REPORT ON
GEOTECHNICAL INVESTIGATION AND FOUNDATION DESIGN
PROPOSED PEDESTRIAN BRIDGES AT HIGHWAY 410
WILLIAMS PARKWAY WIDENING
BRAMPTON, ONTARIO

GEOCRES No. 30M12-197

Prepared for:

AECOM CANADA LTD.

By:

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1. INTRODUCTION

SPL Consultants Limited (SPL) was retained by Aecom Canada Ltd. to undertake a geotechnical investigation for the proposed Williams Parkway Widening, from McLaughlin Road to North Park Drive in Brampton, Ontario.

It is understood that the project will include the following:

1. Construction of noise barrier walls
2. Widening of pedestrian underpass structures at Claypine Park, Waybridge Trail, Major Oaks Park and Lafrance Park
3. New underground utilities.
4. Widening of bridge structure at Etobicoke Creek
5. Pedestrian bridges at Highway 410
6. Widening of existing 4 lane urban section to 6 lane urban section

This report deals only with the above noted Item 5: Pedestrian Bridge at Highway 410. Boreholes for Item 4 are not drilled yet. Geotechnical reports for Items 1 to 3 and Item 6 were submitted earlier under separate covers.

The objective of this investigation was to determine the subsurface conditions at the location of the proposed pedestrian bridges at Highway 410 by means of six (6) exploratory boreholes, and to provide geotechnical recommendations for the construction of the bridges.

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 can cater to the changed design.

This report has been prepared for Aecom Canada Ltd. and City of Brampton. Third party use of this report without SPL Consultants Limited consent is prohibited.

2. FIELD AND LABORATORY WORK

The field investigation consisted of putting down six (6) exploratory boreholes (BH12-1 through to BH12-6) for the proposed pedestrian bridges to depths varying from 9.8 to 29.3 m. The field investigation work of borehole drilling was undertaken between November 7, 2012 and November 27, 2012 by drilling sub-contractors with technical supervision provided by engineering staff from SPL.

The boreholes were generally advanced using truck mounted power drill rigs using hollow-stem augers. The type of drilling method used to advance the boreholes is identified in the respective borehole logs (Drawings 2 to 7).

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 0.75m to 1.5m intervals using a 50mm O.D. split spoon sampler and an automatic SPT hammer, 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 the Borehole Logs.

Shale bedrock was cored at four (4) borehole locations (BH12-1, BH12-2, BH12-5 and BH12-6). The bedrock was cored with HQ-2 double tube wireline equipment providing 63mm diameter rock core samples. The coring was carried out under the full time supervision of a representative from SPL who identified and described the rock samples, noting and recording the percentages of total and solid rock core recovery, RQD values, fracture index and the percentage and thicknesses of hard layers.

Water level observations were made during drilling and in the open boreholes at the completion of the drilling operations. Monitoring wells were installed in four boreholes (BH12-1, BH12-2, BH12-5 and BH12-6) for longer-term (stabilized) groundwater level measurements.

The surface elevations at the borehole locations were surveyed by SPL and were referenced to the geodetic datum.

The geotechnical laboratory testing program consisted of the measurement of the natural moisture content of all samples, grain size analyses on twelve (12) selected soil samples and consistency (Atterberg) limits for three (3) selected samples. Test results are shown on the individual borehole logs and the gradation curves are presented on Drawings 10 and 11.

Two selected soil samples were submitted for chemical analysis for MOE phytoxicological parameters (metals and inorganics) for soil disposal purposes. The soil samples were submitted to AGAT Laboratories in Mississauga, Ontario. The results are attached in **Appendix B** and discussed in Section 4.1 of this report.

Selected three soil samples were subjected to corrosivity testing, including the sulphate (SO_4) resistance requirements for concrete in contact with the soils were evaluated by performing water-soluble sulphate tests. The test results are attached in **Appendix C** and discussed in Section 4.2 of this report.

3. SUBSURFACE CONDITIONS

In the boreholes, the native soils below the fill materials generally consisted of cohesionless soil deposits, overlying shale bedrock. General comments on the samples description are provided on Drawing 1B. More specific details on the subsurface conditions at the individual boring locations are given in the borehole log sheets (Drawings 2 to 7). Generalized sub-surface profiles along the north and south bridges are presented on Drawings 8 and 9. The following notes are, therefore, intended only to summarize the data and to amplify some of the general characteristics of the deposits.

<u>Topsoil / Fill Materials</u>

A surficial layer of topsoil was encountered at BH12-1, BH12-2, BH12-5 and BH12-6 locations, typically varying in thickness from 120 to 350 mm. A surficial granular fill layer, about 300 to 400 mm thick, consisting of sand and gravel was encountered at BH12-3 and BH12-4 locations. Fill material was found in boreholes to depths varying from 0.8m (BH12-3 and BH12-4) to 12.2 m (BH12-6). The fill material was heterogeneous and consisted of clayey silt, silty clay, sandy silt, silty sand and sand. Trace to some inclusions of topsoil / organics were also observed in fill material. The fill was generally in firm to very stiff consistency or in a loose to compact state, with occasional very stiff / dense layers.

Grain size analysis of one (1) sample from silty clay fill (BH12-6/SS9) was conducted and the results are presented on Drawing 10, with the following fractions:

Clay:	26%
Silt:	44%
Sand:	24%
Gravel:	6%

Atterberg limits test of the same sample from silty clay fill (BH12-6/SS9) was conducted. The results are shown on the borehole log and are summarized as follows:

Liquid limit (W_L):	30%
Plastic limit (W_P):	20%
Plasticity index (PI):	10

<u>Cohesionless Soils (sand, silt, gravelly sand, sand and gravel)</u>

Cohesionless soil deposits consisting of sand, silt, gravelly sand and sand and gravel were encountered at all of the borehole locations below fill material (BH12-1, BH12-2, BH12-5 and BH12-6) or glacial till deposits (BH12-3 and BH12-4). Cohesionless soil deposits continued upto the maximum explored depth of 9.8 m in boreholes BH12-3 and BH12-4 and upto depths varying from 23 to 24 m in all other boreholes. Occasional to frequent cobble / boulder were encountered in boreholes generally below a depth of about 15 m. At BH12-6 location, auger refusal was encountered at a depth of 21.4 m and a boulder was cored from 21.4 to 22.9 m. The cohesionless deposit was present in a loose to compact but generally compact state above about Elev. 222 and in a dense to very dense state below this elevation. The cohesionless soils were saturated and water bearing below approximate Elev. 227 m. Occasional layers of silt to sandy silt were also present within the deposits.

Grain size analyses of eight (8) samples from sand, gravely sand and sand and gravel (BH12-1/SS7, BH12-1/SS11, BH12-1/SS17, BH12-2/SS9, BH12-2/SS13, BH12-2/SS14, BH12-3/SS6, and BH124/SS8) were conducted and the results are presented on Drawing 11, with the following fractions:

Clay: 1 to 6%
Silt: 5 to 12%
Sand: 48 to 87%
Gravel: upto 44%

Grain size analysis of a silt sample (BH12-5/SS10) was conducted and the results are presented on Drawing 11, with the following fractions:

Clay: 8%
Silt: 72%
Sand: 20%

<u>Clayey Silt Till</u>

Clayey silt till deposit was encountered in BH12-3 and BH12-4 below the fill material and continued upto a depth of about 3.1 m, overlying sandy silt till. The clayey silt till was present in a stiff to hard consistency, with measured SPT 'N' values ranging from 13 to 30 blows per 300 mm penetration.

Grain size analysis of a clayey silt till sample (BH12-3/SS2) was conducted and the results are presented on Drawing 10, with the following fractions:

Clay: 18%
Silt: 44%
Sand: 36%
Gravel: 2%

Atterberg limits test of same sample from clayey silt till (BH12-3/SS2) were conducted. The results are shown on the borehole log and are summarized as follows:

Liquid limit (W_L): 22%
Plastic limit (W_P): 17%
Plasticity index (PI): 5

<u>Sandy Silt to Silty Sand Till</u>

Sandy silt to silty sand till layer was found in BH12-3 and BH12-4 below the clayey silt till and continued upto depths of 3.1 m and 5.6 m respectively, overlying cohesionless soils deposit. Sandy silt to silty sand till layer was encountered in BH12-1 and BH12-2 below cohesionless soil deposits at depths 23.9 m and 22.9 m respectively, overlying shale bedrock. A layer of sandy silt till of about 1.5 m thick was encountered in BH12-6 at a depth of 13.7 m, embedded within cohesionless soil deposit. The sandy till deposits were present in a compact to very dense state.

Grain size analysis of a sandy silt to silty sand till sample (BH12-3/SS5) was conducted and the results are presented on Drawing 10, with the following fractions:

Clay: 12%
Silt: 40%
Sand: 44%
Gravel: 4%

Atterberg limits test of same sample from sandy silt till (BH12-3/SS5) were conducted. The results are shown on the borehole log and are summarized as follows:

Liquid limit (W_L): 17%
Plastic limit (W_P): 13%
Plasticity index (PI): 4

Shale Bedrock

The shale bedrock of Georgian Bay Formation was encountered in BH12-1, BH12-2, BH12-5 and BH12-6 at depths varying from 24.4 to 25.3 m. Shale bedrock was proven by core drilling. The depth and elevation of the shale bedrock surface in the boreholes are listed on Table 1 below.

Table 1: Depth and Elevation of Shale Bedrock Surface

Borehole No.	Depth of Shale Bedrock Surface below Existing Ground (m)	Approximate Elevation of Shale Bedrock Surface (m)	Notes
BH12-1	25.3	213.0	Bedrock cored upto depth 28.5 m
BH12-2	24.4	214.3	Bedrock cored upto depth 29.3 m
BH12-5	24.4	212.7	Bedrock cored upto depth 27.9 m
BH12-6	24.4	211.5	Bedrock cored upto depth 24.8 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 A. 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 for the cored runs ranged from 17 to 100%. Generally, less core recovery was experienced only near the surface of the rock, where the formation is moderately weathered and was almost full as depth increased.

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 5% to 100%, 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 total length of recovered rock core pieces which are longer than 100mm and expressing their sum total 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 which range from nil to 100%, the rock quality is estimated to be “very poor” to “excellent”, and the average value of more than 50% suggests a rock of generally “fair” 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 nil and 31%. The thickness of these layers varied but was generally varied from 50 to 250mm, but thicker layers have been observed to be as much as 750 to 900 mm at other sites. The layers are actually lenses and they can vary significantly in thickness over short distance. 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 25 to 50mm 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 nil and greater than 25. In BH12-2, broken zone was encountered to a depth of 25.9 m. 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.

Weathering: In general, moderately weathered zone in the bedrock was limited to about 1.5 m from the bedrock surface. 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.

Point Load Index Strength: Point load index strength tests were performed on selected bedrock samples and the test results are presented on the respective rock core logs. 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 a very approximately correlation after Franklin and Hoek.

The equivalent axial unconfined compressive strength of limestone/siltstone samples was inferred to range from 55 to 290 MPa indicating “strong to extremely strong” rock under ISRM strength convention. The inferred axial UCS of the shale was lower than that of limestone/siltstone, ranging from 3 to 34 MPa. These values indicate a “very weak” to “medium strong” rock. The shale can often be broken by hand in the diametral direction, indicating considerable strength anisotropy along bedding planes.

Gas: The Georgian Bay Formation is known to contain pockets of combustible gas. During the rock coring there were no physical indications of the presence of gas in the boreholes. However, appropriate care and monitoring are essential in all confined bedrock excavation work.

<u>Groundwater Conditions</u>

The groundwater table observed in the monitoring wells was at depths ranging from 4.3 to 12.1 m, corresponding to elevations ranging from 225.8 to 226.6 m, as listed on Table 2.

Table 2: Groundwater Levels Observed in Monitoring Wells

BH No.	Date of Drilling	Date of Groundwater Observation	Depth of Groundwater Table (m)	Elevation of Groundwater Table (m)
BH12-1	Nov. 16/12	Jan. 17/13	11.7	226.6
BH12-2	Nov. 21/12	Jan. 04/13	12.1	226.6
BH12-3	Nov. 07/12	Nov. 07/12	4.3	226.3
BH12-4	Nov. 07/12	Nov. 07/12	4.6	226.4
BH12-5	Nov. 27/12	Jan. 04/13	11.3	225.8
BH12-6	Nov. 10/12	Jan. 04/13	9.8	226.1

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to major weather events.

4. GEO-ENVIRONMENTAL FINDINGS AND CHEMICAL TEST RESULTS

4.1 Environmental Soil Quality Test Results

In order to assess options for potential offsite soil disposal at the above captioned site, two (2) soil samples were analysed for metal and inorganic parameters as set out in Ontario Regulation 153, Section XV.1 of the Environmental Protection Act. The soil samples were collected during the advancement of geotechnical boreholes on the site.

The following is the description of the sample name, depth and soil description of the fourteen (2) soil samples submitted for the analysis of metal & inorganic parameters.

- BH12-1/SS3 (1.5-2.0m) was fill soil consisting of moist brown silty sand, trace clay and gravel
- BH12-6/SS5 (3.0-3.6m) was fill soil consisting of moist grey silty clay, sandy, and trace gravel

Soil represented by the above noted soil samples may require removal during the proposed bridge construction. Locations of the boreholes completed by SPL Consultants are shown on **Drawing** 1**.** Detailed descriptions of the subsurface conditions at the borehole locations are presented in the respective borehole logs Drawings 2 to 7**.**

Soil samples were collected and handled in accordance with generally accepted procedures used by the environmental consulting industry. To minimize the potential for cross contamination between soil samples, prior to each sampling event the split spoon sampler used to collect soil samples were brushed clean of soil, washed in municipal water containing phosphate free detergent, rinsed in municipal water and then rinsed with distilled water for each sampling interval. As well, prior to each sampling event, new disposable gloves were used to transfer the samples into plastic bags and glass jars.

The chemical analyses were conducted by AGAT Laboratories located in Mississauga, Ontario. AGAT is a member of the Canadian Association for Laboratory Accreditation (CALA) and meets the requirements of Section 47 of O.Reg. 153/04 certifying that the analytical laboratory be accredited in accordance with the International Standard ISO/IEC 17025 and with standards developed by the Standards Council of Canada. The Certificates of Analysis are attached in **Appendix B**.

For the purposes of soil disposal, if required, the results of chemical analyses were compared to the Full Depth Background Site Condition Standards contained in Table 1, and the Full Depth Generic Site Condition Standards in Potable Ground Water Condition (coarse textured soils) for industrial/commercial/community (ICC) use and residential/parkland/institutional (RPI) use contained in Table 2 and Table 3 of the "Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act", published by the MOE April 15, 2011.

Based on the results of chemical analyses, SPL provides the following conclusions/recommendations:

- Both the samples (BH12-1/SS3 and BH12-6/SS5) had elevated levels of sodium adsorption ratio (SAR) above MOE Table 2 residential/parkland/institutional (RPI) property use standards. Sample BH12-1/SS3 exceeded the Table 2 and 3 ICC Standards for sodium adsorption ratio (SAR) while

BH12-6/SS5 met both Table 2 and 3 industrial/community/commercial (ICC) property use standards for SAR.

- BH12-1/SS3 had elevated levels of electrical conductivity (EC) above MOE Table 2 residential/parkland/institutional (RPI) property use standards but met Table 2 and 3 industrial/community/commercial (ICC) property use standards for EC.
- Acceptance of this material will be at the discretion of the receiving site.
- The results relate to the environmental quality of the soil and do not pertain to the geotechnical suitability of the material.

