranges from 8.5±m (Elev. 155±m) depth under the median of Highway 403 to 2.5±m depth at the shaft locations just outside the MTO ROW.

The objectives of the investigation were to determine the subsurface conditions at the location of the proposed watermain crossing by means of exploratory boreholes, and to provide geotechnical recommendations for the design and construction of the trenchless crossing under Highway 403.

The report is presented in two parts. **Part A** of this report presents the factual borehole data, including review of regional geology, the method of investigation, the field and laboratory work, and describes the subsurface conditions encountered during the investigation. **Part B** interprets the ground and groundwater conditions as relevant to the geotechnical design and construction of the proposed watermain.

This report is provided on the basis of the terms of reference presented above and on the assumption that the design will be in accordance with applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations of this office can be relied upon.

This report has been prepared for the use of Downunder, Delcan, the Regional Municipality of Halton and the Ministry of Transportation. Third party use of this report without SPL Consultants Limited consent is prohibited.

PART A – FACTUAL DATA

2. SITE AND REGIONAL GEOLOGY

The site of the proposed watermain crossing is located on Highway 403, about 1 km south of Dundas Street in Oakville, Ontario. Site Photographs are shown in Appendix A.

Along the east-west alignment of the proposed watermain within the project limits, the existing grade is fairly level which varies from El. 164.2 m in BH13-11 to El. 163.4 m in BH13-8. The watermain alignment, outside the MTO ROW, lies within a Hydro One Networks right of way.

Based on the Physiography of Southern Ontario (1984), the surficial geology of the project site is relatively consistent, typically consisting of Late Wisconsinan Age Halton Glacial Till of red to brown, gritty silty clay to clayey silt till and glaciolacustrine deposits of massive to laminated silt and clay. Ordovician shale interbedded with limestone and calcareous siltstone of the Queenston Formation lie at about 1.5 m below existing ground surface.

3. FIELD AND LABORATORY WORK

3.1 Field Work

The field investigation within the MTO ROW was conducted by SPL on June 4 to 11, 2013, and this consisted of putting down three (3) boreholes (BH13-8, BH13-9 and BH13-10) to depths of 11.1 to 12.3m below existing ground surface at the locations shown on the attached **Drawing No. 1**. It should be noted that the original alignment provided by the Client for the fieldwork was different from the final alignment; it was moved by about 9 m to the south. The ground geodetic elevations and coordinates at the location of these boreholes were surveyed by the Region of Halton surveyors and the geodetic elevations and coordinates were provided to us.

As part of the crossing, Boreholes MW-7 and BH13-7/BH13-7A on the west side and BH13-11/BH13-11A on the east side were drilled just outside of the MTO ROW by the Client on June 17 to 21, 2013, and these were extended to a maximum depth of 9.5 m. Packer tests were conducted within BH13-7A and BH13-11A. BH13-7/7A and BH13-11/11A were supervised by SPL personnel. The geodetic elevations of the ground and coordinates at the locations of these boreholes were surveyed by SPL.

The field investigation work of initial borehole drilling (Boreholes BH13-8, BH13-9 and BH13-10) by SPL was undertaken by Terex Drilling while the drilling of Boreholes of MW-7 and Boreholes BH 13-7/7A and BH13-11/11A was performed by Atcost Drilling. The borehole logging services were provided by engineering staff from SPL. The boreholes were advanced with a power auger machine to the specified depths. The soil stratigraphy was recorded by observing the quality and changes of augered materials which were withdrawn from the boreholes, and by sampling the soils at regular intervals of depth using a 50mm O.D. split spoon sampler, in accordance with the Standard Penetration Test (ASTM D 1586) method. This sampling method recovers samples from the soil strata, and the number of blows required to drive the sampler 0.3m depth into the undisturbed soil (SPT 'N'-values) gives an indication of the compactness condition or consistency of the sampled soil material. The SPT 'N' values are indicated on

3.2 Geotechnical Laboratory Testing

The soil samples were taken to our laboratory where they were re-examined. Representative samples were selected for geotechnical index testing. The testing program consisted of the measurement of the natural moisture content of all samples, grain size analyses on six (6) selected samples and consistency (Atterberg) limits for six (6) plastic soil samples. Test results are shown on the individual borehole logs presented in **Appendix B**. The grain size analysis curves and result of the consistency (Atterberg) limits tests are plotted on Figures 1 to 2 attached to this report in **Appendix C**.

Testing of the rock cores consisted of point load index strength tests on eighty (80) rock samples and unconfined compression tests on five (5) samples. Test results are presented on the borehole log sheets of **Appendix B** and also provided in **Appendix D**.

Slake durability tests were carried out to determine the rock slaking degradation potential. The test is a means of measuring the resistance of shale or other weak rocks to repeated cycles of wetting and drying with abrasion. The test follows ASTM test method D4644-87. There are two cycles of wetting and drying. The test sample is obtained from the rock core which is broken into approximately 10 intact, equi-dimensional rock pieces, each about 40 to 60 grams in weight. The test sample is oven-dried overnight and is then placed in a drum made from 2.0mm square mesh stainless steel wire cloth. The drum is 50% submerged in a water bath at room temperature and is rotated for 10 minutes during which time small fragments of shale or dissolved rock material pass through the openings in the mesh into the water bath. Following the 10 minute first cycle, the test specimen is oven-dried overnight and the immersion and drum rotation process is repeated (2nd cycle). The sample is again oven-dried overnight. At the end of each cycle, the oven dried sample is weighed to determine the mass of sample retained. The slake durability Index [Id(2)] reported is a measurement of the ratio of the retained mass of rock fragments after two cycles of wetting and drying over the initial mass of oven-dried rock material, before the first cycle. Samples with Id(2) near 100% have very high resistance to slaking. Samples with lower Id(2) have less resistance to slaking and are therefore prone to degradation in the field when exposed to the elements over extended periods of time. Three (3) slake durability tests on selected core samples were performed in SPL's laboratory. The samples were tested in a Gilson Slake Durability Test apparatus with twin drums. The test results, along with photographs of the core samples before and after the tests, are provided in **Appendix D**.

Cerchar tests were conducted to measure the relative different hardness of a steel stylus tip and a rock specimen surface. A conical steel 90 degree stylus tip of known Rockwell Hardness (RHC 55) is applied against a saw cut or naturally broken face of a rock core under a constant static load of 70 Newtons and the sample is scratched with this stylus over a distance of 10.0mm. This procedure is repeated 3 times along 3 parallel lines against the rock core surface and then two (2) additional lines are scratched along a pair of transverse lines to assess any directional hardness differences in the rock fabric. Each time the core surface is scratched in this manner, the stylus is changed and a fresh unworn stylus is utilized. The so-called wear-flat diameter of each stylus tip is then measured under a microscope. The arithmetic average wear flat diameter of the five (5) stylus tips used on one core surface is calculated and is converted to units of CAI (Cerchar Abrasivity Index). One (1) Cerchar Abrasivity Index unit (CAI) is equivalent to a wear flat width of 0.1mm. A correction factor is applied to the CAI measured on saw-cut surfaces (CAIs). There is no correction factor applied to CAI measured on core surfaces with natural surfaces / breaks. The test procedure follows ASTM D7625-10 "Standard Test Method for Laboratory Determination of Abrasiveness of Rock Using the Cerchar Method". SPL tested selected five (5) core

samples for Cerchar Abrasivity on saw-cut surface. Full details on the test results, along with photographs of the core surfaces before and after scratching, are provided in **Appendix D**. The testing was performed using a GCTS Model RAA-100 Rock Abrasiveness Apparatus and stylus tips provided by GCTS, hardened to RHC57. The tips were inspected and the wear flat diameters measured under a 200x binocular microscope using GCTS RSS software.

3.3 Corrosivity Analysis

A number of rock samples were selected for chemical analyses to determine their corrosivity potential. Three (3) rock samples were tested for soil corrosivity indicator parameters and water soluble sulphates. The test results are attached in **Appendix F**.

4. SUMMARY OF SUBSURFACE CONDITIONS

4.1 Overview

The boreholes revealed, below the topsoil, the presence of surficial fill and residual soil consisting of silty clay to clayey silt underlain by bedrock.

Bedrock of the Queenston Formation of shale interbedded with limestone/siltstone was contacted at depths ranging between 1.4 and 1.8 m below the existing ground level (El. 162.7 to 161.7 m) in SPL's boreholes drilled in this area. In Downunder's Borehole MW-7 at the west end of the crossing, the bedrock was encountered at a depth of 0.8 m (El. 162.8 m). The bedrock surface appears to be level in this area.

For details of the subsurface conditions encountered at the borehole locations, reference should be made to the individual borehole log sheets and bedrock core log sheets presented in **Appendix B** for SPL's boreholes and Downunder's borehole. The properties of the overburden material and bedrock encountered in Boreholes BH13-7/7A through BH13-11/11A are described in the following sections.

4.2 Topsoil and Fill

Topsoil was encountered at the ground surface in Boreholes BH13-7/7A through BH13-11/11A and the thickness of this material varied from about 170 to 300 mm.

Below the topsoil, 0.3 to 0.8 m thick surficial fill materials were encountered in the boreholes to depths ranging from 0.6 to 1.1 m below existing grade. The fill is composed of silty clay with traces of sand and gravel, rootlets and topsoil. In Borehole BH13-8, the fill is somewhat organic. The consistency of the fill was found to be firm to very stiff, as inferred from SPT 'N' values of 4 to 17 blows per 0.3m penetration. The natural moisture content measured in the test samples from these materials ranged from 15% to 25%.

A grain size distribution test was conducted on a sample of the silty clay fill and the result showed 8% sand, 59% silt and 33% clay size particles. The grain size distribution curve for the sample is presented on Figure 1 in **Appendix C**.

Consistency (Atterberg) limits test on one (1) sample of silty clay indicate a liquid limit of 38, plastic limit of 26 and plasticity index of 12 (see Figure 2 in **Appendix C**). Based on these, the soil is classified as an organic silty clay (OI) based on the Unified Soil Classification System.

4.3 Silty Clay to Clayey Silt

Cohesive silty clay to clayey silt was encountered underlying the fill to depths of 1.4 to 1.8 m below existing grade in Boreholes BH13-7/7A to BH13-11/11A. SPT 'N' values in the cohesive soil were in the range of 19 to greater than 50 blows per 0.3m penetration, corresponding to very stiff to hard consistency, generally hard. Water contents were measured in the test samples to range from 9% to 12%.

In Borehole MW-7 by Downunder, a silty clay deposit was encountered below the topsoil and this extended to a depth of 0.8 m below existing grade. Measured SPT N-value of 11 blows per 0.3 m penetration indicates a stiff consistency.

The silty clay to clayey silt deposit contains traces of gravel, trace to some sand and occasional shale and limestone fragments. This deposit is considered a residual soil from the parent bedrock below.

Five (5) tested samples of the silty clay to clayey silt contain 0% to 2% gravel, 4% to 10% sand, 68% to 70% silt and 22% to 28% clay size particles. The grain size distribution curves for the samples are presented on Figure 1 in **Appendix C**.

Consistency (Atterberg) limits tests on five (5) samples of the silty clay to clayey silt indicate liquid limits of 23 to 28, plastic limits of 17 to 20 and a plasticity index of 6 to 8 (see Figure 2 in **Appendix C**). Based on these, the soil is classified as a low plasticity silty clay (CL) to clayey silt (CL-ML) based on the Unified Soil Classification System.

4.4 Queenston Formation

Bedrock of the Queenston Formation consists of typically slightly weathered to fresh, reddish brown, fine to very fine grained, fissile, weak to medium strong shale interbedded with slightly weathered to fresh grey, fine grained medium strong to very strong, calcareous siltstone and limestone layers. Rock core photographs are presented in **Appendix D**. The inferred top of bedrock level varies from El. 162.8 to 161.7 m. It is noted that variations in the bedrock surface should be expected. It is often difficult to distinguish where the residual soil ends and bedrock begins, particularly where the bedrock surface is highly weathered to weathered. As such, the inferred bedrock surface level should not be considered accurate to better than ± 1 m.

The descriptive terms used on the record of rock cores and throughout this report are explained on the "Explanation of Terms Used in the Bedrock Core Log" sheet in **Appendix B**. In general, the conventions of the International Society for Rock Mechanics (ISRM) are adopted herein. Detailed descriptions of the index properties and results of laboratory testing are presented in the following paragraphs.

Total Core Recovery (TCR)

The total core recovery indicates the total length of rock core recovered expressed as a percentage of the actual length of the core run. The total core recovery was generally good, with values ranging from 83% to 100%. Generally, poorer core recovery was experienced only near the surface of the rock, where the formation is more weathered.

Solid Core Recovery (SCR)

The solid core recovery is the total length of solid, full diameter rock core that was recovered, expressed as a percentage of the length of the core run. Solid core recovery ranged from 15% to 100% with an average value of 82%, and also appears to generally improve with depth. The SCR index was generally influenced by the orientations of the fractures. SCR was low when fractures oblique to the borehole axis were intercepted.

Rock Quality Designation (RQD)

The rock quality designation index is obtained by measuring the length of intact recovered rock core pieces which are longer than 100mm and expressing their sum length as a percentage of the length of the core run. RQD is a function of the frequency of joints, bedding plane partings and fractures in the rock cores. While the use of double tube core barrels provided reasonably good protection of the core during drilling and core retrieval, the fissile nature of the shale greatly influences the RQD values of the rock cores. Consequently, it is believed that the RQD values recorded underestimate the rock quality classification of the laminated fissile shale. On the basis of the recorded RQD values, the top 3± m of the bedrock has RQD values ranging between 0% and 65% indicating "very poor" to "fair" rock quality, while the rock below this depth has RQD ranging from 53% to 100% suggesting "fair" to "excellent" rock quality.

Hard Layers

Based on the visual examination of the rock cores, an attempt was made to identify and record the thickness and percentages of the relatively harder siltstone and limestone layers. The percentage of the "hard layers" per core run ranges between 3 and 20%. The thickness of these layers varied but was generally less than 100mm, except in BH13-11A where most of the hard layers are 100mm thick or greater, up to 150 mm in thickness. This rock formation, however, is known to contain very strong limestone or siltstone layers up to 600mm in thickness. Encountering such thick layers should be anticipated. It is also common to encounter closely spaced groupings of thin strong limestone/siltstone layers which individually may only be a few centimetres thick but collectively can be 1m in thickness.

Fracture Index

When logging the rock cores, the fracture Index (i.e. the number of fractures for each 0.3m length of core) was also recorded. The recorded values range between 0 and greater than 25, but typically 2 to 4. It was observed that the planes of weaknesses along which the cores tended to break, included planes of fissility and bedding, the contact surfaces between shale and siltstone or limestone bands and some oblique and subvertical joints. The joints along the planes of fissility were generally smooth and clean

while those along the bedding surfaces were generally more open and were occasionally infilled with clay. The occasional oblique and subvertical joints, which are found frequently within the siltstone and limestone beds, were often stepped and the joint surfaces were often uneven and rough.

