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REPORT ON

GEOTECHNICAL INVESTIGATION PROPOSED SEWAGE PUMPING STATION AND RELATED SEWER/FORCEMAIN LINES TOWN OF CARLETON PLACE, ONTARIO

Submitted to:

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REPORT



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Maxxam Analytics Report No. R3358428

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed sewage pumping station and related sewer and forcemain lines in Carleton Place, Ontario. The purpose of the subsurface investigation was to determine the soil and groundwater conditions at the site by means of 7 boreholes and 2 probeholes located at the proposed pumping station location as well as along the proposed sewer/forcemain alignment. Based on an interpretation of the factual information obtained, engineering guidelines on the geotechnical design aspects of this project are provided, including construction consideration which could affect design decisions.

The reader is referred to the “Important Information and Limitations of This Report” which follows the text but forms an integral part of this document.



2.0 DESCRIPTION OF PROJECT AND SITE

The following is understood about the project and site:

- The new pumping station will be located on the south side of Highway 7, immediately east of the Rona property.
- The wet wells of the pumping station will extend about 9.5 metres below the existing grade, and about 10.5 metres below the proposed grade.
- The control building and dry well will be constructed immediately adjacent to the wet wells and will consist of an integrated one storey building with one basement level.
- Approximately 350 metres of new sanitary sewer (up to 525 millimetres in diameter) will be constructed to convey flows to the pumping station. The new sewer will be aligned along the east side of the Canadian Tire property, run eastward along the north side of Highway 7 (within the limits of a proposed roadway for a future residential subdivision), then cross under Highway 7 to connect to the new pumping station. The sewer will be installed at depths of about 2 to 6 metres below existing ground surface level. However significant grade raises, of up to 2.5 metres, are proposed along the north portion of the alignment.
- Dual 300 millimetre diameter forcemains will extend northward from the pumping station along the same alignment, but opposite flow direction, as the sanitary sewer. The forcemains will generally be installed at about 1.5 to 2 metres depth below existing ground surface level, which will be about 2 to 3 metres below future ground surface level.
- Three 80 metre long trenchless crossings of Highway 7 are proposed, to accommodate the sanitary sewer and dual forcemains. The bore for the sanitary sewer crossing will be constructed at about 6 metres depth and the forcemain crossings will be constructed at about 3 metres depth, relative to pavement level.
- The trenchless crossings will pass under the light standards along the north and south shoulders of Highway 7 as well as beneath a buried hydro line on the north side of Highway 7 adjacent to the drainage ditch.

The site topography is relatively flat and consists of undeveloped open field. At the trenchless crossings, Highway 7 consists of a four lane divided highway, with a raised paved median.

Based on the results of nearby previous investigations as well as published geological maps, the subsurface conditions are expected to consist of stiff clay and glacial till overlying bedrock, with the bedrock surface generally at about 3 to 4 metres depth and consisting of sandstone of the March Formation.



3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out in two stages, from March 3 through 6, 2015 and from April 14 through 15, 2015. At that time, the test holes (i.e., boreholes and probeholes) were put down at the approximate locations shown on the Site Plan, Figure 1, as described below.

For the first stage of the investigation, six boreholes (numbered 15-1 to 15-6, inclusive) were put down as follows:

- Two boreholes (numbered 15-1 and 15-2) were advanced at the location of the proposed pumping station and wet wells. These boreholes were initially advanced through the overburden to depths of about 5.8 and 5.7 metres below the existing ground surface at which depth practical refusal to augering was encountered. The boreholes were then extended approximately 6.2 and 7.8 metres further, into the bedrock.
- Two boreholes (numbered 15-3 and 15-4) were advanced at the location of the proposed trenchless crossing (i.e., to the south and north of Highway 7, just outside of the right-of-way). These boreholes were initially advanced through the overburden to depths of about 6.1 and 6.4 metres below the existing ground surface where practical refusal to augering was encountered. The boreholes were then extended approximately 1.3 and 1.0 metres further, into the bedrock.
- Two boreholes (numbered 15-5 and 15-6) were advanced along the proposed sewer/forcemain alignment north of Highway 7. Borehole 15-5 was advanced within the overburden to about 7.3 metres depth below the existing ground surface. Borehole 15-6 was advanced through the overburden to a depth of about 3.8 metres where practical refusal to augering was encountered.

For the second stage of the investigation, one borehole (numbered 15-7) and two probeholes (numbered 15-101 and 15-102) were put down for the purpose of better defining the bedrock profile along the trenchless crossing alignment, as follows:

- One borehole (numbered 15-7) was advanced within the median of Highway 7 at the approximate mid-point of the proposed trenchless crossing alignment. This borehole was advanced through the overburden to a depth of about 8.3 metres beneath the existing median surface at which depth practical refusal to augering was encountered.
- Two un-sampled probeholes (numbered 15-101 and 15-102) were advanced within the north and south shoulders of Highway 7, along the proposed trenchless crossing alignment. These probeholes were advanced through the overburden to depths of about 7.4 and 8.0 metres below the existing pavement surface at which depth practical refusal to augering was encountered.

All of the test holes were advanced using either a track-mounted or truck-mounted continuous flight hollow-stem auger drill rig, supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario.

Standard Penetration Tests (SPT) were carried out in the boreholes at regular intervals of depth and samples of the soils encountered were recovered using split spoon sampling equipment.



No sampling of the overburden was carried out within the probeholes. However, the subsurface conditions and approximate depths to strata changes were assessed visually at the time of drilling and by examination of the auger cuttings.

Once refusal was encountered at boreholes 15-1 to 15-4, inclusive, they were advanced further into the bedrock using NQ sized rotary diamond drilling techniques. The core was then sequentially packed into core boxes.

Monitoring wells were sealed into boreholes 15-1, 15-3, and 15-4 to allow subsequent measurement of the groundwater level across the site and for hydraulic conductivity testing. The groundwater levels in these monitoring wells were measured on March 24 and then again on April 14, 2015 (when the second stage of the investigation was carried out).

The field work was supervised by an experienced technician from our staff who located the test holes, directed the drilling operations and in situ testing, logged the test holes, and took custody of the soil and bedrock samples retrieved.

Upon completion of the drilling operations, samples of the soil and bedrock encountered in the boreholes were transported to our laboratory for further examination by the project engineer and for laboratory testing. The laboratory testing included natural water content determinations and Atterberg limit tests.

Two soil samples, one from the native soil in borehole 15-1 and one from the surficial fill material in borehole 15-6, were submitted to Maxxam Analytics for laboratory analysis of petroleum hydrocarbons F1 to F4, metals, and volatile organic compounds, to aid in determining the disposal restrictions/options for soil excavated from the site.

Soil samples from boreholes 15-3 and 15-5 were submitted to EXOVA Environmental Ontario Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The test hole locations were selected, picketed, and surveyed in the field by Golder Associates Ltd. The test hole locations and elevations were surveyed using a Trimble R8 Global Positioning System (GPS) unit. The elevations are referenced to Geodetic datum.



4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is provided as follows:

- The Record of Borehole and Probehole Sheets are provided in Appendix A.
- The results of the rising-head hydraulic conductivity testing are provided in Appendix B.
- The results of the soil quality chemical testing are provided in Appendix C.
- The results of the basic chemical analysis (for sulphate attack and corrosivity assessment) carried out on soil samples from boreholes 15-3 and 15-5 are provided in Appendix D.

The subsurface conditions on this site generally consist of fill and/or topsoil, underlain by very stiff silty clay, glacial till and shaley dolostone bedrock. The bedrock surface was generally inferred/proven at elevations ranging from about 122.2 to 125.9 metres, generally decreasing in elevation from the north/west to east along the alignment.

The following section presents an overview of the subsurface conditions encountered in the boreholes.

4.2 Pavement Structure

About 1.2 to 1.5 metres of pavement structure/subgrade fill were encountered at borehole 15-7 and probeholes 15-101 and 15-102, which were located within the right-of-way (ROW) of Highway 7. The pavement structure at borehole 15-7 generally consists of about 50 millimetres of asphaltic concrete (median paving) underlain by about 1.2 metres of gravelly sand 'crushed stone' base/subbase and about 0.3 metres of sand subgrade fill. The pavement structure at the probeholes generally consists of about 0.4 metres of gravelly sand 'crushed stone' base underlain by about 0.8 metres of sandy gravel 'crushed stone' subbase.

4.3 Topsoil, Fill, and Silty Sand

Topsoil exists at ground surface at most of the boreholes locations (with the exception of those drilled within the ROW of Highway 7) and ranges in thickness from 100 to 400 millimetres (including topsoil fill at borehole 15-3 and 15-5).

Variable soil fill exists at ground surface at borehole 15-6 and beneath the topsoil fill at boreholes 15-3 and 15-5. The fill generally consists of silty sand, sandy gravel, sand or silty clay. Cobbles were also encountered within the sandy gravel fill at borehole 15-3. In addition, several attempts were made to auger through a cobble/boulder layer (potentially an old foundation) encountered at about 0.5 metres depth in the surrounding area of borehole 15-3. Organic matter was encountered within the silty clay fill at borehole 15-6. The thickness of the fill at the borehole locations is between about 0.3 to 0.6 metres.

A thin layer of silty sand was encountered beneath the topsoil at borehole 15-1 with a thickness of about 160 millimetres.



4.4 Silty Clay to Clay

The pavement structure, topsoil, fill, and/or sand are underlain by a deposit of silty clay to clay, and occasional clayey silt (hereafter referred to as silty clay). The upper portion of the silty clay has been weathered to a grey brown crust and extends to depths of between about 3.1 and 4.6 metres below the existing ground/pavement surface (i.e., elevations 125.3 and 126.3 metres). The entire deposit at borehole 15-6 has been weathered to a grey brown colour and extends to a depth of about 2.7 metres below the present ground surface (i.e., elevation 126.9 metres). SPT N values obtained within this deposit range from about 8 to 29 blows per 0.3 metres of penetration indicating a very stiff consistency.

The results of one Atterberg limit test carried out on a sample of the weathered deposit gave a plasticity index value of about 26 percent and a liquid limit value of 52 percent, indicating a soil of high plasticity. Water contents of between about 30 and 33 percent were measured in the weathered silty clay.

The silty clay below the depth of weathering (where present) is grey in colour. The unweathered silty clay was fully penetrated to depths between about 5.5 and 8.2 metres (i.e., elevations 122.7 and 123.3 metres) at boreholes 15-1 to 15-4, inclusive, and 15-7, as well as proven to a depth of about 7.3 metres (i.e., elevation 121.7 metres) at borehole 15-5. SPT N values obtained within this deposit range from about 7 to 13 blows per 0.3 metres of penetration indicating a very stiff consistency.

The results of two Atterberg limits tests carried out on samples of the unweathered silty clay gave plasticity index values of about 24 and 28 percent and liquid limit values of 49 and 51 percent, indicating an intermediate to high plasticity soil. The measured water contents of the silty clay range from approximately 33 to 35 percent.

4.5 Glacial Till

A layer of glacial till was encountered beneath the silty clay at all of the borehole locations, with the exception of borehole 15-5 where it was not encountered within the advancement depth. Based on the retrieved samples (which were limited by the small thickness of the layer) and our general knowledge of the local geological conditions it is considered that the glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand. The glacial till was encountered/inferred at depths between about 2.7 and 8.2 metres below the existing ground/pavement surface and proven to extend to depths of between about 3.8 and 6.4 metres below the existing ground surface. At the pumping station and trenchless crossing sites, the glacial till layer is only about 0.1 to 0.3 metres thick.

One SPT N value of 15 blows per 0.3 metres of penetration was measured in the glacial till, indicating a compact state of packing. Higher N values of greater than 50 blows per 0.3 metres of penetration were also measured in the glacial till; however they likely reflect contact with the bedrock surface, rather than the actual state of packing.

The measured natural water content of one sample of the glacial till was about 10 percent.



4.6 Auger Refusal and Bedrock

Practical refusal to augering was encountered at a depth of about 3.8 metres below the existing ground surface (i.e., elevation 125.9 metres) at borehole 15-6 and between about 7.4 and 8.3 metres below the existing pavement surface (i.e., elevations 122.2 and 122.8 metres) at borehole 15-7 and probeholes 15-101 and 15-102. Auger refusal could indicate cobbles or boulders within the glacial till or more likely the bedrock surface.

Bedrock was encountered beneath the glacial till at depths of between about 5.7 and 6.4 metres below the existing ground surface at boreholes 15-1 to 15-4, inclusive, where it was cored to depths of between about 7.4 and 13.4 metres below the existing ground surface. The following table summarizes the bedrock surface depths and elevations as well as the depths and elevations of bedrock cored at the borehole locations.

| Borehole/Probehole Number | Borehole/Probehole Location | Existing Ground Surface Elevation (m) | Depth to Bedrock (m) | Bedrock Surface Elevation (m) | Depth to Bottom of Cored Bedrock (m) | Elevation of Bottom of Cored Drillhole (m) |
|---------------------------|--|---------------------------------------|----------------------|-------------------------------|--------------------------------------|--|
| 15-1 | Pumping Station/Wet Well | 128.85 | 5.84 | 123.01 | 12.03 | 116.82 |
| 15-2 | Pumping Station/Wet Well | 128.67 | 5.65 | 123.02 | 13.44 | 115.23 |
| 15-3 | South Side of Trenchless Crossing | 129.33 | 6.07 | 123.26 | 7.38 | 121.95 |
| 15-4 | North Side of Trenchless Crossing | 129.14 | 6.35 | 122.79 | 7.39 | 121.75 |
| 15-6 | North End of Sewer/Forcemain Alignment | 129.68 | 3.76* | 125.92* | N/A | N/A |
| 15-7 | Median of Highway 7 along the Trenchless Alignment | 130.88 | 8.31* | 122.57* | N/A | N/A |
| 15-101 | North Shoulder of Highway 7 along the Trenchless Alignment | 130.26 | 7.42* | 122.84* | N/A | N/A |
| 15-102 | South Shoulder of Highway 7 along the Trenchless Alignment | 130.23 | 8.03* | 122.20* | N/A | N/A |

Note: * Depth and elevation to bedrock inferred from auger refusal.

The bedrock generally consists of fresh, thinly to thickly bedded, dark grey to grey, fine to coarse grained, non-porous, shaley dolostone to dolostone. The Rock Quality Designation (RQD) values measured in the bedrock core generally ranged from 79 to 100 percent, indicating a good to excellent quality rock.



4.7 Groundwater and Hydraulic Conductivity Testing

Monitoring wells were installed in boreholes 15-1, 15-3 and 15-4. The groundwater levels were measured on March 24 and April 14, 2015. In situ hydraulic conductivity testing was also carried out on March 24, 2015. The results of the hydraulic conductivity testing are provided in Appendix B.

The following table summarizes the measured groundwater levels and hydraulic conductivities.

| Borehole Number | Borehole Location | Geological Unit | Ground Surface Elevation (m) | Date | Water Level Depth (m) | Water Level Elevation (m) | Calculated Hydraulic Conductivity (cm/sec) |
|-----------------|-----------------------------------|--------------------------|------------------------------|-----------|-----------------------|---------------------------|--|
| 15-1 | Pumping Station/Wet Well | Shaley Dolostone Bedrock | 128.85 | 24-Mar-15 | 0.65 | 128.20 | 1×10^{-2} |
| | | | | 14-Apr-15 | 0.14 | 128.71 | - |
| 15-3 | South Side of Trenchless Crossing | Dolostone Bedrock | 129.33 | 24-Mar-15 | 1.06 | 128.27 | 2×10^{-2} |
| | | | | 14-Apr-15 | 0.54 | 128.79 | - |
| 15-4 | North Side of Trenchless Crossing | Shaley Dolostone Bedrock | 129.14 | 24-Mar-15 | 0.76 | 128.38 | 1×10^{-3} |
| | | | | 14-Apr-15 | 0.34 | 128.80 | - |

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.



5.0 DISCUSSION

5.1 General

This section of the report provides engineering guidelines on the geotechnical and hydrogeological design aspects of the proposed pumping station, wet wells, sanitary sewer, forcemains and trenchless crossings under Highway 7 in Carleton Place, based on our interpretation of the borehole information and project requirements.

Reference should be made to the “Important Information and Limitations of This Report”, which follows the text but forms part of this document.

5.2 Pumping Station

The pumping station will be located on the south side of Highway 7, east of McNeely Avenue, adjacent to the Rona property. The footprint of the pumping station control building and adjacent wet wells will measure about 8 metres wide and 8 metres long in plan dimension. The wet wells will be located on the west side of the pumping station and will be founded at about elevation 119.4 metres, approximately 10 metres below the existing ground surface. The control building/dry well will be located immediately adjacent to the wet wells and will consist of a one storey building within one basement level founded at about elevation 126.7 metres (i.e., about 2 metres below the existing ground surface).

5.2.1 Site Grading

The subsurface conditions in the area of the pump station and wet wells generally consist of topsoil and/or fill underlain by very stiff silty clay and glacial till, with the bedrock surface at about 6 metres depth.

No practical restrictions apply to the thickness of grade raise fill which may be placed on the site, from a foundation design perspective. However, as a general guideline, it is recommended that the grade raise be limited to about 3.5 metres. If higher grade raises are required, their feasibility would need to be reviewed. However, it is understood that the grade raises on the site will be limited to about 1 metre.

As a general guideline regarding the site grading, the preparation for filling of the site should include stripping of the topsoil and fill to improve the settlement performance of structures and services. Topsoil and fill are not suitable as general fill and should be stockpiled separately for re-use in landscaping applications only. In areas with no proposed structures, services, or roadways, these materials may be left in-place provided some settlement of the ground surface following filling can be tolerated.

5.2.2 Excavations

The excavations for the control building/dry well will extend about 2 metres below the existing ground surface, while the excavations for the adjacent inlet chamber and wet wells will extend between about 6 and 10 metres below the existing ground surface. These excavations will be made through the topsoil, thin silty sand layer, silty clay, thin glacial till layer and into the underlying bedrock (at the wet wells).

No unusual problems are anticipated with excavating through the overburden using conventional hydraulic excavating equipment. Where the excavations are carried out in the sensitive silty clay, it is suggested that the excavation equipment be fitted with a smooth bladed bucket (i.e., no teeth), to limit disturbance of the subgrade.



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Excavation into bedrock will require drill and blast procedures, or could possibly be carried out by hoe ramming (although the bedrock is strong/hard and potentially abrasive). It should be noted that the Ministry of Transportation (MTO) sometimes does not allow blasting of the bedrock near their right-of-way. The local conservation authority may also have similar limitations to blasting near creeks where there is fish habitat (as could be the case in the area of the pumping station). These potential restrictions will need to be confirmed prior to construction. Alternatively, the bedrock removal could be carried out with non-exploding expanding agents.

The blasting should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. This will require blast designs by a specialist in this field.

A pre-blast survey should be carried out of all of the surrounding structures and related facilities, as well as nearby utilities. Selected existing interior and exterior cracks in the structures should be identified during the pre-blast survey and should be monitored for lateral or shear movements, such as by means of telltales.

The contractor should be required to submit a complete and detailed blasting design and monitoring proposal prepared by a blasting/vibrations specialist prior to commencing blasting. This would have to be reviewed and accepted in relation to the requirements of the blasting specifications.

The contractor should be limited to only small controlled shots. The following frequency dependent peak vibration limits at the nearest structures are suggested.

| Frequency Range (Hz) | Vibration Limits (millimetres/second) |
|----------------------|---------------------------------------|
| < 10 | 5 |
| 10 to 40 | 5 to 50 (sliding scale) |
| > 40 | 50 |

It is recommended that the monitoring of ground vibration intensities (peak ground vibrations and accelerations) from the blasting and rock removal operations be carried out both in the ground adjacent to the closest structures/services, and within the structures themselves. The monitoring should be carried out on a continuous basis throughout the rock removal process. If practical, blasting should commence at the furthest points from the closest structures or services to assess the ground and air vibration attenuation characteristics and to confirm the anticipated ground vibration levels based on the contractor's blasting proposal.

Blast induced damage to the bedrock must be avoided; otherwise significant rock reinforcement could be required for the excavation walls. It should therefore be planned to either line drill the bedrock along the perimeter of the excavation at a close spacing in advance of blasting so that a clean bedrock face is formed, or to carry out perimeter drilling and pre-shearing of the excavation limits using controlled blasting.

Excavation in the overburden at the site of the pumping station may be carried out at a 1 horizontal to 1 vertical slope (Type 3 soil in accordance with the Occupational Health and Safety Act). If the open cut excavation side slopes cannot be accommodated, consideration can also be given to shoring the excavations. Further details on excavation shoring are provided in Section 5.2.3



Based on the arrangement of the excavations for the pumping station and wet wells, as well as the stiffness of the silty clay, basal instability is not considered an issue.

Excavation within the bedrock may be carried out with a vertical or near vertical side slopes, provided the bedrock is not damaged by the blasting.

5.2.3 Excavation Shoring

Although open cut excavation sides slopes constructed to shallow depth in the overburden soils might be feasible, an engineered shoring could be required given the depth to the bedrock surface. Shoring within the native soils can be achieved using conventional techniques (e.g., soldier pile and lagging, sheet piling, etc.).

The contractor should be made responsible for the design of the shoring. However, this section of the report provides some general guidelines on possible concepts for the shoring, to assist the project designers with:

- Assessing the costs of the shoring.
- Assessing possible impacts of the shoring design and construction on the design of the project.
- Evaluating, at the design stage, the potential for impacts of this shoring on adjacent structures, services, or roadways.

The shoring method(s) chosen to support the excavation sides must take into account: the soil stratigraphy, the groundwater conditions, the methods adopted to manage the groundwater, the permissible ground movements associated with the excavation and construction of the shoring system, and the potential impacts on adjacent structures, services or roadways.

In general, there are three basic shoring methods that are commonly used in this area:

- Steel soldier piles and timber lagging.
- Driven steel sheet piles.
- Continuous concrete (secant or diaphragm) walls, though much less commonly used than the other two systems.