The purpose of this testing was to assess soil disposal options and does not constitute a Phase 2 Environmental Site Assessment as defined in Ontario Regulation 153.

It should be noted that if any aesthetically impacted soils are identified during excavation it is recommended that SPL be notified in order to conduct further assessment and / or testing of the material in question.

This report was prepared for the account of AECOM and the City of Brampton the material in this report reflects SPL's judgment in light of the information available to it at the time of preparation. Any use, which a Third Party not noted above makes of this report, or any reliance on 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.

4.2 Corrosivity / Sulphate Resistance of Concrete

Three soil samples were subjected to corrosivity testing. The test results are presented in **Appendix C** and are also shown on the following Table 3. The need for cathodic protection to gray or ductile cast iron pipe as given in the ANSI/AWWA Rating for soil-test corrosion evaluation is given in the **Appendix C**.

Table 3: Summary of Corrosivity / Sulphate Test Results

Sample	pH	Sulphate (µg/g)	Resistivity (ohm.cm)	Redox Potential (mV)	Sulphides (%)	Assigned Points
BH12-2/SS10	8.93	70.1	4740	120	0.03	7
BH12-3/SS7	8.64	58.8	6100	91	0.03	10.5
BH12-6/SS6	8.22	37.5	3390	95	0.02	6.5

The test results indicated one test result with above 10 assigned points, indicating that the soil is corrosive to gray or ductile cast iron pipe and cathodic protection is required.

Samples were also tested for water soluble sulphate content in order to evaluate the subsoil conditions for possible sulphate attack on concrete. According to Table 3 of CSA Standard, CAN/CSA-A23.1-04 the degree of exposure to sulphate attack is negligible. Therefore normal Portland cement can be used in the subsurface concrete.

5. GEOTECHNICAL DISCUSSION AND RECOMMENDATIONS

It is understood that pedestrian bridges will be constructed over Highway 410 on both the north and the south sides as a part of widening of Williams Parkway. From the available structural drawings, it is understood that the east abutment of the existing Williams Parkway Bridge over Highway 410 is supported on steel H-Piles, while the west abutment and central piers are supported on conventional footings.

5.1 Foundations of Proposed Pedestrian Bridges

For suitability comparison of foundation options, the following types of foundations are listed for discussion purpose:

- Footings
- Micropiles
- Driven piles
- Drilled caissons

Three boreholes (BH12-1, BH12-3 and BH12-5) were drilled for the south pedestrian bridge and three boreholes (BH12-2, BH12-4 and BH12-6) were drilled for the north pedestrian bridge. In the boreholes, the native soils generally consisted of cohesionless soils overlying shale. Cobbles and boulders were encountered in the boreholes during drilling within the cohesionless soil deposits. Stabilized groundwater table at site is at about Elev. 227 m.

In BH12-6, fill material extended to a depth of about 12.2 m below the existing grade. Deep excavation will be required close to existing structures if east abutment of the north bridge is to be supported by conventional footings or mat foundations. As the footings should be constructed in dry, positive dewatering will be required for footings installation below groundwater table. Temporary shoring will also be required for deeper excavation close to the existing major bridge. Due to the close proximity of the existing structure, conventional footings or mat foundation are not considered to be preferred option to support the abutments of the pedestrian bridges. Footings or mat foundations can be used to support the central piers of the bridges. Bridge abutments can be supported by deep foundations such as piles.

Due to the presence of the cohesionless deposits below the groundwater table, installing drilled caissons in the cohesionless deposits will be difficult, due to the groundwater and caving problems from these cohesionless deposits. Sealing of the liner in the shale bedrock will be difficult where limestone cap or boulder is present above bedrock. Coring will be required to pass through the boulders. Drilled caissons are therefore not recommended to support the proposed structures due to constructability issues.

Presence of frequent cobbles and boulders will make installation of driven piles (steel H-piles) very difficult. Vibrations produced during pile driving will also be a concern for the existing bridge structure. Therefore driven plies are not considered to be a preferred foundation option to support the proposed pedestrian bridge abutments.

Micropiles can be drilled through the obstructions or boulders and can be installed in most of ground / bedrock conditions below groundwater table. Based on the boreholes information, micropiles installed in dense cohesionless soils or shale bedrock are considered to be feasible to support the pedestrian bridge abutments.

5.1.1 Footings and Mat Foundations for Central Piers

Conventional footings and/or mat foundations can be considered to support the central piers of the proposed pedestrian structures. The bearing values and the corresponding founding elevations at the borehole locations are summarized on Table 4 below.

Table 4: Bearing capacity Values of Native Soils for Footings and Mat Foundations

Structure	Support Location	Borehole No.	Bearing Capacity at SLS (kPa)	Bearing Capacity at ULS (kPa)	Minimum Depth below Existing Ground (m)	Founding Level At or Below Elevation (m)	Note
South Pedestrian Bridge	Central Pier	BH12-3	200	300	1.2	229.4	Water at 226.3m
North Pedestrian Bridge	Central Pier	BH12-4	200	300	1.2	229.8	Water at 226.4m

A subgrade reaction modulus of 15 MPa/m of the founding native soils can be used for the design of the mat foundations (if adopted).

Lateral resistance of footings can be calculated by assuming a friction coefficient of μ = 0.50 (unfactored) between the footing base and the native soil, and coefficient of passive earth pressure K_p = 3 acting against the side of footing. Passive earth pressure should be ignored for the soil above the frost depth of 1.2 m.

Provided that the subgrade is not disturbed during construction, foundations designed to the specified bearing values are expected to settle less than 25 mm in total at SLS and 19 mm differential.

All footing bases must be inspected by a qualified geotechnical personnel prior to placing concrete to confirm the founding soil conditions and the bearing capacity. Allowance should be made to place a 120 mm thick concrete mud mat in the footing base, immediately after the footing bases are inspected and approved.

5.1.2 Micropiles for Abutments

Based on the borehole information and the existing site conditions, the proposed pedestrian bridge abutments can be supported by micropiles.

A micropile is constructed by drilling a hole, placing reinforcement, and grouting the hole. Micropiles are installed by methods that cause minimal disturbance to the adjacent structures. They can be installed in access restrictive environments in most soil and rock types, with minimal vibrations and noise. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along grout/ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to small pile diameter (typically 150 to 300 mm), end bearing contribution in micropiles is generally neglected in design. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e. pressure grouting or gravity feed.

It is recommended that the micropiles for the pedestrian bridge abutments be installed into the very dense cohesionless soils and/or into bedrock. Typical axial bearing capacity values of 200 to 400 kN per pile at SLS (i.e. 250 to 500 kN per pile at ULS) in soils and 300 to 600 kN per pile at SLS (i.e. 400 to 800 kN per pile at ULS) in bedrock are available, depending on the diameter and length of penetration into the very dense soil or bedrock. The lateral resistances would also depend on the diameter, as well as on the socket length into the bedrock.

For preliminary estimating purposes, the bond strength between the micropile and the native cohesionless soils can be taken as 75 kPa at SLS and 100 kPa at ULS. The skin friction between the pile shaft and the fill materials can be ignored. The bond strength between the micropile and the sound shale bedrock can be taken as 350 kPa at SLS and 450kPa at ULS, but the contribution from the upper relatively fractured 1.2 m should be ignored. These suggested bond values are for preliminary design purpose only, as the actual bond values will depend on the installation and grout procedures of the piles and must be determined by the field load testing. A specialty contractor must be retained to design and construct the micropiles. The specialty contractor should determine the length and size of the piles, based on the design loads, the borehole information and their installation method/procedure.

Field pile load testing will be required to confirm the design bearing capacity. The test piles must be loaded to at least two times its design bearing value at ULS. In order to ignore the group effect, the center-to-centre distance between adjacent micropiles should be at least 3 times its diameter.

The production micropiles must be installed after the pile load testing, only when the design load is confirmed by the pile load test results. The installation and load testing of the test micropile must be monitored by a qualified geotechnical engineer.

If necessary, batter micropiles can be adopted to provide lateral/horizontal resistance.

5.1.3 Driven Piles for Abutments

As mentioned previously, the presence of frequent cobbles and boulders in the soils at the site will make installation of driven H-Piles very difficult. Vibrations produced during pile driving will also be a concern for the existing bridge structure. For the relatively light pedestrian bridge structures, driven piles (steel H-Plies) may not be a preferred foundation option.

Based on the borehole information, the ultimate bearing capacity of the piles (steel H-piles) driven to practical refusal in the shale bedrock can be taken as:

<u>HP 310x110 piles</u>:

Ultimate bearing capacity	= 2400 kN/pile
Factored geotechnical resistance at ULS	= 1200 kN/pile
Bearing capacity at SLS:	= 950 kN/pile

For preliminary design purpose, the practical refusal can be expected to be at about 1 to 2 m below the shale bedrock surface. The depth and elevation of the shale bedrock rock surface at in the boreholes (BH12-1, BH12-2, BH12-5 and BH12-6) are listed on Table 1. The actual depth of the piles must be determined by field PDA testing, which may be shorter or longer than the design depth.

The horizontal spacing of the piles should be at least 3 times the pile size/diameter.

The bearing capacity and the required depth of the piles and the driving criteria for practical refusal must be determined by field pile diving analyzer (PDA) tests. It is recommended that prior to the final design of the foundations, at least four (4) test piles be installed across the site to confirm the available bearing capacity and the required depth of the piles by field pile tests using the Pile Driving Analyzer (PDA). PDA testing should be started at about 2 to 3 m above the design depth of the piles. The depth of the piles will be economized from the results of this initial stage PDA testing. PDA testing is also required at re-tapping at about 1 to 2 weeks after the initial driving, in order to examine the set-up effect on the decrease or increase of pile capacity with time.

The piling contractor should ensure that the pile-driving hammer is powerful enough to achieve the required bearing capacity and depth of the piles, but will not cause damage of the piles during the pile driving. Pile tip protection using flange plates is recommended. The pile toes should be reinforced as per MTO standard. Care must be taken to avoid overdriving and damaging the pile tip, i.e. the structural capacity of the piles should not be exceeded. The possibility of the piles encountering potential obstructions in fill and native soil should be anticipated. Stiffening of the tops of the piles may also be required.

Due to potentially variable soil and bedrock conditions and presence of cobbles and boulder, the actual pile tip elevation will vary. The contractor should allow for some variation in pile length and this aspect should be taken into consideration when ordering the piles.

The pile driving should be observed, on a full time basis, by an experienced soil technician, who will record penetration resistance, pile tip elevation etc. The technician must be supervised by a professional engineer experienced in this type of work.

During the driving process, piles that have already been driven will need to be monitored to determine if heaving occurred due to the effect of driving of the adjacent piles. If this phenomenon occurs, the affected piles will need to be re-driven. Re-tapping to check that relaxation has not occurred will be necessary. Furthermore, it may be necessary to stagger the driving of the piles. The piles should be provided with reinforced tips.

It should be noted that the till is a non-sorted sediment and therefore may contain boulders. Possible large obstructions such as buried concrete pieces and existing foundations are also anticipated in the fill material. Therefore, boulders or obstructions may be encountered during the installation of the piles.

Vibration monitoring of the existing structures will be required during the pile installation.

5.1.4 General Comments on Foundations

All footings and pile caps exposed to seasonal freezing conditions must have at least 1.2 m of soil cover for frost protection.

The existing and proposed bridge foundations design drawings, when available, should be reviewed by SPL to confirm that the existing foundations are not in conflict with the proposed foundations for the new bridges.

It should be noted that the recommended bearing capacities have been calculated by SPL Consultants Limited from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of the underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections provided by SPL Consultants Limited to validate the information for use during the construction stage.

5.2 Retaining Structures and Earth Pressures

Backfill behind retaining structures should consist of non-frost susceptible, free draining granular materials and should conform to the minimum requirements illustrated in OPSD 3101.150. The granular backfill should conform to OPSS 1010 for either Granular ‘A’ or ‘B’ Type I and Type II. To maintain free draining characteristics in these granular fill materials, the maximum percentage passing the No. 200 sieve (75 μm) should be limited to 5%.

The backfill should be placed in accordance with OPSS 501. A perforated subdrain should be installed behind the base of the walls as shown in OPSD 3102.100 to maintain the granular fill in a drained condition. The subdrain should be directed to a positive outlet to the municipal sewer or highway drainage system.

Computation of earth pressures acting against rigid retaining walls and any wingwalls should be in accordance with the Canadian Highway Bridge Design Code, (CHBDC). For design purposes, the following properties can be assumed for backfill.

Compacted Granular 'A' or Granular 'B' Type II

Angle of Internal Friction ϕ=35° (unfactored)
Unit weight = 22 kN/m^3
Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
K_a=0.27	K_a=0.34	K_a=0.40
K_b=0.35	K_b=0.44	K_b=0.50
K_o=0.43	K_o=0.56	K_o=0.62
K*=0.45	K*=0.60	K*=0.66

Compacted Granular 'B' Type I

Angle of Internal Friction ϕ=32° (unfactored)
Unit Weight = 21 kN/m^3
Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
Ka=0.31	Ka=0.39	Ka=0.47
Kb=0.39	Kb=0.49	Kb=0.57
Ko=0.47	Ko=0.62	Ko=0.69
K*=0.54	K*=0.68	K*=0.78

Note: K_a is the coefficient of active earth pressure

K_b is the backfill earth pressure coefficient for an unrestrained structure including compaction efforts

K_o is the coefficient of earth pressure at rest

K* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

These values are based on the assumption that the backfill behind the retaining structure is free-draining granular material and adequate drainage is provided.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. In the case of a rigid frame structure, yielding is unlikely and therefore at rest pressures should be used. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients.

5.3 Approach Embankments

It is understood that the proposed grade after widening of roadway surface and extension of underpass structures will be same as the existing grade. The existing approach embankments are generally about 7.5m to 8m high, with Highway 410 road level at about Elevation 230.5m to 231.0m and the Williams Parkway road level at about Elevation 238.5m to 239.0m.

It is recommended that all existing topsoil, fill materials and any other unsuitable material be removed from the area of the proposed embankments prior to commencing earthwork construction. After stripping, the exposed subgrade should be inspected and approved by a qualified geotechnical engineer. It should then be compacted, where feasible, from the surface using a suitable compactor. With this procedure, conventional 2H:1V or flatter side slopes of embankments should not cause foundation instability of the embankments.

Proper benching of the existing slope should be implemented if and where abutting into the existing earth slopes. This can be constructed in accordance with OPSD 208.01 – Benching of Earth Slope.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill, i.e. select subgrade materials (SSM) or Granular 'B' – OPSS 1010. The embankment fill should be placed on the approved and properly rolled subgrade in lifts not exceeding 300 mm when loosely placed and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). The degree of compaction should be increased to 98% of SPMDD for the upper 1.0 m of subgrade.

The settlement of the new embankment fills under their own weight can be expected to occur, say about 0.25% to 0.5% of the fill thickness. The time rate will depend on the material used for construction. However, if SSM or granular soils are used, the majority of the settlement of will be completed during the construction stage.

5.4 Excavation and Groundwater Control

Excavations can be carried out with heavy hydraulic backhoe. Positive dewatering will be required prior to any excavation in cohesionless soils below groundwater table (about Elev. 227.0 m), otherwise it will result in an unstable base and flowing sides. Water level must be lowered to at least 1 m below the lowest excavation level.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the overburden soil can be classified as Type 3 soil above groundwater table and Type 4 below groundwater table.