Weathering

In general, weathering in the bedrock was limited to the surfaces of major discontinuities. Deeper penetrating weathering has occurred in the zones very close to the bedrock surface, where the degree of weathering is described predominantly as highly weathered to slightly weathered. Below this, the degree of weathering ranged from slightly weathered to fresh as indicated on the Records of Rock Cores. The siltstone and limestone layers were generally fresh with only slight surficial weathering on joint surfaces in the zone close to bedrock surface.

Slake Durability

Slake durability tests were performed on three (3) red-brown shale or shaly mudstone samples. The test results are provided in **Appendix D** and are summarized in Table 4.4a.

Table 4.4a – Summary of Slake Durability Test Results

Sample No.	Sample Description	Slake Durability Index -2nd Cycle [Id(2)]
BH13-8 16'10" (5.13m)	red-brown shale/mudstone	78.8%
BH13-9 22'4" (6.81m)	red-brown shale/mudstone	72.0%
BH13-10 14' (4.27m)	red-brown shale/mudstone	80.0%

The data suggests that the shaly mudstone and red-brown shale material is quite slake-prone. The green-grey interbeds of more limy material, though not tested, are less prone to slaking degradation.

Cerchar Abrasivity

Cerchar Abrasivity tests were performed on five (5) rock core samples. Three (3) of the five rock core samples were red-brown shale or shaly mudstone and the other two (2) were grey-green limy siltstone to limestone. The corrected Cerchar Abrasivity Index (CAI) values and corresponding abrasiveness classification as per ASTM is indicated in Table 4.4b.

Table 4.4b – Summary of Cerchar Abrasivity Test Results

Sample I.D.	Surface Material Description	Corrected CAI	Abrasiveness Classification
BH13-7A (CS-1), 23'5" (7.14m)	Grey limestone	1.8	medium
BH13-8 (CS-2), 23'8" (7.21m)	Red-brown shale	1.6	medium
BH13-9 (CS-3), 27'5" (8.36m)	Grey limestone	1.8	medium
BH13-10 (CS-4), 19'6" (5.94m)	Red-brown shale	1.5	medium
BH13-11A (CS-5), 20'10" (6.35m)	Red-brown limy shale	1.7	medium

From the above results, the tested limestone and shale can be considered as medium abrasive based on ASTM D7625-10.

Uniaxial Compressive Strength (UCS) and Point Load Index Strength

To determine the compressive strength of the intact rock, five (5) samples of suitable length core were selected for uniaxial compression tests. The test results are presented in Table D-1, **Appendix D**. The UCS of the tested samples of limy shale ranged from 19.3 to 36.5 MPa. The test results indicated that the shale samples ranged from "weak" to "medium strong" rock under the ISRM strength convention.

Point load index strength tests were performed on thirty six (36) limestone/siltstone samples and forty four (44) samples of shale, for a total of eighty (80) rock core samples. The test results are presented in Table D-2, **Appendix D**. We have utilized the empirical relationship between unconfined compressive strength (UCS) and point load index strength as follows:

$$UCS [MPa] \approx 24 I_{s(50)}$$

where $I_{s(50)}$ is the point index strength in MPa for a 50mm equivalent diameter core. This is an approximate correlation after Franklin and Hoek.

The equivalent unconfined compressive strength of the limestone/siltstone samples was inferred to range from 35 to 150 MPa in the axial direction and from 30 to 103 MPa in the diametral direction. Those values are indicative of "medium strong" to "very strong" rock under ISRM strength convention. The inferred UCS of the shale embedded with limestone/siltstone was lower than that of limestone/siltstone, ranging from 16 to 72 MPa in the axial direction. These values indicate a generally "weak" to "strong" rock. The low inferred UCS values in the diametral direction for shale are not considered to be representative due to the fissile nature of the rock. The shale can often be broken by hand in the diametral direction, indicating considerable strength anisotropy along bedding planes.

Unit Weight and Modulus

The unit weight (γ) of the tested limy shale samples ranged from 24.8 to 26.2 kN/m³. The Young's Modulus (E) of the tested shale samples ranged from 4.3 to 10.0 GPa and the Poisson's ratio ranged from 0.12 to 0.17.

Secondary Rock Mass Hydraulic Conductivity

Packer tests were performed in 3m long sections at two or three different net water pressures in each test section. The test results are summarized in Table D-3, **Appendix D** and are also presented on the individual borehole log sheets. The measured values range from 6.2×10^{-7} m/s to "no take" ($< 2 \times 10^{-9}$ m/s).

In-situ Stresses

In-situ stress measurements were carried out by a number of investigators in the Toronto area in the Queenston Formation and the measured horizontal stress ranged between 1.7 MPa and greater than 6.9 MPa (References 1, 2, 3 and 5).

Time-Dependent Deformation Characteristics (TDD)

It has been observed both in the laboratory and in the field that upon relief of the high residual horizontal stresses within the Queenston Formation, time-dependent, creep-like deformations take place. These time-dependent deformations persist well beyond the initial elastic deformations and generally exceed the magnitude of the elastic movements. The magnitude of TDD strain can be approximated in the laboratory by free swell tests. TDD measurements were not performed as part of this investigation; however, some reported values of laboratory free swell tests performed by others on samples of the Queenston formation over extended periods of time ranged from 0.03 to 0.26%/Log cycle of time (References 1, 3, 4, 6, 7 and 8).

Gas

The Queenston Formation is known to contain pockets of combustible gas. Although there were no physical indications of the presence of gas in the boreholes during the present investigation, appropriate care and monitoring are essential in all confined bedrock excavation.

4.5 Groundwater Conditions

Two (2) monitoring wells (MW-7 and BH13-11) were installed within this section of the crossing for longer-term monitoring of groundwater levels.

About five weeks after they were installed, the groundwater levels in the monitoring wells installed in the bedrock ranged from 2.7 to 3.0 m below existing ground surface (El. 160.9 to 161.2 m). Groundwater measurements in the monitoring wells are shown on the attached borehole logs and are also summarized on Table 4.5.1.

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Over the long term, seasonal fluctuations in the groundwater level are expected. We recommend that the piezometric levels be further monitored.

Table 4.5.1 - Measured Water Levels in Monitoring Wells

BH No.	Ground Surface Elev. (m)	Soil Type at Screen Location	Depth / Water Level Elevation (m)					
			June 19/13	June 20/13	June 21/13	June 27/13	July 5/13	July 26/13
MW-7	163.6	Shale bedded with siltstone/limestone		Dry	2.4 / 161.2	2.4 / 161.2	2.6 / 161.0	2.7 / 160.9
BH13-11	164.2	Shale bedded with siltstone/limestone	3.6 / 160.6	2.5 / 161.7	2.6 / 161.6	2.8 / 161.4	2.9 / 161.3	3.0 / 161.2

PART B – GEOTECHNICAL INTERPRETATION AND RECOMMENDATIONS

5. GEOTECHNICAL INTERPRETATION AND RECOMMENDATIONS

In this section, the overburden and bedrock conditions are interpreted as relevant to the design and construction of the proposed watermain crossing. Comments relating to construction are intended for the guidance of the design engineers (Delcan) to establish constructability.

The construction methods described in this report must not be considered as being specifications or direct recommendations to the contractors, or as being the only suitable methods. Prospective contractors should evaluate all of the factual information, obtain additional subsurface information as they might deem necessary and should select their construction methods, sequencing and equipment based on their own experience in similar ground conditions. The readers of this report are also reminded that the conditions are known only at the borehole locations and in view of the generally wide spacing of the boreholes, conditions may vary significantly between boreholes.

5.1 Overview of Subsurface Conditions

In simplified terms, the subsurface profile consists of surficial topsoil and fill underlain by silty clay to clayey silt, which is in turn, underlain by the Queenston Formation of shale interbedded with limestone/siltstone. The shale bedrock was encountered at 0.8 to 1.8 m below existing ground surface (El. 162.8 to 161.7 m).

The groundwater table lies between 2.7 m and 3.0 m below existing ground surface in the bedrock (between El. 160.9 and 161.2 m).

5.2 Trenchless Construction

For the installation of the 400 mm HDPE watermain under the Highway 403, trenchless construction is required and will be advanced mainly through the bedrock of the Queenston Formation. Different options were considered and these are as follows:

- Horizontal Directional Drilling (HDD)
- Tunnelling using a Micro Tunnel Boring Machine (MTBM)
- Tunnelling by Hand Mining
- Jack and Bore
- Pipe Jacking with Manual Excavation

We understand that watermain will be 500 mm in diameter for the HDD option or alternatively, the watermain diameter could be 400 mm if installed within a 900 mm jacking pipe for MTBM.

The above list of options with their advantages and disadvantages/limitations are presented in Table E-1 in Appendix E.

Based on the Table E-1, HDD is considered the most cost-effective option and is being considered by the designer for the installation of the watermain. However, MTO requirement of a steel casing under the

highway may consider an MTBM as the second alternative. These two options will be discussed in the next section.

5.2.1 Trenchless Crossing of Highway 403

The length of the watermain crossing under Highway 403 is currently about 136 m (shaft to shaft) and the proposed invert elevation is at about El. 155.0± m under the median of the Highway. At this elevation, the rock cover above the tunnel, based on Borehole BH13-9, is about 6.2 m or approximately 6.9 times the bored diameter of up to 0.9 m. This is considered to be sufficient rock cover.

At about El. 160 to 155 m, the tunnel will be driven through mostly slightly weathered to fresh shale with interbedded limestone and siltstone rock characterized by RQD values ranging between 49% and 100%, suggesting a "fair" to "excellent" rock quality using Deere's classification, but generally "good" rock quality. Overlying the obvert of the tunnel is about 6 m thick zone of "very poor" to "excellent" rock characterized by RQD values of 0 to 92% at the borehole locations. It would be prudent to expect some variability in the rock quality between the boreholes.

In carrying out the trenchless crossing, consideration should be given to (i) the high in-situ horizontal stresses existing in the rock, causing compressive stress concentrations around the floor and crown of the opening and tensile stresses at the sidewalls; (ii) the expected long term deformations causing the rock to squeeze into the tunnel; (iii) the possibility of encountering combustible gas; and (iv) the tendency for the shale to 'slake' or deteriorate over time when exposed to air.

The selection of the equipment and method to remove the overburden soils and bedrock, advance the trenchless crossing and type of temporary ground support should be the contractor's choice. In the selection of these, the contractor should consider the strength range of the intact shaley rock which was measured to range between about 16 and 72 MPa, on the basis of which the rock is classified as "weak" to "strong". Consideration should also be given to the fact that the rock mass contains numerous limestone/siltstone layers which could be a few centimetres thick but collectively can be 1m in thickness, and that the strength of these layers is in the 30 to 150 MPa range ("medium strong" to "very strong").

The preferred option of Horizontal Directional Drilling (HDD) method consists of pilot boring, back reaming and pipe pulling. Drilling begins with a small diameter pilot hole along a designated alignment, using flexible drill rods with remote controlled steering system. After the pilot boring, a back reamer is installed and drilled back through the pilot hole to achieve the required diameter for the pipe to be installed. Typical ratio of diameter of reamer to pipe is 1.3 to 1.5. Special drilling fluid is used to prevent the collapse of borehole as well as providing a lubricant for the drilling and flushing spoils.

The depth from the existing ground surface to the axis of the watermain ranges from 3m to 9m. These cover depths are considered to be sufficient for the proposed pipe. The potential for drill fluid migration to ground surface ("frac-out") is dependent not only on the cover thickness but also the drilling equipment and methods. The contract should have contingency/mitigation plans in the event that inadvertent drilling fluid release takes place. It is recommended to have at least one vacuum truck on standby during the HDD work.

One of the biggest risks associated with the use of HDD for the trenchless pipe installation on this project is the potential for jamming of the product pipe in the bore. This could occur where loose rock blocks in bedrock, fill soils, cobbles and rock slabs in native overburden become dislodged from the bore wall and sink into the drilling fluid. This condition is exacerbated at vertical and horizontal bends in the trajectory since the product pipe may contact the bore wall at tighter radii, potentially dragging and dislodging loose fragments. A jammed pipe can result in complete loss of the bore and product pipe in the worst case scenario. Although risk of this problem cannot be eliminated, it can be reduced by:

- Careful selection/control over-drilling mud specific gravity;
- Maximizing vertical curve radii and eliminating horizontal curves, if possible;
- Back-reaming using a tool which is suited to shale rock to minimize tearing of the rock;
- Back-reaming to greater than 1.5 times the product pipe diameter;
- Checking the bore before pulling the product pipe with a "pig";
- In extreme cases, the pilot hole is grouted to consolidate the rock and then re-drilled along the same trajectory.

This risk and mitigation method should be discussed with the specialist HDD contractor.

Contractors bidding on this project should be required to submit their bore plan and methodology, including specifications for drilling fluid, maximum and minimum pressures utilized during the HDD process, prevention of hydrofracturing, tracking of the bore head, locations of any relief pits, drilling fluid recycling and disposal plan, emergency measures and materials to be used in case of hydrofracture etc., for review by the geotechnical engineer. Reference should be made to OPSS 450 for this method, as well for related specifications. All components of the drilling fluid must meet the requirements of ANSI (American National Standard Institute) Certified 60.

The length of pipe to be installed by HDD method is normally assembled (butt fusion welded if HDPE) and laid out in a single piece for each section, in order to minimize the time and reduce the potential for instability of the drill hole. Sufficient lay-down room must be provided to the contractor for the assembly of the pipe in a single piece beyond the pull-back pit. Inspection of the fusion welds by qualified personnel is required.

The contractor may opt to excavate the tunnel using a Micro-Tunnel Boring Machine (MTBM). The MTBM and cutter head should be designed to accommodate the laminated nature of the Queenston Formation, as well as the relative amount and thickness of the hard layers. On the average, the formation, based on the current boreholes, contains of up to about 20% of hard layers of "strong" to "very strong" rock (but we have seen in other parts of GTA where the hard layers make up as much as 30% of the profile). The corrected Cerchar Abrasivity Index of the hard layers is measured to be from 1.5 to 1.8, corresponding to "medium" abrasiveness.

Some zones of shale will produce more clay-rich 'muck'. This clayey muck can be 'sticky' and can foul up some cutterheads and conveyance equipment. The use of polymers may be needed to reduce the stickiness.

To reduce jacking loads, lubricants should be applied to the pipes. The lubricant usually consists of a mixture of bentonite and water, but polymer lubricants are also often used. Lubricant selection should be the option of the contractor. The annular space between the product pipe and the bore wall must be sufficient to accommodate rock squeeze.