These three options are listed in order of generally increasing stiffness and ability to resist ground movements. Soldier piles and lagging are suitable where the objective is to maintain an essentially vertical excavation wall and where the movements above and behind the shoring need only be sufficiently limited so that relatively flexible features (such as roadways) will not be adversely affected. Where the deflections need to be more strictly limited, such as where heavily loaded foundations lie within the zone of influence of the shoring, continuous concrete shoring can be required. Sheet piling provides an intermediate level of stiffness. Sheet piling and continuous concrete shoring can also provide a hydraulic barrier to 'cut off' groundwater flow.

For all of the above systems, some form of lateral support to the shoring is typically required for excavation depths greater than about 3 metres. Common lateral restraint systems could include:

- Circular or square waler beams, likely with bracing at corner points.
- Interior struts which are connected to the opposite side of the excavation (if not too distant) or to raker piles.
- Soil anchors (i.e., tie-backs), although these are less commonly used for small excavations.



The passive resistance provided to the socketed/embedded portion of the shoring also contributes to the lateral resistance.

Based on the conditions encountered, it is expected that steel sheet pile shoring or soldier pile and lagging, with interior struts or bracing, could feasibly be used for the excavations. However, the final selection of the type of temporary shoring system, and the method of lateral restraint, should be entirely the choice/responsibility of the contractor.

The contractor should be required to submit the shoring system design, including details on the design lateral earth pressures, expected movements, and a monitoring plan, for review prior to the start of shoring construction.

Stockpiling of soil beside the excavations should be avoided; the weight of the stockpiled soil could lead to overstressing of the shoring system.

5.2.4 Ground Movements (Adjacent to Shoring)

Some unavoidable inward horizontal deformation and vertical settlement of the adjacent ground may occur as a result of excavation, installation of the shoring and deflection of the ground support system (including bending of the walls and compression of the struts). The ground movements induced could affect the performance of surface structures or underground utilities adjacent to the excavation.

The resulting ground settlement will depend on the selected shoring alternative. As a preliminary guideline, typical settlements behind sheet pile or soldier pile and lagging shoring are anticipated to be less than about 0.3 percent of the excavation depth, provided that good construction practices are used (e.g., that supports are installed as soon as the support level is reached), and that voids are not left behind the lagging. This guideline would suggest less than about 20 millimetres of ground settlement could occur for the excavations of the inlet chamber and wet wells. Since there are no existing structures or roadways in the immediate vicinity of the proposed pumping station (i.e., within about 15 metres), this settlement should be acceptable.

The construction documents should not specify the specific shoring system that should be used, but rather the permissible deflection level (i.e., 'Performance Level') should be specified, in accordance with Ontario Provincial Standard Specification (OPSS) 539. With the above design approach, it is considered that Performance Level 2 would be specified.

However, the above guidelines are only preliminary and are provided only to assist the owner's designers in carrying out an initial assessment of the expected settlements and the potential impacts of these settlements. A more detailed assessment of the expected settlements should be undertaken by the contractor.

A preconstruction survey should be carried out to if there are any potential adjacent structures that may be affected by ground settlements prior to the commencement of excavation. The magnitude of ground movements adjacent to the shafts should be monitored throughout the construction period. The expected levels of deformation should be established by the contractor and alert levels should be set at which the designers should review the deformation and consider modifications to the design and/or construction procedures.



5.2.5 Groundwater Management

The measured groundwater level in the area of the pumping station is about 0.1 metres below the existing ground surface. Excavations for construction of the pumping station and adjacent wet wells would therefore extend below the groundwater level, and groundwater inflow into the excavations should be expected.

Significant groundwater inflow is not expected for shallow excavations that are entirely within the native silty clay. Therefore, for shallow excavation depths, it should generally be possible to handle the groundwater inflow from the overburden soils by pumping with suitably sized pumps from well filtered sumps within the excavations.

However, the excavation for the wet wells will extend about 10 metres below the existing grade and into the shaley dolostone bedrock, which has a high measured hydraulic conductivity. Therefore, more significant groundwater into the excavation should be expected. For this deeper excavation, it is anticipated that the majority of groundwater inflow will be generated through open discontinuities in the underlying dolostone bedrock.

The Thiem analytical solution for confined conditions was used to conservatively estimate the steady-state groundwater inflow through the bedrock into the pumping station excavation. Preliminary estimates of the volume of groundwater inflow were calculated by assuming the excavation measures 10.6 metres long and 5.6 metres wide at the surface (an additional 1 metre has been allowed for beyond the structure limits on either side of the excavation) by 10.5 metres deep. The groundwater elevation measured at borehole 15-1 in March 2015 (128.2 metres elevation) was used in the assessment (which was the initially measured groundwater level). The hydraulic conductivity measured at borehole 15-1 (1×10^{-2} centimetres per second) was used as the hydraulic conductivity of the entire rockmass. Based on these values and applying a 1.5 factor of safety, the estimated steady-state groundwater inflow would be approximately 1,800,000 Litres per day for the pumping station excavation. Initial inflows may be higher, depending on the excavation schedule and methodology. The predicted radius of influence is approximately 100 metres.

The actual rate of groundwater inflow to the excavation will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, the material (silty clay, glacial till or bedrock), incident precipitation, and the time of year at which the excavation is made (e.g., fluctuation in seasonal groundwater elevation). The pumping station excavation would be best carried out in the drier summer months. Incident precipitation could add approximately 4,100 litres of water per day to the excavation, based on the excavation geometry described above and a 72 millimetre precipitation event (a 10 year event as observed at the Ottawa International Airport weather station). A Permit-To-Take-Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) when a volume greater than 50,000 litres per day is pumped. Based on the predicted inflows for the pumping station excavation, a Permit-To-Take-Water will be required, as the volumes of water to be taken at the site are expected to exceed 50,000 Litres per day.

It is important to note that groundwater inflow into the excavation will decrease over time as the overburden materials and bedrock dewater. However, during the progression to steady-state and once steady-state is reached, short-term increases in groundwater inflows would be expected following precipitation events if the overburden and bedrock is recharged and subsequently drains into the excavation.

Given the relatively high measured hydraulic conductivity of the bedrock (in the range of 10^{-3} to 10^{-2} centimetres per second) construction dewatering will be an important consideration. It may, be necessary to carry out some groundwater pumping in advance of excavation and construction. Pre-dewatering from wells installed in the bedrock in the area of the excavation, for a period of a few weeks, might be a feasible method to lower the



groundwater level and thereby reduce the groundwater inflow into the construction area. This methodology should be considered by the contractor. Larger scale hydraulic testing (i.e., pumping tests) could also be completed in order to obtain hydraulic data that would be more representative of the bulk hydraulic conductivity of the bedrock, and thus increase confidence in the groundwater inflow calculations. Numerical modeling could also be used to obtain better estimates of the groundwater inflow and to evaluate various dewatering scenarios.

5.2.6 Foundations

The boreholes advanced at the pump station (i.e., boreholes 15-1 and 15-2) encountered about 6 metres of overburden overlying dolostone bedrock. The bedrock was encountered at about elevations 123 metres. It is understood that the design founding level for the wet wells will be at about elevation 119.4 metres, and therefore within bedrock.

The foundations of the wet wells may be placed on/within the dolostone bedrock and designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 2,000 kilopascals. Provided the bedrock surface is cleaned of loose/disturbed material, settlements at this bearing stress level should be less than 25 millimetres and, therefore, Serviceability Limit States (SLS) values will not apply. The foundations for the inlet channel, which is proposed at a higher level (i.e., about elevation 123.3 metres), should be carried down to also found on/in the dolostone bedrock (at about elevation 123.0 metres), to avoid differential settlements between these portions of the structure.

The foundations for the control building/dry well, which is also proposed at a higher level (i.e., founding elevation 126.4 to 126.7 metres), can feasibly be founded on a 'raft' foundation within the very stiff silty clay, provided the differential settlement with the wet well can be accommodated. It should be noted that a large portion of the 'raft' foundation may in fact be founded on engineered backfill due to the shoring and excavation arrangement (i.e., it is expected that the east side of the control building excavation will be shored to about elevation 126.4 metre, then sloped downward to the bedrock surface for the wet well excavation and the excavated material would then need to be replaced back up to founding level with compacted engineered fill).

For the proposed founding level, it is considered that the raft foundation can be designed using an SLS gross contact stress in the order of 200 kilopascals and a ULS factored bearing resistance of 275 kilopascals. The post construction *total* settlements of the raft foundation designed for this SLS value would be limited to about 25 millimetres in magnitude. However, it is understood that the SLS gross contact stresses required for the design of the pumping station are more likely to be in the order of 100 to 150 kilopascals. If that is the case, then the total settlements would likely be about half of that value (i.e., about 10 to 15 millimetres).

The corresponding *differential* settlements across the length or width of the portion of the structure founded on the silty clay are estimated at about half of the total settlement, but will also depend greatly on the stiffness of the raft.

The calculated settlements of the foundation on the silty clay will also be entirely differential relative to the portion of the pumping station founded on the bedrock (i.e., the wet wells/inlet chamber) due to the different settlement properties of these materials. The structural designer will therefore need to take this into consideration when designing the foundation connection between these portions of the structure.



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Further, the SLS resistance for the control building/dry well corresponds to a settlement resulting from consolidation of the silty clay. Consolidation of silty clay is a process which takes months or longer and, as such, results from sustained loading. Therefore, the foundation loads to be used in conjunction with the SLS resistance given above should be the full dead load plus sustained live load. The factored dead load plus full factored live load should be used in conjunction with the ULS factored bearing resistance.

The SLS resistance and corresponding settlement estimates are dependent upon the soil at or below founding level not being disturbed during construction. The silty clay subgrade will be very sensitive to disturbance by construction traffic, especially in the presence of water. It should therefore be planned to place a mud slab of lean concrete at the base level immediately upon completion of excavation, to minimize disturbance of the subgrade material.

If the potential differential settlement of the control building/dry well relative to the wet well is not considered acceptable, then consideration could also be given to the following foundation options:

- 1) The foundations for the control building/dry well could also be founded on the bedrock, at about elevation 123 metres, with the area under the floor backfilled to the design slab level with engineered fill.
- 2) The overburden soil could be (fully) excavated down to the bedrock level and replaced up to the founding level using compacted engineered fill or mass (lean) concrete. For this option, the shoring would need to extend a minimum distance of 1 metre outside of the foundation wall and would need to remain permanently in place, in order to adequately support the edge of the structure.

As mentioned previously, a significant portion of the silty clay will already need to be excavated to accommodate the excavation/shoring arrangement. Therefore, the excavations required for the above options are only somewhat more extensive than founding the control building/dry well at the higher elevation within the silty clay.

The foundations for Options 1 and 2 may be designed using the SLS and ULS values provided above for foundations on bedrock and silty clay, respectively. The total and differential settlement for either of the above foundation options would be negligible (i.e., less than about 5 millimetres).

For the control building/dry well, provision should be made for at least 300 millimetres of OPSS Granular A to form the base for the floor slab or raft foundation for either of the above options. The bulk fill required to raise the grade to the underside of the Granular A should consist of OPSS Granular B Type II. The underslab fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

For the wet wells, if the excavated bedrock surface is slightly below the design founding level (because the bedrock bedding thickness necessitates removal to a slightly deeper level), it would be feasible to place mass concrete to fill the excavation up to founding level.



5.2.7 Frost Protection

The native subgrade soils on this site are considered to be frost susceptible. Therefore, all exterior perimeter foundation elements or foundation elements in unheated areas should be provided with a minimum of 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover. Insulation of the bearing surface with high density insulation could be considered as an alternative to earth cover for frost protection.

If any portions of these structures will be unheated, or expected to have below-freezing internal temperatures, additional guidelines will need to be provided.

5.2.8 Seismic Design

The seismic design provisions of the 2012 Ontario Building Code depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level. However, the OBC also permits the Site Class to be specified based solely on the stratigraphy and in situ testing data, rather than from direct measurements of the shear wave velocity.

In the absence of site-specific shear wave velocity testing, a Site Class of C may be used for seismic design for the pump station. It is possible that a better Site Class could be achieved based on site specific shear wave velocity testing and if the control building/dry well foundations were lowered to be closer to the bedrock level. However, the additional cost of that testing may not be justified by the structure savings.

5.2.9 Buoyancy

The wet well and dry well will extend below the observed groundwater level. Given the high hydraulic conductivity, it is not considered practical or desirable to permanently drain the structure backfill and subgrade. The building should therefore be designed to have a factor of safety against buoyancy (with a near-surface groundwater level). This could be accomplished by providing sufficient thickness of the basement walls, in addition to the weight of the remainder of the building, to offset the buoyancy.

Alternatively, rock anchors can be installed into the bedrock to resist uplift loads on the pump station. However, the high groundwater level within the bedrock and the high hydraulic conductivity of the bedrock may make the installation and grouting of the anchors difficult.

The anchors could consist of either grouted or mechanical anchors. In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

- i) Failure of the steel tendon or top anchorage;
- ii) Failure of the grout/tendon bond;
- iii) Failure of the rock/grout bond; and,
- iv) Failure within the rock mass, or rock cone pull-out.

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion. A 'double corrosion protection' system should be specified.



For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ULS design purposes. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance should be calculated based on the buoyant weight of the potential mass of rock which could be mobilised by the anchor. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \phi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

Where: Q_r = Factored uplift resistance of the anchor, kilonewtons;

ϕ = Resistance factor, 0.3;

γ' = Effective unit weight of rock, use 17 kilonewtons per cubic metre;

D = Anchor length in metres; and,

θ = $\frac{1}{2}$ of the apex angle of the rock failure cone, use 30 degrees.

For a group of anchors or for a line of closely spaced anchors the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. Further guidelines can be provided for assessing the anchor resistance for these conditions, once the anchor layout is known.

It is suggested that pull-out tests be carried out on anchors to confirm their pull-out capacity. The pull-out tests should be carried out to 1.3 times the anchor service loads, and at least 10 percent of the anchors should be tested in this manner.

It is suggested that the installation and testing of the anchors be supervised by the geotechnical engineer. The anchor holes should be thoroughly flushed to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to ensure an adequate bond between the grout and the rock.

5.2.10 Backfill to Pumping Station Structure

5.2.10.1 Overburden Excavations

The following guidelines apply to the upper portions of the foundation walls, above the bedrock surface.

The soils at this site are potentially frost susceptible and should not be used as backfill against exterior, unheated, or well insulated foundation elements within the depth of potential frost penetration (1.5 metres) to avoid problems with frost adhesion and heaving. The foundation and basement walls therefore should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for Ontario Provincial Standard Specification (OPSS) Granular B Type I.



To avoid ground settlements around the foundations, which could affect site grading and drainage, all of the backfill materials should be placed in 0.3 metre thick lifts, compacted to at least 95 percent of the material's standard Proctor maximum dry density.

5.2.10.2 Excavations in Bedrock

The following guidelines apply to the deeper portions of the basement walls, which will be constructed in the bedrock.

Where the basement walls will be constructed using formwork, it will be necessary to backfill a narrow gallery between the bedrock face (or shoring) and the outside of the walls. The backfill should consist of 6 millimetre clear stone 'chip', placed by a stone slinger or chute.

In no case should the clear stone chip be placed in direct contact with other soils. For example, surface landscaping or backfill soils placed near the top of the clear stone back fill should be separated from the clear stone with a geotextile.

As a preliminary guideline, it should not be planned to pour the walls directly against the bedrock since groundwater seepage could interfere with concrete placement.

5.2.11 Lateral Earth Pressures on Pumping Station Walls

The walls of the structure will need to be designed to resist lateral earth and water pressures. It is assumed that the structure will be designed to have water-tight foundation walls; i.e., an exterior foundation drainage system will not be provided.

Below the bedrock surface (for walls with a narrow backfilled gallery), the magnitude of the lateral earth pressure will depend on the magnitude of the arching which can develop in the backfill and therefore depends on the width of the backfill, its angle of internal friction, as well as the interface friction angles between the backfill and both the rock/shoring face and the foundation wall. The magnitude of the lateral earth pressure can be calculated as:

$$\sigma_h(z) = \frac{\gamma B}{2 \tan \delta} \left(1 - e^{-2 K \frac{z}{B} \tan \delta} \right) + Kq + u$$

Where: $\sigma_h(z)$ = Lateral earth pressure on the foundation wall at depth 'z', kilopascals;

K = Earth pressure coefficient, use 0.6;

γ = Unit weight of retained soil, use 17 kilonewtons per cubic metre for 6 millimetre clear stone chip;

B = Width of backfill (between wall and bedrock face), metres;

δ = Average interface friction angle at backfill wall and backfill-rock face interfaces, use 15 degrees;

z = Depth below top of the wall, metres;

q = Uniform surcharge at ground surface behind the wall to account for traffic, equipment, or stockpiled soil (not less than 15 kilopascals); and,

u = Hydrostatic groundwater pressure, kilopascals.

Below the groundwater level, effective stress parameters could be used in the above equation.



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Where the walls will be poured directly against the bedrock, they should be designed to resist the hydrostatic groundwater pressure, at a minimum.

Above the bedrock surface, the lateral earth pressure can be calculated as:

$$\sigma_h(z) = K_o (\gamma \cdot z + q) \quad \text{above the groundwater level}$$

$$\sigma_h(z) = K_o [\gamma \cdot d_w + (\gamma - \gamma_w) \cdot (z - d_w) + q] + (z - d_w) \cdot \gamma_w \quad \text{below the groundwater level}$$

Where: $\sigma_h(z)$ = Lateral earth pressure on the wall at depth z , kilopascals;

K_o = At-rest earth pressure coefficient, use 0.5;

γ = Unit weight of retained soil, use 22 kilonewtons per cubic metre;

γ_w = Unit weight of water, 9.81 kilonewtons per cubic metre;

z = Depth below top of the wall, metres;

d_w = Depth to groundwater level, metres; and,

q = Uniform surcharge at ground surface to account for traffic, equipment, or stockpiled soil (not less than 15 kilopascals).

For the wet well foundations which are adjacent to the control building/dry well, the lateral earth pressure calculation should also include a surcharge to account for the load from the control building/dry well foundations.

The lateral earth pressure would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with a maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The combined pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(z) = K_o \gamma z + (K_{AE} - K_o) \gamma (H - z)$$

Where: K_{AE} = The seismic earth pressure coefficient (use 0.7); and,

H = The total depth to the bottom of the foundation wall (metres).

All of the lateral earth pressure equations are given in an unfactored format and will need to be factored for Limit States Design purposes.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible soil/backfill beyond the wall backfill. To reduce the severity of this differential heaving, the backfill adjacent to the wall may have to be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 metres below finished exterior grade at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The granular fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.



5.3 Trenchless Crossings

Three 80 metre long trenchless crossings under Highway 7 are proposed to accommodate the sanitary sewer and dual forcemains. The 300 millimeter diameter forcemains are currently proposed to be installed in steel casings about 900 millimetres in diameter and with casing invert levels between about elevations 127.1 and 127.0 metres at the north and south ends, respectively (i.e., with about 2.6 metres of cover relative to pavement level). The 450 millimetre diameter sanitary sewer has proposed invert levels of between about elevations 124.5 and 124.2 metres at the north and south ends, respectively (i.e., with about 5.5 metres of cover relative to pavement level) and will likely be installed within a similar sized steel casing. The construction of the crossings will therefore be entirely within the very stiff silty clay.

5.3.1 Trenchless Profile Considerations

5.3.1.1 General

The following provides general guidelines with regards depth/cover requirements for trenchless crossings:

- For trenchless crossings in overburden, the cover above the crown of the tunnel/bore should be at least twice the tunnel/bore diameter (and preferably three diameters) relative to the ground surface or settlement sensitive buried utilities. Lesser amounts of cover could jeopardize the stability of the working face (depending on the method) or lead to excessive ground movements.
- For trenchless crossings entirely within bedrock, one tunnel/bore diameter of competent rock cover above the crown would generally be acceptable provided that man-entry into the tunnel is not planned. Any highly fractured rock directly beneath the bedrock surface should be ignored in that calculation. A lesser thickness of bedrock cover could potentially compromise the stability of the tunnel roof.
- The bore should be separated from existing trenchless service crossings or adjacent structural foundations (e.g., bridge foundations) by a horizontal or vertical distance equal to at least twice the diameter of the bore. The objectives of having this separation are: to avoid providing too narrow of a 'pillar' of soil or rock between two adjacent trenchless crossings which could be unstable/overstressed under the weight of overlying soil and bedrock; and, to avoid the bore possibly intersecting other existing services or foundations during drilling, should there be 'drift' from the intended alignment.

In addition to the above guidelines, the following issues may need to be considered:

- There are limitations on the curvature of the profile of some boring methods (such as Horizontal Directional Drilling) which can require longer or deeper bores.
- The final bore diameter for trenchless installations will depend on the type of product pipe being installed (e.g., PVC, steel or concrete), the installation methodology (e.g., jack and bore, pipe ramming, etc.), the ground conditions, and the local availability of the boring equipment.
- The Ministry of Transportation (MTO) often requires that crossings beneath their highways (especially four lane, divided, controlled-access 'freeways') be installed within a grouted steel casing to protect the highway against the risk of blowout.



5.3.1.2 *Forcemain Crossings*

It is understood that the forcemains will be installed in dual crossings within 900 millimetre diameter steel casings, with an invert level of about elevation 127.0 metres, through the very stiff silty clay. The proposed profile depth for the forcemain crossings should therefore be feasible. However, there is minimal overburden cover (i.e., less than one bore diameter) at north and south ends of the crossing (i.e., at the location of the Highway 7 ditches). As mentioned above, this could impact the stability of the working face or lead to excessive ground movements at these locations. Since it is likely that any settlement in those areas could be simply repaired, the proposed profile should be acceptable. However consideration could be given to shortening the bores such that the entry and exit shafts are located in these ditch areas.