Possible large obstructions such as buried concrete pieces are also anticipated in the fill material. Provisions must be made in the excavation contract for the removal of possible obstructions in the fill material and cobble / boulders in cohesionless deposits. Coring of boulders or limestone cap above shale bedrock may be required for foundation installation.

5.5 Temporary Shoring

It is understood that the proposed excavations may be supported by a temporary shoring system consisting of timber lagging and soldier piles. Positive dewatering will be required for lagging installation below groundwater table.

Temporary protection systems should be designed and constructed in accordance with OPSS 539. The soil parameters estimated to be applicable for this design are as follows:

1) Earth Pressure Coefficients

   (a) where movement must be minimal K=0.45

   (b) where minor movement (.002H) can be tolerated K=0.25

   (c) passive earth pressure for soldier piles (unfactored) Kp=4

2) For stability check

   $\phi = 32^\circ$

   c= 0

   $\gamma = 21$ kN/m^3

   Surcharge is to be determined by shoring contractor.

3) For earth anchors

An allowable bond value of 48 kPa is suggested; this value depends on anchor installation methods and grouting procedures. Gravity poured concrete can result in low bond values while pressure grouted anchors will give higher values and produce a more satisfactory anchor.

An allowable bearing value of 400 kPa can be used for soldier pile caissons in compact or dense soil, provided concrete is poured in clean dry caisson holes. If a slurry procedure and tremie concrete is used, an allowable bearing capacity of 200 kPa can be adopted.

Casing will be required during the construction of the tiebacks to prevent caving of soils. The soldier piles should be installed in pre-augered holes taken below the deepest excavation. The holes should be filled with concrete below the excavation level and half bag mix above the base of the excavation. The concrete strength must be specified by the shoring designer. Temporary liners may be required to help prevent the sand from caving during the installation period. Positive measures may be required to prevent the loss of soil through the spaces between the lagging boards. This could probably be achieved by placing well-graded sand and gravel behind the lagging boards or by installing a geotextile filter cloth.

Soil anchors will be required to support the shoring. The anchors must be of a length that meets the Canadian Foundation Manual recommendations. It is important to note that the minimum length lies beyond the (45 - ϕ/2 + .15H) line drawn from the base of the soldier pile and the overall stability of the system must be checked at each anchor level, where ϕ is the soil friction angle and H is the shoring height.

The top anchor must not be placed lower than 3.0 metres below the top of level ground surface. Anchors will require casing when penetrating through wet sand and silt layers. The bond value of 48 KPa is suggested but this value is arbitrary since the contractors installation procedures will determine the actual soil to concrete bond value. Hence, the contractor must decide on a capacity and confirm its availability. All anchors must be tested as indicated in the Foundation Manual, 4th edition.

Adhesion on the buried caisson shaft or behind the shoring system must be neglected when designing this shoring system.

Movement of the shoring system is inevitable. Vertical movements will result from the vertical load on the soldier piles resulting from the inclined tiebacks and inward horizontal movement results from earth and water pressures. The magnitude of this movement can be controlled by sound construction practices, and it is anticipated that the horizontal movement will be in the range of 0.1 to 0.25% of the shoring height (H). Therefore, assuming H=7 metres movements of 17.5 mm should be expected. Vertical movements increase the horizontal movements because of the reduced stress in the inclined anchors and must be kept well below this value.

To ensure that movements of the shoring are within an acceptable range, monitoring must be carried out. Vertical and horizontal targets on the soldier piles must be located and surveyed before excavation begins. Weekly readings during excavation should show that the movements will be within those predicted; if not, the monitoring results will enable directions to be given to improve the shoring.

The recommended performance level for temporary shoring is 2 (maximum horizontal displacement of 25 mm).

6. GENERAL COMMENTS AND LIMITATIONS OF REPORT

SPL Consultants Limited should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, SPL Consultants Limited will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole and test pit results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

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.

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.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

Yours very truly,

SPL CONSULTANTS LIMITED

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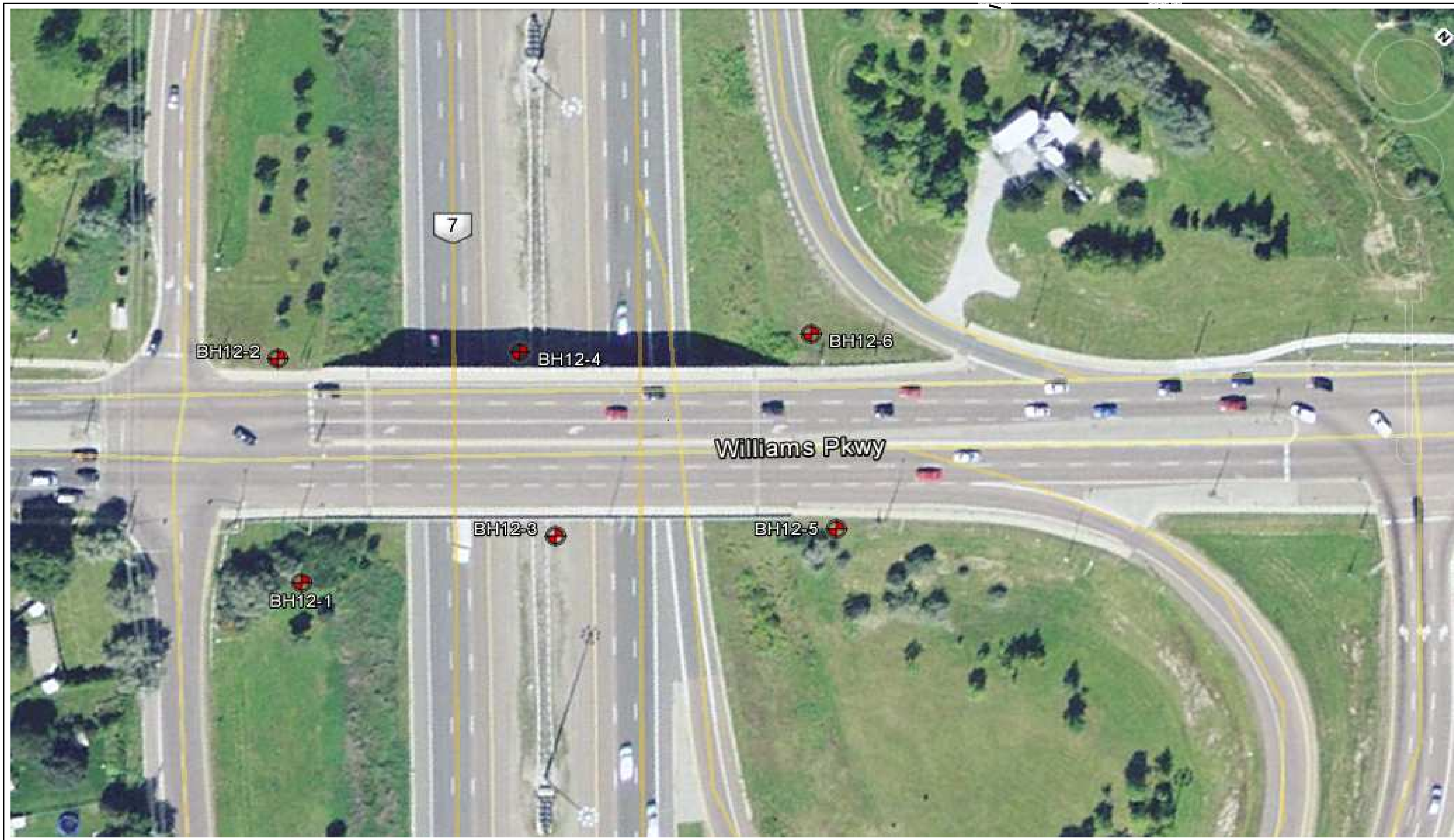
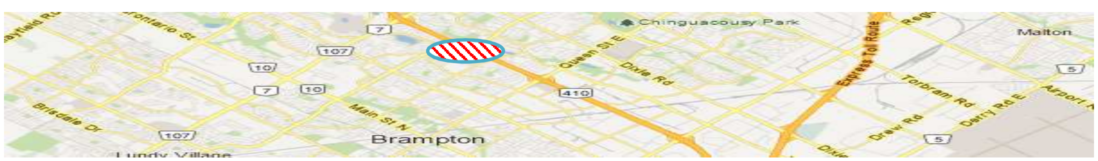
Drawings





Client:	AECOM CANADA LTD.			Project No:	1122-110	Drawing No:	1
Drawn:	AH	Approved:	FZ	Title:	Borehole Location Plan		
Date:	Aug. 2012	Scale:	as shown	Project:	Geotechnical Investigation-Williams Parkway, Brampton, Ontario		
Original Size:	Tabloid	Rev:	1	SPL Consultants Limited Geotechnical Environmental Materials Hydrogeology			

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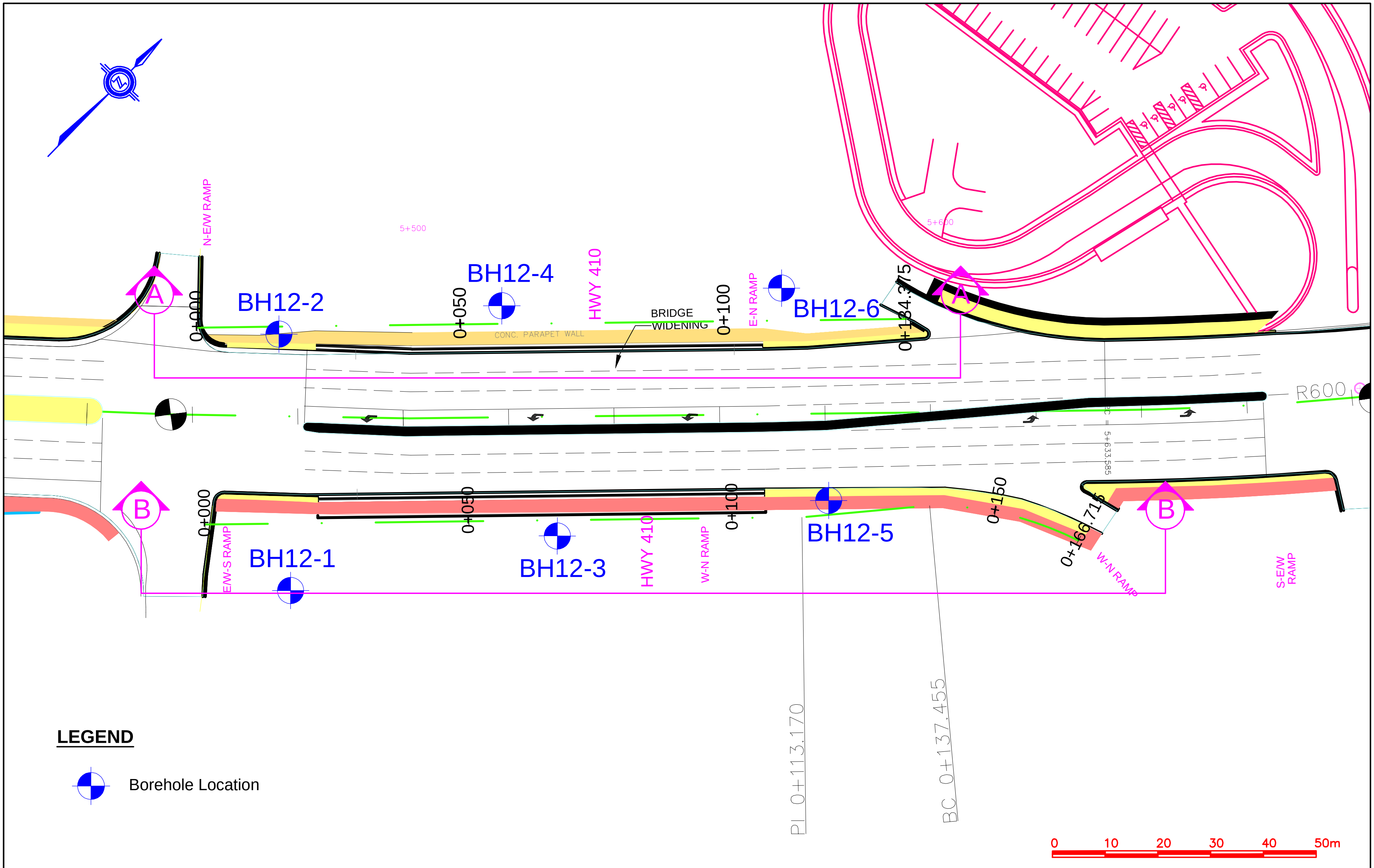

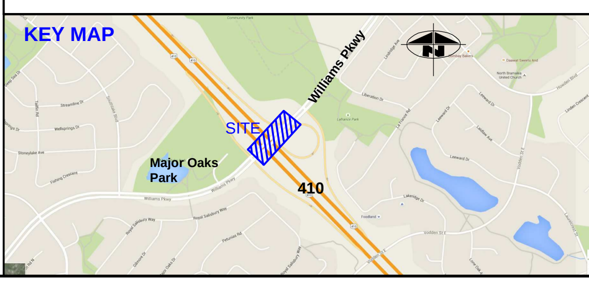

Client: AECOM CANADA LTD.		Project No.: 1122-110B	Drawing No.: 1A
Drawn: ZMO	Approved: AS	Title: Borehole Location Plan	
Date: September 04, 2015	Scale: As Shown	Project: Geotechnical Investigation and Foundation Design - Proposed Pedestrian Bridges at Hwy 410 Williams Parkway Widening, Brampton, ON	
MTO Geocres No: 30M12-179	Rev: N/A	SPL Consultants Limited Geotechnical * Environmental * Materials * Hydrogeology	

Drawing 1B: Notes On Sample Descriptions

1. All sample descriptions included in this report generally follow the Unified Soil Classification. Laboratory grain size analyses provided by SPL also follow the same system. Different classification systems may be used by others, such as the system by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Please note that, with the exception of those samples where a grain size analysis and/or Atterberg Limits testing have been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

0.002 | 0.006 | 0.02 | 0.06 | 0.2 | 0.6 | 2.0 | 6.0 | 20 | 60 | 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

UNIFIED SOIL CLASSIFICATION

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional preliminary geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

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LOG OF BOREHOLE BH12-1

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4840990 E 600523

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter: 203mm/63mm
Date: Nov/16/2012

REF. NO.: 1122-110
ENCL NO.: 2

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
238.3						
238.1 / 0.2	**TOPSOIL:** 150 mm					
	FILL: clayey silt, some sand to sandy, trace topsoil / rootlets and asphalt fragments, brownish grey, moist, stiff to hard	1	SS	8		
		2	SS	50/ 125mm		
236.8 / 1.5	**FILL:** silty sand, trace clay, occasional gravel, brown, moist, compact to very dense	3	SS	55		
		4	SS	30		
		5	SS	37		
	trace topsoil pockets and rootlets at 4.6m	6	SS	28		
232.2 / 6.1	**SAND:** trace to some silt, trace clay, contains clayey silt pockets, brown, moist, dense to very dense	7	SS	30		0 79 16 5
		8	SS	30		
		9	SS	30		
		10	SS	32	W. L. 226.6 m Jan 17, 2013	
	grey, wet below 12.2m	11	SS	32		0 84 12 4
		12	SS	35		
		13	SS	34		

ELEVATION: 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223, 222

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE — 50 100 150 200 250
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (Mg/m^3)

Continued Next Page

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: + 3 , × 3 : Numbers refer to Sensitivity ○ ε=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-1

PROJECT: Proposed Widening of Williams Parkway	**DRILLING DATA**
CLIENT: AECOM Canada Ltd.	Method: Hollow Stem Auger/HQ Casing
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario	Diameter: 203mm/63mm
DATUM: Geodetic	Date: Nov/16/2012
BH LOCATION: (See Drawing 1) N 4840990 E 600523	

REF. NO.: 1122-110
ENCL NO.: 2

(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND: trace to some silt, trace clay, contains clayey silt pockets, brown, moist, dense to very dense(Continued)		14	NR	35		221		
			15	SS	79		220	○	
							219		
217.9			16	SS	40		218	○	
20.4	**GRAVELLY SAND:** some silt, occasional cobble / boulder, trace clay, grey, wet, very dense.		17	SS	50/ 75mm		217		36 48 12 4
							216		
214.4			18	SS	51		215		
23.9	**SANDY SILT TILL:** trace clay, trace gravel, trace shale fragments, grey, saturated, very dense.		19	SS	55/ 150mm		214	○	
213.0							213		
25.3	**SHALE BEDROCK:** grey, interbedded with siltstone and limestone (Georgian Bay Formation) rock coring started at 25.5 m		R1	CORE			212		
	Refer to Log of Rock Core		R2	CORE			211		
209.8			R3	CORE			210		
28.5	**END OF BOREHOLE**								

Notes:
1) Auger drilling ended at 25.3 m and rock coring started at 25.5 m using 63 mm HQ core barrel.
2) 50mm monitoring well installed at 24.4 m after completion.