Between the jacking pipe and the pipe segments, a compressible low strength, low modulus grout material (such as foamed or cellular grout) should be placed that will absorb these long term deformations (without transferring high stresses to the permanent watermain pipe).

If reinforced concrete jacking pipe is used as the temporary lining to house the watermain, it will need to be discussed with MTO since a welded steel liner may also be required to contain the pressure and release of water in the event of a watermain leak.

The Queenston Formation is known to contain pockets of combustible gas. Although during the present investigation there were no physical indications of the presence of gas in the borehole, the monitoring of the gas in the tunnel should be a mandatory part of the contract and proper ventilation systems will obviously be needed.

5.2.2 Shafts

We understand that two (2) shafts, one at each end of the crossing, will be constructed for the trenchless crossing. The subsurface conditions encountered in the boreholes at or near these shafts are shown in **Drawing No. 1**, and are briefly summarized in Table 5.2.2.

Table 5.2.2 - Subsurface Conditions at Shafts

Approximate Station	BH No.	Soil/Rock Type	Water Table in Piezometer
1+570	13-7/7A	0.6m topsoil/fill underlain by hard silty clay over bedrock at 1.7 to 2.5 m (Elev. 161.1m) for HDD or to 8.5 m (Elev. 155.1m) for MTBM, below ground surface.	2.7 m (El. 160.9) below ground surface
1+690	13-11/11A	0.6m topsoil/fill underlain by hard silty clay over bedrock at 1.5 to 2.5 m (Elev. 162.7m) for HDD or to 9.1 m (Elev. 155.1m) for MTBM, below ground surface.	3.0 m (El. 161.2) below ground surface

The fill would be classified as Type 3 Soils defined by the Occupational Health and Safety Act of Ontario above the groundwater table. The native silty clay to clayey silt falls into the category of Type 2 Soils.

Excavation progress in the hard silty clay to clayey silt and the highly weathered to slightly weathered shale may be slow and as such, heavy equipment will be required.

Vertical cuts in the overburden soils should be supported with shoring. Sheet piling is not recommended as it is difficult to be driven into the bedrock. Alternatively, soldier piles and lagging or similar methods are recommended. The soldier piles and lagging could be supported by tie-back anchors or struts. Because of anticipated difficulties to install the soldier piles by driving, they will have to be installed in pre-augered holes. The earth pressure distribution shown on **Drawing No. 2** should be utilized for multiple strut or tie-back shored system. All shoring designs should be in accordance with the 4th Edition of the Canadian Foundation Engineering Manual and must be reviewed by a qualified geotechnical engineer. Allowable bond stress for rock anchors is 400 kPa (at least 3m into sound rock). An allowable bearing capacity of 3000 kPa can be used for soldier piles which are installed at least 1m into sound bedrock. No water pressure has been assumed to act against the shoring. Lagging walls are assumed to permit drainage of perched water between the lagging boards. The surcharge must account for construction machinery. If shoring is to be carried out over the winter months or if the excavation is to be left open for any period during below zero temperature, shored walls must be protected against frost penetration by means of insulation or heated hoarding. No loss of ground should be permitted during augering for soldier piles and the drilling contractor should be warned of the potential for obstructions within the residual soil such as rock fragments or slabs.

The excavation into fresh, sound bedrock can be performed using near-vertical sidewalls (say 10V:1H) provided that:

- All OHSA requirements regarding worker safety are met during the course of the work.
- The bedrock excavation is stepped-in from the inside face of the secant pile wall by at least 1.2m and the rock in the pile toe zone (which is offering passive restraint to the piles) is continuously reinforced at each king pile, or closer spacing, by means of a row of inclined rock anchors fitted with steel bearing plates designed to resist the pile toe reactions. Alternatively, the contractor may opt to extend the soldier piles to minimum 1m below the base of the excavation in order to avoid the 1.2m benching at the bedrock surface.
- the rock face is scaled of all loose and potentially spalling material (including slaked rock as the excavation faces dry out over time) and is fully covered with a welded wire mesh (or at least 60mm of fibre-reinforced shotcrete).

Considering the measured hydraulic conductivity of the rock mass as 2.1×10^{-7} m/s or lower, the rate of groundwater seepage through the bedrock is expected to be slow, and can be handled by gravity drainage and pumping from filtered sumps established at the base of the shaft. Some increased seepage may occur along the overburden to bedrock contact as well as along limestone interbeds within the shale.

5.2.3 Settlement Monitoring

Monitoring of settlements at some distance above the tunnel invert is recommended underneath the pavement of Highway 403. The settlement monitoring system should consist of deep/shallow settlement points installed at different depths into the rock/soil and surface settlement points installed

at the traffic lanes of the road. These should be installed along the tunnel alignment. Ground movements of these points should be monitored at regular intervals during the tunnelling operations. In the unlikely event that unacceptable ground movements are observed, the tunnelling and ground support operations should be immediately modified.

With good workmanship, loss of ground and soil relaxation can be minimized, and it should be possible to keep settlements at road level to less than 10mm.

A settlement monitoring program for any trenchless crossing alternative underpassing Highway 403 is required as per MTO's Guidelines for Foundation Engineering – Tunnelling Speciality for Corridor Encroachment Permit Application. The settlement monitoring system should consist of deep/shallow and surface settlement points installed at different depths into the rock/soil in the road, shoulder and near utilities. These should be installed along the tunnel alignment but care must be taken so as to stay a safe distance above the obvert to minimize the hydrofracturing potential into an instrument hole.

The installation locations, details and monitoring frequency and accuracy are presented in **Drawing Nos. 3 and 4**. A minimum of two (2) sets of repeatable baseline readings should be taken on all of the settlement points well in advance of the start of the tunnelling. Settlement monitoring should be conducted at least three (3) times daily. Readings may be reduced to twice daily when the tunnel face is greater than 50m away from the monitoring points. The frequency of readings can then be reduced to daily for a minimum of two (2) weeks, twice weekly for a period of one month and then once monthly for the following six months.

Should settlement monitoring indicate excessive ground movement, immediate changes to the tunnelling method and ground support procedures must be adopted. The following table details the recommended 'Review/Alert Levels' for in-ground settlement rods.

Table 5.2.3 - Settlement 'Review/Alert Levels'

Ground Movement as Measured in Settlement Rods	Notes
<10 mm	Proceed. No action required.
Review Level: 10 mm	Immediately notify MTO & the geotechnical engineer for further assessment; Proceed with caution.
Alert Level: >15 mm	Halt tunnelling until further assessment is carried out by the MTO & geotechnical engineer; Carry out immediate remedial work to the settlement zone as approved by the MTO.

A preconstruction condition survey of Highway 403 pavement in the proximity of the tunneling, as well as the existing utilities in the immediate vicinity of the tunnel alignment, should be carried out prior to start of construction.

5.2.4 Lateral Earth Pressure

Lateral Earth Pressure in Overburden Soils

The earth pressure distribution on shafts can be taken as hydrostatic, i.e. which is increasing uniformly with depth according to the formula:

$$P_h = K \cdot \gamma \cdot h + K \cdot q$$

where

P_h = horizontal pressure at depth h (kN/m²)

γ = unit weight of soil (kN/ m³), taken as 21 kN/ m³

h = depth below ground surface (m)

q = surcharge load at ground surface (kPa)

K = coefficient of lateral earth pressure, assumed to be 0.50

Lateral Earth Pressure in Bedrock

Shafts which extend below the surface of the bedrock and the walls of which are poured in direct contact with the bedrock will be subject to "rock squeeze".

Although in-situ stress measurements were not made at this site, it is known that bedrock belonging to the Queenston Formation contains high horizontal stresses, the magnitude of which varies between 1.7 and greater than 6.9 MPa. As a result of the relief of this high horizontal stress, significant elastic displacements occur during and after the excavation. Of these, the long term, time dependant displacements are of greater importance. These are estimated to be of the order of 0.05% of the height of the excavation per log cycle of time (e.g. 5mm per log cycle of time (in days) for a 10m deep excavation or about a total of 21mm over a period of 50 years). Approximately 50% of the displacement (i.e. 10mm) is expected to occur during the first 100 days following excavation. It is essential, however, that precise measurement of the rock squeeze be conducted at regular intervals of time during the course of the excavation in order to be able to better predict the dissipation of rock strains over time. This is normally done using a convergence gauge with measurement points consisting of stainless steel eye bolts which are grouted into the faces of the excavation at varying depths.

The shaft should not be designed to resist these displacements. Rather a layer of compressible material must be placed between the structure and the rock. This compressible layer could be either a synthetic material (e.g. suitable expanded polystyrene) or foamed "cellular grout". Properties and proposed thicknesses of the compressible material should be submitted to a qualified engineer to evaluate its stiffness and assess its suitability. Certain rigid polystyrene insulation products are considered to be excessively stiff for this application.

Provided that rock squeeze is allowed to dissipate by delaying construction of permanent concrete walls or by applying a compressible void former, the lateral earth pressures acting on this bedrock for concrete cast against rock with no backfill can be assumed to be a uniform pressure equal to the maximum overburden lateral earth pressure calculated at the overburden to rock interface, plus the hydrostatic forces.

5.3 Corrosivity Test Results

Three (3) rock samples from Boreholes BH13-8, BH13-9 and BH13-10 at depths from about 6 to 8 m were submitted for corrosivity tests. The summary of results is presented on Table 5.3.1 and the test results are attached in **Appendix F**.

Table 5.3.1 Summary of Corrosivity Test Results

Parameter	BH13-8/RC2 (bottom)	BH13-9/RC5	BH13-10/RC3 (top)
pH	9.94	9.56	9.39
Sulphate ($\mu\text{g/g}$)	121	48	38
Resistivity (ohm.cm)	4200	5710	4370
Redox Potential (mV)	200	189	177
Sulphides (%)	0.02	0.02	0.02

The corrosivity of the rock was evaluated using the 10 points method which is based on five material properties: sulphides, resistivity, pH, redox potential and moisture content.

The need for cathodic protection to gray or ductile cast iron pipe as given in the ANSI/AWWA Rating for corrosion evaluation is given in the **Appendix F**. A summary of the evaluation based on the test values is summarized in Table 5.3.2 below.

Table 5.3.2 Summary of Test Results for Cathodic Protection

Borehole / Sample Number	Assigned Points
BH13-8/RC2	7
BH13-9/RC5	7
BH13-10/RC3	7

The assigned points for the test results of three rock samples are less than 10, indicating that the rock is not unusually corrosive to gray or ductile cast iron pipes.

The test revealed that the sulphate concentrations in the rock samples were 38 to 121 $\mu\text{g/g}$. The category of severity of attack is "negligible" based on CSA Standard A23.1, Concrete Materials and Methods of Concrete Construction. The final selection of the type of concrete should be made by the Engineer taking into account all aspects of design considerations.

6. LIMITATIONS OF REPORT

The statement of limitations, as provided in **Appendix G**, forms an integral part of this report.

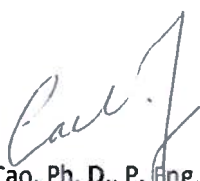
Thank you for the opportunity to be of service to you. Should you have any questions or require further clarification on any aspect of this report, please do not hesitate to contact this office.

Yours very truly,


SPL CONSULTANTS LIMITED


Ramon Miranda, P. Eng.




Laifa Cao, Ph. D., P. Eng.




Scott M. Peaker, P. Eng.

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Drawings

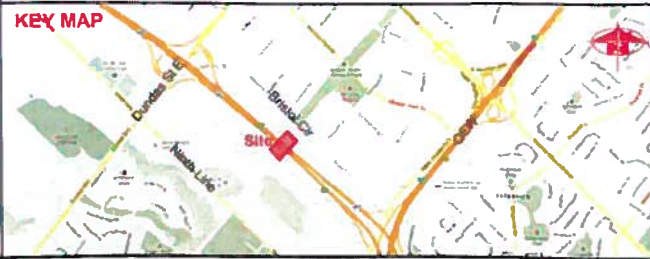
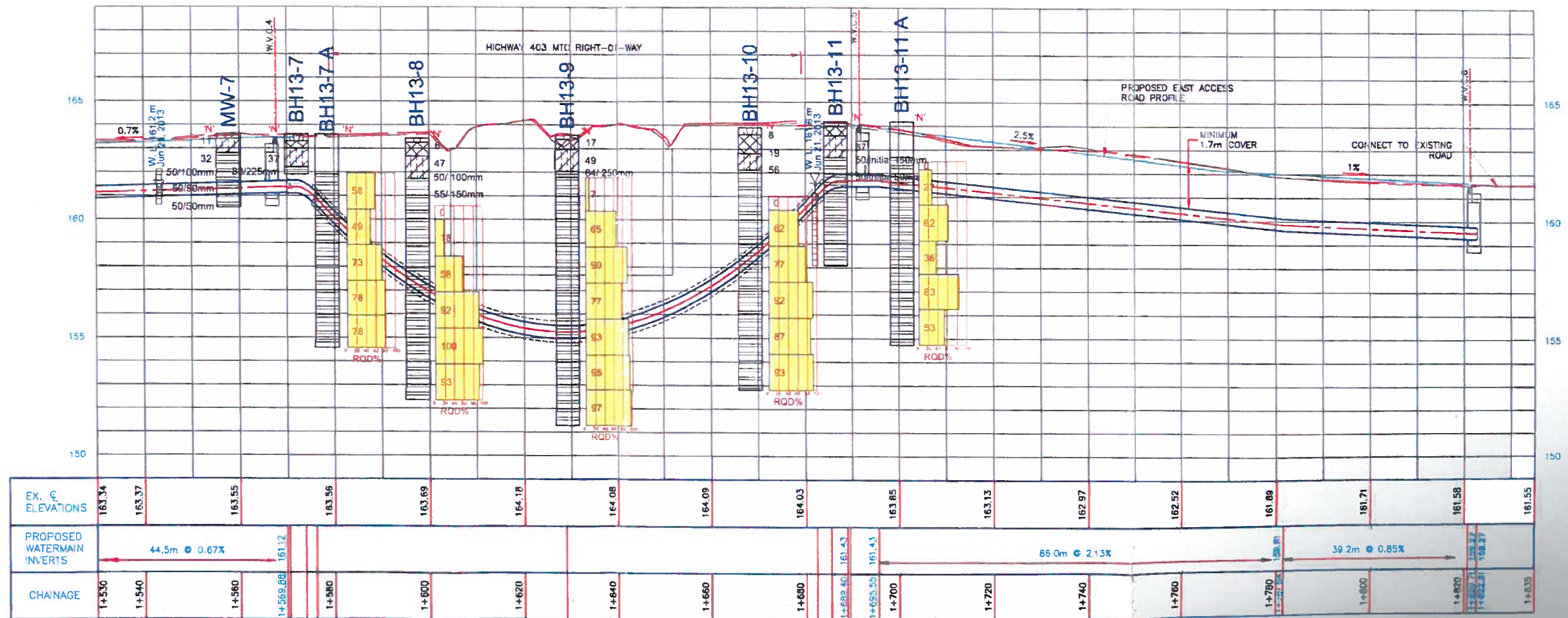
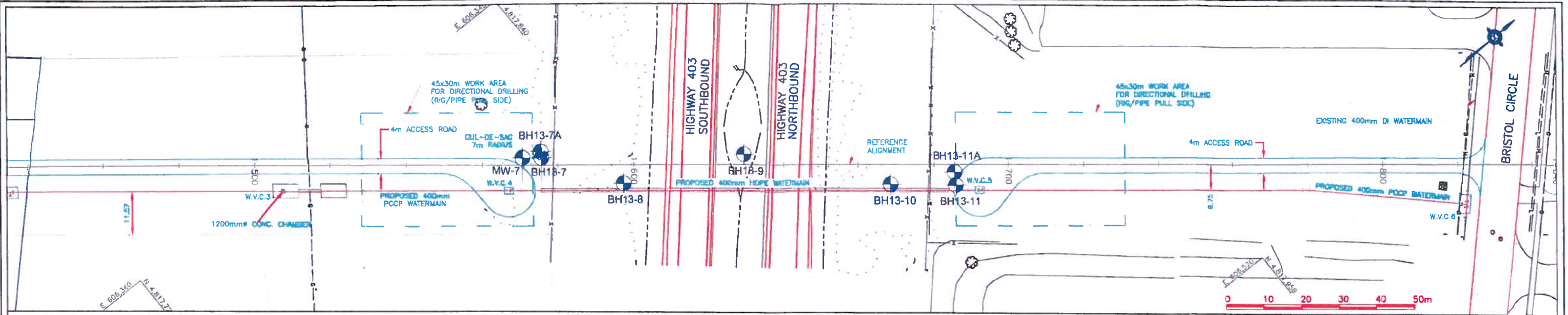
Borehole Location Plan and Geological Section (Drawing No. 1)