5.3.1.3 *Sanitary Sewer Crossing*

It is understood that the sanitary sewer will be installed at an invert level between about 124.2 and 124.5 metres, which will also be through the very stiff silty clay. The proposed profile depth is also considered feasible; however the bedrock surface is indicated to be between elevations 122.2 and 123.3 metres along the length of the bore (and the till level slightly above that) which is within about 0.9 metre of the invert level at the south (low) end of the crossing. This clearance would be even less if a casing similar in size to that of the forcemain crossings' is used. Therefore, there is some potential that the bedrock surface could be encountered along the crossing (due to even a minor upward undulation in the bedrock surface), which could be very problematic for the trenchless construction. To lower the risk of encountering a mixed face condition (i.e., a soil/bedrock interface), consideration could be given to raising the profile (if hydraulically feasible) or to lowering the profile of the crossing to be entirely within the bedrock. A least one bore diameter of intact rock cover would be required above the crown of pipe for the latter option.

It is noted however that, with the current boreholes/probeholes, which were advanced at a maximum 25 metre spacing along the alignment, bedrock was not encountered within the elevation horizon of the trenchless construction.

5.3.2 *Trenchless Construction Methods*

The diameter of the pipe, length of the crossing, and anticipated subsurface conditions will limit the type of trenchless installation techniques that would be economically viable at this site. Ultimately, the choice of equipment and the method of trenchless installation is the Contractor's responsibility. The following section provides a discussion on trenchless construction methods which are considered feasible based on the anticipated ground conditions. The contractor is not limited to the methods discussed herein and, regardless of the method selected, must consider the suitability of the specific equipment and techniques relative to the ground conditions at the site.

For the anticipated pipe sizes, it is considered that contractors would generally consider one of the following trenchless methods:

- Pipe Jacking and Horizontal Auger Boring;
- Pipe Ramming;
- Horizontal Direction Drilling (HDD); or,
- Rock Boring.



For the proposed sanitary sewer and forcemains to be installed for this project, the discussion below has been based on the above construction methods. Tunnelling and microtunnelling, although technically feasible based on the ground conditions at this site, have not been considered herein because of the planned pipe size, and costs.

Pipe Jacking and Horizontal Auger Boring

A pipe jacking operation involves pushing an oversized liner pipe (casing) horizontally into the ground by jacking. Soil/pipe friction reduction is typically achieved with lubrication, and different types of bentonite and/or polymers can be used for this purpose. The spoil is generally removed from within the casing using an auger boring machine. The cutting head is driven by, and is positioned at, the leading end of an auger string that is established within the casing pipe. The product pipe would then be installed within the casing, once the bore is complete. For this method, the profile needs to be approximately horizontal, and jacking and receiving pits/shafts are required. There can be limited ability to steer the casing during jacking.

Construction of the crossings using this methodology should be feasible in this silty clay deposit for both the forcemains and deeper sanitary sewer, since no significant obstructions to the casing advancement (such as cobbles or boulders) are expected (and provided there are no upward undulations of the bedrock surface into the sanitary sewer installation horizon). Cover equal to at least twice the diameter of the casing is ideal for this method since, with lesser cover, there is less confining pressure at the face of the bore and therefore a greater potential for ground loss or face instability. Some ground movement around the casing will, however, be unavoidable.

To accommodate the carrier pipe installation and allow for potential misalignment during casing advancement, steel casings are typically chosen to be about 1.5 times the diameter of the carrier pipe. But a minimum diameter of 0.9 m is commonly selected, so as to allow worker-entry in case obstructions are encountered and need to be removed manually. The selected carrier pipe must be capable of supporting the overburden and roadway loads, hydrostatic pressures, and the installation forces and pressures from grout used to fill the annulus between the carrier pipe and the casing.

Pipe Ramming

Pipe ramming uses a pneumatic ramming tool to hammer up to 1500 millimetre diameter steel pipes or casings into the ground, in sequential spliced sections from one pit to the other, over distances of up to about 60 metres. The leading edge of the carrier pipe is almost always open and its shape has to allow a small overcut to reduce friction between the carrier pipe and the soil, to improve the load conditions on the pipe, and to direct the soil into the pipe interior instead of compacting it outside the pipe. Soil/pipe friction reduction is typically achieved with lubrication, and different types of bentonite and/or polymers can be used for this purpose. Depending on the length of the installation, the soils inside the pipe can be removed either during or after the installation by augering (most commonly), compressed air, or water jetting. For this method, the profile needs to be approximately horizontal and launching and receiving pits are required.

Pipe ramming is not-steerable, meaning that once the bore has begun there is little control of the line and grade of installation. Installation accuracy (vertically and horizontally) is usually about +/- 1% of the length of the bore, but subsurface obstructions or improperly aligned pipes may result in significant deviations from the desired line and grade. Another drawback is the possibility of significant soil disturbance and the potential for heave at the surface if a blockage is created at the end of the installed pipe below the travelled right of way.



For the ground conditions at this site, pipe ramming should be generally feasible. However, for the deep sanitary sewer it may not be possible to achieve the required tolerance for gravity flow of the pipe over an 80 metre long crossing with this method (unless a significantly oversized casing is used so as to allow correction of the profile during product pipe installation).

Horizontal Directional Drilling

Horizontal Directional Drilling (HDD) involves the drilling of a pilot hole, using a steerable drill bit on a flexible string of drill rods, while the bore is supported using a bentonite slurry. Once the pilot hole is complete, the bore would be reamed to a larger diameter, and then the pipe would be pulled through the bore (using the drill rods to pull the pipe into place).

The ground conditions at this site are considered to be suitable for the use of HDD methods. For the deep sanitary sewer, this would only be the case if there are no upward undulations of the bedrock into the sewer horizon.

HDD methods are more vulnerable to being impacted by the presence of obstructions, in comparison to auger-boring and pipe ramming. However, based on the subsurface conditions on this site, no obstructions such as cobbles or boulders in the overburden are expected.

The escape of slurry and cuttings outside of the drill hole is possible, depending on the circulation pressure, and the ground conditions at this site. The loss of drilling materials to the surrounding soil can lead to a 'frac-out' where the slurry and cuttings emerge at ground surface. However, due to the low hydraulic conductivity of the silty clay, a 'frac-out' is probably unlikely. Nonetheless, HDD contractors must closely monitor the rate of slurry and cuttings return as well as carry out continuous inspections of the ground surface above the pipe, to alert for any indications of possible 'frac-outs'.

Although HDD equipment is steerable, the precision of the drilling is limited. This methodology would *not* therefore be preferred if there is a very tight tolerance for the crossings, as may be the case for the sanitary sewer.

One advantage of this methodology is however that deep entrance and exit pits could potentially be avoided.

HDD methods may also be less likely to generate ground movements, since the cuttings are removed as part of the boring advancement.

The use of HDD methods would, however, likely require the use of more flexible pipe materials, to accommodate the curving profile. It is also not generally feasible to install a permanent steel casing, around the product pipe, which is a common requirement of MTO.

Rock Boring

Rock boring generally involves the initial drilling of a pilot hole in the bedrock and then back-reaming the bore to the required diameter. The equipment is generally not steerable and is best suited to construction along an approximately horizontal profile. Conventional drilling methods use roller-cone bits, an air-percussion hammer/down-the-hole-hammer, or disk cutters. Bores of up to 1828 millimetre diameter have been completed locally.

This method would only be considered if it is decided to lower the profile depth of the sanitary sewer to within the bedrock.



5.3.3 Evaluation of Trenchless Crossing Methods

5.3.3.1 *Forcemains*

The results of the geotechnical investigation indicate that the boring for the forcemain crossings will be carried out within the very stiff silty clay. Given the size of the pipe, the ground conditions, and the locally available contractors and equipment, it is expected that this crossing would likely be accomplished using pipe jacking and horizontal auger boring techniques. Pipe ramming could also be considered, however, it would likely be more costly. HDD could also be considered however, as mentioned previously, the MTO often required that crossings beneath highways be installed within a grouted steel casing to protect the highway against risk of blowout. Since HDD is carried out without the use of a casing, this method may not be feasible. This potential constraint should be confirmed with MTO so that the specification can state whether HDD methods would be permitted.

The following discussions are based on the expectation that the work will be carried out using pipe jacking and horizontal auger boring. If other methods would be used, these guidelines would need to be re-evaluated.

5.3.3.2 *Sanitary Sewer*

Similarly, the results of the geotechnical investigation indicate that the boring for the sanitary sewer crossing will be carried out within the very stiff silty clay. Therefore, as above, it is expected that this crossing would likely be accomplished using pipe jacking and horizontal auger boring techniques. Pipe ramming could also be considered but is limited by the lack of profile control and higher costs. Likewise, HDD has limitations with regards to grade control and cannot be carried out with a casing. Rock boring is also considered a feasible option provided the grade of the sewer is lowered adequately to be entirely within the bedrock, such as might be considered as a measure to reduce the risk of encountering an upward undulation of the bedrock along the currently proposed profile.

Should the bedrock be encountered while constructing the crossing using pipe jacking and horizontal auger boring, it would likely be necessary to withdraw the auger string to allow worker access to the face so that the bedrock could be removed manually, which would be costly and likely induce settlement of the highway. Given the limited clearance with the bedrock, the current profile involves a potential risk of such difficulties. It is also noted however that the bedrock elevation beneath the central portion of the bore (where encountering bedrock would be the most problematic) is actually lower than at the ends (i.e., the least amount of clearance is in the areas outside of the roadway platform).

5.3.4 Ground Behaviour and Settlements

The silty clay deposit at this site is considered to be very stiff in consistency. In accordance with the Terzaghi's Tunnelman's Ground Classification System, the ground at this site would be classified as 'ravelling' or 'firm'.

To reduce the risk of ground settlement/subsidence, trenchless installations require a minimum depth of overburden cover over the tunnel crown. As the depth of overburden cover decreases, the risk of concentrated subsidence increases, as does the risk of extreme events such as sinkholes forming at the ground surface. In Ontario the general practice is to maintain a depth of cover equivalent to 2 to 3 tunnel diameters. The proposed pipe profiles for the forcemains and sanitary sewer satisfy this requirement, however as mentioned previously, the north and south ends of the forcemain crossings have less than one tunnel diameter of cover in the area of the Highway 7 ditches. Therefore, there could be potential for ground settlements in these areas. There could also be some horizontal shifting of the ditch side slope, on the receiving pit side, which could need to be repaired.



The potential settlement also depends on the construction practices. To avoid excessive settlements:

- Augering ahead of the carrier pipe should not be permitted.
- Pauses/delays in the jacking operation should be minimized.

Based on the proposed pipe profiles and provided good construction procedures are used, excessive settlements of the pavement surface are not expected for this site. The settlements of the pavement surface would not be expected to exceed 10 millimetres, and would probably be less than 5 millimetres.

Should, however, the casing encounter obstructions which prevent its advancement and the casing needs to be cleaned-out so that hand-mining can be used to remove the obstruction, the potential for larger settlements to occur will need to be evaluated and monitored.

Furthermore, it is expected that the tunnel construction will not result in measureable ground surface settlements beyond a distance on either side of the tunnel alignment exceeding the tunnel depth. There are no structures or known utilities located within this area of influence (with the possible exception of light poles along Highway 7); however, there are utilities at shallower depth which cross above the tunnel alignment (i.e., a buried hydro line within the Highway 7 right-of-way).

5.3.5 Instrumentation and Monitoring

A settlement monitoring program needs to be implemented for this project so as to:

- Document the effects of the trenchless construction.
- Obtain prior warning of ground movements that could occur due to the construction methods.
- Allow adjustments to be made to the construction methods such that the settlement limits established are not exceeded, recognizing however that there is typically some delay between the tunnel construction and the full manifestation of the ground surface settlements.

The reference standard for preparation of this monitoring plan is MTO's "Guidelines for Foundation Engineering – Tunnelling Specialty For Corridor Encroachment Permit Application." The appendix to that document ('Settlement Monitoring Guidelines – Tunnelling') outlines typical requirements for settlement monitoring. The plan should incorporate the following key components:

- Installation or establishment of survey benchmarks by a qualified surveyor outside of the zone of influence of the works.
- Installation of surface settlement pins across the roadway at minimum 5 metre intervals along the centreline of the tunnel alignment. The surface monitoring points installed on the unpaved right-of-way will need to be founded below frost penetration depths if work is carried out during or just before the winter.
- Precise elevation surveys (+/- 3 millimetre accuracy) of benchmarks and surface settlement pins at the following times:
 - Two complete sets of daily baseline readings taken prior to the work commencing.



- A complete set of readings at a minimum frequency of 3 times daily at approximately equally spaced intervals during the day, from the start to finish of trenchless operations, including during work stoppages (non-operation periods).
- A final set of readings approximately 2 weeks after the completion of the work.

In the event that difficult/unexpected ground conditions are encountered, or there is a mechanical breakdown, and the drilling equipment needs to be removed from the bore, increased monitoring could be required. As an initial guideline, a complete set of readings taken at one-hour intervals until the situation is resolved and regular operations are resumed could be required. Monitoring should also be extended after the completion of construction if the post-construction readings indicate some movement.

A detailed settlement monitoring plan should be submitted by the contractor.

Input to specifications can be provided, upon request.

MTO typically requires that settlements be limited to no more than 10 to 15 millimetres. If greater settlement occurs, the contractor could be required to repair the pavement.

5.3.6 Liner Design

The casing must be designed to accommodate hoop stress and sufficient nominal wall thickness must be provided to meet this design requirement. The casing should conform to OPSS 1802 (Smooth Walled Steel Pipe) and at least the nominal wall thickness provided in Table 1 of this specification will apply.

The casing must also be designed to withstand the jacking stresses. Additional wall thickness could be required.

5.4 Shaft Construction

Based on the expectation that pipe jacking and horizontal auger boring will be the selected construction method the excavations for the shafts will extend to be about 6 metres depth below the existing ground surface. It is assumed that one shaft will be constructed for all of the trenchless crossings, increasing in excavation depth as required. The excavations at the shafts will be through the topsoil, fill, very stiff silty clay, glacial till and ultimately to the bedrock surface at about 6 metres depth (since the shaft typically needs to extend to about 1 metre below the casing level to accommodate the boring equipment).

The following general comments are offered in regards to construction of the shafts for the crossings:

- Although very short-term open-cut excavation sides slopes constructed to shallow depth in the overburden soils might be feasible, it is generally considered that the shafts will need to be shored (due to their depth, the duration that the shafts will be open, and the proximity of existing site features such as Highway 7).
- Shoring of the overburden soils can likely be achieved using conventional techniques (e.g., soldier piles and lagging, sheet piling, etc.). Guidelines for the shaft excavations and shoring are provided in the following sections.
- The silty clay is susceptible to disturbance from construction traffic. A granular (or concrete) working pad will be required at the base of the shafts, if founded in the silty clay. The type of material and the thickness of the pad will depend on the contractor's equipment; however, a 0.3 metre thickness of OPSS



Granular B Type II with geotextile should generally be appropriate. A mud slab of lean concrete would also be suitable.

5.4.1 Excavations and Excavation Shoring

General recommendations for excavation through the overburden are provided in Section 5.2.2 and are equally applicable to these shafts. As described in Section 5.2.2, basal instability is not considered a concern for the shaft excavations.

5.4.2 Excavation Shoring

Due to the proximity of the existing roadway and services, and the depth of excavation, it is expected that the excavations for the receiving and jacking shafts will likely require vertical (or near vertical) walls supported by an engineered shoring system. The recommendations provided in Section 5.2.3 with regards to excavation shoring for the pumping station are also applicable to the shafts for the trenchless crossing. However, the following provides some additional considerations.

Based on the conditions encountered, it is expected that rigid steel sheet pile shoring or soldier piles and lagging, with interior struts or bracing, would also be appropriate for the crossing. The use of double-stacked trench boxes with liner plates may also be feasible at this site. However, the final selection of the type of temporary shoring system, and the method of lateral restraint, should be entirely the choice/responsibility of the contractor.

For the shaft excavations, although there is a need to limit the ground movements around the shoring to avoid potential damage to existing roads and buried infrastructure, it cannot be predicted with certainty which shoring system would be proposed by the bidding contractors as being the most feasible and economic. It is therefore proposed that the contract documents specify a restriction on the ground movements and allow the contractor to propose the shoring system to be used (as discussed below in Section 5.4.3).

The contractor should be required to submit the shoring system design, including details on the design lateral earth pressures, expected movements, and a monitoring plan, for review prior to the start of shoring construction.

The selected shoring method will need to be designed to resist jacking or ramming forces applied to the wall as a backstop.

Stockpiling of soil beside the excavations should be avoided; the weight of the stockpiled soil could lead to overstressing of the shoring system.

5.4.3 Ground Movements (Adjacent to Shoring)

Some unavoidable inward horizontal deformation and vertical settlement of the adjacent ground may occur as a result of excavation, installation of the shoring, deflection of the ground support system (including bending of the walls and compression of the struts), as well as deformation of the soil in which the toes of the walls are embedded. The ground movements induced could affect the performance of surface structures or underground utilities adjacent to the excavation, within a horizontal distance of about three times the shaft depth.

The resulting ground settlement will depend on the selected shoring alternative. As a preliminary guideline, typical settlements behind sheet pile or soldier pile and lagging shoring are anticipated to be less than about 0.3 percent of the excavation depth, provided that good construction practices are used (e.g., that supports are installed as soon as the support level is reached), and that voids are not left behind the lagging. This guideline would suggest



less than about 20 millimetres of ground settlement could occur for the shafts on either side of Highway 7. Provided the shafts are constructed outside of the MTO right-of-way, Highway 7 shouldn't be impacted.

As discussed in Section 5.2.4, the construction documents should not specify the specific shoring system that should be used, but rather the permissible deflection level (i.e., 'Performance Level') should be specified, in accordance with OPSS 539. With the above design approach, it is considered that Performance Level 2 would be specified (subject to the approval of MTO).

However, the above guidelines are only preliminary and are provided only to assist the owner's designers in carrying out an initial assessment of the expected settlements and the potential impacts of these settlements. A more detailed assessment of the expected settlements should be undertaken by the contractor. Should the preliminary assessment carried out using this estimated settlement indicate unacceptably large settlements to adjacent structures, roadways, or utilities, then a more detailed assessment should be carried out at the design stage (prior to tender) to better assess the shoring requirements, or a more rigid form of shoring should be selected.

A preconstruction survey should be carried out of any adjacent structures (e.g., pavements) prior to the commencement of excavation. The magnitude of ground movements adjacent to the shafts should be monitored throughout the construction period. The expected levels of deformation should be established by the contractor and alert levels should be set at which the designers should review the deformation and consider modifications to the design and/or construction procedures.

5.4.4 Groundwater Management

The measured groundwater levels are typically within about 0.3 metres of the ground surface at the pit locations. Excavations for construction of the shafts would therefore extend below the groundwater level, and groundwater inflow into the excavations should be expected.

The shafts will primarily be made in silty clay and however the floor of the excavation will likely extend to the bedrock surface. It should generally be possible to handle the groundwater inflow from the overburden soils by pumping from well filtered sumps within the excavations. However, as previously discussed in Section 5.2.5, significant groundwater inflows are anticipated from the dolostone bedrock.

The Thiem analytical solution for confined conditions was used to conservatively estimate the steady-state groundwater inflow through the exposed bedrock into the shaft at the south end of the crossing. The hydraulic conductivity of the bedrock in this area was measured as 2×10^{-2} centimetres per second, based on the results of the in situ hydraulic conductivity testing completed at borehole 15-3. Preliminary estimates of the volume of groundwater inflow were calculated by assuming the shaft measures 15 metres long by 10 metres wide at the surface by 6 metres deep. The groundwater elevation measured at borehole 15-3 in March 2015 (128.3 metres elevation) was used in the assessment. The groundwater level in the bedrock would be lowered to 122.8 metres elevation, 0.5 metres below the bedrock elevation for the construction of the shaft. Based on these values and applying a 1.5 factor of safety, the estimated steady-state groundwater inflow would be approximately 2,400,000 Litres per day for the southern shaft. In addition, it can be conservatively assumed that a similar groundwater inflow would be expected at the north shaft (i.e., the estimated inflow for both shafts combined could be in the order of 4,800,000 Litres per day). Initial inflows may be higher, depending on the excavation schedule and methodology. The predicted radius of influence is approximately 100 metres.



The actual rate of groundwater inflow to the excavations will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavations, the material (silty clay, glacial till or bedrock), incident precipitation, and the time of year at which the excavations are made (e.g., fluctuation in seasonal groundwater elevation). The excavations would be best carried out in the drier summer months. Incident precipitation could add approximately 10,800 Litres of water per day to the excavations, based on the excavation geometry described above and a 72 millimetre precipitation event (a 10 year event as observed at the Ottawa International Airport weather station). As previously mentioned in Section 5.2.5, the expected groundwater inflows will exceed 50,000 Litres per day and therefore a PTTW will be required.

It is important to note that groundwater inflow into the excavation will decrease over time as the overburden materials and bedrock dewater. During the progression to steady-state and once steady-state is reached, short-term increases in groundwater inflows would be expected following precipitation events if the overburden or bedrock is recharged and subsequently drains into the excavation.

The dolostone bedrock within the vicinity of the site has measured hydraulic conductivities in the range of 10^{-3} to 10^{-2} centimetres per second, which are high. Therefore, significant groundwater inflows are expected for the shaft construction down to the bedrock surface. It may, in fact, be necessary to carry out some groundwater pumping in advance of excavation and construction, similar to the proposed groundwater control measures recommended for the pumping station excavation. Pre-pumping from wells installed in the bedrock in the area of the shaft might be a feasible method of lowering the groundwater level and thereby reducing the groundwater inflow into the construction area. This methodology should be considered by the contractor.

Even if the contractor doesn't plan to excavate the shafts right to the bedrock surface, lowering of the groundwater level in the bedrock in advance of the excavation will be required so as to avoid basal heaving of the floor of the excavation due to hydrostatic pressure. Basal heaving could lead to disturbance of the pipe subgrade (and post-construction settlements) along with flooding of the excavation, and destabilizing of the shoring.

5.5 Sanitary Sewer and Forcemain Construction

Approximately 350 metres of new sanitary sewer (up to 525 millimetres in diameter) will be constructed to convey flows to the pumping station. The new sewer will be aligned along the east side of the Canadian Tire property, run eastward along the north side of Highway 7, then cross under Highway 7 to connect to the new pumping station. A short piece of new sewer will also be needed to the south of the pumping station. The sewer will be installed at depths of about 3 to 6 metres.