Water Level Readings:

Date	W. L. Depth(m)
Nov. 30/12	12.4
Jan. 17/13	11.7

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity; ● QUICK TRIAXIAL × LAB VANE; 50 100 150 200 250

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

POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (Mg/m^3)

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF ROCK CORE BH12-1

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4840990 E 600523

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter:
Date: 16/11/2012

REF. NO.: 1122-110
ENCL NO.: 2A

(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE NUMBER	CORE SAMPLE SIZE (mm)	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)
212.8 25.5	**GEORGIAN BAY FORMATION BEDROCK** Moderately weathered to fresh, laminated to thinly bedded, dark grey to grey, very weak to medium strong, **SHALE** (0-69%), thinly laminated to medium bedded with slightly weather to fresh, light grey, medium strong to extremely strong **SILTSTONE** and **LIMESTONE** (0-31%).		R1	63	97	33	0	17	>25 3 9	moderately weathered upto 25.6 m slightly weathered to fresh below 25.6m		3			
212.0 26.3			R2	63	100	60	20	56	5 2 2 4 3			57			
210.5 27.8			R3	63	100	71	31	54	3 1			290			
209.8 28.5	**END OF BOREHOLE** Notes: 1) 50mm monitoring well installed at 24.4 m after completion. Water Level Readings: Date W. L. Depth(m) Nov. 30/12 12.4														

E = Modulus of Elasticity

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-2

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841024 E 600488

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter: 203mm
Date: Nov/21/2012

REF. NO.: 1122-110
ENCL NO.: 3

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
238.7 / 238.4 0.0 / 0.3	**TOPSOIL:** 300mm **FILL:** sandy silt, trace to some clay, trace topsoil / rootlets, greyish brown, moist, loose to compact	1	SS	5	~20	
		2	SS	15	~10	
237.2 / 1.5	**FILL:** sand, some silt, occasional gravel, brown, moist, loose to compact	3	SS	52	~4	
		4	SS	5	~7	
		5	SS	8	~6	
234.1 / 4.6	**SAND:** trace silt, trace clay, trace gravel, brown, moist, compact	6	SS	18	~6	
	greyish brown, very moist to wet below 6.1m	7	SS	23	~9	
		8	SS	22	~12	
		9	SS	22	~19	1 87 9 3
		10	SS	19	~21	
	grey, wet below 9.1 m	11	SS	17	~16	
	some gravel to gravelly below 13.7 m	12	SS	14	~17	
223.5 / 15.2	**SAND AND GRAVEL:** trace silt, occasional cobble / boulder, grey, wet, compact	13	SS	26	~16	44 50 5 1

W. L. 226.6 m
Jan 04, 2013

Chart headers: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE (50 100 150 200 250); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (Mg/m³); STRATA PLOT; GROUND WATER CONDITIONS; ELEVATION 238–223.

Continued Next Page

GROUNDWATER ELEVATIONS
Shallow/ Single Installation   Deep/Dual Installation

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity   ○ ε=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-2

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841024 E 600488

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter: 203mm
Date: Nov/21/2012

REF. NO.: 1122-110
ENCL NO.: 3

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	ELEVATION	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
221.9					222		
16.8	**SAND:** trace silt, grey, wet, dense to very dense	14	SS	33		○	0 87 9 4
					221		
		15	SS	50/ 125mm	220		
	trace to some gravel below 19.8 m				219		
		16	SS	68		○	
					218		
217.4 / 21.3	**SAND AND GRAVEL:** some silt, occasional cobble / boulder, grey, wet, very dense	17	SS	100/ 125mm	217		
					216		
215.8 / 22.9	**SILTY SAND TILL:** trace to some clay, trace gravel, trace shale fragments, greyish brown to grey, wet, very dense	18	SS	50/ 75mm		○	
					215		
214.3 / 24.4	**SHALE BEDROCK:** grey, interbedded with siltstone and limestone (Georgian Bay Formation) Refer to Log of Rock Core				214		
		1	CORE		213		
		2	CORE		212		
		3	CORE		211		
		4	CORE		210		
209.4 / 29.3	**END OF BOREHOLE:** 1) Auger drilling ended at 24.5 m and rock coring started at 24.8 m using 63 mm HQ core barrel. 2) 50mm monitoring well installed at 24.4 m after completion. Water Level Readings: Date W. L. Depth(m) Nov. 30/12 12.6 Jan. 04/13 12.1						

Other chart headings: SOIL PROFILE; SAMPLES; STRATA PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE (50 100 150 200 250); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (Mg/m^3)

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

LOG OF ROCK CORE BH12-2

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841024 E 600488

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter:
Date: 21/11/2012

REF. NO.: 1122-110
ENCL NO.: 3A

(m) ELEV / DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE NUMBER	CORE SAMPLE SIZE (mm)	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)
214.3 / 24.4	**GEORGIAN BAY FORMATION BEDROCK** Moderately weathered to fresh, laminated to thinly bedded, dark grey to grey, very weak to medium strong, **SHALE** (95-81%), thinly laminated to medium bedded with slightly weather to fresh, light grey, medium strong to very strong **SILTSTONE** and **LIMESTONE** (5-19%).		1	63	17	5	5	0	>25	slightly weathered upto 27.4 m					
212.8 / 25.9			2	63	55	50	0	45	6, 12, 8, 0			22, 24			
211.3 / 27.4			3	63	100	77	0	68	>25, 7, 8	moderately weathered upto 27.7 m, slightly weathered to fresh below		60, 74			
210.5 / 28.2			4	63	100	98	19	95	3, 6, 5			92			
209.4 / 29.3	**END OF BOREHOLE** Notes: 1) 50mm monitoring well installed at 24.4 m after completion. Water Level Readings: Date W. L. Depth(m) Nov. 30/12 12.6														

E = Modulus of Elasticity

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-3

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841033 E 600553

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 203mm
Date: Nov/07/2012

REF. NO.: 1122-110
ENCL NO.: 4

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT / LIMITS	POCKET PEN. (Cu) (kPa)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
230.6 / 0.0	300mm **GRANULAR FILL** sand and gravel, trace silt, hydrocarbon odour, greyish brown, moist, compact.	1	SS	16		230			
230.3 / 0.3	**FILL:** clayey silt, sandy, trace gravel, contains sand pockets, dark grey, moist, very stiff.	2	SS	22				>225	2 36 44 18
229.8 / 0.8	**CLAYEY SILT TILL:** sandy, trace gravel, grey, moist, very stiff to hard.	3	SS	30		229			
		4	SS	20		228		>225	
227.6 / 3.1	**SANDY SILT TILL:** some clay, trace gravel, contains clayey silt seams, grey, moist, compact.	5	SS	12		227			4 44 40 12
226.5 / 4.1	**SAND:** trace silt, trace clay, occasional gravel, brown to greyish brown, wet, compact. disturbed at 4.6 m	6	SS disturbed		W. L. 226.3 m upon completion				0 86 8 6
						225			
		7	SS	17		224			
		8	SS	11		223			
						222			
		9	SS	11		221			
220.9 / 9.8	**END OF BOREHOLE** Notes: 1) Water level at 4.3 m and borehole caved to 4.6m upon completion.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE; 50 100 150 200 250
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
NATURAL UNIT WT (Mg/m^3)

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-4

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841057 E 600515

DRILLING DATA
Method: Hollow Stem Auger
Diameter: 203mm
Date: Nov/07/2012

REF. NO.: 1122-110
ENCL NO.: 5

(m) ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	POCKET PEN. (Cu) (kPa)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
231.0								
0.0	400mm **GRANULAR FILL**	1	SS	11				
230.6 / 0.4	sand and gravel, trace silt, trace clay, brown, wet, compact.							
230.2 / 0.8	**FILL:** clayey silt, sandy, trace gravel, dark grey, moist, stiff. **CLAYEY SILT TILL:** trace gravel, grey, moist, stiff to very stiff	2	SS	29		230	>225	
		3	SS	13		229	150	
		4	SS	14				
227.9 / 3.1	**SANDY SILT TILL:** trace clay, trace gravel, grey, moist, compact to dense.	5	SS	37		228		
						227		
	50mm sand seam at 4.6m	6	SS	20	W. L. 226.4 m upon completion			
225.4 / 5.6	**SAND:** trace silt, trace clay, brown, wet, compact.					225		
		7	SS	15				
223.8 / 7.2	**GRAVELLY SAND:** trace silt, trace clay, contains clayey silt seams/ pockets, grey, wet, compact to very dense.					224		
		8	SS	25		223		34 63 3
						222		
		9	SS	53				
221.3 / 9.8	**END OF BOREHOLE** Notes: 1) Borehole caved to 4.6m and was wet at bottom upon completion.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE; 50 100 150 200 250
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
NATURAL UNIT WT (Mg/m^3)

SPL SOIL LOG 1122-110 410 BRIDGE BOREHOLE LOGS.GPJ SPL.GDT 8/3/13

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: + 3, × 3 : Numbers refer to Sensitivity ○ ε=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-5

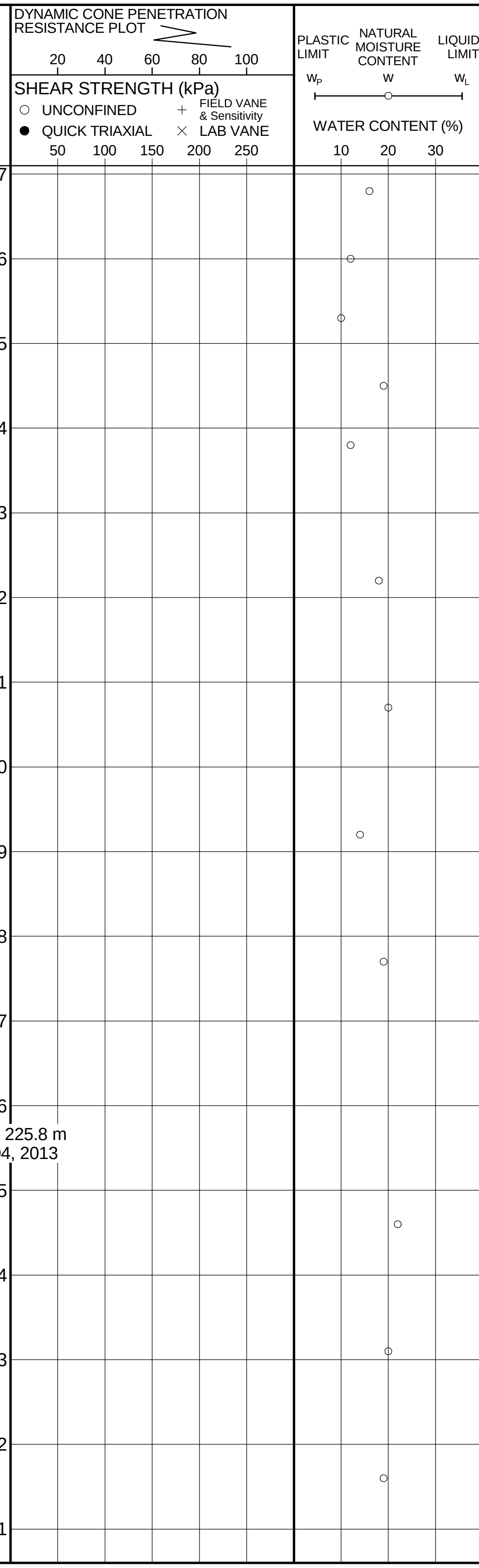
PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841074 E 600585

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter: 203mm/63mm
Date: Nov/27/2012

REF. NO.: 1122-110
ENCL NO.: 6

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
237.1 / 0.0	**TOPSOIL:** 350 mm	1	SS	6	
236.8 / 0.4	**FILL:** sandy silt, trace to some clay, brownish grey, moist, loose to compact; trace gravel, occasional pockets of clayey sil below 0.8 m	2	SS	12	
235.6 / 1.5	**FILL:** silty sand to sandy silt, trace clay, occasional gravel, brown, wet, loose; some clay, trace organics, trace gravel below 1.8 m	3	SS	8	
234.5 / 2.6	**FILL:** clayey silt, sandy, trace to some gravel, brown, moist, compact; trace to some sand, trace organics, trace gravel below 3.0 m	4	SS	10	
		5	SS	16	
232.5 / 4.6	**FILL:** sandy silt, some clay, trace gravel, brown, moist, loose to compact; clayey below 6.1 m	6	SS	15	
		7	SS	6	
229.5 / 7.6	**SAND AND GRAVEL:** trace silt, brown, wet, very dense	8	SS	55	
228.0 / 9.1	**SAND:** trace to some silt, trace clay, brown, wet, compact	9	SS	23	
226.4 / 10.7	**SILT:** some sand to sandy, trace clay, occasional sand seams, brown, wet, dense	10	SS	43	0 20 72 8
224.9 / 12.2	**SAND:** trace to some silt, occasional silt seams, brown, wet, dense	11	SS	44	
		12	SS	38	
221.9 / 15.2	**SANDY GRAVEL:** trace silt, occasional cobble / boulder, grey, wet, very dense	13	SS	68	

W. L. 225.8 m
Jan 04, 2013



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GROUNDWATER ELEVATIONS
Shallow/ Single Installation   Deep/Dual Installation

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity   ○ ε=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-5

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841074 E 600585

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter: 203mm/63mm
Date: Nov/27/2012

REF. NO.: 1122-110
ENCL NO.: 6

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	ELEVATION
220.3					
16.8	**SAND:** trace to some silt, brown, wet, very dense	14	SS	52	220
					219
	silty, occasional cobble, grey below 18.3 m	15	SS	50/ 150 mm	
					218
217.3 / 19.8	**SAND AND GRAVEL:** trace silt, occasional cobble / boulder, grey, wet, very dense	16	SS	50/ 150 mm	217
					216
		17	SS	92	215
		18	SS	50/ 125 mm	214
					213
212.7 / 24.4	**SHALE BEDROCK:** grey, interbedded with siltstone and limestone (Georgian Bay Formation) rock coring started at 24.8 m	19	SS	100/ 125 mm	212
	Refer to Log of Rock Core	1	CORE		211
		2	CORE		210
209.2 / 27.9	**END OF BOREHOLE**				

Notes:
1) Auger drilling ended at 24.5 m and rock coring started at 24.8 m using 63 mm HQ core barrel.
2) 50mm monitoring well installed at 24.4 m after completion.