Earth Pressure Distribution (Drawing No. 2)

Preliminary Layout of Ground Monitoring Arrays (Drawing No. 3)

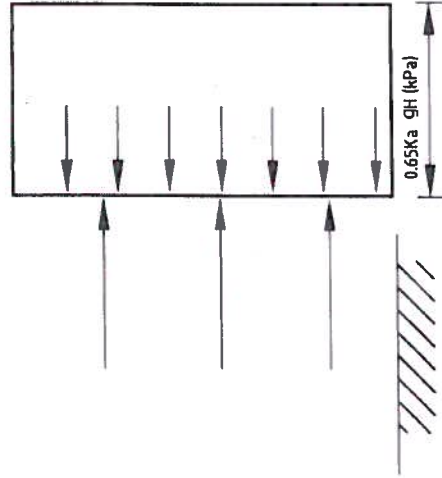
Installation Details of Ground Monitoring Arrays (Drawing No. 4)

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LEGEND	
	Topsoil
	Silty Clay
	Fill
	Shale

Client: Downunder Geotechnical Ltd		Project No.: 1549-110	Drawing No.: 1
Drawn: ZMO	Approved: RM	Title: Borehole Location Plan and Geological Section	
Date: August, 2013	Scale: As Shown	Project: Geotechnical Investigation - Highway 403 Watermain - Crossing, South of Dundas St. W, Oakville, ON	
Original Size: Tabloid	Rev: N/A	SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	



g = unit weight of soil = 21.0 kN/m³

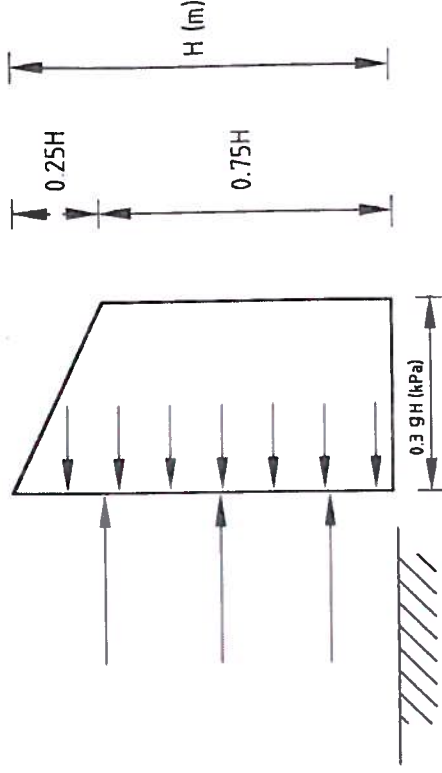
g' = submerged unit weight of soil (i.e. below ground water level) = 11.2 kN/m³

$K_a = 0.3$

IN NON-COHESIVE SOILS (SANDS AND SILTS)

Notes:


1. Check system for partial excavation condition.
2. If the free water level is above the base of the excavation, the hydrostatic pressure must be added to the above pressure distribution.
3. If surcharge loadings are present near the excavation, these must be included in the lateral pressure calculation.

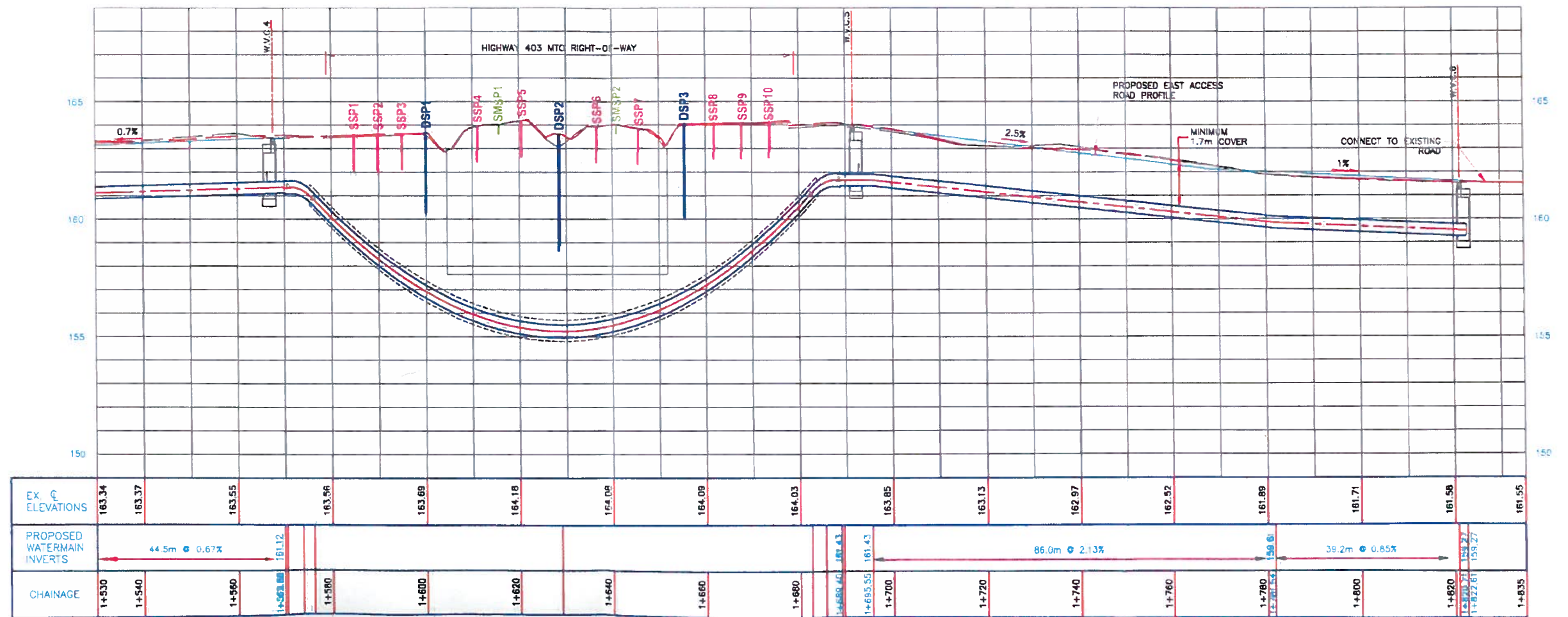
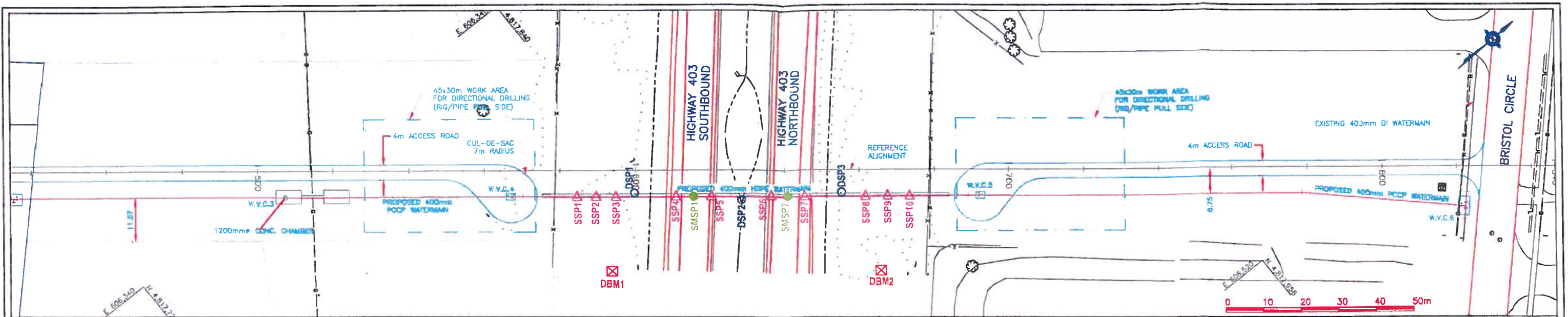


g = unit weight of soil = 21.5 kN/m³

g' = submerged unit weight of soil (i.e. below ground water level) = 11.7 kN/m³


ON BRACED SHEETING IN COHESIVE CLAYS OR CLAYEY SOILS

Client:	Downunder Geotechnical Ltd	Project No.:	1549-110	Drawing No.:	2
Drawn:	ZMO	Approved:	RM	EARTH PRESSURE DISTRIBUTION	
Date:	August 26, 2013	Scale:	Not To Scale	Project:	
Original Size:	Letter	Rev:	N/A	Geotechnical Investigation - Highway 403 Watermain - Crossing, South of Dundas St. W, Oakville, ON	
				 SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	



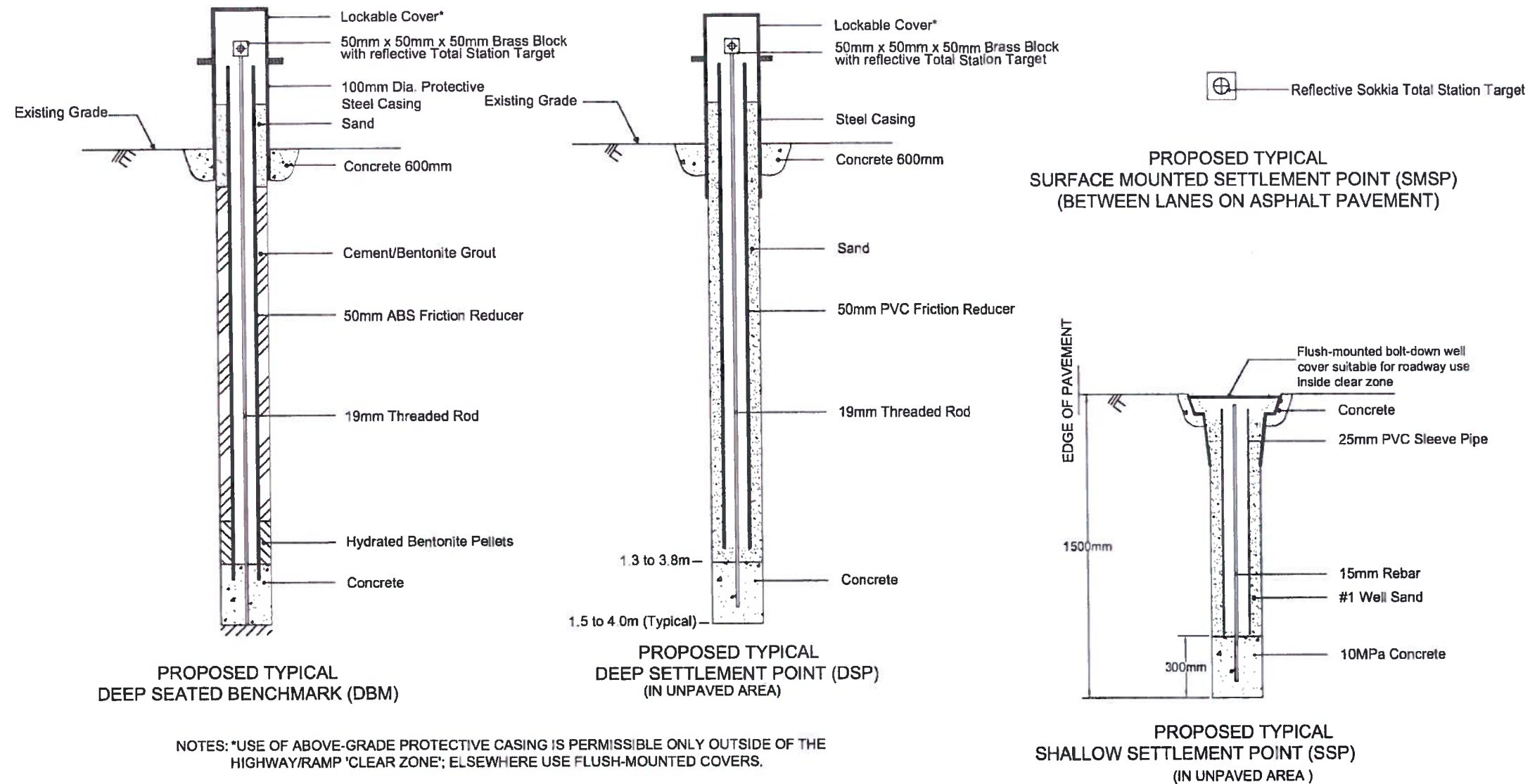
LEGEND

- DBM - Deep Benchmark
- SSP - Shallow Settlement Point
- SMSP - Surface Mounted Settlement Point
- DSP - Deep Settlement Point

Client: Downunder Geotechnical Ltd		Project No.: 1549-110	Drawing No.: 3
Drawn: ZMO	Approved: RM	Title: Preliminary Layout of Ground Monitoring Arrays	
Date: Oct 28, 2013	Scale: As Shown	Project: Geotechnical Investigation - Highway 403 Watermain - Crossing, South of Dundas St. W, Oakville, ON	
Original Size: Tabloid	Rev: N/A	 SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	

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TYPICAL INSTALLATION DETAILS



NOTES: *USE OF ABOVE-GRADE PROTECTIVE CASING IS PERMISSIBLE ONLY OUTSIDE OF THE HIGHWAY/RAMP 'CLEAR ZONE'; ELSEWHERE USE FLUSH-MOUNTED COVERS.