5.5.1 Excavations

Open cut excavations for construction of the sanitary sewer and forcemains on the north and south sides of Highway 7 will extend through surficial materials and into the underlying native silty clay and potentially into the glacial till at the north end of the alignment. Excavations deeper than about 0.1 metres will likely extend below the groundwater level.

Based on the current profile and the borehole information, it appears that bedrock excavation will not be required for the open cut sewer and forcemain installation. However, the sewer inverts are only marginally above the bedrock level and upward undulations in the bedrock surface between boreholes are possible.



No unusual problems are anticipated with excavating the overburden using conventional hydraulic excavating equipment. However, it should be expected that boulders will be encountered within the glacial till. Boulders larger than 0.3 metres in size should be removed from the excavation side slopes.

In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the topsoil, surficial fill, very stiff silty clay and glacial till be classified as Type 3 soils. Accordingly, side slopes in these materials may be temporarily sloped at no steeper than 1 horizontal to 1 vertical. Alternatively, the excavations could be carried out using steeper side slopes with all manual labour carried out within a fully braced, steel trench box for worker safety.

For the sanitary sewer excavations, it may be necessary to actively lower the groundwater level in the underlying bedrock in advance of the excavation. The design excavation levels (where above the bedrock surface) would leave only a very limited thickness of overburden between the excavation floor level and the surface of the underlying bedrock. The groundwater level in the bedrock was measured within about 0.1 metres of the ground surface. There is therefore the potential for basal heaving of the floor of the excavation, since the piezometric pressure in the bedrock would exceed the weight of the overlying silty clay and glacial till. Heaving of the excavation floor could lead to disturbance of the pipe subgrade (and post-construction settlements) along with flooding and destabilization of the excavation. It should therefore be planned to depressurize the underlying bedrock in advance of excavation, similarly to the proposed groundwater control measures recommended for the pumping station and trenchless crossing shaft excavations. It is noted, however, that pre-pumping from wells installed in the bedrock in the area of the trenchless crossing shafts might be a feasible method of lowering the groundwater level along the sewer alignment as well. This methodology could be considered by the contractor. As mentioned in Sections 5.2.5 and 5.4.4 a PTTW should also be obtained from the provincial MOECC for this work.

Basal instability (i.e., shearing of the clay under the excavation/trench box due to the weight of retained soil) is not considered a concern for the sanitary sewer and forcemain excavations.

5.5.2 Pipe Bedding and Cover

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for the sewers. The excavations will extend into the silty clay, and some unavoidable disturbance to the subgrade surface may occur. Therefore, a sub-bedding layer consisting of 300 millimetres of compacted OPSS Granular B Type II beneath the Granular A or some thickness of additional Granular A bedding may be required. The bedding material should in all cases extend to the spring line of the pipe and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support or surface settlement.

Cover material, from spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.



5.5.3 Trench Backfill

It should generally be possible to re-use the drier silty clay and glacial till as trench backfill.

Where the trench will be covered with hard surfaced areas, the type of native material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

The width of the trenches for the sanitary sewer and forcemain lines on the north side of Highway 7, within the easement between proposed structures of the planned residential development, should be limited as not to undermine the foundations of future houses (i.e., the trenches should be constructed to be entirely outside of the 1 horizontal to 1 vertical projection from the edge of the house foundations). It will be important that the excavation be controlled/supervised to ensure that this requirement is met.

The above recommendations regarding trench backfill are also based on the assumption that the forcemains will not be founded within the backfill of the deeper sanitary sewer.

5.5.4 Impermeable Cutoffs

Impervious dykes or cut-offs should be constructed at 100 metre intervals in the service trenches to reduce groundwater lowering at the site due to the "french drain" effect of the granular bedding and surround for the service pipes (which could otherwise lead to surrounding groundwater level lowering in the silty clay deposits and settlement of nearby structures). It is important that these barriers extend from trench wall to trench wall and that they fully penetrate the granular materials to the trench bottom. The dykes should be at least 1.5 metres wide and could be constructed using relatively dry (i.e., compactable) grey brown silty clay from the weathered zone.

5.5.5 Thrust Restraint

Thrust restraint for the forcemains could be provided either by means of friction between the forcemain pipe and the granular bedding (restrained joints) or by thrust blocks.

For preliminary design purposes, a coefficient of friction of 0.25 may be used between the granular pipe bedding and the forcemain, assuming that the pipe bedding and surround are adequately compacted in place and in intimate contact with the pipe.

The allowable bearing pressure for thrust blocks installed within compacted granular backfill and/or bearing on the native inorganic soils may be taken as 100 kilopascals. Thrust blocks should not be designed to bear against uncompacted fill.



5.6 Groundwater Management Considerations

5.6.1 Permit to Take Water

Based on the predicted inflows for the pumping station excavation and shaft construction at the trenchless crossings as well as for the sewer installation, a Permit-To-Take-Water (PTTW) will be required, because the volumes of water to be taken at the site are expected to exceed 50,000 Litres per day. The maximum water taking allowed under a Category 2 Permit to Take Water (up to 400,000 Litres per day) will be exceeded; therefore a Category 3 Permit would be required.

A PTTW application package can be prepared for submission to the Environmental Assessment and Approvals Branch of the MOECC once the timing of construction is better known. The PTTW application would include a technical study, which is required to evaluate the rate of groundwater taking and any potential impacts due to the groundwater taking.

Due to the large quantity of expected groundwater inflow and associated volume of water that will need to be discharged, further groundwater sampling and testing should be carried out, once an outlet with sufficient capacity has been identified to confirm that the groundwater quality is appropriate for the discharge (i.e., meets the regulatory requirements). However, the dewatering or excavation contractor should be made responsible for obtaining the necessary permits for discharge (to a sewer or drainage system) and ensuring compliance with the applicable MOECC or municipal requirements.

Due the large estimated groundwater inflows at this site, consideration should be given to carrying out pumping tests at the locations of the pumping station and trenchless crossing shaft to better confirm/refine the groundwater inflow estimates. This could be carried out as part of the PTTW scope of work.

5.6.2 Impacts due to Dewatering

The following provides a preliminary assessment of the impacts of dewatering at the site.

Temporary groundwater level lowering will be required for (or induced by) the construction of the pumping station excavation and shaft construction. For the PTTW, the impacts of temporary groundwater level lowering will need to be evaluated with regards to:

- Impacts to nearby groundwater users and receptors (e.g., nearby creek);
- The potential for mobilization of contaminants; and,
- Impacts to nearby structures due to ground settlements.

The first two items will be evaluated as part of the separate scope of work needed for preparing the PTTW application; however the following provides some preliminary considerations with regards to the impacts on adjacent structures.

The investigation determined that the site, and likely the adjacent properties, is underlain by sensitive silty clay materials, which can settle if subjected to significant reduction in pore pressure over an extended period of time. Commercial buildings have been identified within the predicted radius of influences of the dewatering needed for the pumping station excavation and shaft construction. Given the relatively limited groundwater level lowering anticipated during construction dewatering, the distance of other buildings from the proposed pumping station excavation, and the stiffness of the silty clay, settlement of the sensitive silty clay deposit is not expected to be significant and impacts to nearby structures are not anticipated. However, this assessment should be confirmed as part of the PTTW application study, when the duration and extent of groundwater lowering is known.



Even if it is expected that any settlements resulting from the calculated drawdowns would be minimal in magnitude, and that no damage to the nearby commercial and residential buildings, and services or impact on their serviceability would be expected, it would be prudent to carry out a precise survey/monitoring of the elevations of selected structures in the nearby developed areas underlain by silty clay. This work could be carried out by the contractor.

The purpose of this program would be to protect the Town of Carleton Place from damage claims that are unrelated to the construction and to provide baseline information against which such claims can be evaluated.

A more detailed evaluation of the potential impacts due to dewatering such as to nearby groundwater users and receptors, to the potential for mobilization of contaminants, and to impacts on nearby structures due to ground settlements will need to be confirmed once more details with regards to the construction methods (i.e. excavation shoring, dewatering plan, etc.) and duration are known. This assessment will be required as part of the PTTW application.

5.7 Soil Management Considerations

During the geotechnical subsurface investigation, soil samples were collected for potential submission to a laboratory for analytical testing. Two representative samples were selected for analysis based on location (coverage across the site), material type (fill versus native soil) and location relative to the inferred groundwater table.

A sample from borehole 15-1 was collected from the south side of Highway 7 (pumping station site) from the native silty clay material from a depth of 3.8 to 4.4 metres below ground surface. A sample from borehole 15-6 was collected from the north side of Highway 7 from silty clay fill from a depth of 0.1 to 0.7 metres below ground surface.

Excavated soil generated during future construction activities at the site will require management which may include beneficial reuse or disposal, depending on its environmental quality.

5.7.1 Regulatory Framework

The management of excavated soil/fill in the Province of Ontario is regulated under the ***Environmental Protection Act (1990)*** which is managed and enforced by the Ontario Ministry of the Environment and Climate Change (MOECC) through a number of environmental regulations and guidelines. Relevant regulations and guidelines with respect to the management of excavated materials include the following:

- ***Ontario Regulation 347, 1990, as amended – General Waste Management.*** This regulation provides classifications for the various regulated waste streams, including excavated soil (Section 2 of the Regulation). Under this regulation excavated soil or fill is designated as a waste and is subject to Part V of the Environmental Protection Act (waste management), unless it is exempted under Section 3 of the Regulation, which includes a provision for inert fill. Inert fill is defined under the regulation as “*earth or rock fill or waste of a similar nature that contains no putrescible materials or soluble or decomposable chemical substances*” (O.Reg. 347). In addition, O.Reg. 347 requires Toxicity Characteristic Leaching Procedure (TCLP) testing to be performed for soil disposal purposes to characterise if the soil is classified as hazardous or non-hazardous solid waste.



- **Ontario Regulation 153/04, as amended – Record of Site Condition:** The Record of Site Condition Regulation (Part XV.1 of the Environmental Protection Act) outlines the approved process for the assessment, investigation, remediation and risk assessment for contaminated sites. This regulation also provides the soil and groundwater quality standards used to evaluate contaminated sites in Ontario for the purpose of filing a record of site condition (*Soil, Ground Water and Sediment Standards for Use under Part XV.1 of the Environmental Protection Act (MOE 2011)*). These standards have been updated most recently in April 2011. The Regulation also contains a process for the testing and movement of imported and exported soil in the context of completion of a record of site condition. Although not directly applicable to classifying excess soil, these standards can be used by a Qualified Person as a comparative set of standards for evaluating excess soil material, where appropriate.
- **MOECC Guideline: *Management of Excess Soil – A Guide for Best Management Practices (2014)*:** The Best Management Practices document outlines the most current accepted approach to managing excess soil in the Province of Ontario and was developed around the concept of beneficial reuse of excess soil outside the waste stream. Beneficial reuse of excess soil can be considered when it can be demonstrated that the excess soil is appropriate and suitable for reuse at a specific Receiving Site, as determined by a Qualified Person. In certain instances pre-consultation with the local municipality and Conservation Authority may be warranted. The guidance document outlines specific considerations for source sites, receiving sites and temporary storage sites including the requirements for a soil management plan (source site) and fill management plan (receiving site).

Based on Golder's interpretation of the above referenced provincial Regulations and Guidelines, current accepted industry practice, and supplementary guidance provided by the MOECC, the following subsections outline the recommended approach for classifying excavated materials.

5.7.2 Applicable Site Condition Standards

5.7.2.1 On-Site Considerations

Based on the current use of the site as agricultural and roadway use (Highway 7) and the proposed use of the site (infrastructure corridor), the following are considered the applicable site condition standards for the evaluation of soil which will remain on the site following construction of the sewage pumping station and associated lines:

- *Soil, Ground Water and Sediment Standards for Use under Part XV.1 of the Environmental Protection Act (MOECC 2009)*, as required under Ontario Regulation 153/04, as amended (April 15, 2011).
 - Table 2: Potable Ground Water Condition Standards (Industrial/Commercial/Community property use, coarse grained material).

5.7.2.2 Off-Site Considerations (Excess Soil)

Excess soil should be managed in accordance with the MOECC guidance document entitled "*Management of Excess Soil – A Guide for Best Management Practices*", dated January 2014 (on and off-site management guidance). In instances where excess soil cannot be beneficially reused and becomes a waste, soil quality needs to be evaluated in comparison to the following:

- Ontario Regulation 347 General Waste Management: Schedule 4 Leachate Quality Criteria; and,
- Any applicable disposal testing required by the licensed waste disposal facility.



5.7.2.3 Applying Standards for Material Classification

The following section outlines how the above documents will be applied to soil classification for the project:

- Material which meets the MOECC Table 1 Standards may be considered as “inert fill” and is likely suitable for re-use off-site as clean fill without further considerations or can be reused on the site;
- Material which meets the standards applicable to the site (MOECC Table 2 Standards) could be re-used on the project site. If on-site reuse is not possible, a suitable re-use or disposal location should be identified in consultation with a Qualified Person. This may include beneficial reuse as general fill at a location other than a waste disposal site as prescribed in the MOECC best practices for excavated soil management (i.e., at other similar sites) or disposal at a licensed waste disposal site.

5.7.3 Soil Analytical Results

A total of two soil samples were submitted for laboratory analysis of the following: petroleum hydrocarbons fraction 1 to fraction 4 including benzene, toluene, ethylbenzene and xylenes (PHCs F1-F4 + BTEX), volatile organic compounds (VOCs) and metals. The soil analytical results were compared to applicable on-site Standards (MOECC Table 2) and comparative off-site Standards (MOECC Table 1), which are summarized below. Laboratory reports of analysis for the soil samples are included in Appendix C:

- There were no parameter exceedances of the on-site Standards (MOECC Table 2).
- The soil sample from borehole 15-1 had a concentration of barium that exceeded the MOECC Table 1 Standards. The concentration of barium in borehole 15-1 is 270 µg/g. The MOECC Table 1 standard for barium is 220 µg/g.

5.7.4 Soil Management Options

Based on the limited analytical data obtained from the analysis of the two soil samples on site, the following recommendations are made for management of excess soils at the site:

- There were no exceedances of the MOECC Table 2 Standards (applicable for on-site soil reuse). Therefore the soil located on-site could be reused on the project site.
- The soil sample collected from borehole 15-1 did not meet the MOECC Table 1 Standards and therefore excess soil that is removed from that area of the site (i.e., the clay that is excavated at the pumping station and presumably from the rest of the project) may not be considered as inert fill at other sites. A suitable receiving site will need to be identified in consultation with a Qualified Person, as outlined in the MOECC best management practices guidance document for excavated material. In the absence of a suitable fill receiving site, landfill disposal would be required. It would therefore be preferred to re-use the excavated soil on-site, if possible.

Although efforts have been made to identify areas of soil impacts at the site during the subsurface investigations, there is still the potential that contaminated soil exists between testing locations. As such, a contingency plan should be developed by the contractor to deal with unexpected soil impacts; thereby reducing the risk to project costs and delays in the event that additional impacts are identified.



5.8 Pavement

In preparation for pavement construction, all unsuitable material (e.g., topsoil and fill materials containing organic or deleterious material) should be excavated from all pavement areas at the pumping station site.

Pavement areas requiring grade raising to proposed subgrade level should be filled using acceptable OPSS 1010 Select Subgrade Material (SSM) or OPSS 212 Earth Borrow. The SSM or Earth Borrow should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

The surface of the subgrade or fill should be crowned to promote drainage of the pavement granular structure. Perforated pipe subdrains should be provided at subgrade level extending from the catch basins for a distance of at least 3 metres in four orthogonal directions, or longitudinally where parallel to a curb.

The pavement structure for car parking areas should consist of:

| Pavement Component | Thickness (millimetres) |
|---------------------------------|-------------------------|
| Asphaltic Concrete | 50 |
| OPSS Granular A Base | 150 |
| OPSS Granular B Type II Subbase | 300 |

The pavement structure for truck traffic areas should consist of:

| Pavement Component | Thickness (millimetres) |
|---------------------------------|-------------------------|
| Asphaltic Concrete | 90 |
| OPSS Granular A Base | 150 |
| OPSS Granular B Type II Subbase | 450 |

The granular base and subbase materials should be uniformly compacted as per OPSS 501, Method A. The asphaltic concrete should be compacted in accordance with the procedures outlined in OPSS 310

The composition of the asphaltic concrete pavement should be as follows:

- Superpave 12.5 mm Surface Course 40 mm
- Superpave 19 mm Base Course 50 mm

The asphaltic cement should consist of PG 58-34 and the design of the mixes should be based on a Traffic Category B.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.



5.9 Corrosion and Cement Type

Soil samples from boreholes 15-3 and 15-5 were submitted to EXOVA Laboratories Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of the testing are provided in Appendix D.

The results indicate that concrete made with Type GU Portland cement should be acceptable for substructures. The results also indicate a potential for corrosion of exposed ferrous metal, which should be considered in the design of substructures. In addition, the trenchless crossings will pass beneath Highway 7 which is presumably heavily salted during the winter months of the year thereby creating a very corrosive environment for the buried pipes. This should also be taken into consideration in the design.

5.10 Trees

It should be noted that the silty clay encountered at the site is potentially highly sensitive to water depletion by trees of high water demand during periods of dry weather. When trees draw water from the silty clay, the silty clay undergoes shrinkage which can result in settlement of adjacent structures. The zone of influence of a tree is considered to be approximately equal to the height of the tree. Therefore trees which have a high water demand should not be planted closer to the pumping station than the ultimate height of the tree. The restrictions on trees can be waived where the footings will be founded below the silty clay deposit.

5.11 Contractor Submissions

The contractor's submissions requirements for this project should include:

- A excavation procedure and shoring design, which includes:
 - The lateral earth pressures used for the design;
 - An evaluation of the expected ground movements around the shoring; and,
 - A shoring movement monitoring plan.
- A preconstruction survey of the adjacent existing structures (e.g., RONA, light standards).
- A vibration monitoring plan.
- The equipment and methods proposed for the trenchless construction.
- A calculation of the required jacking force, in comparison to the jacking capacity of the equipment.
- Documentation that the backstop can resist the required jacking force.
- A settlement monitoring program.
- A dewatering plan for the pumping station and trenchless crossing shafts as well as the sewer construction.
- Soil and groundwater management plan.



6.0 ADDITIONAL CONSIDERATIONS

The soils at this site are sensitive to disturbance from ponded water, construction traffic and frost.

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill as well as sewer bedding and backfill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction point of view.

At the time of the writing of this report, only preliminary details of the project were available. Golder Associates should be retained to review the contract document and the guidelines provided in this report once additional details are known.

Due the large estimated groundwater inflows at this site, consideration should be given to carrying out pumping tests at the locations of the pumping station and trenchless crossing shaft to better confirm/refine the groundwater inflow estimates. This could be carried out as part of the PTTW scope of work.

In addition, a suitably sized sedimentation pond will need to be incorporated into the construction design to manage the expected groundwater pumping requirements. This pond may need to be very large.

It should also be noted that the soil and rock samples retrieved as part of the geotechnical investigation are generally only maintained for a period of 3 months following issuance of the report.

The groundwater level monitoring devices (i.e., standpipe piezometers or wells) installed at the site will require decommissioning at the time of construction in accordance with Ontario Regulation 128/03. However, it is expected that most of the wells can be more economically abandoned as part of the construction contract (which should be included in the specifications). If that is not the case or is not considered feasible, abandonment of the monitoring wells can be carried out separately.



GEOTECHNICAL INVESTIGATION - PROPOSED PUMPING STATION AND RELATED SEWER/FORCEMAIN LINES

7.0 CLOSURE

We trust this report satisfies your current requirements. If you have any questions regarding this report, please contact the undersigned.

GOLDER ASSOCIATES LTD.