Water Level Readings:

Date	W. L. Depth(m)
Nov. 30/12	12.1
Jan. 04/13	11.3

Natural moisture content at sample 14 (elev. 220): approx. 20 %

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE (50 100 150 200 250); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30); POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (Mg/m³); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: +3, ×3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF ROCK CORE BH12-5

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841074 E 600585

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter:
Date: 27/11/2012

REF. NO.: 1122-110
ENCL NO.: 6A

(m) ELEV / DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE NUMBER	CORE SAMPLE SIZE (mm)	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)
212.3 / 24.8	**GEORGIAN BAY FORMATION BEDROCK** Moderately weathered to fresh, laminated to thinly bedded, dark grey to grey, weak to medium strong, **SHALE** (91-72%), thinly laminated to medium bedded with slightly weather to fresh, light grey, medium strong to very strong **SILTSTONE** and **LIMESTONE** (9-28%).		1	63	96	80	9	50	6, 4, 10, 8, 4	slightly to moderately weathered		31, 116			
210.8 / 26.3			2	63	100	98	28	95	4, 5, 7, 2, 4	slightly weathred to fresh		30, 213			
209.2 / 27.9	**END OF BOREHOLE** Notes: 1) 50mm monitoring well installed at 24.4 m after completion. Water Level Readings: Date W. L. Depth(m) Nov. 30/12 12.1														

E = Modulus of Elasticity

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-6

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841096 E 600550

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter: 203mm/63mm
Date: Nov/10/2012

REF. NO.: 1122-110
ENCL NO.: 7

SOIL PROFILE

(m) ELEV DEPTH	DESCRIPTION
235.9	
235.8 / 0.1	**TOPSOIL:** 120 mm
	FILL: sandy silt, trace gravel, trace organics, brown, moist, compact
	some gravel below 0.8 m
234.9 / 1.0	**FILL:** clayey silt, some sand, trace organics, brown, moist, very stiff
234.4 / 1.5	**FILL:** sand and gravel, grey to dark grey, very moist, compact
233.6 / 2.3	**FILL:** clayey silt, trace gravel, some sand, trace organics, grey, moist, firm
232.9 / 3.0	**FILL:** silty clay, sandy, some organics, trace gravel, grey, moist, firm to very stiff
	trace organics below 7.6 m
223.7 / 12.2	**SAND:** trace gravel, some silt, brownish grey to grey, wet, loose
222.2 / 13.7	**SANDY SILT TILL:** trace to some clay, grey, wet, compact
220.7 / 15.2	**SAND AND GRAVEL TO SANDY GRAVEL:** trace silt, occasional cobble / boulder, grey, wet, dense to very dense

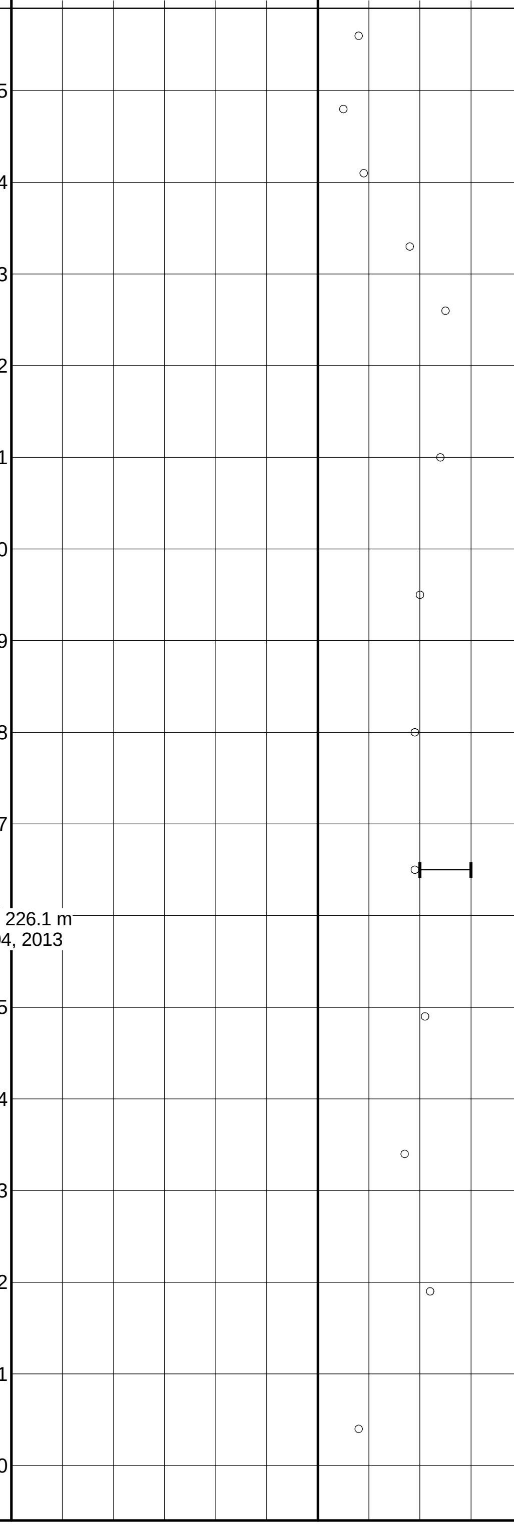
SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	10	
2	SS	17	
3	SS	48	
4	SS	6	
5	SS	6	
6	SS	16	
7	SS	6	
8	SS	9	
9	SS	14	6 24 44 26
10	SS	12	
11	SS	10	
12	SS	16	
13	SS	43	

GROUND WATER CONDITIONS: W. L. 226.1 m Jan 04, 2013

ELEVATION: 235, 234, 233, 232, 231, 230, 229, 228, 227, 225, 224, 223, 222, 221, 220

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE; 50 100 150 200 250
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 10 20 30
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (Mg/m^3)



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GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF BOREHOLE BH12-6

PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
PROJECT LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841096 E 600550

DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter: 203mm/63mm
Date: Nov/10/2012

REF. NO.: 1122-110
ENCL NO.: 7

(m) ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS 0.3 m	ELEVATION	NATURAL MOISTURE CONTENT (WATER CONTENT %)
	SAND AND GRAVEL TO SANDY GRAVEL: trace silt, occasional cobble / boulder, grey, wet, dense to very dense(Continued)	14	SS	49	219	≈9
					218	
		15	SS	51	217	≈6
	frequent cobble / boulder below 19.5 m	16	SS	50/ 50 mm	216	≈14
					215	
214.5 21.4	boulder cored from 21.4 to 22.9 m	17	SS	50/ 125 mm		≈13
		R1	CORE		214	
213.0 22.9	**SAND AND GRAVEL:** trace silt, grey, wet, very dense				213	
					212	
211.5 24.4 211.1 24.8	**SHALE BEDROCK:** grey, interbedded with siltstone and limestone (Georgian Bay Formation) Refer to Log of Rock Core	R2	CORE			

END OF BOREHOLE:
1) Auger drilling ended at 24.4 m and rock coring started at 24.4 m using 63 mm HQ core barrel.
2) 50mm monitoring well installed at 24.4 m after completion.
Water Level Readings:

Date	W. L. Depth(m)
Nov. 30/12	10.3
Jan. 04/13	9.8

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE (50 100 150 200 250); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (Mg/m^3); REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

LOG OF ROCK CORE BH12-6

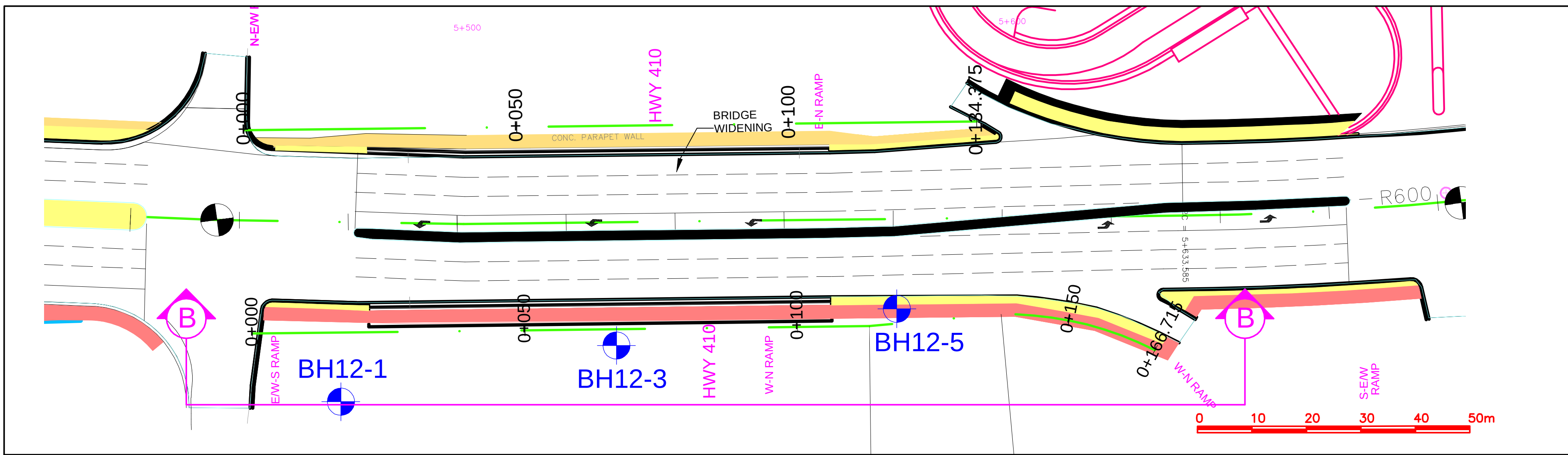
PROJECT: Proposed Widening of Williams Parkway
CLIENT: AECOM Canada Ltd.
LOCATION: Williams Parkway / Hwy 410, Brampton, Ontario
DATUM: Geodetic
BH LOCATION: (See Drawing 1) N 4841096 E 600550

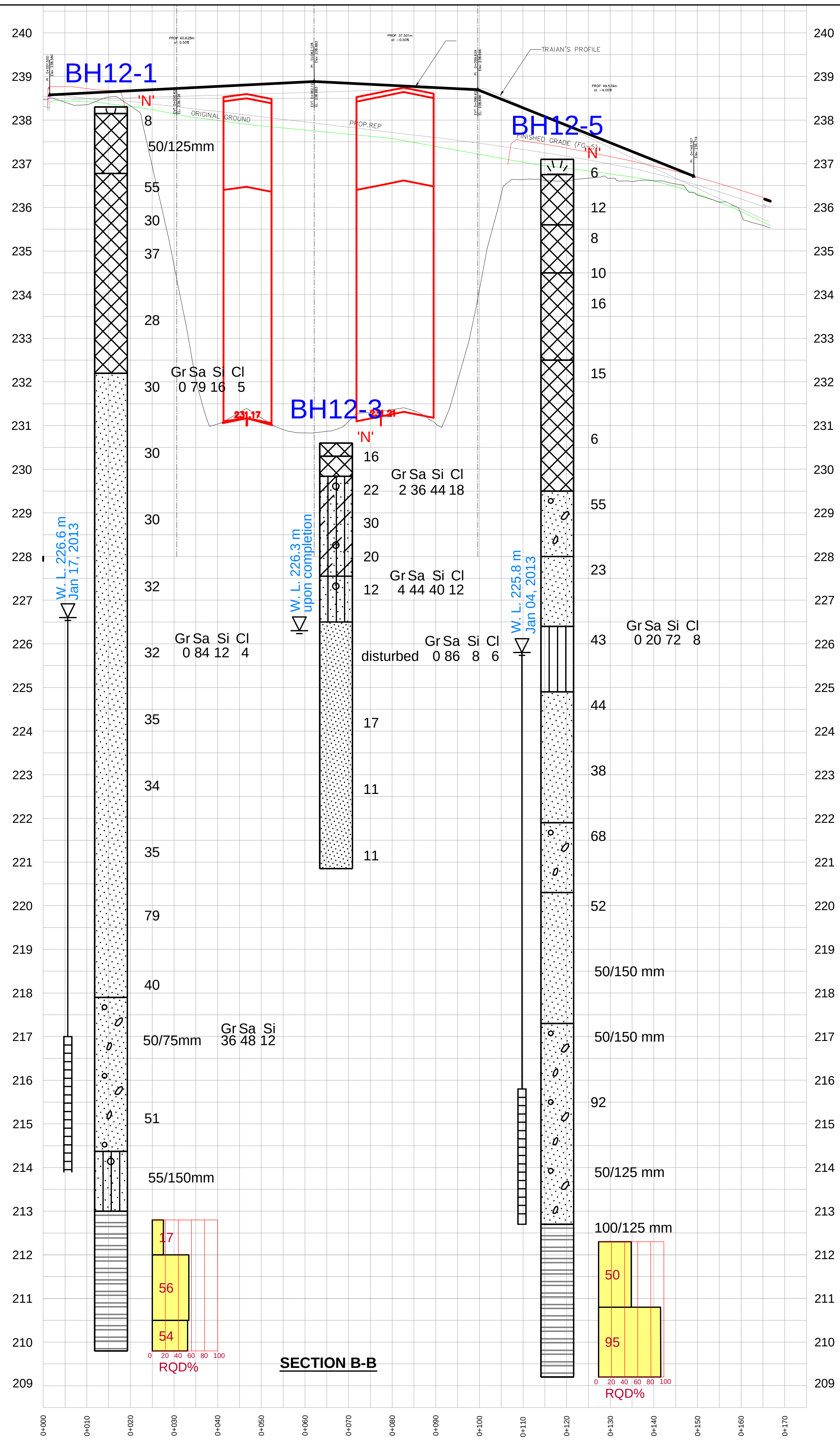
DRILLING DATA
Method: Hollow Stem Auger/HQ Casing
Diameter:
Date: 10/11/2012

REF. NO.: 1122-110
ENCL NO.: 7A

(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE NUMBER	CORE SAMPLE SIZE (mm)	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)
211.5															
211.1 24.4	**GEORGIAN BAY FORMATION BEDROCK**		R2	63	100	100	0	100	8			34			
24.8	Moderately weathered to fresh, laminated to thinly bedded, dark grey to grey, weak to medium strong, **SHALE** (100%), thinly laminated to medium bedded with slightly weather to fresh, light grey, medium strong to very strong **SILTSTONE** and **LIMESTONE** (0%). **END OF BOREHOLE** Notes: 1) 50mm monitoring well installed at 24.4 m after completion. Water Level Readings: Date W. L. Depth(m) Nov. 30/12 10.3														