TABLE 1
FREQUENCY AND ACCURACY OF MONITORING

Installation Schedule	Baseline Reading	Monitoring Schedule	Monitoring Duration
At least one week prior to start of tunnelling	Minimum of two (2) sets of readings prior to tunnelling. Accuracy of readings should be 0.5mm or better.	Three (3) times per day including during work stoppages (eg. weekends). Readings may be reduced to twice daily when the tunnel face is greater than 50m away from the monitoring point. Minimum readings are as above within 50m meters.	On completion of tunnelling, monitoring is to be maintained at least once daily for a minimum of two weeks; then twice weekly for a period of one month; then once month for the following six months.
Note: - During each monitoring visit, all monitoring points are to be recorded except where the tunnel face is greater than 50m from the instrument. - The above outline is recommended for all installed monitoring devices including the Deep and Shallow (Surface) Settlements Points.			

NOTES:

1) Deep settlement points are to be founded into the bedrock; but no closer than 3m above the crown of the tunnel.

2) Accurate ground surface elevations and depths to the proposed tunnel obvert are required prior to the installation of any Deep Settlement Points.

3) Deep Seated Benchmarks are to be founded a minimum of 0.3m into the bedrock and located at least 20m away from the tunnel alignment; Confirmed by grinding auger and split spoon refusal on the shale bedrock.

4) Shallow Settlement Points are to be flush with or recessed below the surrounding paved shoulder to protect settlement points and passing traffic from potential damage.

SETTLEMENT CRITERIA:

Definition	Movement
Review Level	≥ 10mm for SSP & DSP ≥ 5mm for SMSP
-Immediately notify MTO & the geotechnical engineer for further assessment; Proceed with caution.	
Alert Level	≥ 15mm for SSP & DSP ≥ 10mm for SMSP
- Halt tunnelling until further assessment is carried out by the MTO & geotechnical engineer; Carry out immediate remedial work to the settlement zone as approved by the MTO.	

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Installation Details of Ground Monitoring Arrays

Geotechnical Investigation - Highway 403 Watermain - Crossing, South of Dundas St. W, Oakville, ON

Scale: NOT TO SCALE
Project No.: 1549-110

Date: Oct. 2013
Drawing No.: 4

PROJECT: Hwy 403 Watermain Crossing CLIENT: DownUnder Geotechnical Ltd PROJECT LOCATION: South of Dundas St. W., Oakville, Ontario DATUM: Geodetic BH LOCATION: See Borehole Location Plan N 4817892.11 E 606443.21	DRILLING DATA Method: Solid Stem Augers/Rock core Diameter: 115mm/63mm Date: Jun/11/2013 REF. NO.: 1549-110 ENCL. NO.: 5
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SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	POCKET PEN (kgf/cm²)	NATURAL UNIT WT (kg/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE							
163.91 0.00	TOPSOIL: 300mm														
163.61 0.30	FILL: silty clay, trace sand, trace topsoil / rootlets, greyish brown to reddish brown, moist, firm to very stiff.		1	SS	8										
162.81 1.10	CLAYEY SILT: trace sand, trace gravel, trace shale fragments, reddish brown, moist, hard. (residual soil)		2	SS	19										
162.11 1.80	QUEENSTON FORMATION: shale interbedded with limestone and siltstone		3	SS	56										
	Refer to Rock Core Log		1	RC											
			2	RC											
			3	RC											
			4	RC											
			5	RC											

SPL SOIL LOG 2DIG 1549-110-JULY30.GPJ SPL.GDT 11/6/13

Continued Next Page


GROUNDWATER ELEVATIONS

GRAPH NOTES

+ 3 x 3 : Numbers refer to Sensitivity ○ = 3% Strain at Failure

Shallow/ Single Installation Deep/Dual Installation

PROJECT: Hwy 403 Watermain Crossing CLIENT: DownUnder Geotechnical Ltd PROJECT LOCATION: South of Dundas St., W., Oakville, Ontario DATUM: Geodetic BH LOCATION: See Borehole Location Plan N 4817892 11 E 606443 21	DRILLING DATA Method: Solid Stem Augers/Rock core Diameter: 115mm/63mm Date: Jun/11/2013 REF. NO.: 1549-110 ENCL NO.: 5
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SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	POCKET PEN (Cp) (kPa)	NATURAL UNIT WT (Mg/m ³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)				
(m)	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa)								WATER CONTENT (%)			
ELEV DEPTH								20	40								60	80	100
152.81	QUEENSTON FORMATION: shale interbedded with limestone and siltstone(Continued)		6	RC															
							153												

SPL SOIL LOG 2D(G) 1549-110-JULY30 GRJ SPL.GDT 11/6/13

GROUNDWATER ELEVATIONS

GRAPH NOTES

+ 3, X 3. Numbers refer to Sensitivity

○ 3% Strain at Failure

Shallow Single Installation Deep/Dual Installation

PROJECT: Hwy 403 Watermain Crossing			DRILLING DATA								REF. NO.: 1549-110					
CLIENT: DownUnder Geotechnical Ltd			Method: Solid Stem Augers/Rock core								ENCL. NO.: 5					
LOCATION: South of Dundas St. W., Oakville, Ontario			Diameter: 115mm/63mm													
DATUM: Geodetic			Date: 6/11/2013													
BH LOCATION: See Borehole Location Plan N 4817892 11 E 606443.21																
(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE		TOTAL CORE RECOVERY (%)	SOLID CORE RECOVERY (%)	HARD LAYER (%)	RQD (%)	FRACTURE INDEX (per 0.3 m)	DISCONTINUITIES AND WEATHERING	HYDRAULIC CONDUCTIVITY (cm/sec)	POINT LOAD TEST UCS AXIAL (MPa)*	POINT LOAD TEST UCS DIAMETRAL (MPa)*	UNIAxIAL COMPRESSION (MPa)	DENSITY (g/cc) E (GPa)	
			NUMBER	SIZE												
162.11 1.80																
161.04 2.87	QUEENSTON FORMATION Highly weathered to fresh, laminated to thinly bedded, reddish brown, weak to medium strong, SHALE (80-92%), thinly laminated to medium bedded with slightly weathered to fresh, light grey, medium strong to very strong SILTSTONE and LIMESTONE (8-20%) Siltstone and limestone (hard) layers generally less than 50mm thick except at the following depths: Depth Thickness 4.70m 100mm 5.77m 110mm 7.11m 100mm 7.80m 60mm 8.66m 100mm 8.85m 80mm 9.19m 70mm 9.50m 60mm 9.58m 80mm 10.69m 60mm		1	HQ	83	38	8	0	22	Highly weathered to slightly weathered Soft layers (highly weathered): 2.87m-3.07m (Fragmented zone: 3.07m-3.18m)		58	30			
160.43 3.48									5	Slightly weathered to fresh Fracture 4.80m-4.88m Fragmented zone: 4.85m-4.95m						
			2	HQ	100	93	15	62	3			62	29			
									4							
									2							
158.91 5.00									12							
									7	Slightly weathered to fresh						
			3	HQ	100	95	13	77	2	Soft layers (moderately weathered): 5.68m-5.69m Fracture 5.72m-5.79m Fragmented zone: 5.16m-5.23m		65	9			
									5							
									0			70	44	36.5	2.6	10.042
157.38 6.53									2							
									3	Slightly weathered to fresh						
			4	HQ	100	92	15	92	3	Soft layers (moderately weathered): 7.38m-7.40m Fracture 6.53m-7.21m		57	30			
									2							
155.86 8.05									3							
									2	Slightly weathered to fresh						
			5	HQ	100	93	20	87	2	Soft layers (moderately to slightly weathered): 8.62m-8.64m; 8.75m-8.78m 9.46m-9.47m						
									3			170	79			
154.33 9.58									1							
									2	Slightly weathered to fresh						
			6	HQ	100	93	15	93	0			63	47			
									3							
152.81 11.10	END OF BOREHOLE								1							

SPL ROCK CORE LOG 1549-110-JULY30 GPJ SPL GDT 11/8/13

SPL SOIL LOG 2DIG 1549-110-JULY30 GPJ SPL GDT 11/6/13

○ $\epsilon = 3\%$ Strain at Failure

Shallow/Single Installation   Deep/Dual Installation  

PROJECT: Hwy 403 Watermain Crossing				DRILLING DATA			
CLIENT: DownUnder Geotechnical Ltd				Method: Solid Stem Augers/Rock core			
PROJECT LOCATION: South of Dundas St. W, Oakville, Ontario				Diameter: 115mm/63mm		REF. NO.: 1549-110	
DATUM: Geodetic				Date: Jun/17/2013		ENCL NO.: 7	
BH LOCATION: See Borehole Location Plan N 4817907 44 E 606451.22							
SOIL PROFILE			SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT		
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION
154.18	Augering without sampling to 1.68m						164
162.50	QUEENSTON FORMATION: shale interbedded with limestone and siltstone Refer to Rock Core Log		1	RC			163
1.68			2	RC			162
			3	RC			161
			4	RC			160
			5	RC			159
			6	RC			158
154.73							157
9.45	END OF BOREHOLE						156
	Note: 1) Borehole was drilled 2.5m north of BH13-11.						155

SPL SOIL LOG 2DIG 1549-110-JULY30 GPJ SPL GDT 11/6/13

Continued Next Page
GROUNDWATER ELEVATIONS

GRAPH NOTES

+ 3, x 3. Numbers refer to Sensitivity

○ 6-3% Strain at Failure

Shallow Single Installation  Deep/Dual Installation 

PROJECT: Hwy 403 Watermain Crossing					DRILLING DATA							
CLIENT: DownUnder Geotechnical Ltd					Method: Solid Stem Augers/Rock core							
PROJECT LOCATION: South of Dundas St W, Oakville, Ontario					Diameter: 115mm/63mm		REF NO: 1549-110					
DATUM: Geodetic					Date: Jun/17/2013		ENCL NO: 7					
BH LOCATION: See Borehole Location Plan N 4817907.44 E 606451.22												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		POCKET PEN (C _u) (kPa)	NATURAL UNIT WT (Mg/m ³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT NUMBER	TYPE	"N" BLOWS 0.3m	GROUND WATER CONDITIONS	ELEVATION	20 40 60 80 100	W _p — W — W _L	WATER CONTENT (%)			
	2) Packer tests conducted from 3.40m to 9.45m in borehole 13-11A											GR SA SI CL