Susan Trickey, P.Eng.
Geotechnical Engineer

Mike Cunningham, P.Eng.
Principal, Geotechnical Engineer



SAT/MIC/md/ob

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Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



APPENDIX A

Method of Soil Classification

**Abbreviations and Terms Used on Records of Boreholes
and Test Pits**

List of Symbols

Lithological and Geotechnical Rock Description Terminology

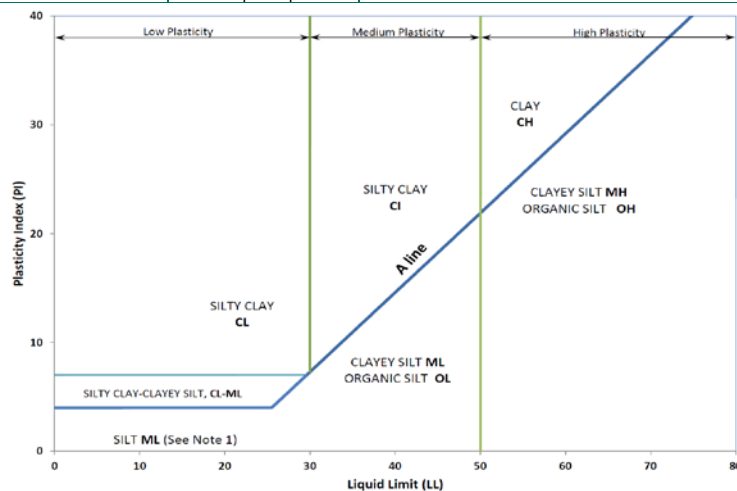
Record of Borehole and Probehole Sheets



METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

| Organic or Inorganic | Soil Group | Type of Soil | | Gradation or Plasticity | $Cu = \frac{D_{60}}{D_{10}}$ | | $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ | | | Organic Content | USCS Group Symbol | Group Name | | | |
|--|--|--|---|---|------------------------------|------------------|--|----------------|--------------|-----------------|------------------------------|-----------------|------------------------|--------------|--|
| INORGANIC (Organic Content $\leq 30\%$ by mass) | COARSE-GRAINED SOILS ($>50\%$ by mass is larger than 0.075 mm) | GRAVELS ($>50\%$ by mass of coarse fraction is larger than 4.75 mm) | Gravels with $\leq 12\%$ fines (by mass) | Poorly Graded | <4 | | ≤ 1 or ≥ 3 | | | $\leq 30\%$ | GP | GRAVEL | | | |
| | | | | Well Graded | ≥ 4 | | 1 to 3 | | | | GW | GRAVEL | | | |
| | | | Gravels with $>12\%$ fines (by mass) | Below A Line | n/a | | | | | | GM | SILTY GRAVEL | | | |
| | | | | Above A Line | n/a | | | | | | GC | CLAYEY GRAVEL | | | |
| | | SANDS ($\geq 50\%$ by mass of coarse fraction is smaller than 4.75 mm) | Sands with $\leq 12\%$ fines (by mass) | Poorly Graded | <6 | | ≤ 1 or ≥ 3 | | | | SP | SAND | | | |
| | | | | Well Graded | ≥ 6 | | 1 to 3 | | | | SW | SAND | | | |
| | | | Sands with $>12\%$ fines (by mass) | Below A Line | n/a | | | | | | SM | SILTY SAND | | | |
| | | | | Above A Line | n/a | | | | | | SC | CLAYEY SAND | | | |
| | | Organic or Inorganic | Soil Group | Type of Soil | | Laboratory Tests | Field Indicators | | | | | Organic Content | USCS Group Symbol | Primary Name | |
| | | INORGANIC (Organic Content $\leq 30\%$ by mass) | FINE-GRAINED SOILS ($\geq 50\%$ by mass is smaller than 0.075 mm) | SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below) | Liquid Limit <50 | Rapid | None | None | >6 mm | | N/A (can't roll 3 mm thread) | $<5\%$ | ML | SILT | |
| Slow | None to Low | | | | | Dull | 3mm to 6 mm | None to low | $<5\%$ | ML | CLAYEY SILT | | | | |
| Slow to very slow | Low to medium | | | | | Dull to slight | 3mm to 6 mm | Low | 5% to 30% | OL | ORGANIC SILT | | | | |
| Liquid Limit ≥ 50 | Slow to very slow | | | | Low to medium | Slight | 3mm to 6 mm | Low to medium | $<5\%$ | MH | CLAYEY SILT | | | | |
| | None | | | | Medium to high | Dull to slight | 1 mm to 3 mm | Medium to high | 5% to 30% | OH | ORGANIC SILT | | | | |
| CLAYS (PI and LL plot above A-Line on Plasticity Chart below) | Liquid Limit <30 | | | None | Low to medium | Slight to shiny | ~ 3 mm | Low to medium | 0% to 30% | CL | SILTY CLAY | | | | |
| | Liquid Limit 30 to 50 | | | None | Medium to high | Slight to shiny | 1 mm to 3 mm | Medium | (see Note 2) | CI | SILTY CLAY | | | | |
| | Liquid Limit ≥ 50 | | | None | High | Shiny | <1 mm | High | | CH | CLAY | | | | |
| HIGHLY ORGANIC SOILS (Organic Content $>30\%$ by mass) | | | | Peat and mineral soil mixtures | | | | | | | 30% to 75% | PT | SILTY PEAT, SANDY PEAT | | |
| | | | | Predominantly peat, may contain some mineral soil, fibrous or amorphous peat | | | | | | | 75% to 100% | | PEAT | | |



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.

Note 2 – For soils with $<5\%$ organic content, include the descriptor “trace organics” for soils with between 5% and 30% organic content include the prefix “organic” before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.

For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between “clean” and “dirty” sand or gravel.

For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.

A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.



ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

| Soil Constituent | Particle Size Description | Millimetres | Inches (US Std. Sieve Size) |
|------------------|---------------------------|---|--|
| BOULDERS | Not Applicable | >300 | >12 |
| COBBLES | Not Applicable | 75 to 300 | 3 to 12 |
| GRAVEL | Coarse Fine | 19 to 75 4.75 to 19 | 0.75 to 3 (4) to 0.75 |
| SAND | Coarse Medium Fine | 2.00 to 4.75 0.425 to 2.00 0.075 to 0.425 | (10) to (4) (40) to (10) (200) to (40) |
| SILT/CLAY | Classified by plasticity | <0.075 | < (200) |

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

| Percentage by Mass | Modifier |
|--------------------|--|
| >35 | Use 'and' to combine major constituents (i.e., SAND and GRAVEL, SAND and CLAY) |
| > 12 to 35 | Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable |
| > 5 to 12 | some |
| ≤ 5 | trace |

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d :

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

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

SAMPLES

| | |
|----------|--|
| AS | Auger sample |
| BS | Block sample |
| CS | Chunk sample |
| DO or DP | Seamless open ended, driven or pushed tube sampler – note size |
| DS | Denison type sample |
| FS | Foil sample |
| RC | Rock core |
| SC | Soil core |
| SS | Split spoon sampler – note size |
| ST | Slotted tube |
| TO | Thin-walled, open – note size |
| TP | Thin-walled, piston – note size |
| WS | Wash sample |

SOIL TESTS

| | |
|-----------------|---|
| w | water content |
| PL, w_p | plastic limit |
| LL, w_L | liquid limit |
| C | consolidation (oedometer) test |
| CHEM | chemical analysis (refer to text) |
| CID | consolidated isotropically drained triaxial test ¹ |
| CIU | consolidated isotropically undrained triaxial test with porewater pressure measurement ¹ |
| D_R | relative density (specific gravity, G_s) |
| DS | direct shear test |
| GS | specific gravity |
| M | sieve analysis for particle size |
| MH | combined sieve and hydrometer (H) analysis |
| MPC | Modified Proctor compaction test |
| SPC | Standard Proctor compaction test |
| OC | organic content test |
| SO ₄ | concentration of water-soluble sulphates |
| UC | unconfined compression test |
| UU | unconsolidated undrained triaxial test |
| V (FV) | field vane (LV-laboratory vane test) |
| Y | unit weight |

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

NON-COHESIVE (COHESIONLESS) SOILS

Compactness²

| Term | SPT 'N' (blows/0.3m) ¹ |
|------------|-----------------------------------|
| Very Loose | 0 - 4 |
| Loose | 4 to 10 |
| Compact | 10 to 30 |
| Dense | 30 to 50 |
| Very Dense | >50 |

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
- Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N_{60} values.

Field Moisture Condition

| Term | Description |
|-------|---|
| Dry | Soil flows freely through fingers. |
| Moist | Soils are darker than in the dry condition and may feel cool. |
| Wet | As moist, but with free water forming on hands when handled. |

COHESIVE SOILS

Consistency

| Term | Undrained Shear Strength (kPa) | SPT 'N' ¹ (blows/0.3m) |
|------------|--------------------------------|-----------------------------------|
| Very Soft | <12 | 0 to 2 |
| Soft | 12 to 25 | 2 to 4 |
| Firm | 25 to 50 | 4 to 8 |
| Stiff | 50 to 100 | 8 to 15 |
| Very Stiff | 100 to 200 | 15 to 30 |
| Hard | >200 | >30 |

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

Water Content

| Term | Description |
|-------------|--|
| $w < PL$ | Material is estimated to be drier than the Plastic Limit. |
| $w \sim PL$ | Material is estimated to be close to the Plastic Limit. |
| $w > PL$ | Material is estimated to be wetter than the Plastic Limit. |



LIST OF SYMBOLS

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

I. GENERAL

| | |
|-------------|---------------------------------------|
| π | 3.1416 |
| $\ln x$ | natural logarithm of x |
| \log_{10} | x or log x, logarithm of x to base 10 |
| g | acceleration due to gravity |
| t | time |

II. STRESS AND STRAIN

| | |
|--------------------------------|--|
| γ | shear strain |
| Δ | change in, e.g. in stress: $\Delta \sigma$ |
| ε | linear strain |
| ε_v | volumetric strain |
| η | coefficient of viscosity |
| ν | Poisson's ratio |
| σ | total stress |
| σ' | effective stress ($\sigma' = \sigma - u$) |
| σ'_{vo} | initial effective overburden stress |
| $\sigma_1, \sigma_2, \sigma_3$ | principal stress (major, intermediate, minor) |
| σ_{oct} | mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$ |
| τ | shear stress |
| u | porewater pressure |
| E | modulus of deformation |
| G | shear modulus of deformation |
| K | bulk modulus of compressibility |

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

| | |
|-------------|---|
| C_c | compression index (normally consolidated range) |
| C_r | recompression index (over-consolidated range) |
| C_s | swelling index |
| C_α | secondary compression index |
| m_v | coefficient of volume change |
| C_v | coefficient of consolidation (vertical direction) |
| C_h | coefficient of consolidation (horizontal direction) |
| T_v | time factor (vertical direction) |
| U | degree of consolidation |
| σ'_p | pre-consolidation stress |
| OCR | over-consolidation ratio = σ'_p / σ'_{vo} |

(d) Shear Strength

| | |
|------------------|--|
| τ_p, τ_r | peak and residual shear strength |
| ϕ' | effective angle of internal friction |
| δ | angle of interface friction |
| μ | coefficient of friction = $\tan \delta$ |
| c' | effective cohesion |
| C_u, S_u | undrained shear strength ($\phi = 0$ analysis) |
| p | mean total stress $(\sigma_1 + \sigma_3)/2$ |
| p' | mean effective stress $(\sigma'_1 + \sigma'_3)/2$ |
| q | $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$ |
| q_u | compressive strength $(\sigma_1 - \sigma_3)$ |
| S_t | sensitivity |

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

| <u>Description</u> | <u>Spacing</u> |
|--------------------|------------------|
| Very wide | Greater than 3 m |
| Wide | 1 m to 3 m |
| Moderately close | 0.3 m to 1 m |
| Close | 50 mm to 300 mm |
| Very close | Less than 50 mm |

GRAIN SIZE

| <u>Term</u> | <u>Size*</u> |
|---------------------|-------------------------|
| Very Coarse Grained | Greater than 60 mm |
| Coarse Grained | 2 mm to 60 mm |
| Medium Grained | 60 microns to 2 mm |
| Fine Grained | 2 microns to 60 microns |
| Very Fine Grained | Less than 2 microns |

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

| | |
|---------------------|-------------------|
| JN Joint | PL Planar |
| FLT Fault | CU Curved |
| SH Shear | UN Undulating |
| VN Vein | IR Irregular |
| FR Fracture | K Slickensided |
| SY Stylolite | PO Polished |
| BD Bedding | SM Smooth |
| FO Foliation | SR Slightly Rough |
| CO Contact | RO Rough |
| AXJ Axial Joint | VR Very Rough |
| KV Karstic Void | |
| MB Mechanical Break | |

PROJECT: 1419005

RECORD OF BOREHOLE: 15-1

SHEET 1 OF 2

LOCATION: See Site Plan

BORING DATE: March 4-5, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | | | SAMPLES | | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | | | | HYDRAULIC CONDUCTIVITY, k, cm/s | | | | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION | |
|-----------------------|---|--|-------------|-----------------------|---------|------|---|----------------|--|------------|------------------------------------|---|--|----------------------------|----------------------------|---|--|
| | | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.30m | | | | | | | | | | |
| | | | | | | | | SHEAR STRENGTH | | | | WATER CONTENT PERCENT | | | | | |
| | | | | | | | | 20 40 | | 60 80 | | 10 ⁻⁸ 10 ⁻⁶ 10 ⁻⁴ 10 ⁻² | | 20 40 60 80 | | | |
| | | | | | | | | | | | | | | | | | |
| 0 | | GROUND SURFACE | | 128.85 | | | | | | | | | | | | | |
| | Power Auger 200 mm Diam. (Hollow Stem) | TOPSOIL - (SM) SILTY SAND; dark brown to black, with organic matter and rootlets; non-cohesive, moist/frozen | | 0.00 | | | | | | | | | | | | | |
| | | | | 128.55 | | | | | | | | | | | | | |
| | | (SM) SILTY SAND; brown to dark brown; non-cohesive, moist/frozen | | 0.30 | | | | | | | | | | | | | |
| | | | | 0.46 | | | | | | | | | | | | | |
| | | (CI/CH) SILTY CLAY to CLAY, trace sand; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff | | | | | | | | | | | | | | | |
| 1 | | | | | 1 | SS | 8 | | | | | | | | | | |
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| 2 | | | | | 2 | SS | 22 | | | | | | | | | | |
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| | | | | | 3 | SS | 13 | | | | | | | | | | |
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DEPTH SCALE

1 : 50



LOGGED: RI

CHECKED: SAT

MIS-BHS 001 1419005.GPJ GAL-MIS.GDT 05/14/15 JM

PROJECT: 1419005

RECORD OF DRILLHOLE: 15-1

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: March 4-5, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | FLUSH | RECOVERY TOTAL CORE % | SOLID CORE % | R.Q.D. % | FRACT. INDEX PER 0.25 m | B Angle | DIP w.r.t. CORE AXIS | DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION | Joon | Jr | Ja | HYDRAULIC CONDUCTIVITY K, cm/sec | Diametral Point Load Index (MPa) | RMC -Q AVG. | |
|-----------------------|-----------------|---|--------------|-----------------------|---------|-------|-----------------------------|-----------------|-------------|----------------------------------|---------|----------------------------|---|------|----|----|--|---|-------------------|---|
| | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | BEDROCK SURFACE | | 123.01 | | | | | | | | | | | | | | | | |
| 6 | | Fresh, thinly to thickly bedded, dark grey, fine to coarse grained, non-porous SHALEY DOLOSTONE, with thin to thick interbeds of dark grey, slightly calcareous sandstone | | 5.84 | 1 | 100 | | | | | | | .BD., .BD., .BD., | | | | | | | Bentonite Seal |
| | | - Broken core from 6.24 m to 6.36 m | | | | | | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | .BD., | | | | | | | Silica Sand |
| | | | | | | | | | | | | | .BD., | | | | | | | |
| 8 | | | | | 2 | 100 | | | | | | | .BD., | | | | | | | 32 mm Diam. PVC #10 Slot Screen |
| | | | | | | | | | | | | | .JN., .BD., .BD., .BD., .BD., | | | | | | | Silica Sand |
| 9 | | | | | 3 | 100 | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | .JN., | | | | | | | Bentonite Seal |
| 11 | | | | | 4 | 100 | | | | | | | | | | | | | | |
| 12 | | End of Drillhole | | 116.82 | | | | | | | | | | | | | | | | |
| | | | | 12.03 | | | | | | | | | | | | | | | | WL in Screen at Elev. 128.20 m on Mar. 24, 2015 |
| 13 | | | | | | | | | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: RI

CHECKED: SAT

MIS-RCK 004 1419005.GPJ GAL-MISS.GDT 05/14/15 JM

PROJECT: 1419005

RECORD OF BOREHOLE: 15-2

SHEET 1 OF 2





LOCATION: See Site Plan

BORING DATE: March 6, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | | | SAMPLES | | | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | | | | HYDRAULIC CONDUCTIVITY, k, cm/s | | | | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION |
|-----------------------|---|--|---|--|----------------|------|-------------|---|----|--------------------------------|----|------------------------------------|------------------|------------------|------------------|----------------------------|---|
| | | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.30m | | | | | | | | | | |
| | | | | | | | | SHEAR STRENGTH | | | | WATER CONTENT PERCENT | | | | | |
| | | | | | | | | Cu, kPa | | nat V. + Q - rem V. ⊕ U - ● | | Wp | | W | | | |
| | | | | | | | | 20 | 40 | 60 | 80 | 10 ⁻⁸ | 10 ⁻⁶ | 10 ⁻⁴ | 10 ⁻² | | |
| 0 | | GROUND SURFACE | | 128.67 | | | | | | | | | | | | | |
| | Power Auger 200 mm Diam. (Hollow Stem) | TOPSOIL - (CL/CI) SILTY CLAY; dark brown to black, with organic matter and rootlets; cohesive, w~PL/frozen |  | 0.00 | | | | | | | | | | | | | |
| | | (CI/CH) SILTY CLAY to CLAY, trace sand; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff |  | 128.34 0.33 | | | | | | | | | | | | | |
| 1 | | | | | 1 | SS | 14 | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| 2 | | | | | 2 | SS | 29 | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| 3 | | | | | 3 | SS | 18 | | | | | | | | | | |
| | | | (CI/CH) SILTY CLAY to CLAY; grey, with shells; cohesive, w>PL; very stiff |  | 125.62 3.05 | | | | | | | | | | | | |
| 4 | | | | | 4 | SS | 13 | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| 5 | | | | 5 | SS | 11 | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| 6 | | | | 6 | SS | 8 | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| 7 | | | | 7 | SS | >50 | | | | | | | | | | | |
| | | (SM) SILTY SAND, some gravel; grey (GLACIAL TILL); non-cohesive, wet, dense |  | 123.18 5.49 | | | | | | | | | | | | | |
| 8 | | Borehole continued on RECORD OF DRILLHOLE 15-2 | | 5.65 | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: RI

CHECKED: SAT

MIS-BHS 001 1419005.GPJ GAL-MIS.GDT 05/14/15 JM

PROJECT: 1419005

RECORD OF DRILLHOLE: 15-2

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: March 6, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. | RUN No. | COLOUR % RETURN | JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols. | | | | | | | | | | HYDRAULIC CONDUCTIVITY | | | | Diametral Point Load Index (MPa) | RMC -Q AVG. | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
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| | | | | | | | TOTAL CORE % | SOLID CORE % | | | B Angle | DIP w.r.t. CORE AXIS | TYPE AND SURFACE DESCRIPTION | Joon | Jr | Ja | 10 10 10 10 10 | 10 10 10 10 10 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
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| | | BEDROCK SURFACE | | 123.02 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: RI

CHECKED: SAT

MIS-RCK 004 1419005.GPJ GAL-MISS.GDT 05/14/15 JM

PROJECT: 1419005

RECORD OF BOREHOLE: 15-3

SHEET 1 OF 2




LOCATION: See Site Plan

BORING DATE: March 5-6, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | | | SAMPLES | | | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | | | | HYDRAULIC CONDUCTIVITY, k, cm/s | | | | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION | |
|-----------------------|---|---|---|-----------------------|---------|------|-------------|---|----|----|----|------------------------------------|------------------|------------------|------------------|----------------------------|---|--|
| | | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.30m | | | | | | | | | | | |
| | | | | | | | | SHEAR STRENGTH Cu, kPa | | | | WATER CONTENT PERCENT | | | | | | |
| | | | | | | | | | | | | | | | | | | |
| | | | | | | | | 20 | 40 | 60 | 80 | 10 ⁻⁸ | 10 ⁻⁶ | 10 ⁻⁴ | 10 ⁻² | | | |
| | | | | | | | | 20 | 40 | 60 | 80 | Wp | W | | Wi | | | |
| | | | | | | | | 20 | 40 | 60 | 80 | 20 | 40 | 60 | 80 | | | |
| 0 | | GROUND SURFACE | | 129.33 | | | | | | | | | | | | | | |
| | Power Auger 200 mm Diam. (Hollow Stem) | TOPSOIL/FILL - (SM) SILTY SAND; dark brown, with organic matter and rootlets; non-cohesive, frozen |  | 0.00 | | | | | | | | | | | | | | |
| 0.10 | | | | | | | | | | | | | | | | | | |
| | | | | A | GRAB | - | | | | | | | | | | | | |
| 128.88 | | | | | | | | | | | | | | | | | | |
| 0.45 | | | | | | | | | | | | | | | | | | |
| | | FILL - (SM) SILTY SAND, trace gravel; brown to dark brown; non-cohesive, frozen |  | 128.64 | | | | | | | | | | | | | | |
| 0.69 | | | | | | | | | | | | | | | | | | |
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| | | FILL - (GW) sandy GRAVEL; brown, with cobbles; non-cohesive, moist (CI/CH) SILTY CLAY to CLAY, trace sand; grey brown, with rootlets (WEATHERED CRUST); cohesive, w>PL, very stiff |  | | | | | | | | | | | | | | | |
| | | | | 1 | SS | 14 | | | | | | | | | | | | |
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| | 2 | | | SS | 16 | | | | | | | | | | | | | |
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DEPTH SCALE

1 : 50



LOGGED: RI

CHECKED: SAT

MIS-BHS 001 1419005.GPJ GAL-MIS.GDT 05/14/15 JM

PROJECT: 1419005

RECORD OF DRILLHOLE: 15-3

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: March 5-6, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

| DEPTH SCALE METRES | DRILLING RECORD | | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN | FLUSH | RECOVERY | | R.Q.D. % | FRACT INDEX PER 0.25 m | DISCONTINUITY DATA | | | | | HYDRAULIC CONDUCTIVITY | | | | Diametral Point Load Index (MPa) | RMC -Q AVG. | | | | | |
|-----------------------|-------------------------|---------------------------|---|--------------|-----------------------|---------|--------------------|-------|-----------------|-----------------|-------------|---------------------------------|---------------------------------|------|----|----|-----------|---------------------------|----------------------|------|----|---|-------------------|--|----|-----------|----------------------|----------------------|
| | | | | | | | | | TOTAL CORE % | SOLID CORE % | | | TYPE AND SURFACE DESCRIPTION | Joon | Jr | Ja | K, cm/sec | 10 10 10 10 | 10 10 10 10 | | | | | | | | | |
| | B Angle | DIP w.r.t CORE AXIS | | | | | | | | | | | | | | | | | | Joon | Jr | | | | Ja | K, cm/sec | 10 10 10 10 | 10 10 10 10 |
| | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | BEDROCK SURFACE | | 123.26 | | | | | | | | | | | | | | | | | | | | | | | |
| | Rotary Drill NQ Core | | Fresh, thinly to medium bedded, dark grey to grey, fine grained, non-porous DOLOSTONE, with thin interlaminations to thin interbeds of dolomitic sandstone, occasional thin interlaminations of dark grey shale | | 6.07 | 1 | | 100 | | | | | | | | | | | | | | | | | | | | |
| 7 | | | - Thin (~2-3 mm) near-vertical calcite vein from 7.10 m to 7.25 m | | 121.95 | | | | | | | | | | | | | | | | | | | | | | | |
| | | | End of Drillhole | | 7.38 | | | | | | | | | | | | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
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Bentonite Seal