E = Modulus of Elasticity

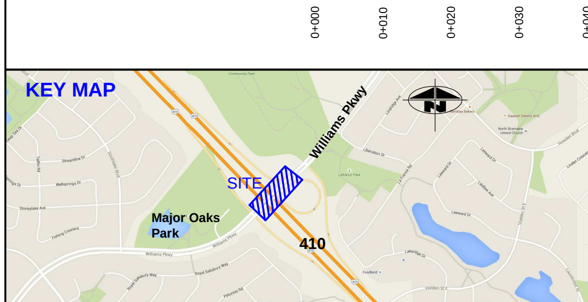




LEGEND

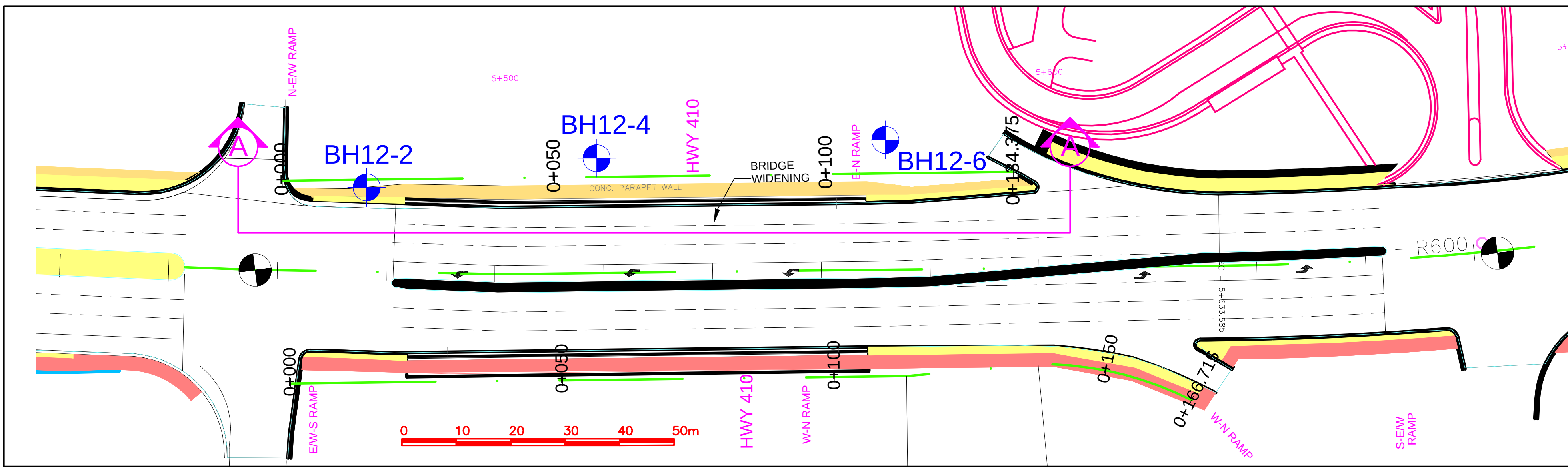
- Topsoil
- Fill
- Sand
- Sand and Gravel
- Sandy Silt Till
- Silty Sand Till
- Clayey Silt Till
- Gravelly Sand
- Silt
- Shale Georgian Bay Formation

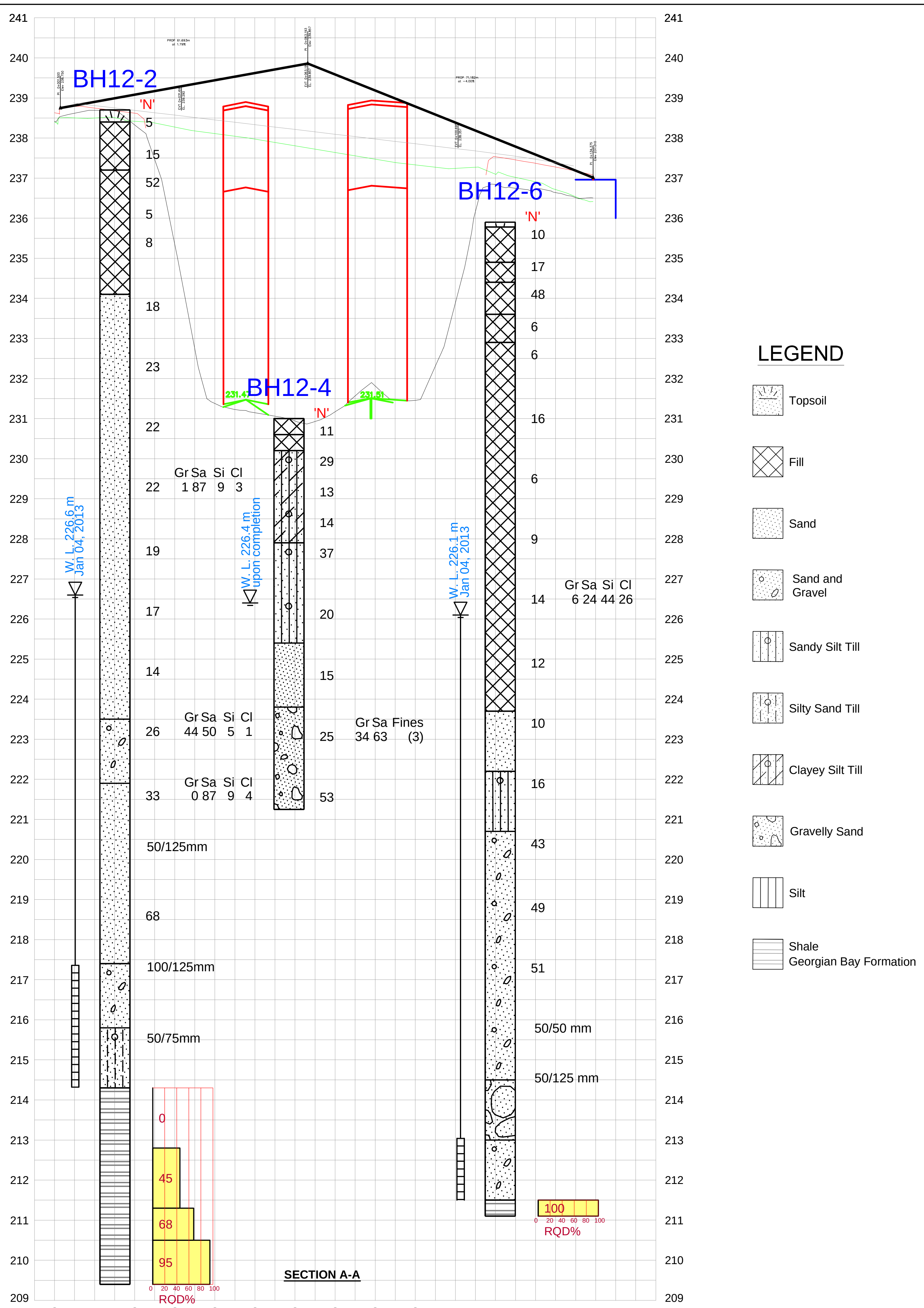
KEY MAP

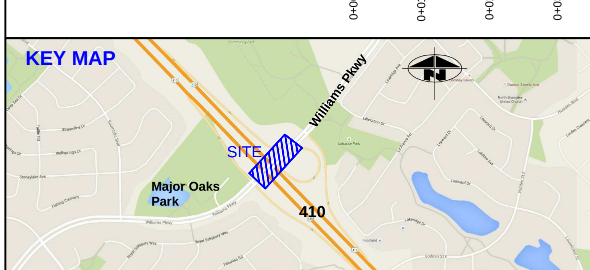


Client: AECOM CANADA LTD.		Project No.: 1122-110B	Drawing No.: 8
Drawn: ZMO	Approved: AS	Title: Geological Section - Section B-B	
Date: September 04, 2015	Scale: As Shown	Project: Geotechnical Investigation and Foundation Design - Proposed Pedestrian Bridges at Hwy 410 Williams Parkway Widening, Brampton, ON	
MTO Geocres No: 30M12-179	Rev: N/A	SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	

T:\Geotech\1100 GeoProjects\1122-110 - Williams Pkwy Widening\Drawings\410\1122-110-Profile -Nov11-2015.dwg

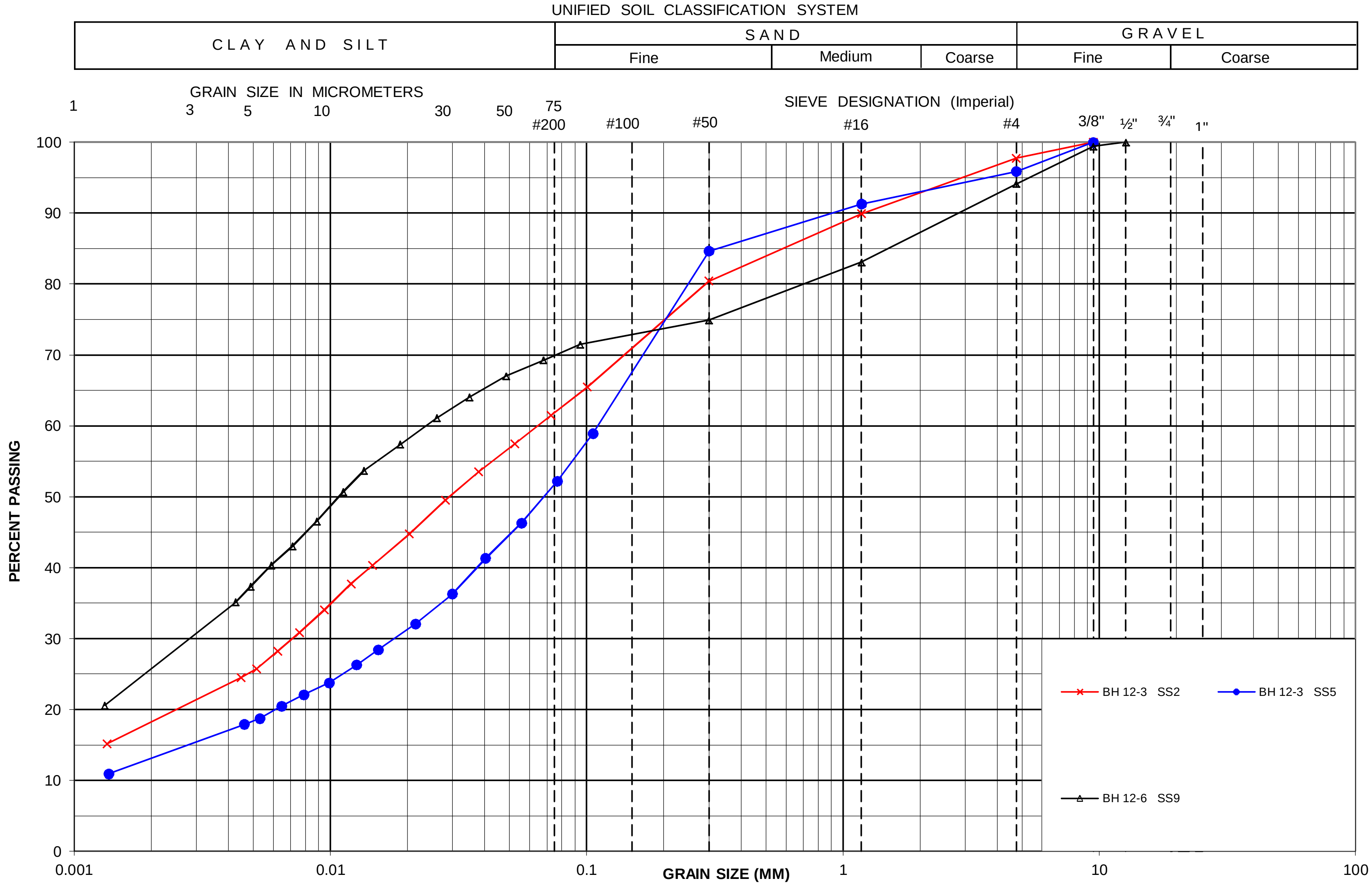






Client: AECOM CANADA LTD.		Project No.: 1122-110B	Drawing No.: 9
Drawn: ZMO	Approved: AS	Title: Geological Section - Section A-A	
Date: September 04, 2015	Scale: As Shown	Project: Geotechnical Investigation and Foundation Design - Proposed Pedestrian Bridges at Hwy 410 Williams Parkway Widening, Brampton, ON	
MTO Geocres No: 30M12-179	Rev: N/A	SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	

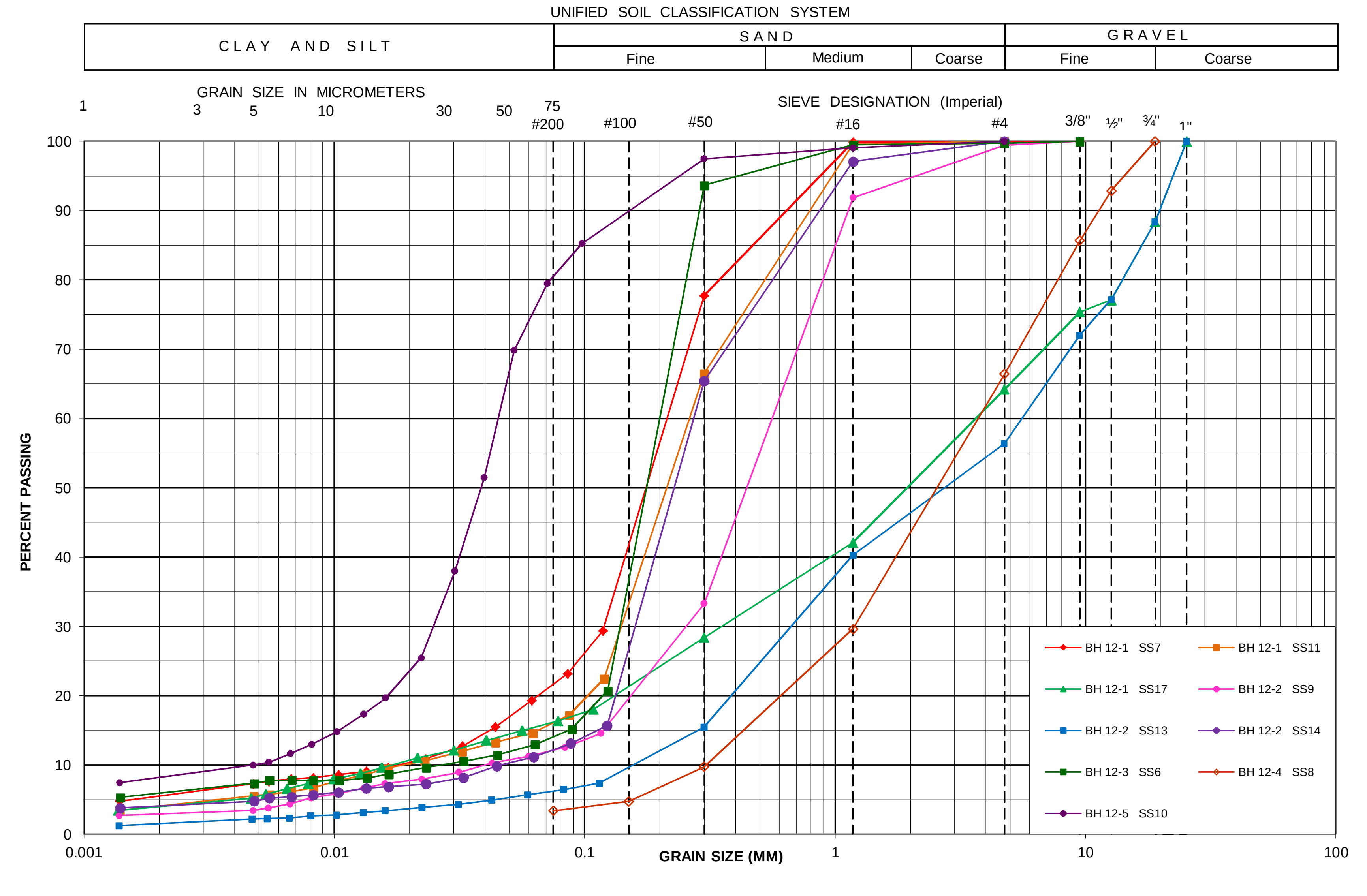
T:\Geotech\1100 GeoProjects\1122-110 - Williams Pkwy Widening\Drawings\410\1122-110-Profile -Nov11-2015.dwg



SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

GRAIN SIZE DISTRIBUTION

Figure No:	10
Project No.	1122-110
Date :	January 11, 2013



SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

GRAIN SIZE DISTRIBUTION

Figure No: 11
Project No. 1122-110
Date : January 11, 2013

Appendix A

Explanation of Terms Used in the Record of Rock Core Log
General Comments on Shale Bedrock in Toronto Area
Photographs of Bedrock Cores

Explanation of Terms Used in the Bedrock Core Log

Strength (ISRM)

Term	Grade	Description	Unconfined Compressive Strength (MPa)	(psi)
Extremely weak rock	RO	Indented by thumbnail	0.25-1.0	36-145
Very weak	R1	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0	145-725
Weak rock	R2	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0-25	725-3625
Medium Strong	R3	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25-50	3625-7250
Strong rock	R4	Specimen require more than one blow of geological hammer to fracture it	50-100	7250-14500
Very strong rock	R5	Specimen requires many blows of geological hammer to fracture it	100-250	14500-36250
Extremely strong rock	R6	Specimen can only be chipped with geological hammer	>250	>36250

Bedding (Geological Society Eng. Group Working Party, 1970. Q.J. of Eng. Geol. Vol. 3)

Term	Bed Thickness	
Very thickly bedded	>2 m	>6.5 ft
Thickly bedded	600 mm-2 m	2.00-6.50 ft
Medium bedded	200 mm-600 mm	0.65-2.00 ft
Thinly bedded	60 mm-200 mm	0.20-0.65 ft
Very thinly bedded	20 mm-60 mm	0.06-0.20 ft
Laminated	6 mm-20 mm	0.02-0.06 ft
Thinly laminated	<6 mm	<0.02 ft

TCR (Total Core Recovery)

Sum of lengths of rock core recovered from a core run, divided by the length of the core run and expressed as a percentage.