SPL SOIL LOG 2013G 1549-110 JUL Y30 GPJ SPL GDT 11/6/13

GROUNDWATER ELEVATIONS

GRAPH NOTES

+ 3 × 3 Numbers refer to Sensitivity

○ @ -3% Strain at Failure

Shallow Single Installation  Deep/Dual Installation 

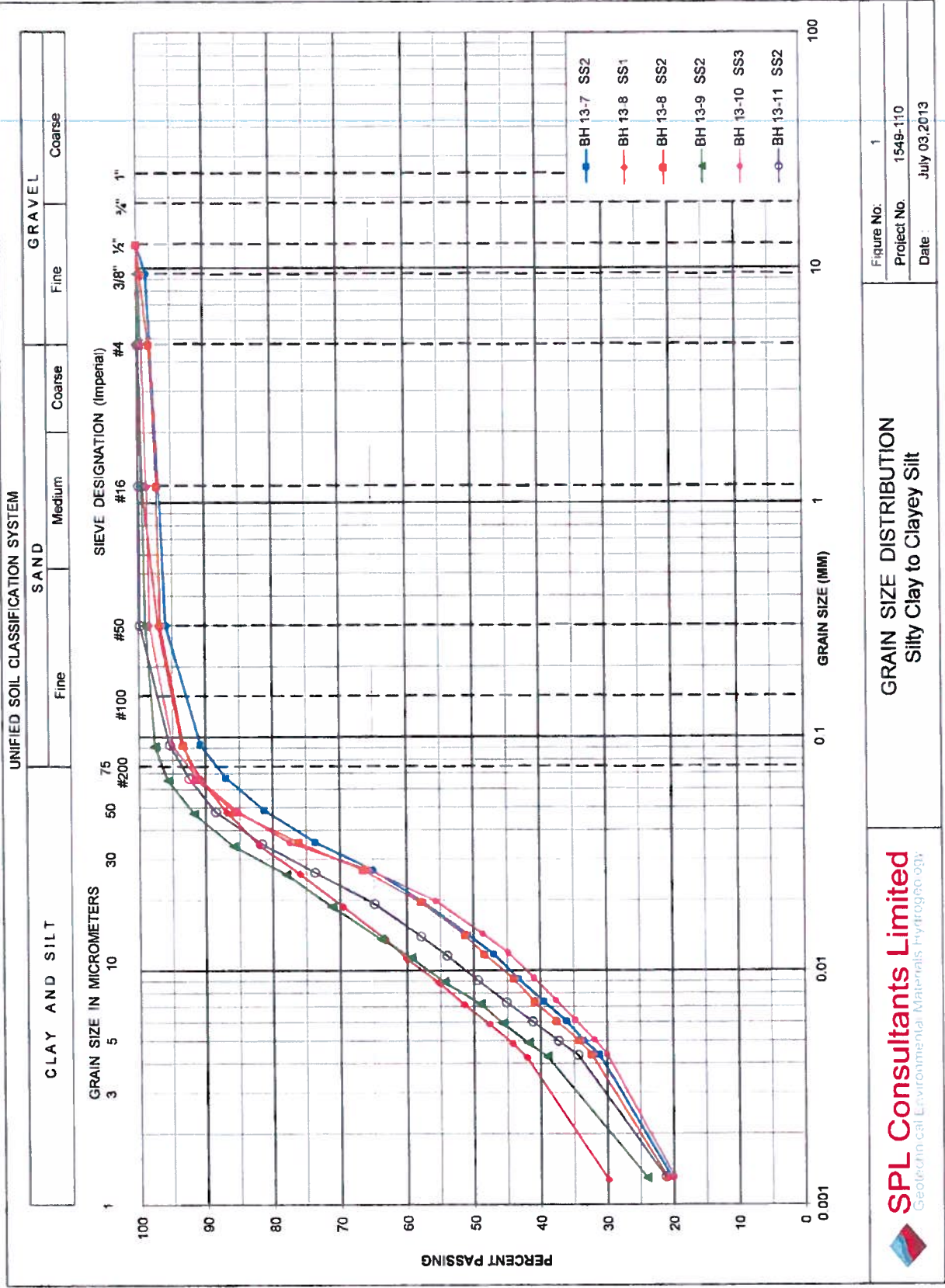
E = Modulus of Elasticity

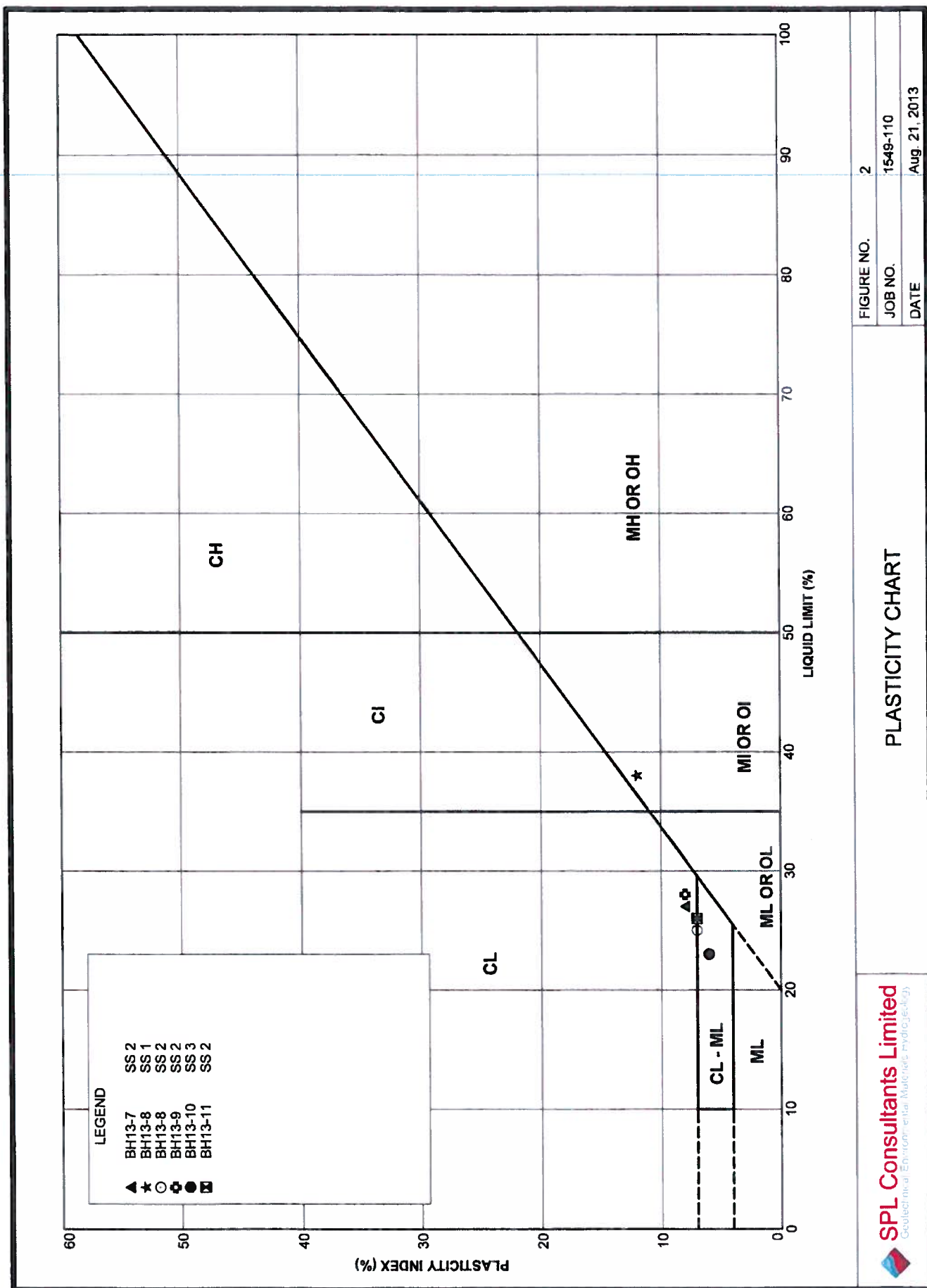
RECORD OF BOREHOLE No MW-7										Project No.: D13103		
Project: Proposed Watermain, Ninth Line to Bristol Circle										Drilling Date: 20/06/2013		
Location: Oakville, Ontario										Drilling Method: Solid Stem Augering		
Client: Delcan Corporation										Hole Diameter: 110mm		
										DATUM: Geodetic		
① POINT LOAD INDEX <small>(s(50) MPa)</small> SHEAR STRENGTH kPa <small>▲ Field Vane</small> 50 100 150 200	UNIT WEIGHT γ kN/m ³	PLASTIC LIMIT <small>W_p</small> NATURAL MOISTURE CONTENT <small>W</small> LIQUID LIMIT <small>W_L</small> WATER CONTENT (%) 10 20 30 40	SAMPLES				SOIL PROFILE		ELEVATION SCALE	DEPTH (m)	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
			NUMBER	TYPE	"N" VALUES	RECOVERY	STRAT PLOT	SOIL DESCRIPTION				
GROUND SURFACE ELEVATION 163.6 m												
			1	SS	11	[Pattern]	[Pattern]	20cm TOPSOIL brown SILTY CLAY damp	163	1	[Pattern]	
			2	SS	32	[Pattern]	[Pattern]	brownish red completely weathered SHALE BEDROCK		162	[Pattern]	
			3	SS	50/ 10cm	[Pattern]	[Pattern]	damp		2	[Pattern]	
			4	SS	50/ 5cm	[Pattern]	[Pattern]	weathered SHALE BEDROCK occasional grey Limestone layers		161	[Pattern]	
			5	SS	50/ 5cm	[Pattern]	[Pattern]	END OF BOREHOLE GROUNDWATER IN OPEN BOREHOLE On completion: none June 21: 2.4m June 27: 2.4m July 5: 2.6m July 26: 2.7m		3	[Pattern]	

Appendix C

Grain Size Distribution Curves (Figure 1)

Plasticity Chart (Figure 2)





Appendix D

Photographs of Rock Core

Results of Unconfined Compression Failure Tests (Table D-1)

Results of Point Load Index Strength Tests (Table D-2)

Results of Packer Water Pressure Acceptance Testing in Boreholes (Table D-3)

Results of Slake Durability Test Results

Results of Cerchar Abrasivity Test Results

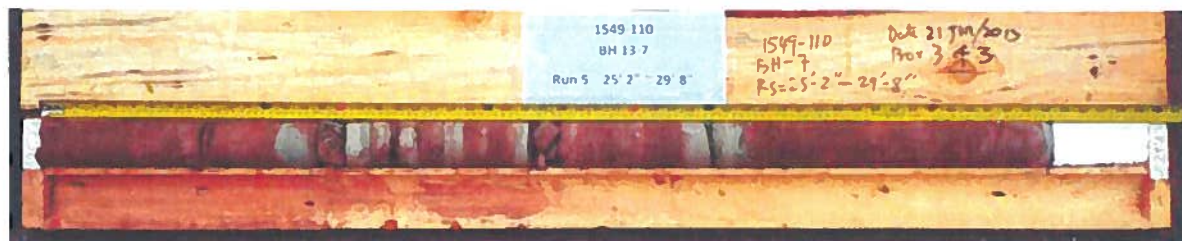
BH13-7A



Rock Core Photo: Run 1, Run 2, Run 3 and Run 4

Run 1: 5' 5" – 10' 5" (1.65m – 3.18m) Run 2: 10' 5" – 15' 4" (3.18m – 4.68m)

Run 3: 15' 4" – 20' 4" (4.68m – 6.20m) Run 4: 20' 4" – 25' 2" (6.20m – 7.67m)



Rock Core Photo: Run 5

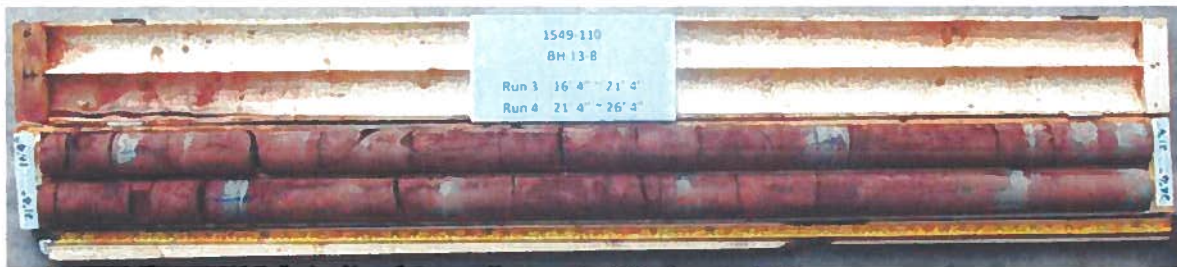
Run 5: 25' 2" – 29' 8" (7.67m – 9.04m)

BH13-8



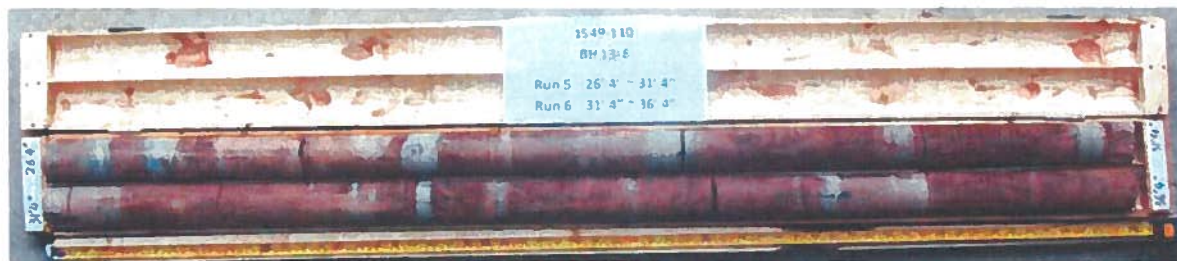
Rock Core Photo: Run 1 and Run 2

Run 1: 9' 4" – 11' 4" (2.84m – 3.45m) Run 2: 11' 4" – 16' 4" (3.45m – 4.98m)



Rock Core Photo: Run 3 and Run 4

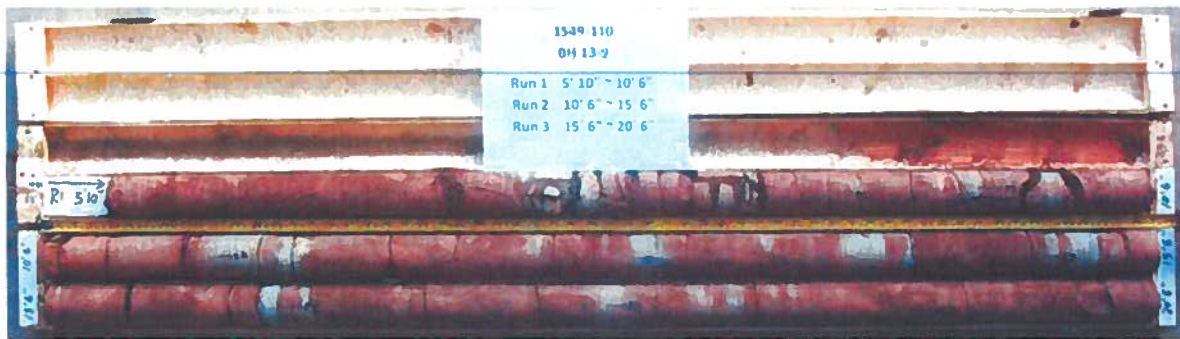
Run 3: 16' 4" – 21' 4" (4.98m – 6.50m) Run 4: 21' 4" – 26' 4" (6.50m – 8.03m)



Rock Core Photo: Run 5 and Run 6

Run 5: 26' 4" – 31' 4" (8.03m – 9.55m) Run 6: 31' 4" – 36' 4" (9.55m – 11.07m)

BH13-9



Rock Core Photo: Run 1, Run 2 and Run 3

Run 1: 5' 10" – 10' 6" (1.78m – 3.20m) Run 2: 10' 6" – 15' 6" (3.20m – 4.72m)

Run 3: 15' 6" – 20' 6" (4.72m – 6.25m)



Rock Core Photo: Run 4, Run 5, Run6 and Run 7

Run 4: 20' 6" – 25' 6" (6.25m – 7.77m) Run 5: 25' 6" – 30' 6" (7.77m – 9.30m)

Run 6: 30' 6" – 35' 4" (9.30m – 10.77m) Run 7: 35' 4" – 40' 4" (10.77m – 12.29m)

BH13-10



Rock Core Photo: Run 1 and Run 2

Run 1: 9' 5" - 11' 5" (2.87m - 3.48m) Run 2: 11' 5" - 16' 5" (3.48m - 5.00m)



Rock Core Photo: Run 3 and Run 4

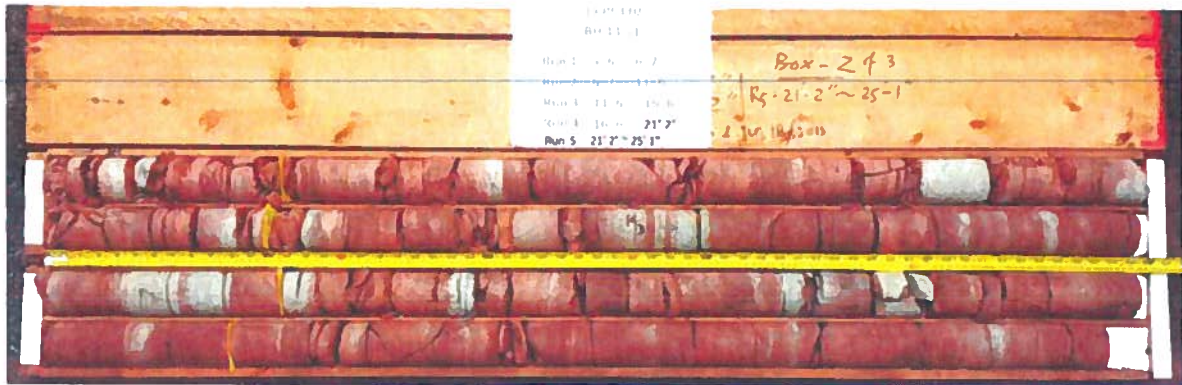
Run 3: 16' 5" - 21' 5" (5.00m - 6.53m) Run 4: 21' 5" - 26' 5" (6.53m - 8.05m)



Rock Core Photo: Run 5 and Run 6

Run 5: 26' 5" - 31' 5" (8.05m - 9.58m) Run 6: 31' 5" - 36' 5" (9.58m - 11.10m)

BH13-11A



Rock Core Photo: Run 1, Run 2, Run 3, Run 4 and Run 5

Run 1: 5' 6" – 6' 7" (1.68m – 2.01m) Run 2: 6' 7" – 11' 6" (2.01m – 3.51m)