Silica Sand

32 mm Diam. PVC #10 Slot Screen

WL in Screen at
Elev. 128.27 m on
Mar. 24, 2015

DEPTH SCALE

1 : 50



LOGGED: RI

CHECKED: SAT

MIS-RCK 004 1419005.GPJ GAL-MISS.GDT 05/14/15 JM

PROJECT: 1419005

RECORD OF BOREHOLE: 15-4

SHEET 1 OF 2

LOCATION: See Site Plan

BORING DATE: March 3-4, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | | | SAMPLES | | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | | | | HYDRAULIC CONDUCTIVITY, k, cm/s | | | | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION |
|-----------------------|---|---|-------------|-----------------------|---------|------|---|---------------------------|----|--------------------------------|------------------------------------|--|------------------|------------------|----------------------------|---|
| | | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.30m | SHEAR STRENGTH Cu, kPa | | nat V. + Q - rem V. ⊕ U - ● | | WATER CONTENT PERCENT Wp — W — Wl | | | | |
| | | | | | | | | 20 | 40 | 60 | 80 | 10 ⁻⁸ | 10 ⁻⁶ | 10 ⁻⁴ | | |
| 0 | Power Auger 200 mm Diam. (Hollow Stem) | GROUND SURFACE | | 129.14 | | | | | | | | | | | | |
| | | TOPSOIL - (SM) SILTY SAND; dark brown to black, with organic matter and rootlets; non-cohesive, moist/frozen | | 0.00 | | | | | | | | | | | | |
| | | | | 128.78 | | | | | | | | | | | | |
| | | (CI/CH) SILTY CLAY to CLAY, trace sand; grey brown, with rootlets (WEATHERED CRUST); cohesive, w>PL, very stiff | | 0.36 | | | | | | | | | | | | |
| 1 | | | 1 | SS | 14 | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| 2 | | 2 | SS | 11 | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| 3 | | 3 | SS | 18 | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| 4 | | | | | | | | | | | | | | | | |
| | (CI/CH) SILTY CLAY to CLAY; grey, with shells; cohesive, w>PL, very stiff | | 126.09 | | | | | | | | | | | | | |
| | | 4 | SS | 9 | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| 5 | | 5 | SS | 9 | | | | | | | | | | | | |
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DEPTH SCALE

1 : 50



LOGGED: RI

CHECKED: SAT

MIS-BHS 001 1419005.GPJ GAL-MIS.GDT 05/14/15 JM

PROJECT: 1419005

RECORD OF BOREHOLE: 15-6

SHEET 1 OF 1


LOCATION: See Site Plan

BORING DATE: March 3, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | | | SAMPLES | | | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | | | | HYDRAULIC CONDUCTIVITY, k, cm/s | | | | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION | | |
|-----------------------|---|---|---|-----------------------|---------|------|-------------|---|----|----------------------------------|----|------------------------------------|------------------|------------------|------------------|----------------------------|---|----|--|
| | | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.30m | | | | | | | | | | | | |
| | | | | | | | | SHEAR STRENGTH | | | | WATER CONTENT PERCENT | | | | | | | |
| | | | | | | | | Cu, kPa | | nat V. + Q - ● rem V. ⊕ U - ○ | | Wp | | W | | | | Wi | |
| | | | | | | | | 20 | 40 | 60 | 80 | 10 ⁻⁸ | 10 ⁻⁶ | 10 ⁻⁴ | 10 ⁻² | | | | |
| | | | | | | | | | | | | | | | | | | | |
| 0 | | GROUND SURFACE | | 129.68 | | | | | | | | | | | | | | | |
| | Power Auger 200 mm Diam. (Hollow Stem) | FILL - (GW) sandy GRAVEL, angular; grey; non-cohesive, moist/frozen FILL - (CL/CI) SILTY CLAY, some sand; dark brown, with organic matter; cohesive, w~PL, frozen (CI/CH) SILTY CLAY to CLAY, trace sand; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff |  | 0.00 | | | | | | | | | | | | | | | |
| | | | | 0.13 | | | | | | | | | | | | | | | |
| | | | | 129.15 | 1 | SS | 48 | | | | | | | | | | | | |
| | | | | 0.53 | | | | | | | | | | | | | | | |
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DEPTH SCALE

1 : 50



LOGGED: RI

CHECKED: SAT

MIS-BHS 001 1419005.GPJ GAL-MIS.GDT 05/14/15 JM

PROJECT: 1419005












RECORD OF PROBEHOLE: 15-101

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: April 14, 2015

DATUM: Geodetic

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | | SAMPLES | | | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | | | | HYDRAULIC CONDUCTIVITY, k, cm/s | | | | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION | |
|-----------------------|---|--|---|-----------------------|--------|------|---|---------------------------|----|----|------------------------------------|-----------------------|--------|------------|----------------------------|---|--------|
| | | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.30m | | | | | | | | | | |
| | | | | | | | | SHEAR STRENGTH Cu, kPa | | | | WATER CONTENT PERCENT | | | | | |
| | | | | | | | | 20 | 40 | 60 | 80 | nat V. rem V. | + ⊕ | Q - U - | | | ● ○ |
| | | | | | | | | | | | | | | | | | |
| 0 | | GROUND SURFACE | | 130.26 | | | | | | | | | | | | | |
| | Power Auger 200 mm Diam. (Hollow Stem) | FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE) |  | 0.00 | | | | | | | | | | | | | |
| | | FILL - (GW) sandy GRAVEL, angular; grey (PAVEMENT STRUCTURE) |  | 0.40 | | | | | | | | | | | | | |
| 1 | | | | | | | | | | | | | | | | | |
| | | Possible Silty Clay to Clay; grey brown to grey |  | 1.22 | | | | | | | | | | | | | |
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| 6 | | |  | | | | | | | | | | | | | | |
| 7 | | |  | | | | | | | | | | | | | | |
| | | Possible Glacial Till |  | 123.10 7.16 | | | | | | | | | | | | | |
| | | |  | 122.84 7.42 | | | | | | | | | | | | | |
| | | End of Borehole Auger Refusal | | | | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | | | | | | |
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DEPTH SCALE

1 : 50



LOGGED: HEC

CHECKED: SAT

MIS-BHS 001 1419005.GPJ GAL-MIS.GDT 05/14/15 JM

PROJECT: 1419005






RECORD OF PROBEHOLE: 15-102

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: April 14, 2015

DATUM: Geodetic

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | | SAMPLES | | | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | | | | HYDRAULIC CONDUCTIVITY, k, cm/s | | | | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION | |
|-----------------------|---|--|---|-----------------------|--------|------|---|---------------------------|----|----|------------------------------------|-----------------------|--------|------------|----------------------------|---|--------|
| | | DESCRIPTION | STRATA PLOT | ELEV. DEPTH (m) | NUMBER | TYPE | BLOWS/0.30m | | | | | | | | | | |
| | | | | | | | | SHEAR STRENGTH Cu, kPa | | | | WATER CONTENT PERCENT | | | | | |
| | | | | | | | | 20 | 40 | 60 | 80 | nat V. rem V. | + ⊕ | Q - U - | | | ● ○ |
| | | | | | | | | | | | | | | | | | |
| 0 | | GROUND SURFACE | | 130.23 | | | | | | | | | | | | | |
| | Power Auger 200 mm Diam. (Hollow Stem) | FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE) |  | 0.00 | | | | | | | | | | | | | |
| | | FILL - (GW) sandy GRAVEL, angular; grey (PAVEMENT STRUCTURE) |  | 0.40 | | | | | | | | | | | | | |
| 1 | | | | | | | | | | | | | | | | | |
| | | Possible Clayey Silt, some sand; brown; cohesive |  | 1.22 | | | | | | | | | | | | | |
| 2 | | | | | | | | | | | | | | | | | |
| | | Possible Silty Clay to Clay; grey brown to grey |  | 2.00 | | | | | | | | | | | | | |
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| 6 | | | | | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | | | | | |
| 8 | | Possible Glacial Till |  | 122.36 7.87 | | | | | | | | | | | | | |
| | | End of Borehole Auger Refusal | | 8.03 | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: HEC

CHECKED: SAT

MIS-BHS 001 1419005.GPJ GAL-MIS.GDT 05/14/15 JM



APPENDIX B

Results of Hydraulic Conductivity

HVORSLEV SLUG TEST ANALYSIS **RIISING HEAD TEST 15-1**

INTERVAL (metres below ground surface)

Top of Interval = 7.16
Bottom of Interval = 8.69

$$K = \frac{r_c^2}{2L_e} \ln \left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e} \right)^2} \right] \left[\frac{\ln \left(\frac{h_1}{h_2} \right)}{(t_2 - t_1)} \right] \quad \text{where } K = (\text{m/sec})$$

where:

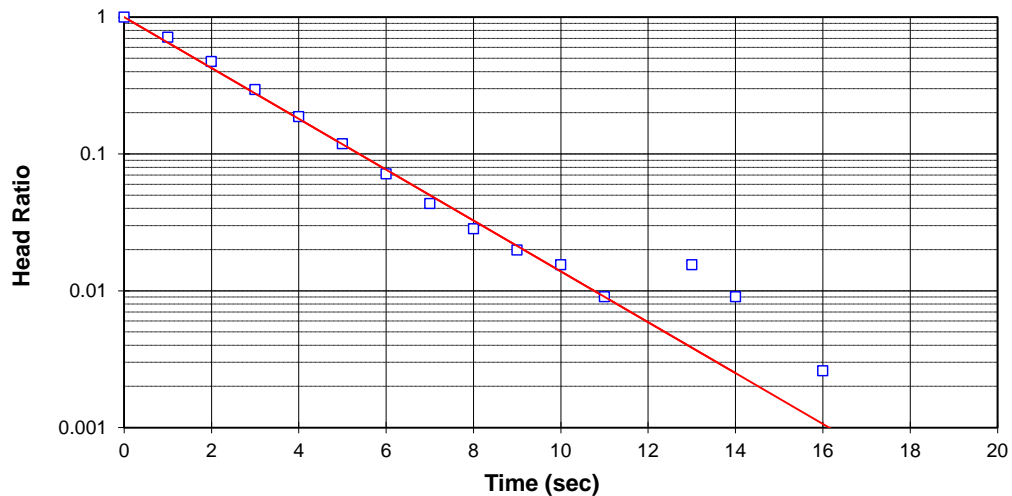
- r_c = casing radius (metres)
- R_e = filter pack radius (metres)
- L_e = length of screened interval (metres)
- t = time (seconds)
- h_t = head at time t (metres)

INPUT PARAMETERS

$r_c = 0.02$
 $R_e = 0.04$
 $L_e = 1.5$
 $t_1 = 0$
 $t_2 = 14$
 $h_1/h_0 = 1.00$
 $h_2/h_0 = 0.00$

RESULTS

$K = 1\text{E-}04 \text{ m/sec}$
 $K = 1\text{E-}02 \text{ cm/sec}$



Project Name: **Carleton Place Pumping Station CP**
Project No.: **1419005**
Test Date: **24-03-15**

Analysis By: **DH**
Checked By: **BTB**
Analysis Date: **26-03-15**

Golder Associates Ltd.

HVORSLEV SLUG TEST ANALYSIS **RIISING HEAD TEST 15-3**

INTERVAL (metres below ground surface)

Top of Interval = 6.62
Bottom of Interval = 7.38

$$K = \frac{r_c^2}{2L_e} \ln \left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e} \right)^2} \right] \left[\frac{\ln \left(\frac{h_1}{h_2} \right)}{(t_2 - t_1)} \right] \text{ where } K = (\text{m/sec})$$

where:

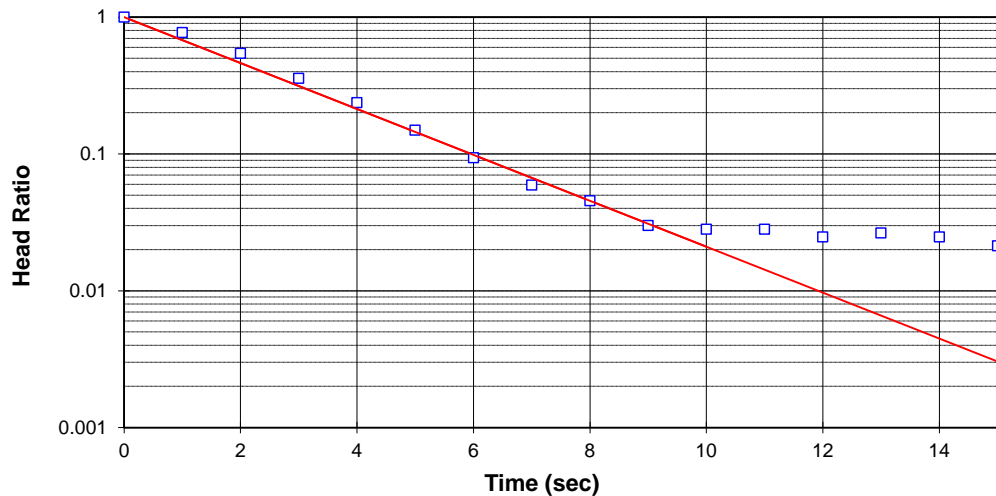
- r_c = casing radius (metres)
- R_e = filter pack radius (metres)
- L_e = length of screened interval (metres)
- t = time (seconds)
- h_t = head at time t (metres)

INPUT PARAMETERS

$r_c = 0.02$
 $R_e = 0.04$
 $L_e = 0.8$
 $t_1 = 0$
 $t_2 = 10$
 $h_1/h_0 = 1.00$
 $h_2/h_0 = 0.02$

RESULTS

$K = 2\text{E-}04 \text{ m/sec}$
 $K = 2\text{E-}02 \text{ cm/sec}$



Project Name: Carleton Place Pumping Station CP
Project No.: 1419005
Test Date: 24-03-15

Analysis By: DH
Checked By: BTB
Analysis Date: 26-03-15

Golder Associates Ltd.

HVORSLEV SLUG TEST ANALYSIS FALLING HEAD TEST 15-4

INTERVAL (metres below ground surface)

**Top of Interval = 6.63
Bottom of Interval = 7.39**

$$K = \frac{r_c^2}{2L_e} \ln \left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e} \right)^2} \right] \left[\frac{\ln \left(\frac{h_1}{h_2} \right)}{(t_2 - t_1)} \right] \quad \text{where } K = (\text{m/sec})$$

where:

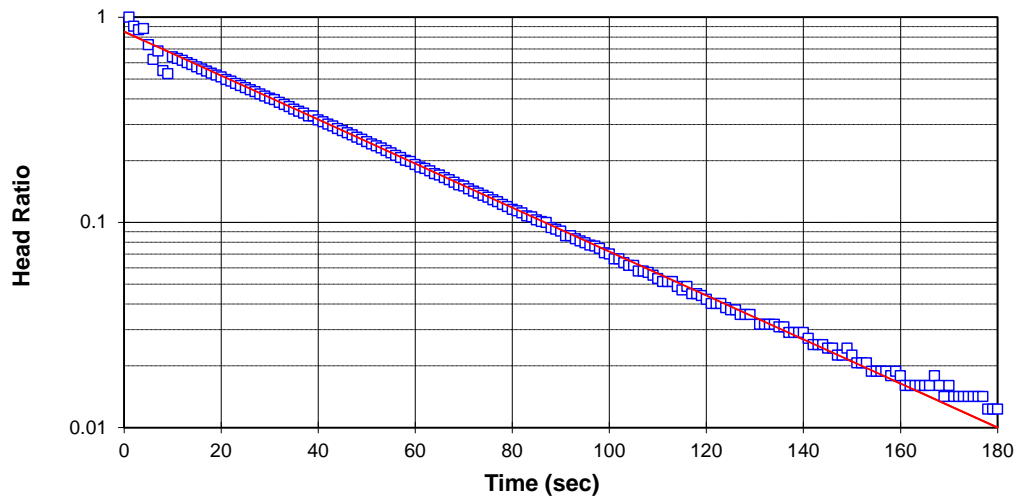
- r_c = casing radius (metres)
- R_e = filter pack radius (metres)
- L_e = length of screened interval (metres)
- t = time (seconds)
- h_t = head at time t (metres)

INPUT PARAMETERS

$r_c = 1.6\text{E-}02$
 $R_e = 3.8\text{E-}02$
 $L_e = 0.8$
 $t_1 = 0$
 $t_2 = 180$
 $h_1/h_0 = 0.85$
 $h_2/h_0 = 0.01$

RESULTS

K= 1E-05 m/sec
K= 1E-03 cm/sec



Project Name: **Carleton Place Pumping Station CP**
 Project No.: **1419005**
 Test Date: **24-03-15**

Analysis By: **DH**
 Checked By: **BTB**
 Analysis Date: **27-03-15**

Golder Associates Ltd.



APPENDIX C

**Results of Soil Quality Testing
Maxxam Analytics Report No. R3358428**

| Parameter | Units | RDL | Ontario (2011) Table 1 Standards | Ontario (2011) Table 2 Standards | BH 15-1 SA#5 04-March-2015 3.81 - 4.42 m | BH 15-6 SA#1 03-March-2015 0.13 - 0.74 m |
|--|-------|------|--|--|--|--|
| VOCs | | | | | | |
| Acetone | µg/g | 0.5 | 0.5 | 16 | <0.50 | <0.50 |
| Benzene | µg/g | 0.02 | 0.02 | 0.32 | <0.020 | <0.020 |
| Bromodichloromethane | µg/g | 0.05 | 0.05 | 1.5 | <0.050 | <0.050 |
| Bromoform | µg/g | 0.05 | 0.05 | 0.61 | <0.050 | <0.050 |
| Bromomethane | µg/g | 0.05 | 0.05 | 0.05 | <0.050 | <0.050 |
| Carbon Tetrachloride | µg/g | 0.05 | 0.05 | 0.21 | <0.050 | <0.050 |
| Chlorobenzene | µg/g | 0.05 | 0.05 | 2.4 | <0.050 | <0.050 |
| Chloroform | µg/g | 0.05 | 0.05 | 0.47 | <0.050 | <0.050 |
| Dibromochloromethane | µg/g | 0.05 | 0.05 | 2.3 | <0.050 | <0.050 |
| 1,1-Dichloroethane | µg/g | 0.05 | 0.05 | 0.47 | <0.050 | <0.050 |
| 1,2-Dichloroethane | µg/g | 0.05 | 0.05 | 0.05 | <0.050 | <0.050 |
| 1,1-Dichloroethylene | µg/g | 0.05 | 0.05 | 0.064 | <0.050 | <0.050 |
| Cis-1,2-Dichloroethylene | µg/g | 0.05 | 0.05 | 1.9 | <0.050 | <0.050 |
| Trans-1,2-Dichloroethylene | µg/g | 0.05 | 0.05 | 1.3 | <0.050 | <0.050 |
| 1,2-Dichloropropane | µg/g | 0.05 | 0.05 | 0.16 | <0.050 | <0.050 |
| Cis-1,3-Dichloropropylene | µg/g | 0.03 | NV | NV | <0.030 | <0.030 |
| Trans-1,3-Dichloropropylene | µg/g | 0.04 | NV | NV | <0.040 | <0.040 |
| Ethylbenzene | µg/g | 0.02 | 0.05 | 1.1 | <0.020 | <0.020 |
| Ethylene Dibromide | µg/g | 0.05 | 0.05 | 0.05 | <0.050 | <0.050 |
| Methyl Ethyl Ketone | µg/g | 0.5 | 0.5 | 70 | <0.50 | <0.50 |
| Methylene Chloride | µg/g | 0.05 | 0.05 | 1.6 | <0.050 | <0.050 |
| Methyl Isobutyl Ketone | µg/g | 0.5 | 0.5 | 31 | <0.50 | <0.50 |
| Methyl-t-Butyl Ether | µg/g | 0.05 | 0.05 | 1.6 | <0.050 | <0.050 |
| Styrene | µg/g | 0.05 | 0.05 | 34 | <0.050 | <0.050 |
| 1,1,1,2-Tetrachloroethane | µg/g | 0.05 | 0.05 | 0.087 | <0.050 | <0.050 |
| 1,1,2,2-Tetrachloroethane | µg/g | 0.05 | 0.05 | 0.05 | <0.050 | <0.050 |
| Toluene | µg/g | 0.02 | 0.2 | 6.4 | <0.020 | <0.020 |
| Tetrachloroethylene | µg/g | 0.05 | 0.05 | 1.9 | <0.050 | <0.050 |
| 1,1,1-Trichloroethane | µg/g | 0.05 | 0.05 | 6.1 | <0.050 | <0.050 |
| 1,1,2-Trichloroethane | µg/g | 0.05 | 0.05 | 0.05 | <0.050 | <0.050 |
| Trichloroethylene | µg/g | 0.05 | 0.05 | 0.55 | <0.050 | <0.050 |
| Vinyl Chloride | µg/g | 0.02 | 0.02 | 0.032 | <0.020 | <0.020 |
| m-Xylene & p-Xylene | µg/g | 0.02 | NV | NV | <0.020 | <0.020 |
| o-Xylene | µg/g | 0.02 | NV | NV | <0.020 | <0.020 |
| Total Xylenes ⁽²⁾ | µg/g | 0.02 | 0.05 | 26 | <0.020 | <0.020 |
| Dichlorodifluoromethane | µg/g | 0.05 | 0.05 | 16 | <0.050 | <0.050 |
| Hexane(n) | µg/g | 0.05 | 0.05 | 46 | <0.050 | <0.050 |
| Trichlorofluoromethane | µg/g | 0.05 | 0.25 | 4 | <0.050 | <0.050 |
| 1,3-Dichloropropene (cis + trans) ⁽¹⁾ | µg/g | 0.05 | 0.05 | 0.059 | <0.050 | <0.050 |

| Parameter | Units | RDL | Ontario (2011) Table 1 Standards | Ontario (2011) Table 2 Standards | BH 15-1 SA#5 04-March-2015 3.81 - 4.42 m | BH 15-6 SA#1 03-March-2015 0.13 - 0.74 m |
|---------------------------|-------|------|--|--|--|--|
| Metals | | | | | | |
| Antimony | µg/g | 0.2 | 1.3 | 40 | <0.20 | <0.20 |
| Arsenic | µg/g | 1 | 18 | 18 | <1.0 | <1.0 |
| Barium | µg/g | 0.5 | 220 | 670 | 270 | 98 |
| Beryllium | µg/g | 0.2 | 2.5 | 8 | 0.88 | 0.47 |
| Boron (Hot Water Soluble) | µg/g | 0.05 | NV | 2 | 0.14 | 0.16 |
| Cadmium | µg/g | 0.1 | 1.2 | 1.9 | 0.12 | 0.13 |
| Chromium | µg/g | 1 | 70 | 160 | 52 | 22 |
| Chromium VI | µg/g | 0.2 | 0.66 | 8 | <0.2 | <0.2 |
| Cobalt | µg/g | 0.1 | 21 | 80 | 16 | 6.6 |
| Copper | µg/g | 0.5 | 92 | 230 | 32 | 8.2 |
| Lead | µg/g | 1 | 120 | 120 | 7.5 | 5.6 |
| Mercury | µg/g | 0.05 | 0.27 | 3.9 | <0.050 | <0.050 |
| Molybdenum | µg/g | 0.5 | 2 | 40 | 0.97 | <0.50 |
| Nickel | µg/g | 0.5 | 82 | 270 | 31 | 12 |
| Selenium | µg/g | 0.5 | 1.5 | 5.5 | <0.50 | <0.50 |
| Silver | µg/g | 0.2 | 0.5 | 40 | <0.20 | <0.20 |
| Thallium | µg/g | 0.05 | 1 | 3.3 | 0.33 | 0.12 |
| Vanadium | µg/g | 5 | 86 | 86 | 78 | 41 |
| Zinc | µg/g | 5 | 290 | 340 | 100 | 53 |
| Boron (Total) | µg/g | 5 | 36 | 120 | 10 | <5.0 |
| Uranium | µg/g | 0.05 | 2.5 | 33 | 1.1 | 0.55 |
| PHCs | | | | | | |
| F1 (C6-C10) | µg/g | 10 | 25 | 55 | <10 | <10 |
| F1 (C6-C10) - BTEX | µg/g | 10 | 25 | 55 | <10 | <10 |
| F2 (C10-C16) | µg/g | 10 | 10 | 230 | <10 | <10 |
| F3 (C16-C34) | µg/g | 50 | 240 | 1700 | <50 | <50 |
| F4 (C34-C50) | µg/g | 50 | 120 | 3300 | <50 | <50 |
| Reached Baseline at C50 | -- | -- | NV | NV | YES | YES |