SCR (Solid Core Rocovery)

Sum length of solid, full diameter drill core recovered expressed as a percentage of the total length of the core run.

RQD (Rock Quality Designation, after Deere, 1968)

Sum of lengths of pieces of rock core measured along centreline of core equal to or greater than 100 mm from a core run, divided by the length of the core run and expressed as a percentage. Core fractured by drilling is considered intact. RQD normally quoted for N-size or H-size core.

RQD(%)	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very poor

Weathering (ISRM)

Term	Grade	Description
Fresh	W1	No visible sign of rock material weathering
Slightly weathered	W2	Discolouration indicates weathering of rock material and discontinuity surface. All the rock material may be discoloured by weathering and may be somewhat weaker than in its fresh condition
Moderately weathered	W3	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a corestone
Highly weathered	W4	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones
Completely weathered	W5	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact
Residual soil	W6	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported

(FI) Fracture Index

Expressed as the number of discontinuities per 300mm (1 ft). Excludes drill-induced fractures and fragmented zones. Reported as ">25" if frequency exceeds 25 fractures/0.3m.

Broken Zone

Zone of full diameter core of very low RQD which may include some drill-induced fractures.

Fragmented Zone

Zone where core is less than full diameter and RQD = 0.

Discontinuity Spacing (ISRM)

Term	Average Spacing	
Extremely widely spaced	>6 m	>20.00 ft
Very widely spaced	2 m-6 m	6.50-20.00 ft
Widely spaced	600 mm-2 m	2.00-6.50 ft
Moderately spaced	200 mm-600 mm	0.65-2.00 ft
Closely spaced	60 mm-200 mm	0.20-0.65 ft
Very closely spaced	20 mm-60 mm	0.06-0.20 ft
Extremely closely spaced	<20 mm	>0.06 ft

Note: Excludes drill-induced fractures and fragmented rock.

Discontinuity Orientation

Discontinuity, fracture and bedding plane orientations are cited as the acute angle measured with respect to the core axis. Fractures perpendicular to the core axis are at 90° and those parallel to the core axis are at 0°.

Abbreviations

B – Bedding	J – Joint	PL – Planar
R – Ridged / Rough	SM - Smooth	

General Comments – Bedrock in Metro Toronto Area

The bedrock that makes spread footings or caissons a popular choice for high-rise foundation support is a shale or shale limestone composition. The highest member, the Queenston Formation, is generally found west of Toronto, while the Georgian Bay Formation underlies most of Metro Toronto, with the Collingwood Formation east of Toronto. The Queenston is, relatively speaking, the weaker of the three formations that are likely to support caissons or footings.

The Georgian Bay as well as the Queenston and Collingwood Formation are of Middle Ordovician Age. It is defined as the rock unit that overlies the bluish grey shales of the Collingwood Formation and is in turn overlain by the red shale of the Queenston Formation. The Georgian Bay Formation consists of bluish and grey shale with interbeds of sandstone, limestone and dolostone. Towards the west where the Georgian Bay formation underlies the Queenston Formation, the limestone content increases significantly and limestone and/or sandstone may comprise as much as 70 to 90 percent of the bedrock. The hard layers are usually less than about 100 to 150 mm thick but some layers are much thicker. The thicker layers have been observed to be as much as 750 to 900 mm at some sites. The layers are actually lenses and they can vary significantly in thickness over short distances.

The upper portion of the bedrock is commonly weathered for a depth of 600 to 1000 mm and within this weathered zone hard limestone layers or lenses are common. These hard limestone layers can result in contractual problems for augers, and can provide misleading bedrock elevations. Where the weathering is more extensive a shale till layer may be found above the bedrock. In the sound bedrock, the limestone, sandstone, dolostone is hard to very hard.

Stress relief features such as folds and faults are common in the bedrock. In these features, the rock is heavily fractured and sheared, and contains layers of shale rubble and clay. Weathering is much deeper than the surrounding rock in these features and often there is a lateral migration of the stress relief features resulting in sound unweathered bedrock overlying fractured and weather bedrock. The stress relief features are usually in the order of 4 to 6 m wide, but the depth can vary from 4 to 5 m to in excess of 10 m. These features occur randomly.

The bedrock contains significant high locked in horizontal stresses. These stresses can impose significant loads on tunnel walls but the slower rate of construction for basements allows for a relaxation of these stresses and they are not normally a problem for basement construction.

Groundwater seepage below the top 1000 mm is generally small, however, at several locations in Toronto and Mississauga large quantities have been encountered.

Bedding joints in the bedrock are very close-to-close, smooth planar in the shale and rough planar in the limestone. Significant vertical jointing is common.

Where the bedrock was cored, a detailed description of the rock core is appended to the borehole log.

Design features related to the bedrock are discussed in other sections of this report, and these general comments must be considered with these comments.

Methane gas exists in the bedrock, normally below the top 1000 mm and more concentrated with depth. Appropriate care and monitoring is essential in all confined bedrock excavations, particularly caissons and tunnels.

Photo A1: BH12-1 Bedrock Core

Run 1 83' 7" ~ 86' 1"

Run 2 86' 1" ~ 91' 1"

Run 3 91' 1" ~ 93' 7"

Photo A2: BH12-2 Bedrock Core
Run 1: 80' – 85'
Run 2: 85' – 90'

Photo A3: BH12-2 Bedrock Core
Run 3: 90' – 92'7"
Run 4: 92'7" – 96'1"

Photo A4: BH12-5 Bedrock Core
Run 1: 81'6" – 86'6"
Run 2: 86'6" – 91'6"

Photo A5: BH12-6 Boulder / Bedrock Core
Run 1: 70’ – 75’ (Boulder)
Run 2: 80’ – 81’2” (Shale Bedrock)

Appendix B

Environmental Testing of Soil - Certificates of Analyses

Certificate of Analysis

AGAT WORK ORDER: 12T672355
PROJECT NO: 1122-110

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

CLIENT NAME: SPL CONSULTANTS **ATTENTION TO: Alka Sangar**

O. Reg. 153(511) - Metals & Inorganics (Soil)

DATE RECEIVED: 2012-12-11 **DATE REPORTED: 2012-12-18**

Parameter	Unit	G / S	RDL	BH12-130(SS-2)	BH12-1-SS3
			SAMPLE TYPE:	Soil	Soil
			DATE SAMPLED:	12/10/2012	12/10/2012
				4013563	4013564
Antimony	µg/g		0.8	<0.8	<0.8
Arsenic	µg/g		1	4	3
Barium	µg/g		2	44	20
Beryllium	µg/g		0.5	<0.5	<0.5
Boron	µg/g		5	<5	<5
Boron (Hot Water Soluble)	µg/g		0.10	0.23	0.10
Cadmium	µg/g		0.5	<0.5	<0.5
Chromium	µg/g		2	13	9
Cobalt	µg/g		0.5	6.3	4.2
Copper	µg/g		1	26	19
Lead	µg/g		1	10	5
Molybdenum	µg/g		0.5	<0.5	<0.5
Nickel	µg/g		1	13	7
Selenium	µg/g		0.4	<0.4	<0.4
Silver	µg/g		0.2	<0.2	<0.2
Thallium	µg/g		0.4	<0.4	<0.4
Uranium	µg/g		0.5	<0.5	<0.5
Vanadium	µg/g		1	21	16
Zinc	µg/g		5	45	26
Chromium VI	µg/g		0.2	<0.2	<0.2
Cyanide	µg/g		0.040	<0.040	<0.040
Mercury	µg/g		0.10	<0.10	<0.10
Electrical Conductivity (2:1)	mS/cm		0.005	0.658	0.748
Sodium Adsorption Ratio	NA		NA	6.63	14.0
pH, 2:1 CaCl2 Extraction	pH Units		NA	7.72	7.97

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard
4013563-4013564 EC & SAR were determined on the DI water extract obtained from the 2:1 leaching procedure (2 parts DI water:1 part soil). pH was determined on the 0.01M CaCl2 extract prepared at 2:1 ratio.

Certified By: Tony Yaach

AGAT Laboratories

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Quality Assurance

CLIENT NAME: SPL CONSULTANTS **AGAT WORK ORDER: 12T672355**

PROJECT NO: 1122-110 **ATTENTION TO: Alka Sangar**

Soil Analysis

RPT Date: Dec 18, 2012			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper
O. Reg. 153(511) - Metals & Inorganics (Soil)															
Antimony	1		< 0.8	< 0.8	0.0%	< 0.8	112%	70%	130%	83%	80%	120%	83%	70%	130%
Arsenic	1		4	4	0.0%	< 1	100%	70%	130%	109%	80%	120%	111%	70%	130%
Barium	1		64	63	1.6%	< 2	100%	70%	130%	103%	80%	120%	108%	70%	130%
Beryllium	1		0.5	0.6	18.2%	< 0.5	88%	70%	130%	98%	80%	120%	93%	70%	130%
Boron	1		< 5	< 5	0.0%	< 5	82%	70%	130%	110%	80%	120%	96%	70%	130%
Boron (Hot Water Soluble)	1		<0.10	<0.10	0.0%	< 0.10	113%	60%	140%	92%	70%	130%	94%	60%	140%
Cadmium	1		< 0.5	< 0.5	0.0%	< 0.5	99%	70%	130%	94%	80%	120%	101%	70%	130%
Chromium	1		20	20	0.0%	< 2	101%	70%	130%	115%	80%	120%	111%	70%	130%
Cobalt	1		7.6	7.6	0.0%	< 0.5	98%	70%	130%	104%	80%	120%	102%	70%	130%
Copper	1		22	22	0.0%	< 1	103%	70%	130%	115%	80%	120%	97%	70%	130%
Lead	1		8	8	0.0%	< 1	103%	70%	130%	101%	80%	120%	96%	70%	130%
Molybdenum	1		< 0.5	< 0.5	0.0%	< 0.5	100%	70%	130%	101%	80%	120%	107%	70%	130%
Nickel	1		16	17	6.1%	< 1	104%	70%	130%	104%	80%	120%	99%	70%	130%
Selenium	1		< 0.4	< 0.4	0.0%	< 0.4	80%	70%	130%	100%	80%	120%	107%	70%	130%
Silver	1		< 0.2	< 0.2	0.0%	< 0.2	86%	70%	130%	104%	80%	120%	107%	70%	130%
Thallium	1		< 0.4	< 0.4	0.0%	< 0.4	92%	70%	130%	98%	80%	120%	102%	70%	130%
Uranium	1		< 0.5	< 0.5	0.0%	< 0.5	97%	70%	130%	91%	80%	120%	87%	70%	130%
Vanadium	1		31	31	0.0%	< 1	101%	70%	130%	110%	80%	120%	114%	70%	130%
Zinc	1		37	37	0.0%	< 5	97%	70%	130%	107%	80%	120%	100%	70%	130%
Chromium VI	1		< 0.2	< 0.2	0.0%	< 0.2	91%	70%	130%	98%	80%	120%	96%	70%	130%
Cyanide	1		< 0.040	< 0.040	0.0%	< 0.040	103%	70%	130%	110%	80%	120%	95%	70%	130%
Mercury	1		< 0.10	< 0.10	0.0%	< 0.10	102%	70%	130%	98%	80%	120%	93%	70%	130%
Electrical Conductivity (2:1)	1		1.80	1.85	2.7%	< 0.005	99%	90%	110%	NA			NA		
Sodium Adsorption Ratio	1		2.68	2.53	5.8%	NA	NA			NA			NA		
pH, 2:1 CaCl2 Extraction	1		7.69	7.73	0.5%	NA	98%	90%	110%	NA			NA		

Comments: NA signifies Not Applicable.