Run 3: 11' 6" – 16' 6" (3.51m – 5.03m) Run 4: 16' 6" – 21' 2" (5.03m – 6.45m)

Run 5: 21' 2" – 25' 1" (6.45m – 7.65m)



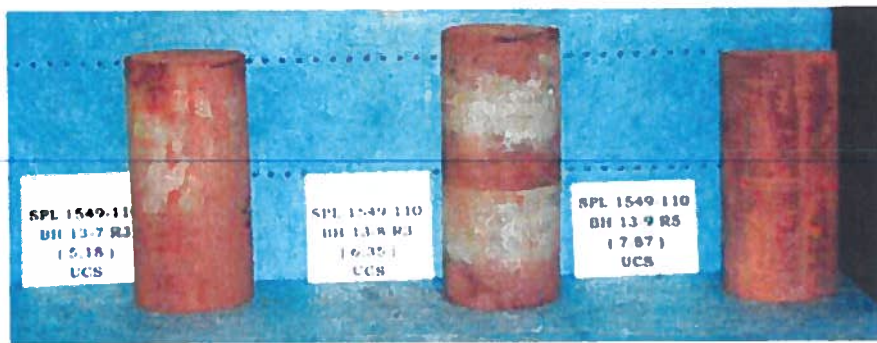
Rock Core Photo: Run 5 and Run 6

Run 5: 25' 1" – 26' 0" (7.65m – 7.92m) Run 6: 26' 0" – 31' 0" (7.92m – 9.45m)

Table D-1 : Unconfined Compression Failure Test Results
(by Rock Mechanics Laboratory of Queen's University)

Borehole No.	Sample Depth/Elevation (m)	Rock Type	Unconfined Compressive Strength (MPa)	Young's Modulus (Gpa)	Poisson's Ratio	Bulk Density(g/cm ³)
BH 13-7A	5.18/158.42	Limy Shale	24.1	4.659	0.12	2.58
BH 13-8	6.35/157.36	Limy Shale	19.3	5.013	0.14	2.56
BH 13-9	7.87/155.66	Limy Shale	24.3	6.219	0.12	2.62
BH 13-10	6.27/157.63	Limy Shale	36.5	10.042	0.12	2.60
BH 13-11A	8.20/155.98	Limy Shale	21.3	4.331	0.17	2.48

Pre-Test Samples



Post-Test Samples

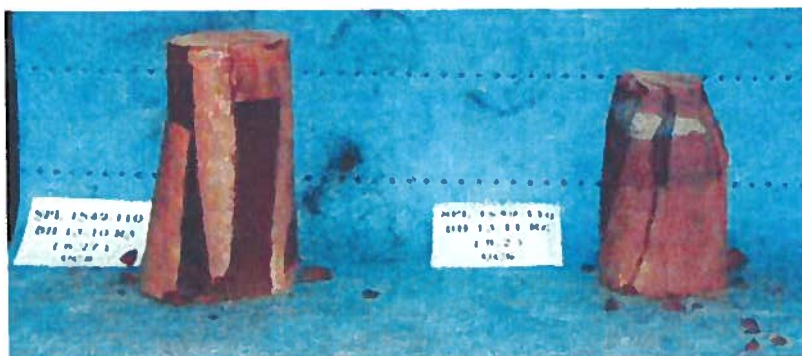
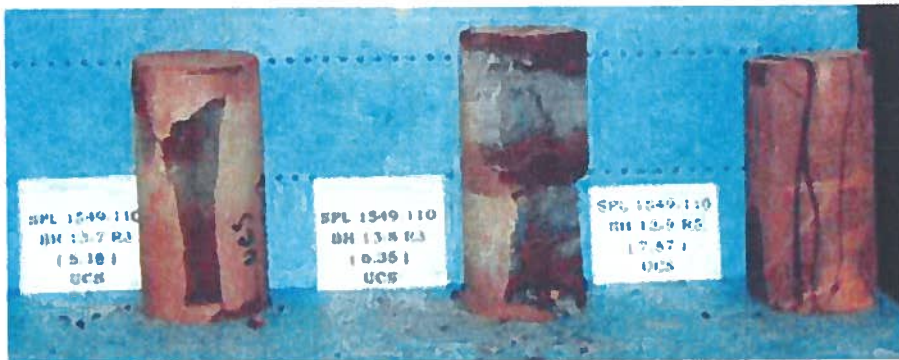


Table D-2: Results of Point Load Index Strength Tests

BH No.	Depth (m)	Elevation (m)	Rock Type	Point Load Index $I_{s(50)}$ (MPa)		Approximate Uniaxial Compressive Strength (MPa)*
				Diametral	Axial	
BH13-7A	3.0	160.6	Limestone	4.4		106
	3.0	160.6	Limestone		6.4	153
	4.5	159.2	Siltstone/Shale		1.6	40
	4.6	159.0	Siltstone/Shale		1.5	36
	4.9	158.7	Shale	0.2		5
	4.9	158.7	Shale		1.3	31
	5.4	158.2	Shale	0.8		18
	5.4	158.2	Shale		2.0	49
	7.3	156.3	Shale	0.6		15
	7.3	156.3	Shale		1.8	43
	7.8	155.8	Shale	0.4		9
	7.8	155.8	Shale		1.5	36
	8.7	154.9	Shale	0.4		11
	8.7	154.9	Shale		1.1	27

* Approximate uniaxial compressive strength based on the relationship $UCS \approx 24 I_{s(50)} \text{ Mpa}$.

Table D-2: Results of Point Load Index Strength Tests

BH No.	Depth (m)	Elevation (m)	Rock Type	Point Load Index $I_{s(50)}$ (MPa)		Approximate Uniaxial Compressive Strength (MPa)*
				Diametral	Axial	
BH13-8	3.2	160.4	Siltstone/Limestone	2.0		48
	3.2	160.4	Siltstone/Limestone		4.1	98
	3.7	159.9	Siltstone/Shale	1.3		30
	3.7	159.9	Siltstone/Shale		1.5	35
	5.3	158.4	Siltstone/Shale	2.1		51
	5.3	158.4	Siltstone/Shale		2.4	58
	6.3	157.4	Siltstone/Shale	0.6		15
	6.3	157.4	Siltstone/Shale		2.7	66
	6.8	156.9	Shale	0.6		15
	6.8	156.9	Shale		2.1	50
	7.5	156.3	Shale	1.1		26
	7.5	156.3	Shale		2.5	59
	9.1	154.7	Siltstone/Shale	1.2		29
	9.1	154.7	Siltstone/Shale		2.4	59
	10.0	153.9	Siltstone/Shale	1.1		27
	10.0	153.9	Siltstone/Shale		2.1	49
	10.7	153.1	Siltstone/Shale	1.1		27
	10.7	153.1	Siltstone/Shale		2.7	65

* Approximate uniaxial compressive strength based on the relationship $UCS \approx 24 I_{s(50)}$ Mpa.

Table D-2: Results of Point Load Index Strength Tests

BH No.	Depth (m)	Elevation (m)	Rock Type	Point Load Index $I_{s(50)}$ (MPa)		Approximate Uniaxial Compressive Strength (MPa)*
				Diametral	Axial	
BH13-9	2.7	160.9	Shale	0.6		14
	2.7	160.9	Shale		0.7	16
	3.4	160.1	Limestone/Shale	1.4		35
	3.4	160.1	Limestone/Shale		3.8	91
	4.6	159.0	Shale	0.1		3 **
	4.6	159.0	Shale		1.4	33
	5.6	158.0	Siltstone/Shale	0.6		14
	5.6	158.0	Siltstone/Shale		2.5	61
	6.9	156.6	Shale	0.1		2 **
	6.9	156.6	Shale		2.0	48
	8.7	154.9	Limestone/Shale	0.8		20
	8.7	154.9	Limestone/Shale		3.0	72
	9.4	154.2	Shale	1.5		36
	9.4	154.2	Shale		1.7	42
	11.3	152.2	Siltstone/Shale	1.3		30
	11.3	152.2	Siltstone/Shale		2.3	56

* Approximate uniaxial compressive strength based on the relationship $UCS \approx 24 I_{s(50)}$ Mpa.

** Failure along bedding plane parting, UCS is therefore not considered to be representative.

Table D-2: Results of Point Load Index Strength Tests

BH No.	Depth (m)	Elevation (m)	Rock Type	Point Load Index $I_{s(50)}$ (MPa)		Approximate Uniaxial Compressive Strength (MPa)*
				Diametral	Axial	
BH13-10	3.4	160.5	Siltstone/Shale	1.3		30
	3.4	160.5	Siltstone/Shale		2.4	58
	4.5	159.5	limestone/Shale	1.2		29
	4.5	159.5	limestone/Shale		2.6	62
	5.5	158.4	limestone/Shale	0.4		9
	5.5	158.4	limestone/Shale		3.1	75
	6.2	157.7	limestone/Shale	1.8		44
	6.2	157.7	limestone/Shale		2.5	61
	7.4	156.5	Shale	1.3		30
	7.4	156.5	Shale		2.4	57
	9.2	154.7	Limestone	3.3		79
	9.2	154.7	Limestone		7.1	170
	9.9	154.0	Siltstone/Shale	1.9		47
	9.9	154.0	Siltstone/Shale		2.6	63

* Approximate uniaxial compressive strength based on the relationship $UCS \approx 24 I_{s(50)}$ Mpa.

Table D-2: Results of Point Load Index Strength Tests

BH No.	Depth (m)	Elevation (m)	Rock Type	Point Load Index $I_{s(50)}$ (MPa)		Approximate Uniaxial Compressive Strength (MPa)*
				Diametral	Axial	
BH13-11A	2.9	161.3	limestone	4.3		103
	2.9	161.3	limestone		2.7	65
	3.7	160.5	Shale	0.1		3 **
	3.7	160.5	Shale		1.8	44
	4.1	160.1	Shale	0.1		3 **
	4.1	160.1	Shale		1.9	45
	4.3	159.8	Shale	0.1		2 **
	4.3	159.8	Shale		0.8	19
	4.8	159.4	Siltstone/Shale	0.1		3 **
	4.8	159.4	Siltstone		4.0	96
	5.6	158.6	Shale	0.7		17
	5.6	158.6	Shale		1.7	40
	6.4	157.8	Shale	0.1		3 **
	6.4	157.8	Shale		1.5	35
	7.1	157.1	Shale	0.3		8
	7.1	157.1	Shale		1.5	37
	8.9	155.3	limestone	1.6		38
	8.9	155.3	limestone		6.1	148

* Approximate uniaxial compressive strength based on the relationship $UCS \approx 24 I_{s(50)}$ Mpa

** Failure along bedding plane parting, UCS is therefore not considered to be representative.

Table D-3: Results of Packer Water Pressure Acceptance Testing in Borehole

Borehole Number	Run No.	Packer Interval Depth BGL (Elevation)	Water Pressure (kPa)	Secondary Hydraulic Conductivity (m/s)
BH13-7A	2 and 3	3.15 - 6.20 (157.46 - 160.51)	115	No take
			460	2.1×10^{-7}
BH13-7A	4 and 5	6.20 - 9.04 (154.62 - 157.46)	144	No take
			488	2.3×10^{-9}
			833	3.7×10^{-7}
BH13-11A	3, 4 and 5	3.40-6.45 (157.73 - 160.78)	117	No take
			186	6.2×10^{-7}
BH13-11A	5 and 6	6.40 - 9.45 (154.73 - 157.78)	147	3.9×10^{-7}
			491	3.6×10^{-7}

Slake Durability Test Results (ASTM D 4644-87)

Project No.: 1549-110

Project: Highway 403 Watermain – Crossing, South of Dundas St. W, Oakville ON

Evaluator: B.W.
Testing Dates: Jul.15-Jul.18/2013

Sample No.	Sample Description	Slake Durability Index - 1st Cycle [Id(1)]	Slake Durability Index - 2nd Cycle [Id(2)]	Description of Retained sample after 2nd Cycle
BH13-8 SL-1 Run-3(16' 4" ~17' 4")	reddish brown shale/limy shale	92.3%	78.8%	Type 2
BH13-9 SL-2 Run-4(22' 0" ~22' 9")	reddish brown shale/limy shale	88.1%	72.0%	Type 2
BH13-10 SL-3 Run-2(13' 6" ~14' 7")	reddish brown shale/limy shale	90.8%	80.0%	Type 2

Sample description after 2nd cycle (ASTM D 4644-87):

Type 1 - retained pieces remain virtually unchanged

Type 2 - retained materials consist of large and small pieces

Type 3 - retained material is exclusively small fragments



SPL Consultants Limited

Geotechnical Environmental Materials Hydrogeology

1549-110

BH13-8

16' 4" ~ 17' 4"

SL-1(Run-3)

Before Test



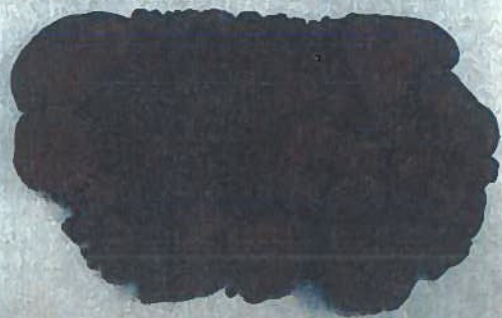
1549-110

BH13-8

16' 4" ~ 17' 4"

SL-1(Run-3)

After 2nd Cycle



Slake Durability Test Sample (before test and after test): SL-1

1549-110

BH13-9

22' 0" ~ 22' 9"

SL-2(Run-4)

Before Test



1549-110

BH13-9

22' 0" ~ 22' 9"

SL-2(Run-4)

After 2nd Cycle



Slake Durability Test Sample (before test and after test): SL-2

1549-110

BH13-10

13' 6" ~ 14' 7"

SL-3(Run-2)

Before Test



1549-110

BH13-10

13' 6" ~ 14' 7"

SL-3(Run-2)

After 2nd Cycle



Slake Durability Test Sample (before test and after test): SL-3

CERCHAR Abrasiveness Test Results on Rock Core Samples

Project No. : 1549-110
 Project : Highway 403 Watermain – Crossing , South of Dundas St. W, Oakville ON
 Evaluator: B.W
 Testing Dates: Jul.15-Jul.17/2013