Notes:

RDL - Reportable detection limit

NV - No value

PAH - Polycyclic aromatic hydrocarbon

PHC - Petroleum hydrocarbon

VOC - Volatile organic compound

(1) Value for 1,3-dichloropropene used for isomers cis and trans-1,3-dichloropropylene

(2) Total xylenes used to screen m/p-xylene and o-xylene

Screening:**Bold and shaded** Indicates exceedance of the Ontario (2011) Table 1 StandardUnderlined and shaded Indicates exceedance of the Ontario (2011) Table 2 StandardReference:

Ontario Ministry of the Environment (Ontario) April 2011. Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act. Table 1 Full Depth Background Site Condition Standards.
Residential/Parkland/Institutional/Industrial/Commercial/Community Property Use [coarse-grained soil]

Ontario Ministry of the Environment (Ontario) April 2011. Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act. Table 2 Full Depth Generic Site Condition Standards in a Potable Groundwater Condition.
Industrial/Commercial/Community Property Use [coarse-grained soil]

Your Project #: 1419005
Site#: TOWN PUMPING STATION
Site Location: CARLETON PLACE-HWY 7
Your C.O.C. #: na

Attention: Susan Trickey

Golder Associates Ltd
1931 Robertson Rd
Ottawa, ON
K2H 5B7

Report Date: 2015/03/18

Report #: R3358428

Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B542638

Received: 2015/03/10, 16:22

Sample Matrix: Soil
Samples Received: 2

| Analyses | Quantity | Date Extracted | Date Analyzed | Laboratory Method | Reference |
|---|----------|-------------------|------------------|-------------------|----------------------|
| Hot Water Extractable Boron (1) | 2 | 2015/03/16 | 2015/03/18 | CAM SOP-00408 | R153 Ana. Prot. 2011 |
| 1,3-Dichloropropene Sum (1) | 2 | N/A | 2015/03/17 | CAM SOP-00226 | EPA 8260 |
| Hexavalent Chromium in Soil by IC (1, 2) | 2 | 2015/03/13 | 2015/03/18 | CAM SOP-00436 | EPA 3060/7199 m |
| Petroleum Hydro. CCME F1 & BTEX in Soil (1) | 2 | 2015/03/12 | 2015/03/16 | CAM SOP-00315 | CCME PHC-CWS m |
| Petroleum Hydrocarbons F2-F4 in Soil (1) | 2 | 2015/03/13 | 2015/03/16 | CAM SOP-00316 | CCME CWS m |
| Strong Acid Leachable Metals by ICPMS (1) | 2 | 2015/03/16 | 2015/03/16 | CAM SOP-00447 | EPA 6020A m |
| Moisture (1) | 2 | N/A | 2015/03/16 | CAM SOP-00445 | Carter 2nd ed 51.2 m |
| Volatile Organic Compounds in Soil (1) | 2 | 2015/03/12 | 2015/03/14 | CAM SOP-00228 | EPA 8260 m |

Remarks:

Maxxam Analytics has performed all analytical testing herein in accordance with ISO 17025 and the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act. All methodologies comply with this document and are validated for use in the laboratory. The methods and techniques employed in this analysis conform to the performance criteria (detection limits, accuracy and precision) as outlined in the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act.

The CWS PHC methods employed by Maxxam conform to all prescribed elements of the reference method and performance based elements have been validated. All modifications have been validated and proven equivalent following the 'Alberta Environment Draft Addenda to the CWS-PHC, Appendix 6, Validation of Alternate Methods'. Documentation is available upon request. Maxxam has made the following improvements to the CWS-PHC reference benchmark method: (i) Headspace for F1; and, (ii) Mechanical extraction for F2-F4. Note: F4G cannot be added to the C6 to C50 hydrocarbons. The extraction date for samples field preserved with methanol for F1 and Volatile Organic Compounds is considered to be the date sampled.

Maxxam Analytics is accredited for all specific parameters as required by Ontario Regulation 153/04. Maxxam Analytics is limited in liability to the actual cost of analysis unless otherwise agreed in writing. There is no other warranty expressed or implied. Samples will be retained at Maxxam Analytics for three weeks from receipt of data or as per contract.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

(1) This test was performed by Maxxam Analytics Mississauga

(2) Soils are reported on a dry weight basis unless otherwise specified.

Attention: Susan Trickey

Golder Associates Ltd
1931 Robertson Rd
Ottawa, ON
K2H 5B7

Your Project #: 1419005
Site#: TOWN PUMPING STATION
Site Location: CARLETON PLACE-HWY 7
Your C.O.C. #: na

Report Date: 2015/03/18

Report #: R3358428

Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B542638

Received: 2015/03/10, 16:22

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Parnian Baber, Project Manager

Email: pbaber@maxxam.ca

Phone# (613) 274-0573

=====

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam Job #: B542638
Report Date: 2015/03/18

Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

O.REG 153 METALS PACKAGE (SOIL)

| Maxxam ID | | ZV1497 | ZV1498 | | |
|----------------------------------|-------|-----------------|-----------------|-------|----------|
| Sampling Date | | 2015/03/04 | 2015/03/03 | | |
| COC Number | | na | na | | |
| | Units | BH 15-1 SA#5 | BH 15-6 SA#1 | RDL | QC Batch |
| Inorganics | | | | | |
| Chromium (VI) | ug/g | <0.2 | <0.2 | 0.2 | 3947504 |
| Metals | | | | | |
| Hot Water Ext. Boron (B) | ug/g | 0.14 | 0.16 | 0.050 | 3949646 |
| Acid Extractable Antimony (Sb) | ug/g | <0.20 | <0.20 | 0.20 | 3949559 |
| Acid Extractable Arsenic (As) | ug/g | <1.0 | <1.0 | 1.0 | 3949559 |
| Acid Extractable Barium (Ba) | ug/g | 270 | 98 | 0.50 | 3949559 |
| Acid Extractable Beryllium (Be) | ug/g | 0.88 | 0.47 | 0.20 | 3949559 |
| Acid Extractable Boron (B) | ug/g | 10 | <5.0 | 5.0 | 3949559 |
| Acid Extractable Cadmium (Cd) | ug/g | 0.12 | 0.13 | 0.10 | 3949559 |
| Acid Extractable Chromium (Cr) | ug/g | 52 | 22 | 1.0 | 3949559 |
| Acid Extractable Cobalt (Co) | ug/g | 16 | 6.6 | 0.10 | 3949559 |
| Acid Extractable Copper (Cu) | ug/g | 32 | 8.2 | 0.50 | 3949559 |
| Acid Extractable Lead (Pb) | ug/g | 7.5 | 5.6 | 1.0 | 3949559 |
| Acid Extractable Molybdenum (Mo) | ug/g | 0.97 | <0.50 | 0.50 | 3949559 |
| Acid Extractable Nickel (Ni) | ug/g | 31 | 12 | 0.50 | 3949559 |
| Acid Extractable Selenium (Se) | ug/g | <0.50 | <0.50 | 0.50 | 3949559 |
| Acid Extractable Silver (Ag) | ug/g | <0.20 | <0.20 | 0.20 | 3949559 |
| Acid Extractable Thallium (Tl) | ug/g | 0.33 | 0.12 | 0.050 | 3949559 |
| Acid Extractable Uranium (U) | ug/g | 1.1 | 0.55 | 0.050 | 3949559 |
| Acid Extractable Vanadium (V) | ug/g | 78 | 41 | 5.0 | 3949559 |
| Acid Extractable Zinc (Zn) | ug/g | 100 | 53 | 5.0 | 3949559 |
| Acid Extractable Mercury (Hg) | ug/g | <0.050 | <0.050 | 0.050 | 3949559 |
| RDL = Reportable Detection Limit | | | | | |
| QC Batch = Quality Control Batch | | | | | |

Maxxam Job #: B542638
Report Date: 2015/03/18

Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

O.REG 153 PETROLEUM HYDROCARBONS (SOIL)

| | | | | | |
|-----------------------------------|-------|-----------------|-----------------|-----|----------|
| Maxxam ID | | ZV1497 | ZV1498 | | |
| Sampling Date | | 2015/03/04 | 2015/03/03 | | |
| COC Number | | na | na | | |
| | Units | BH 15-1 SA#5 | BH 15-6 SA#1 | RDL | QC Batch |
| Inorganics | | | | | |
| Moisture | % | 25 | 16 | 1.0 | 3949445 |
| BTEX & F1 Hydrocarbons | | | | | |
| F1 (C6-C10) | ug/g | <10 | <10 | 10 | 3949679 |
| F1 (C6-C10) - BTEX | ug/g | <10 | <10 | 10 | 3949679 |
| F2-F4 Hydrocarbons | | | | | |
| F2 (C10-C16 Hydrocarbons) | ug/g | <10 | <10 | 10 | 3947768 |
| F3 (C16-C34 Hydrocarbons) | ug/g | <50 | <50 | 50 | 3947768 |
| F4 (C34-C50 Hydrocarbons) | ug/g | <50 | <50 | 50 | 3947768 |
| Reached Baseline at C50 | ug/g | Yes | Yes | | 3947768 |
| Surrogate Recovery (%) | | | | | |
| 1,4-Difluorobenzene | % | 102 | 106 | | 3949679 |
| 4-Bromofluorobenzene | % | 93 | 98 | | 3949679 |
| D10-Ethylbenzene | % | 78 | 69 | | 3949679 |
| D4-1,2-Dichloroethane | % | 91 | 94 | | 3949679 |
| o-Terphenyl | % | 94 | 92 | | 3947768 |
| RDL = Reportable Detection Limit | | | | | |
| QC Batch = Quality Control Batch | | | | | |

Maxxam Job #: B542638
Report Date: 2015/03/18

Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

O.REG 153 VOLATILE ORGANICS (SOIL)

| | | | | | |
|-------------------------------------|--------------|-------------------------|-------------------------|------------|-----------------|
| Maxxam ID | | ZV1497 | ZV1498 | | |
| Sampling Date | | 2015/03/04 | 2015/03/03 | | |
| COC Number | | na | na | | |
| | Units | BH 15-1 SA#5 | BH 15-6 SA#1 | RDL | QC Batch |
| Calculated Parameters | | | | | |
| 1,3-Dichloropropene (cis+trans) | ug/g | <0.050 | <0.050 | 0.050 | 3944229 |
| Volatile Organics | | | | | |
| Acetone (2-Propanone) | ug/g | <0.50 | <0.50 | 0.50 | 3945801 |
| Benzene | ug/g | <0.020 | <0.020 | 0.020 | 3945801 |
| Bromodichloromethane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Bromoform | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Bromomethane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Carbon Tetrachloride | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Chlorobenzene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Chloroform | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Dibromochloromethane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,2-Dichlorobenzene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,3-Dichlorobenzene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,4-Dichlorobenzene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Dichlorodifluoromethane (FREON 12) | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,1-Dichloroethane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,2-Dichloroethane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,1-Dichloroethylene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| cis-1,2-Dichloroethylene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| trans-1,2-Dichloroethylene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,2-Dichloropropane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| cis-1,3-Dichloropropene | ug/g | <0.030 | <0.030 | 0.030 | 3945801 |
| trans-1,3-Dichloropropene | ug/g | <0.040 | <0.040 | 0.040 | 3945801 |
| Ethylbenzene | ug/g | <0.020 | <0.020 | 0.020 | 3945801 |
| Ethylene Dibromide | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Hexane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Methylene Chloride(Dichloromethane) | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Methyl Isobutyl Ketone | ug/g | <0.50 | <0.50 | 0.50 | 3945801 |
| Methyl Ethyl Ketone (2-Butanone) | ug/g | <0.50 | <0.50 | 0.50 | 3945801 |
| Methyl t-butyl ether (MTBE) | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Styrene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,1,1,2-Tetrachloroethane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,1,2,2-Tetrachloroethane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| RDL = Reportable Detection Limit | | | | | |
| QC Batch = Quality Control Batch | | | | | |

Maxxam Job #: B542638
Report Date: 2015/03/18

Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

O.REG 153 VOLATILE ORGANICS (SOIL)

| Maxxam ID | | ZV1497 | ZV1498 | | |
|-----------------------------------|-------|-----------------|-----------------|-------|----------|
| Sampling Date | | 2015/03/04 | 2015/03/03 | | |
| COC Number | | na | na | | |
| | Units | BH 15-1 SA#5 | BH 15-6 SA#1 | RDL | QC Batch |
| Tetrachloroethylene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Toluene | ug/g | <0.020 | <0.020 | 0.020 | 3945801 |
| 1,1,1-Trichloroethane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| 1,1,2-Trichloroethane | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Trichloroethylene | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Vinyl Chloride | ug/g | <0.020 | <0.020 | 0.020 | 3945801 |
| p+m-Xylene | ug/g | <0.020 | <0.020 | 0.020 | 3945801 |
| o-Xylene | ug/g | <0.020 | <0.020 | 0.020 | 3945801 |
| Total Xylenes | ug/g | <0.020 | <0.020 | 0.020 | 3945801 |
| Trichlorofluoromethane (FREON 11) | ug/g | <0.050 | <0.050 | 0.050 | 3945801 |
| Surrogate Recovery (%) | | | | | |
| 4-Bromofluorobenzene | % | 90 | 87 | | 3945801 |
| D10-o-Xylene | % | 120 | 106 | | 3945801 |
| D4-1,2-Dichloroethane | % | 111 | 102 | | 3945801 |
| D8-Toluene | % | 100 | 97 | | 3945801 |
| RDL = Reportable Detection Limit | | | | | |
| QC Batch = Quality Control Batch | | | | | |

Maxxam Job #: B542638
Report Date: 2015/03/18

Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

TEST SUMMARY

Maxxam ID: ZV1497
Sample ID: BH 15-1 SA#5
Matrix: Soil

Collected: 2015/03/04
Shipped:
Received: 2015/03/10

| Test Description | Instrumentation | Batch | Extracted | Date Analyzed | Analyst |
|---|-----------------|---------|------------|---------------|-----------------------|
| Hot Water Extractable Boron | ICP | 3949646 | 2015/03/16 | 2015/03/18 | Suban Kanapathippalai |
| 1,3-Dichloropropene Sum | CALC | 3944229 | N/A | 2015/03/17 | Automated Statchk |
| Hexavalent Chromium in Soil by IC | IC/SPEC | 3947504 | 2015/03/13 | 2015/03/18 | Manoj Gera |
| Petroleum Hydro. CCME F1 & BTEX in Soil | HSGC/MSFD | 3949679 | 2015/03/12 | 2015/03/16 | Anca Ganea |
| Petroleum Hydrocarbons F2-F4 in Soil | GC/FID | 3947768 | 2015/03/13 | 2015/03/16 | (Kent) Maolin Li |
| Strong Acid Leachable Metals by ICPMS | ICP/MS | 3949559 | 2015/03/16 | 2015/03/16 | Grace Bu |
| Moisture | BAL | 3949445 | N/A | 2015/03/16 | Valentina Kaftani |
| Volatile Organic Compounds in Soil | GC/MS | 3945801 | 2015/03/12 | 2015/03/14 | Anna Gabrielyan |

Maxxam ID: ZV1498
Sample ID: BH 15-6 SA#1
Matrix: Soil

Collected: 2015/03/03
Shipped:
Received: 2015/03/10

| Test Description | Instrumentation | Batch | Extracted | Date Analyzed | Analyst |
|---|-----------------|---------|------------|---------------|-----------------------|
| Hot Water Extractable Boron | ICP | 3949646 | 2015/03/16 | 2015/03/18 | Suban Kanapathippalai |
| 1,3-Dichloropropene Sum | CALC | 3944229 | N/A | 2015/03/17 | Automated Statchk |
| Hexavalent Chromium in Soil by IC | IC/SPEC | 3947504 | 2015/03/13 | 2015/03/18 | Manoj Gera |
| Petroleum Hydro. CCME F1 & BTEX in Soil | HSGC/MSFD | 3949679 | 2015/03/12 | 2015/03/16 | Anca Ganea |
| Petroleum Hydrocarbons F2-F4 in Soil | GC/FID | 3947768 | 2015/03/13 | 2015/03/16 | (Kent) Maolin Li |
| Strong Acid Leachable Metals by ICPMS | ICP/MS | 3949559 | 2015/03/16 | 2015/03/16 | Grace Bu |
| Moisture | BAL | 3949445 | N/A | 2015/03/16 | Valentina Kaftani |
| Volatile Organic Compounds in Soil | GC/MS | 3945801 | 2015/03/12 | 2015/03/14 | Anna Gabrielyan |

Maxxam Job #: B542638
Report Date: 2015/03/18

Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

| | |
|-----------|-------|
| Package 1 | 3.7°C |
|-----------|-------|

Results relate only to the items tested.

Maxxam Job #: B542638
Report Date: 2015/03/18

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

| QC Batch | Parameter | Date | Matrix Spike | | Spiked Blank | | Method Blank | | RPD | | QC Standard | |
|----------|------------------------------------|------------|--------------|-----------|--------------|-----------|--------------|-------|-----------|-----------|-------------|-----------|
| | | | % Recovery | QC Limits | % Recovery | QC Limits | Value | Units | Value (%) | QC Limits | % Recovery | QC Limits |
| 3945801 | 4-Bromofluorobenzene | 2015/03/12 | 100 | 60 - 140 | 106 | 60 - 140 | 96 | % | | | | |
| 3945801 | D10-o-Xylene | 2015/03/12 | 95 | 60 - 130 | 97 | 60 - 130 | 75 | % | | | | |
| 3945801 | D4-1,2-Dichloroethane | 2015/03/12 | 102 | 60 - 140 | 97 | 60 - 140 | 110 | % | | | | |
| 3945801 | D8-Toluene | 2015/03/12 | 107 | 60 - 140 | 103 | 60 - 140 | 93 | % | | | | |
| 3947768 | o-Terphenyl | 2015/03/16 | 101 | 60 - 130 | 99 | 60 - 130 | 92 | % | | | | |
| 3949679 | 1,4-Difluorobenzene | 2015/03/16 | 108 | 60 - 140 | 105 | 60 - 140 | 113 | % | | | | |
| 3949679 | 4-Bromofluorobenzene | 2015/03/16 | 96 | 60 - 140 | 93 | 60 - 140 | 91 | % | | | | |
| 3949679 | D10-Ethylbenzene | 2015/03/16 | 76 | 60 - 140 | 72 | 60 - 140 | 73 | % | | | | |
| 3949679 | D4-1,2-Dichloroethane | 2015/03/16 | 94 | 60 - 140 | 92 | 60 - 140 | 108 | % | | | | |
| 3945801 | 1,1,1,2-Tetrachloroethane | 2015/03/12 | 108 | 60 - 140 | 103 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,1,1-Trichloroethane | 2015/03/12 | 107 | 60 - 140 | 97 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,1,2,2-Tetrachloroethane | 2015/03/12 | 108 | 60 - 140 | 106 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,1,2-Trichloroethane | 2015/03/12 | 106 | 60 - 140 | 97 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,1-Dichloroethane | 2015/03/12 | 110 | 60 - 140 | 104 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,1-Dichloroethylene | 2015/03/12 | 124 | 60 - 140 | 107 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,2-Dichlorobenzene | 2015/03/12 | 106 | 60 - 140 | 102 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,2-Dichloroethane | 2015/03/12 | 104 | 60 - 140 | 98 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,2-Dichloropropane | 2015/03/12 | 105 | 60 - 140 | 94 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,3-Dichlorobenzene | 2015/03/12 | 103 | 60 - 140 | 97 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | 1,4-Dichlorobenzene | 2015/03/12 | 103 | 60 - 140 | 98 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Acetone (2-Propanone) | 2015/03/12 | 97 | 60 - 140 | 96 | 60 - 140 | <0.50 | ug/g | NC | 50 | | |
| 3945801 | Benzene | 2015/03/12 | 102 | 60 - 140 | 95 | 60 - 130 | <0.020 | ug/g | NC | 50 | | |
| 3945801 | Bromodichloromethane | 2015/03/12 | 103 | 60 - 140 | 96 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Bromoform | 2015/03/12 | 97 | 60 - 140 | 101 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Bromomethane | 2015/03/12 | 107 | 60 - 140 | 96 | 60 - 140 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Carbon Tetrachloride | 2015/03/12 | 111 | 60 - 140 | 102 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Chlorobenzene | 2015/03/12 | 103 | 60 - 140 | 99 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Chloroform | 2015/03/12 | 108 | 60 - 140 | 101 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | cis-1,2-Dichloroethylene | 2015/03/12 | 106 | 60 - 140 | 101 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | cis-1,3-Dichloropropene | 2015/03/12 | 80 | 60 - 140 | 72 | 60 - 130 | <0.030 | ug/g | NC | 50 | | |
| 3945801 | Dibromochloromethane | 2015/03/12 | 104 | 60 - 140 | 103 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Dichlorodifluoromethane (FREON 12) | 2015/03/12 | 100 | 60 - 140 | 96 | 60 - 140 | <0.050 | ug/g | NC | 50 | | |