Certified By:

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Method Summary

CLIENT NAME: SPL CONSULTANTS **AGAT WORK ORDER: 12T672355**

PROJECT NO: 1122-110 **ATTENTION TO: Alka Sangar**

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Antimony	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Arsenic	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Barium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Beryllium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Boron	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Boron (Hot Water Soluble)	MET-93-6104	EPA SW 846 6010C; MSA, Part 3, Ch.21	ICP/OES
Cadmium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Chromium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Cobalt	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Copper	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Lead	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Molybdenum	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Nickel	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Selenium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Silver	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Thallium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Uranium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Vanadium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Zinc	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Chromium VI	INOR-93-6029	SM 3500 B; MSA Part 3, Ch. 25	SPECTROPHOTOMETER
Cyanide	INOR-93-6052	MOE CN-3015 & E 3009 A;SM 4500 CN	TECHNICON AUTO ANALYZER
Mercury	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Sodium Adsorption Ratio	INOR-93-6007	McKeague 4.12 & 3.26 & EPA SW-846 6010C	ICP/OES
pH, 2:1 CaCl2 Extraction	INOR-93-6031	MSA part 3 & SM 4500-H+ B	PH METER

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Certificate of Analysis

AGAT WORK ORDER: 12T672352
PROJECT NO: 1122-110

CLIENT NAME: SPL CONSULTANTS
ATTENTION TO: Alka Sangar

O. Reg. 153(511) - Metals & Inorganics (Soil)

DATE RECEIVED: 2012-12-11
DATE REPORTED: 2012-12-18

			SAMPLE DESCRIPTION:	12-132(SS-6)	12-103(SS-3)	12-11(SS-6)	12-6 (SS-5)	12-115 (SS-3)	12-105 (SS-2)	12-113 (SS-2)	12-123 (SS-4)
			SAMPLE TYPE:	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			DATE SAMPLED:	12/10/2012	12/10/2012	12/10/2012	12/10/2012	12/10/2012	12/10/2012	12/10/2012	12/10/2012
Parameter	Unit	G / S	RDL	4013539	4013540	4013541	4013542	4013543	4013544	4013545	4013546
Antimony	µg/g		0.8	<0.8	<0.8	<0.8	<0.8	<0.8	<0.8	<0.8	<0.8
Arsenic	µg/g		1	3	5	4	6	4	5	6	5
Barium	µg/g		2	18	64	58	46	73	52	53	69
Beryllium	µg/g		0.5	<0.5	0.6	0.5	0.9	0.6	<0.5	0.6	0.7
Boron	µg/g		5	<5	6	<5	9	7	7	6	6
Boron (Hot Water Soluble)	µg/g		0.10	<0.10	0.26	0.17	0.21	0.27	0.20	0.13	0.24
Cadmium	µg/g		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Chromium	µg/g		2	6	17	16	24	20	14	17	19
Cobalt	µg/g		0.5	3.9	10.1	8.7	13.7	11.2	8.4	11.1	10.7
Copper	µg/g		1	18	31	34	31	29	30	43	30
Lead	µg/g		1	5	10	11	7	8	7	9	12
Molybdenum	µg/g		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Nickel	µg/g		1	7	20	18	31	22	17	22	21
Selenium	µg/g		0.4	<0.4	<0.4	0.4	<0.4	<0.4	<0.4	<0.4	<0.4
Silver	µg/g		0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Thallium	µg/g		0.4	<0.4	<0.4	<0.4	<0.4	<0.4	<0.4	<0.4	<0.4
Uranium	µg/g		0.5	<0.5	<0.5	<0.5	<0.5	0.6	<0.5	<0.5	<0.5
Vanadium	µg/g		1	9	24	23	30	27	20	24	26
Zinc	µg/g		5	24	53	80	71	55	45	55	61
Chromium VI	µg/g		0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Cyanide	µg/g		0.040	<0.040	<0.040	<0.040	<0.040	<0.040	<0.040	<0.040	<0.040
Mercury	µg/g		0.10	<0.10	<0.10	<0.10	<0.10	<0.10	<0.10	<0.10	<0.10
Electrical Conductivity (2:1)	mS/cm		0.005	0.095	0.231	0.387	0.355	0.380	0.861	0.444	0.525
Sodium Adsorption Ratio	NA		NA	0.715	0.197	0.338	5.33	1.04	8.43	5.66	2.30
pH, 2:1 CaCl2 Extraction	pH Units		NA	7.85	7.64	7.49	7.69	7.78	7.94	7.79	7.67

Certified By:

Certificate of Analysis

AGAT WORK ORDER: 12T672352
PROJECT NO: 1122-110

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CLIENT NAME: SPL CONSULTANTS

ATTENTION TO: Alka Sangar

O. Reg. 153(511) - Metals & Inorganics (Soil)

DATE RECEIVED: 2012-12-11

DATE REPORTED: 2012-12-18

			SAMPLE DESCRIPTION:	12-119 (SS-3)	12-108 (SS-4)	12-126 (SS-5)	12-136 (SS-2)
			SAMPLE TYPE:	Soil	Soil	Soil	Soil
			DATE SAMPLED:	12/10/2012	12/10/2012	12/10/2012	12/10/2012
Parameter	Unit	G / S	RDL	4013547	4013548	4013549	4013550
Antimony	µg/g		0.8	<0.8	<0.8	<0.8	<0.8
Arsenic	µg/g		1	5	5	2	5
Barium	µg/g		2	62	132	12	65
Beryllium	µg/g		0.5	0.6	0.7	<0.5	0.6
Boron	µg/g		5	7	13	<5	6
Boron (Hot Water Soluble)	µg/g		0.10	0.12	1.84	<0.10	0.11
Cadmium	µg/g		0.5	<0.5	<0.5	<0.5	<0.5
Chromium	µg/g		2	17	17	5	17
Cobalt	µg/g		0.5	10.8	10.2	2.9	11.9
Copper	µg/g		1	32	15	11	31
Lead	µg/g		1	8	8	3	8
Molybdenum	µg/g		0.5	<0.5	0.7	<0.5	<0.5
Nickel	µg/g		1	22	23	5	22
Selenium	µg/g		0.4	<0.4	<0.4	<0.4	<0.4
Silver	µg/g		0.2	<0.2	<0.2	<0.2	<0.2
Thallium	µg/g		0.4	<0.4	<0.4	<0.4	<0.4
Uranium	µg/g		0.5	<0.5	0.8	<0.5	<0.5
Vanadium	µg/g		1	23	23	10	23
Zinc	µg/g		5	54	49	16	64
Chromium VI	µg/g		0.2	<0.2	<0.2	<0.2	<0.2
Cyanide	µg/g		0.040	<0.040	<0.040	<0.040	<0.040
Mercury	µg/g		0.10	<0.10	<0.10	<0.10	<0.10
Electrical Conductivity (2:1)	mS/cm		0.005	0.250	0.456	0.141	0.187
Sodium Adsorption Ratio	NA		NA	0.861	0.961	1.61	0.919
pH, 2:1 CaCl2 Extraction	pH Units		NA	7.80	7.71	7.81	7.70

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

4013539-4013550 EC & SAR were determined on the DI water extract obtained from the 2:1 leaching procedure (2 parts DI water:1 part soil). pH was determined on the 0.01M CaCl2 extract prepared at 2:1 ratio.

Certified By:

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Quality Assurance

CLIENT NAME: SPL CONSULTANTS

AGAT WORK ORDER: 12T672352

PROJECT NO: 1122-110

ATTENTION TO: Alka Sangar

Soil Analysis

RPT Date: Dec 18, 2012			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper
O. Reg. 153(511) - Metals & Inorganics (Soil)															
Antimony	1	4013540	< 0.8	< 0.8	0.0%	< 0.8	102%	70%	130%	80%	80%	120%	75%	70%	130%
Arsenic	1	4013540	5	5	0.0%	< 1	99%	70%	130%	116%	80%	120%	102%	70%	130%
Barium	1	4013540	64	64	0.0%	< 2	99%	70%	130%	108%	80%	120%	94%	70%	130%
Beryllium	1	4013540	0.6	0.6	0.0%	< 0.5	90%	70%	130%	113%	80%	120%	103%	70%	130%
Boron	1	4013540	6	6	0.0%	< 5	77%	70%	130%	112%	80%	120%	99%	70%	130%
Boron (Hot Water Soluble)	1	4013539	<0.10	<0.10	0.0%	< 0.10	98%	60%	140%	93%	70%	130%	101%	60%	140%
Cadmium	1	4013540	< 0.5	< 0.5	0.0%	< 0.5	103%	70%	130%	118%	80%	120%	104%	70%	130%
Chromium	1	4013540	17	17	0.0%	< 2	93%	70%	130%	110%	80%	120%	100%	70%	130%
Cobalt	1	4013540	10.1	9.9	2.0%	< 0.5	101%	70%	130%	112%	80%	120%	96%	70%	130%
Copper	1	4013540	31	30	3.3%	< 1	103%	70%	130%	120%	80%	120%	103%	70%	130%
Lead	1	4013540	10	11	9.5%	< 1	102%	70%	130%	111%	80%	120%	99%	70%	130%
Molybdenum	1	4013540	< 0.5	< 0.5	0.0%	< 0.5	101%	70%	130%	114%	80%	120%	107%	70%	130%
Nickel	1	4013540	20	27	29.8%	< 1	105%	70%	130%	111%	80%	120%	97%	70%	130%
Selenium	1	4013540	< 0.4	< 0.4	0.0%	< 0.4	98%	70%	130%	118%	80%	120%	106%	70%	130%
Silver	1	4013540	< 0.2	< 0.2	0.0%	< 0.2	81%	70%	130%	118%	80%	120%	104%	70%	130%
Thallium	1	4013540	< 0.4	< 0.4	0.0%	< 0.4	92%	70%	130%	107%	80%	120%	99%	70%	130%
Uranium	1	4013540	< 0.5	< 0.5	0.0%	< 0.5	92%	70%	130%	104%	80%	120%	97%	70%	130%
Vanadium	1	4013540	24	24	0.0%	< 1	107%	70%	130%	111%	80%	120%	98%	70%	130%
Zinc	1	4013540	53	54	1.9%	< 5	97%	70%	130%	115%	80%	120%	101%	70%	130%
Chromium VI	1	4013545	< 0.2	< 0.2	0.0%	< 0.2	91%	70%	130%	99%	80%	120%	99%	70%	130%
Cyanide	1		< 0.040	< 0.040	0.0%	< 0.040	103%	70%	130%	110%	80%	120%	95%	70%	130%
Mercury	1	4013540	< 0.10	< 0.10	0.0%	< 0.10	101%	70%	130%	118%	80%	120%	106%	70%	130%
Electrical Conductivity (2:1)	1	4013539	0.095	0.099	4.1%	< 0.005	99%	90%	110%	NA			NA		
Sodium Adsorption Ratio	1	4013539	0.713	0.756	5.8%	NA	NA			NA			NA		
pH, 2:1 CaCl2 Extraction	1	4013540	7.64	7.60	0.5%	NA	98%	90%	110%	NA			NA		

Comments: NA signifies Not Applicable.

Certified By:

AGAT Laboratories

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Method Summary

CLIENT NAME: SPL CONSULTANTS

AGAT WORK ORDER: 12T672352

PROJECT NO: 1122-110

ATTENTION TO: Alka Sangar

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Antimony	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Arsenic	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Barium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Beryllium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Boron	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Boron (Hot Water Soluble)	MET-93-6104	EPA SW 846 6010C; MSA, Part 3, Ch.21	ICP/OES
Cadmium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Chromium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Cobalt	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Copper	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Lead	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Molybdenum	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Nickel	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Selenium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Silver	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Thallium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Uranium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Vanadium	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Zinc	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Chromium VI	INOR-93-6029	SM 3500 B; MSA Part 3, Ch. 25	SPECTROPHOTOMETER
Cyanide	INOR-93-6052	MOE CN-3015 & E 3009 A;SM 4500 CN	TECHNICON AUTO ANALYZER
Mercury	MET-93-6103	EPA SW-846 3050B & 6020A	ICP-MS
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Sodium Adsorption Ratio	INOR-93-6007	McKeague 4.12 & 3.26 & EPA SW-846 6010C	ICP/OES
pH, 2:1 CaCl2 Extraction	INOR-93-6031	MSA part 3 & SM 4500-H+ B	PH METER

Appendix C

Sulphate and Corrosivity Testing of Soil - Certificates of Analyses

Certificate of Analysis

AGAT WORK ORDER: 13T682212
PROJECT NO: 1122-10

AGAT Laboratories
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CLIENT NAME: SPL CONSULTANTS

ATTENTION TO: Alka Sangar

Corrosivity Package

DATE RECEIVED: 2013-01-23

DATE REPORTED: 2013-01-30

			SAMPLE DESCRIPTION:	BH12-2 SS10	BH12-3 SS7	BH12-12 SS8	BH12-6 SS6
			SAMPLE TYPE:	Soil	Soil	Soil	Soil
			DATE SAMPLED:	1/22/2013	1/22/2013	1/22/2013	1/22/2013
Parameter	Unit	G / S	RDL	4089048	4089049	4089051	4089053
Sulphide*	%		0.01	0.03	0.03	0.03	0.02
Chloride (2:1)	µg/g		2.00	21.5	24.6	55.8	82.2
Sulphate (2:1)	µg/g		2.00	70.1	58.8	28.9	37.5
pH (2:1)	pH Units		N/A	8.93	8.64	8.09	8.22
Electrical Conductivity (2:1)	mS/cm		0.005	0.211	0.164	0.289	0.295
Resistivity (2:1)	ohm.cm		1	4740	6100	3460	3390
Redox Potential (2:1)	mV		5	120	91	102	95

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard
4089048-4089053 * 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:

Certificate of Analysis

AGAT WORK ORDER: 13T682212
PROJECT NO: 1122-10

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CLIENT NAME: SPL CONSULTANTS

ATTENTION TO: Alka Sangar

pH, Sulphate (soil)

DATE RECEIVED: 2013-01-23

DATE REPORTED: 2013-01-30

			SAMPLE DESCRIPTION:	BH12-102 SS4	BH12-106 SS4	BH12-114 SS5	BH12-119 SS6	BH12-126 SS6	BH12-128 SS4	BH12-130 SS5	BH12-134 SS6
			SAMPLE TYPE:	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			DATE SAMPLED:	1/22/2013	1/22/2013	1/22/2013	1/22/2013	1/22/2013	1/22/2013	1/22/2013	1/22/2013
Parameter	Unit	G / S	RDL	4089033	4089041	4089042	4089043	4089044	4089045	4089046	4089047
Sulphate (2:1)	µg/g		2.00	29.4	39.8	462	312	237	15.1	230	376
pH, 2:1 CaCl2 Extraction	pH Units			7.90	7.86	7.84	7.92	8.09	7.94	7.86	7.87

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

Certified By:

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Quality Assurance

CLIENT NAME: SPL CONSULTANTS

AGAT WORK ORDER: 13T682212

PROJECT NO: 1122-10

ATTENTION TO: Alka Sangar

Soil Analysis

RPT Date: Jan 30, 2013

PARAMETER	Batch	Sample Id	DUPLICATE Dup #1	DUPLICATE Dup #2	DUPLICATE RPD	Method Blank	REFERENCE MATERIAL Measured Value	REFERENCE MATERIAL Acceptable Limits Lower	REFERENCE MATERIAL Acceptable Limits Upper	METHOD BLANK SPIKE Recovery	METHOD BLANK SPIKE Acceptable Limits Lower	METHOD BLANK SPIKE Acceptable Limits Upper	MATRIX SPIKE Recovery	MATRIX SPIKE Acceptable Limits Lower	MATRIX SPIKE Acceptable Limits Upper
pH, Sulphate (soil)															
Sulphate (2:1)	40890	4089045	30.1	28.9	4.3%	< 2	94%	80%	120%	98%	80%	120%	99%	70%	130%
pH, 2:1 CaCl2 Extraction	1	4089033	7.86	7.88	0.3%	NA	102%	80%	120%	NA	0%	0%	NA	0%	0%
Corrosivity Package															
Sulphide*	1	4089048	0.03	0.03	0.0%	< 0.01	102%	80%	120%	NA			NA		
Chloride (2:1)	40890	4089045	122	113	7.8%	< 2	91%	80%	120%	94%	80%	120%	103%	70%	130%
Sulphate (2:1)	40890	4089045	30.1	28.9	4.3%	< 2	94%	80%	120%	98%	80%	120%	99%	70%	130%
pH (2:1)	1	4089048	8.93	8.95	0.2%	N/A	100%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	1	4089048	0.211	0.224	6.0%	< 0.005	109%	90%	110%	NA			NA		
Redox Potential (2:1)	1	4089048	120	129	7.2%	< 5	102%	70%	130%	NA			NA		

Comments: NA - Not Applicable.

Certified By:

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Method Summary

CLIENT NAME: SPL CONSULTANTS

AGAT WORK ORDER: 13T682212

PROJECT NO: 1122-10

ATTENTION TO: Alka Sangar

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
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH, 2:1 CaCl2 Extraction	INOR-93-6031	MSA part 3 & SM 4500-H+ B	PH METER