Sample I.D.	Core Sample Description	Surface Material Description	Surface Prep.	Moisture Condition	CAI** (mm)					Average CAI* (mm)	Abrasiveness Classification
					Line 1	Line 2	Line 3	Line 4	Line 5		
BH13-7A (CS-1) 23'5"	limestone/shale	limestone	S	A	1.82	1.77	1.90	1.68	1.58	1.8	Medium
BH13-8(CS-2) 23'8"	shale	shale	S	A	1.55	1.48	1.46	1.72	1.61	1.6	Medium
BH13-9 (CS-3) 27'5"	limestone/shale	limestone	S	A	1.59	1.64	1.91	1.79	1.82	1.8	Medium
BH13-10 (CS-4) 19'6"	shale	shale	S	A	1.64	1.61	1.40	1.57	1.45	1.5	Medium
BH13-11A (CS-5) 20'10"	limy shale	limy shale	S	A	1.78	1.71	1.71	1.51	1.63	1.7	Medium

Notes: Test Apparatus - GCTS Model RAA-100 Rock Abrasivity Apparatus
 Stylus hardness = Rockwell HRC 55
 Lines 1 through 3 are parallel, Lines 4 and 5 are transverse
 S - indicates saw-cut core surface N - indicates natural core surface
 A - Indicates air-dried core sample F - Indicates field moisture condition O - indicates oven dried moisture condition
 ** CAI = $(0.99CAI_s) + 0.48$, **CAI measured used GCTS RSS software

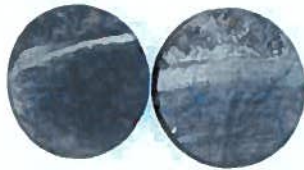
Classification of CERCHAR Abrasiveness Index Test Results (ASTM D7625-10)

Classification	Average CAI *
Very low abrasiveness	0.30-0.50
Low abrasiveness	0.50-1.00
Medium Abrasiveness	1.00-2.00
High Abrasiveness	2.00-4.00
Extreme Abrasiveness	4.00-6.00
Quartzitic	6.0-7.0

* for Rockwell Hardness = C55

1549-110
BH13-7A

23' 5"
CS-1(Run-4)
before test



1549-110
BH13-7A

23' 5"
CS-1(Run-4)
after test



CERCHAR Abrasiveness Test: CS-1 (23' 5")

1549-110
BH13-8

23' 8"
CS-2(Run-4)
before test



1549-110
BH13-8

23' 8"
CS-2(Run-4)
after test



CERCHAR Abrasiveness Test: CS-2 (23' 8")

1549-110
BH13-9
27' 5"
CS-3(Run-5)
before test

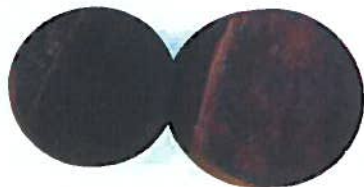


1549-110
BH13-9
27' 5"
CS-3(Run-5)
after test



CERCHAR Abrasiveness Test: CS-3 (27' 5")

1549-110
BH13-10
19' 6"
CS-4(Run-3)
before test



1549-110
BH13-10
19' 6"
CS-4(Run-3)
after test



CERCHAR Abrasiveness Test: CS-4 (19' 6")

1549-110
BH13-11A

20' 10"
CS-5(Run-4)
before test



1549-110
BH13-11A

20' 10"
CS-5(Run-4)
after test



CERCHAR Abrasiveness Test: CS-5 (20'10")

Appendix E

Tunnelling Options in Queenston Shale Bedrock

K

Option	Construction Method	Application Range		Limitation	Cost Comparison
		Length	Diameter		
1	Jack and Bore / Auger Boring	Up to ~ 60m	0.1m and 1.1m	<p>Limited length estimated due to large jacking forces in the shale with limestone layers</p> <p>Difficulty in penetrating thick limestone layers</p> <p>Due to rock squeeze liner must be designed to withstand horizontal pressures. Grout between casing and pipe must be designed to limit load transfer (ie. weak strength grout)</p>	Low
2	Pipe Jacking w/ Manual Excavation	Up to ~60m	1.5m	<p>High capital cost and setup</p> <p>Requires groundwater control and discharge due to workers inside the tunnel</p> <p>Specialized operation requiring good operator skill and experience</p> <p>Limited length estimated due to large jacking forces in the shale with limestone layers</p> <p>Possible refusal in thick limestone layers</p> <p>Expensive corrective action if mis-aligned</p> <p>Due to rock squeeze liner must be designed to withstand horizontal pressures. Grout between casing and pipe must be designed to limit load transfer (ie. weak strength grout)</p>	Medium to High

Table E-1: SUMMARY OF TUNNELLING OPTIONS IN QUEENSTON SHALE BEDROCK

Option	Construction Method	Application Range		Description	Temporary Support	Permanent Lining	Alignment Control	Advantage	Limitation	Cost Comparison
		Length	Diameter							
1	Jack and Bore / Auger Boring	Up to ~ 60m	0.1 to 1.5m	A horizontal borehole is advanced from a drive shaft to an exit shaft by the use of a continuous flight auger. Spoil is transported back to the drive shaft by rotating the auger inside a steel casing. The casing is jacked in placed simultaneously during the augering operation.	Provided by the steel casing during the jack and bore operations	After the steel casing installation, the pipe is installed inside the casing and the gap between the casing and the pipe is grouted.	By hydraulic jacks in shafts pushing steel casing Not very good control in mixed face conditions	Technique commonly used locally Skilled labour, equipment and contractor available locally Relatively lower cost	Limited length estimated due to large jacking forces in the shale with limestone layers Difficulty in penetrating thick limestone layers Due to rock squeeze liner must be designed to withstand horizontal pressures. Grout between casing and pipe must be designed to limit load transfer (ie. weak strength grout)	Low
2	Pipe Jacking w/ Manual Excavation	Up to ~60m	1.5 to 3.7m	A prefabricated pipe is installed through the ground from a drive shaft to an exit shaft. The pipe is pushed by jacks located in the drive shaft and the jacking force is transmitted through the pipe to the face of the excavation. Excavation of the face is carried out by manual methods.	Not required	Pipe installed during the jacking operations (concrete pipe is not recommended due to rock squeeze), or the pipe can be installed within the larger tunnel lining and the gap grouted.	Very tight alignment and grade tolerance Fair control due to rock conditions	Local contractors available	High capital cost and setup Requires groundwater control and discharge due to workers inside the tunnel Specialized operation requiring good operator skill and experience Limited length estimated due to large jacking forces in the shale with limestone layers Possible refusal in thick limestone layers Expensive corrective action if mis-aligned Due to rock squeeze liner must be designed to withstand horizontal pressures. Grout between casing and pipe must be designed to limit load transfer (ie. weak strength grout)	Medium to High

Table E-1: SUMMARY OF TUNNELLING OPTIONS IN QUEENSTON SHALE BEDROCK

Option	Construction Method	Application Range		Description	Temporary Support	Permanent Lining	Alignment Control	Advantage	Limitation	Cost Comparison
		Length	Diameter							
3	Horizontal Directional Drilling	Up to 1,800m	0.1 to 1.2m	A curved horizontal borehole is advanced using a rotary cutting head with drilling fluid to stabilize the bore. The borehole is advanced to the exit shaft and then over reamed to produce a tunnel about 1.5 times larger than the permanent pipe. The gap between the pipe and rock face is filled with pressurized grout.	Pressurized grout or HDPE/steel casing with pressurized grout.	Pipe installed in pressurized grout, or within casing.	Alignment controlled by instrumented drill casing. Fair control	Shaft construction is carried out after HDD completed Shallower shaft required than other methods No groundwater control required during HDD Fast construction schedule Over reamed hole addresses rock squeezing concerns. Horizontal rock pressures are limited by designing the grout to limit load transfer	Size of drill area for grout mixing quite large HDD drill shaft must be tracked in front of the drill to monitor alignment and grade	Medium
4	Microtunnelling	Up to 200m (with internal jacking stations)	0.6 to 3.5m	A tunnel is advanced using a MicroTunnel Boring Machine (MTBM). As the TBM advances temporary or permanent support is installed as spoils are removed.	Jacking Pipe.	Pipe installed during the tunnelling operations, or the pipe can be installed within the larger tunnel lining and the gap grouted.	Can tolerate some misalignment Good control	Method can be executed with any ground condition Can excavate thick limestone layers, assuming proper rock cutter head provided Rock squeeze can be mitigated by overbreaking the rock to allow a gap between the jacking pipe and the rock	High capital cost and setup Specialized operation requiring good operator skill and experience Requires large jacking frame to deal with jacking forces If overbreak is not carried out , the liner must be designed to withstand horizontal pressures to allow for rock squeeze Potential for higher settlements due to overbreak	High
5	Tunnelling by Hand Mining	none	1.5m+	The tunnel is advanced using manual methods or small excavation equipment. Temporary ground support is installed as the tunnel is advanced. Groundwater control is required to minimize water leakage into the tunnel. Workers are required inside the tunnel to carry out excavation and/or spoil removal.	Temporary support with steel ribs with wood lagging, or steel/concrete segmental liner.	Pipe installed within the temporary lining. Gap between is grouted.	Good control by surveying.	Skilled labour, equipment and contractor available locally Lower capital cost and setup cost	Requires groundwater control and disposal Extra effort required where limestone layers are encountered Longer construction schedule Due to rock squeeze liner must be designed to withstand horizontal pressures. Grout between casing and pipe must be designed to limit load transfer	Medium

Appendix F

Corrosivity Test Results

CLIENT NAME: SPL CONSULTANTS
6221 HIGHWAY 7 WEST UNIT 16
VAUGHAN, ON L4H0K8
(905) 856-0065

ATTENTION TO: Nirogini Nalliah

PROJECT NO: 1549-110

AGAT WORK ORDER: 13T731429

SOIL ANALYSIS REVIEWED BY: Anthony Dapaah, PhD (Chem), Inorganic Lab Manager

DATE REPORTED: Jul 09, 2013

PAGES (INCLUDING COVER): 4

VERSION*: 2

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

***NOTES**

VERSION 2 Sample IDs revised on July 9th, 2013

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

AGAT Laboratories (V2)

Member of: Association of Professional Engineers, Geologists and Geophysicists of Alberta (APEGGA)
Western Enviro-Agricultural Laboratory Association (WEALA)
Environmental Services Association of Alberta (ESAA)

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Results relate only to the items tested and to all the items tested



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 13T731429

PROJECT NO: 1549-110

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905) 712-5100
FAX (905) 712-5122
http://www.agatlabs.com

CLIENT NAME: SPL CONSULTANTS

ATTENTION TO: Nirogini Nalliah

Corrosivity Package										
DATE RECEIVED: 2013-07-02					DATE REPORTED: 2013-07-09					
Parameter	Unit	G / S: A	G / S: B	G / S: C	G / S: D	BH13-10 Run		BH13-9, Run 5		BH13-8, Run
						SAMPLE DESCRIPTION: SAMPLE TYPE:	3(top) Rock	6/28/2013 Rock	2(bottom) Rock	
						DATE SAMPLED:	6/28/2013	4527698	4527699	
						RDL	4507857			
						RDL	0.01	0.02	0.02	0.02
Sulphide*	%									
Chloride (2:1)	µg/g	NA	NA	NA	NA		20	20	30	45
Sulphate (2:1)	µg/g						20	38	48	121
pH (2:1)	pH Units					N/A	9.39	9.56	9.56	9.94
Electrical Conductivity (2:1)	mS/cm	0.7	1.4	0.7	1.4	0.005	0.229[<A]	0.175[<A]	0.238[<A]	0.238[<A]
Resistivity (2:1)	ohm.cm					1	4370	5710	4200	4200
Redox Potential (2:1)	mV					5	177	189	200	200

Comments: RDL - Reported Detection Limit. G / S - Guideline / Standard. A Refers to T2(RPI) - Current, B Refers to T2(ICC) - Current, C Refers to T3(RPI) - Current, D Refers to T3(ICC) - Current. 4507857-4527699 * Analysis was performed at AGAT's Mining Division.

EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water : 1 part soil).

Certified By:

Stony Heath

Quality Assurance

CLIENT NAME: SPL CONSULTANTS

AGAT WORK ORDER: 13T731429

PROJECT NO: 1549-110

ATTENTION TO: Nirogini Nalliah

Soil Analysis

RPT Date: Jul 09, 2013

RPT Date: Jul 09, 2013			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Corrosivity Package															
Sulphide*	1		0.02	0.02	0.0%	< 0.01	100%	80%	120%	NA			NA		
Chloride (2:1)	4511184		<20	<20	0.0%	< 2	104%	80%	120%	96%	80%	120%	105%	70%	130%
Sulphate (2:1)	4511184		<20	<20	0.0%	< 2	101%	80%	120%	104%	80%	120%	108%	70%	130%
pH (2:1)	1		8.78	8.67	1.3%	N/A	102%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	1		0.219	0.219	0.0%	< 0.005	103%	90%	110%	NA			NA		
Redox Potential (2:1)	1		187	188	0.5%	< 5	108%	70%	130%	NA			NA		

Comments: NA - Not Applicable

Certified By:



AGAT QUALITY ASSURANCE REPORT (V2)

Page 3 of 4

AGAT Laboratories is accredited to ISO/IEC 17025 by the Canadian Association for Laboratory Accreditation Inc. (CALA) and/or Standards Council of Canada (SCC) for specific tests listed on the scope of accreditation. AGAT Laboratories (Mississauga) is also accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA) for specific drinking water tests. Accreditations are location and parameter specific. A complete listing of parameters for each location is available from www.cala.ca and/or www.scc.ca. The tests in this report may not necessarily be included in the scope of accreditation.

Results relate only to the items tested and to all the items tested

Method Summary

CLIENT NAME: SPL CONSULTANTS

PROJECT NO: 1549-110

AGAT WORK ORDER: 13T731429

ATTENTION TO: Nirogini Nalliah

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulphide*	MIN-200-12000	ASTM E1915-07a	LECO C_S
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR 1036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR 1036		CALCULATION
Redox Potential (2:1)		McKeague 4.12 & SM 2510 B	REDOX POTENTIAL ELECTRODE

Table 1.
Soil evaluation for ductile/cast iron pipe using 10-P method (after ANSI/AWWA C105/A21.5-99, 1999; DIPRA, 2000).

Soil	Values and characteristics	Points
Resistivity (Ω -cm)	< 1,500	10
	$\geq 1,500 - 1,800$	8
	> 1,800 - 2,100	5
	> 2,100 - 2,500	2
	> 2,500 - 3,000	1
	> 3,000	0
pH	0 - 2	5
	2 - 4	3
	4 - 6.5	0
	6.5 - 7.5	0
	7.5 - 8.5	0
	>8.5	3
Redox potential (mV)	> +100	0
	+50 - +100	3.5
	0 - +50	4
	< 0	5
Sulfides	Positive	3.5
	Trace	2
	Negative	0
Moisture	Poor drainage (continually wet)	2
	Fair drainage (generally moist)	1
	Good drainage (generally dry)	0

Appendix G

Statement of Limitations

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to SPL Consultants Limited at the time of preparation. Unless otherwise agreed in writing by SPL Consultants Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