Maxxam Job #: B542638
Report Date: 2015/03/18

QUALITY ASSURANCE REPORT(CONT'D)

Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

| QC Batch | Parameter | Date | Matrix Spike | | Spiked Blank | | Method Blank | | RPD | | QC Standard | |
|----------|-------------------------------------|------------|--------------|-----------|--------------|-----------|--------------|-------|-----------|-----------|-------------|-----------|
| | | | % Recovery | QC Limits | % Recovery | QC Limits | Value | Units | Value (%) | QC Limits | % Recovery | QC Limits |
| 3945801 | Ethylbenzene | 2015/03/12 | 96 | 60 - 140 | 89 | 60 - 130 | <0.020 | ug/g | NC | 50 | | |
| 3945801 | Ethylene Dibromide | 2015/03/12 | 102 | 60 - 140 | 101 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Hexane | 2015/03/12 | 116 | 60 - 140 | 101 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Methyl Ethyl Ketone (2-Butanone) | 2015/03/12 | 92 | 60 - 140 | 99 | 60 - 140 | <0.50 | ug/g | NC | 50 | | |
| 3945801 | Methyl Isobutyl Ketone | 2015/03/12 | 96 | 60 - 140 | 96 | 60 - 130 | <0.50 | ug/g | NC | 50 | | |
| 3945801 | Methyl t-butyl ether (MTBE) | 2015/03/12 | 93 | 60 - 140 | 87 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Methylene Chloride(Dichloromethane) | 2015/03/12 | 117 | 60 - 140 | 109 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | o-Xylene | 2015/03/12 | 97 | 60 - 140 | 92 | 60 - 130 | <0.020 | ug/g | NC | 50 | | |
| 3945801 | p+m-Xylene | 2015/03/12 | 99 | 60 - 140 | 92 | 60 - 130 | <0.020 | ug/g | NC | 50 | | |
| 3945801 | Styrene | 2015/03/12 | 105 | 60 - 140 | 99 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Tetrachloroethylene | 2015/03/12 | 119 | 60 - 140 | 113 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Toluene | 2015/03/12 | 105 | 60 - 140 | 96 | 60 - 130 | <0.020 | ug/g | NC | 50 | | |
| 3945801 | Total Xylenes | 2015/03/12 | | | | | <0.020 | ug/g | NC | 50 | | |
| 3945801 | trans-1,2-Dichloroethylene | 2015/03/12 | 109 | 60 - 140 | 104 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | trans-1,3-Dichloropropene | 2015/03/12 | 78 | 60 - 140 | 67 | 60 - 130 | <0.040 | ug/g | NC | 50 | | |
| 3945801 | Trichloroethylene | 2015/03/12 | 108 | 60 - 140 | 104 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Trichlorofluoromethane (FREON 11) | 2015/03/12 | 116 | 60 - 140 | 104 | 60 - 130 | <0.050 | ug/g | NC | 50 | | |
| 3945801 | Vinyl Chloride | 2015/03/12 | 110 | 60 - 140 | 101 | 60 - 130 | <0.020 | ug/g | NC | 50 | | |
| 3947504 | Chromium (VI) | 2015/03/18 | 56 (1) | 75 - 125 | 101 | 80 - 120 | <0.2 | ug/g | NC | 35 | 97 | 80 - 120 |
| 3947768 | F2 (C10-C16 Hydrocarbons) | 2015/03/16 | 92 | 50 - 130 | 89 | 80 - 120 | <10 | ug/g | NC | 30 | | |
| 3947768 | F3 (C16-C34 Hydrocarbons) | 2015/03/16 | 99 | 50 - 130 | 98 | 80 - 120 | <50 | ug/g | NC | 30 | | |
| 3947768 | F4 (C34-C50 Hydrocarbons) | 2015/03/16 | 106 | 50 - 130 | 104 | 80 - 120 | <50 | ug/g | NC | 30 | | |
| 3949445 | Moisture | 2015/03/16 | | | | | | | 7.2 | 20 | | |
| 3949559 | Acid Extractable Antimony (Sb) | 2015/03/16 | 97 | 75 - 125 | 100 | 80 - 120 | <0.20 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Arsenic (As) | 2015/03/16 | 99 | 75 - 125 | 102 | 80 - 120 | <1.0 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Barium (Ba) | 2015/03/16 | NC | 75 - 125 | 104 | 80 - 120 | <0.50 | ug/g | 4.0 | 30 | | |
| 3949559 | Acid Extractable Beryllium (Be) | 2015/03/16 | 97 | 75 - 125 | 100 | 80 - 120 | <0.20 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Boron (B) | 2015/03/16 | 94 | 75 - 125 | 108 | 80 - 120 | <5.0 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Cadmium (Cd) | 2015/03/16 | 103 | 75 - 125 | 103 | 80 - 120 | <0.10 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Chromium (Cr) | 2015/03/16 | NC | 75 - 125 | 101 | 80 - 120 | <1.0 | ug/g | 0.22 | 30 | | |
| 3949559 | Acid Extractable Cobalt (Co) | 2015/03/16 | 98 | 75 - 125 | 102 | 80 - 120 | <0.10 | ug/g | 1.5 | 30 | | |
| 3949559 | Acid Extractable Copper (Cu) | 2015/03/16 | 100 | 75 - 125 | 102 | 80 - 120 | <0.50 | ug/g | 0.17 | 30 | | |

Maxxam Job #: B542638
Report Date: 2015/03/18

QUALITY ASSURANCE REPORT(CONT'D)

Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

| QC Batch | Parameter | Date | Matrix Spike | | Spiked Blank | | Method Blank | | RPD | | QC Standard | |
|----------|----------------------------------|------------|--------------|-----------|--------------|-----------|--------------|-------|-----------|-----------|-------------|-----------|
| | | | % Recovery | QC Limits | % Recovery | QC Limits | Value | Units | Value (%) | QC Limits | % Recovery | QC Limits |
| 3949559 | Acid Extractable Lead (Pb) | 2015/03/16 | NC | 75 - 125 | 102 | 80 - 120 | <1.0 | ug/g | 1.8 | 30 | | |
| 3949559 | Acid Extractable Mercury (Hg) | 2015/03/16 | 101 | 75 - 125 | 106 | 80 - 120 | <0.050 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Molybdenum (Mo) | 2015/03/16 | 101 | 75 - 125 | 101 | 80 - 120 | <0.50 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Nickel (Ni) | 2015/03/16 | 95 | 75 - 125 | 103 | 80 - 120 | <0.50 | ug/g | 0.92 | 30 | | |
| 3949559 | Acid Extractable Selenium (Se) | 2015/03/16 | 96 | 75 - 125 | 102 | 80 - 120 | <0.50 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Silver (Ag) | 2015/03/16 | 102 | 75 - 125 | 102 | 80 - 120 | <0.20 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Thallium (Tl) | 2015/03/16 | 98 | 75 - 125 | 99 | 80 - 120 | <0.050 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Uranium (U) | 2015/03/16 | 96 | 75 - 125 | 96 | 80 - 120 | <0.050 | ug/g | 0.24 | 30 | | |
| 3949559 | Acid Extractable Vanadium (V) | 2015/03/16 | NC | 75 - 125 | 99 | 80 - 120 | <5.0 | ug/g | NC | 30 | | |
| 3949559 | Acid Extractable Zinc (Zn) | 2015/03/16 | NC | 75 - 125 | 98 | 80 - 120 | <5.0 | ug/g | 4.2 | 30 | | |
| 3949646 | Hot Water Ext. Boron (B) | 2015/03/18 | 98 | 75 - 125 | 98 | 75 - 125 | <0.050 | ug/g | NC | 40 | | |
| 3949679 | F1 (C6-C10) - BTEX | 2015/03/16 | | | | | <10 | ug/g | NC | 30 | | |
| 3949679 | F1 (C6-C10) | 2015/03/16 | 93 | 60 - 140 | 98 | 80 - 120 | <10 | ug/g | NC | 30 | | |

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

QC Standard: A sample of known concentration prepared by an external agency under stringent conditions. Used as an independent check of method accuracy.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

Surrogate: A pure or isotopically labeled compound whose behavior mirrors the analytes of interest. Used to evaluate extraction efficiency.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of the native sample concentration).

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (one or both samples < 5x RDL).

(1) The matrix spike recovery was below the lower control limit. This may be due in part to the reducing environment of the sample. The matrix spike was reanalyzed to confirm result.

Maxxam Job #: B542638
Report Date: 2015/03/18

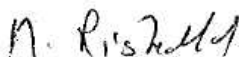
Golder Associates Ltd
Client Project #: 1419005
Site Location: CARLETON PLACE-HWY 7
Sampler Initials: RI

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).



Ewa Pranjić, M.Sc., C.Chem, Scientific Specialist



Medhat Riskallah, Manager, Hydrocarbon Department

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.



6740 Campbell Rd Mississauga, ON L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5778 Toll Free: (800) 563-6266

CHAIN OF CUSTODY RECORD

Page 1 of 1

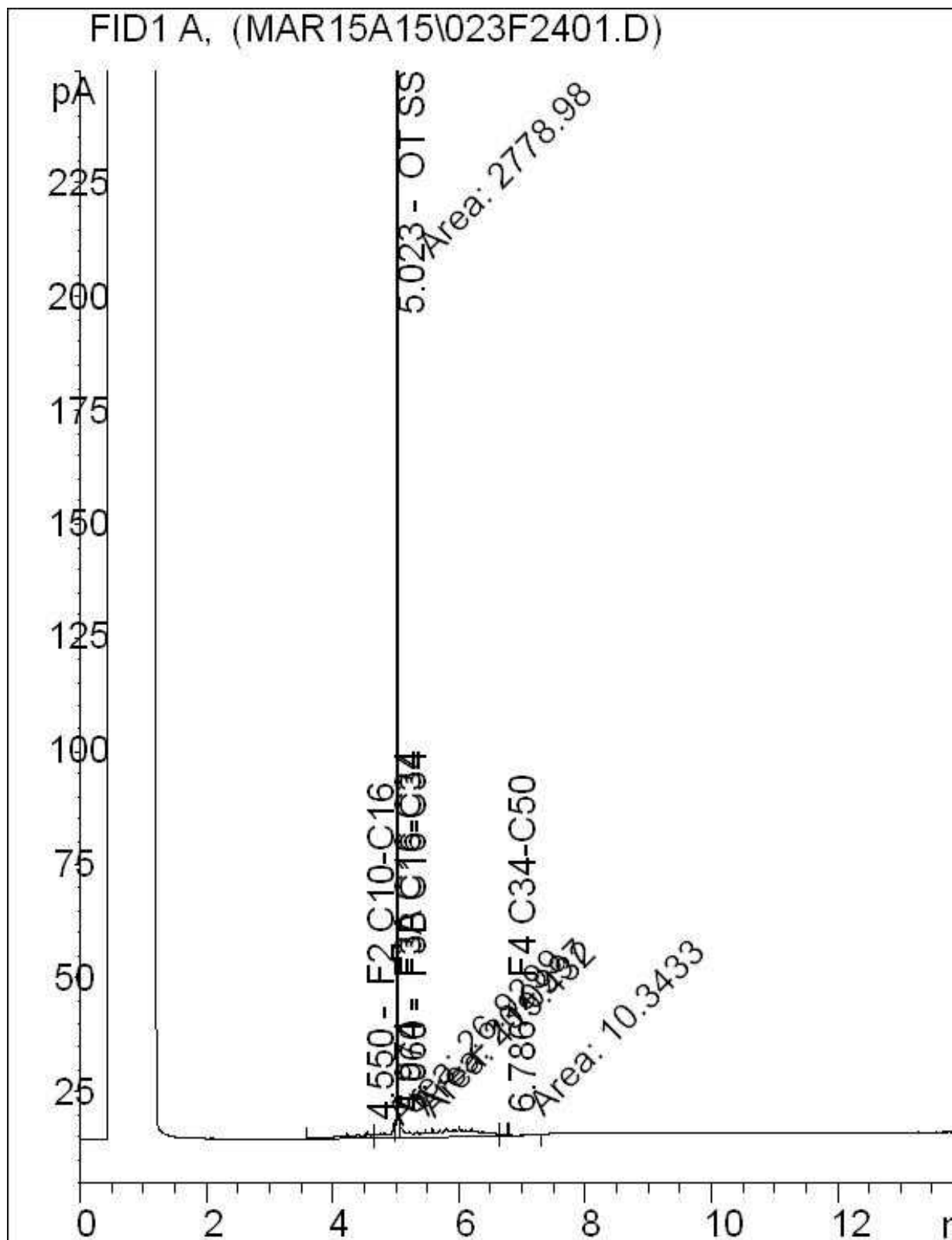
| INVOICE INFORMATION: | | REPORT INFORMATION (if differs from invoice): | | PROJECT INFORMATION: | | MAXXAM JOB NUMBER: | |
|---|------------------------|---|-----------------------------|--------------------------------------|-------------------------------------|---|---|
| Company Name: | Golder Associates Ltd. | Company Name: | | Quotation # | | CHAIN OF CUSTODY # : | |
| Contact Name: | Susan Trickey | Contact Name: | | P.O. # | | | |
| Address: | 1931 Robertson Road | Address: | | Project # | 1419005 | | |
| | Ottawa, Ontario | | | Project Name: | Town/Pumping Station/Carleton Place | | |
| Phone: | 613-592-9600 | Phone: | | Location: | Carleton Place - Hwy 7 | | |
| Fax: | 613-592-9601 | Fax: | 613-592-9601 | Sampled By: | Rob Ireland | | |
| Email: | strickey@golder.com | Email: | | | | | |
| REGULATORY CRITERIA | | ANALYSIS REQUESTED (Please be specific): | | | | TURNAROUND TIME (TAT) REQUIRED: | |
| Note: For regulated drinking water samples - please use the Drinking Water Chain of Custody Form | | | | | | PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS | |
| <input type="checkbox"/> MISA <input type="checkbox"/> PWQO <input type="checkbox"/> Reg. 558 | | | | | | Regular (Standard) TAT: <input checked="" type="checkbox"/> 5 to 7 Working Days | |
| <input checked="" type="checkbox"/> Table 1 <input checked="" type="checkbox"/> Table 2 <input checked="" type="checkbox"/> Table 3 <input checked="" type="checkbox"/> Table 6 | | | | | | Rush TAT: Rush Confirmation # (call Lab for #) <input type="checkbox"/> 1 day <input type="checkbox"/> 2 days <input type="checkbox"/> 3 days | |
| Other (specify): | | | | | | DATE Required: TIME Required: | |
| Report Criteria on C of A ? <input type="checkbox"/> | | | | | | Please note that TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details. | |
| SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM | | | | | | # of Cont. COMMENTS / TAT COMMENTS | |
| Sample Identification | Date Sampled | Time Sampled | Matrix (GW, SW, Soil, etc.) | Regulated Drinking Water ? (Y / N) | Metals Field Filtered ? (Y / N) | Metals (including Hg, CrVI) | |
| 1 BH 15-1 SA #5 | Mar. 4, 2015 | | Soil | | | | 4 |
| 2 BH 15-6 SA #1 | Mar. 3, 2015 | | Soil | | | | 4 |
| 3 | | | | | | | |
| 4 | | | | | | | |
| 5 | | | | | | | |
| 6 | | | | | | | |
| 7 | | | | | | | |
| 8 | | | | | | | |
| 9 | | | | | | | |
| 10 | | | | | | | |
| 11 | | | | | | | |
| 12 | | | | | | | |
| RELINQUISHED BY: (Signature/Print) | | RECEIVED BY: (Signature/Print) | | Date: | Time: | # JARS USED AND NOT SUBMITTED | Laboratory Use Only Temperature (°C) on Receipt |
| B. McParlan | | Kelsey Pilon | | March 10, 2015 | 15:15 | | |
| | | K. Pilon | | 2015/03/10 | 16:22 | | |
| | | A. B. B. B. | | 2015/03/11 | 09:32 | | 3, 4, 4. |

* MANDATORY SECTIONS IN GREY MUST BE FILLED OUT. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS

Maxxam Analytics International Corporation o/a Maxxam Analytics

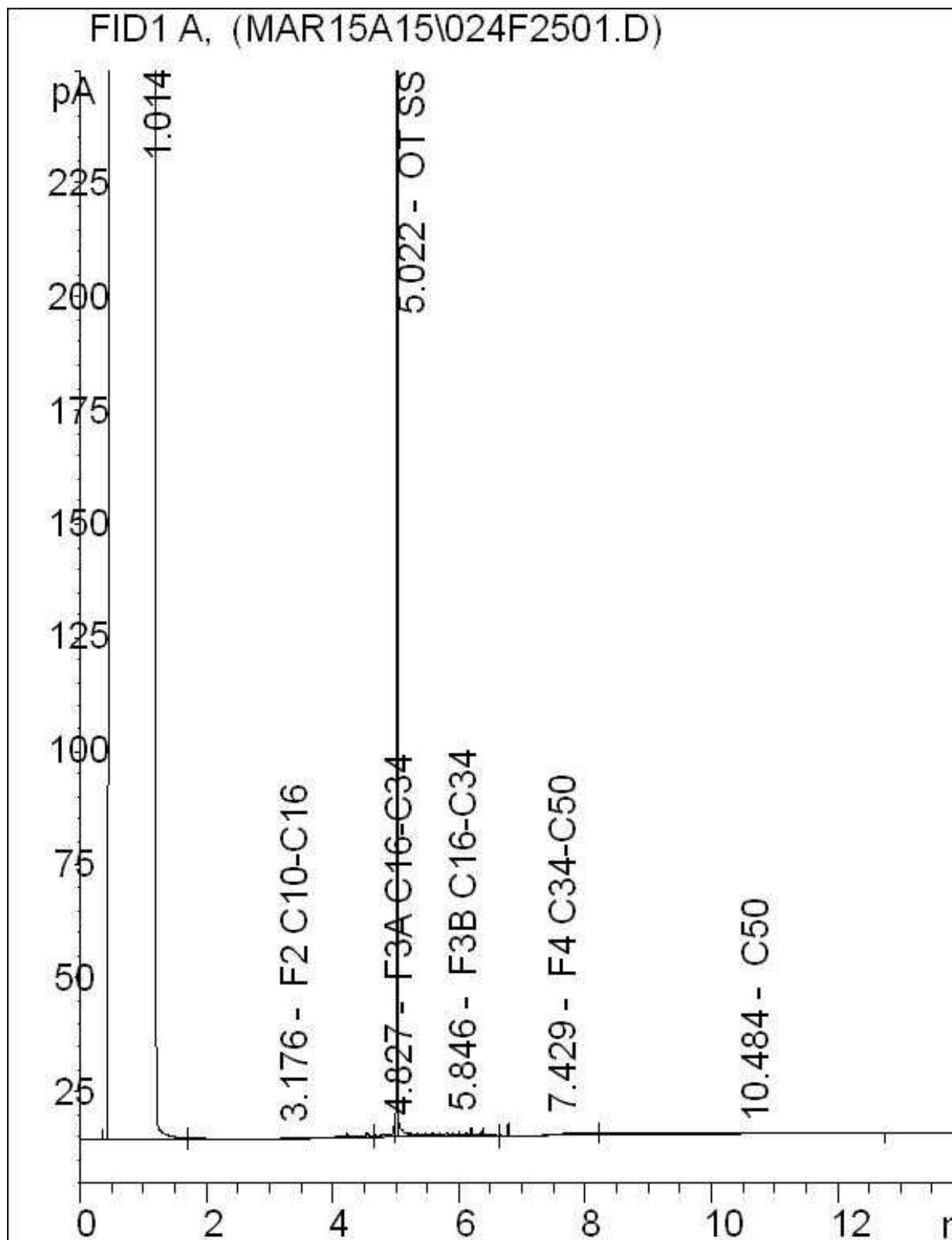
White: Maxxam Yellow: Mail Pink: Client

Petroleum Hydrocarbons F2-F4 in Soil Chromatogram



Note: This information is provided for reference purposes only. Should detailed chemist interpretation or fingerprinting be required, please contact the laboratory.

Petroleum Hydrocarbons F2-F4 in Soil Chromatogram



Note: This information is provided for reference purposes only. Should detailed chemist interpretation or fingerprinting be required, please contact the laboratory.



APPENDIX D

Results of Chemical Analysis

EXOVA Environmental Ontario Report No. 1504100

Client: Golder Associates Ltd. (Ottawa)
 1931 Robertson Road
 Ottawa, ON
 K2H 5B7
 Attention: Ms. Susan Trickey
 PO#:
 Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1504100
 Date Submitted: 2015-03-18
 Date Reported: 2015-03-23
 Project: 1419005
 COC #: 794827

| Group | Analyte | MRL | Units | Guideline | Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D. | |
|-------------------|-------------------------|-------|--------|-----------|--|--|
| | | | | | 1164620 Soil 2015-03-05 BH 15-3 SA4 / 3.05-3.66m | 1164621 Soil 2015-03-03 BH 15-5 SA1 / 1.52-2.13m |
| Agri. - Soil | pH | 2.0 | | | 8.1 | 7.8 |
| General Chemistry | Cl | 0.002 | % | | 0.012 | 0.004 |
| | Electrical Conductivity | 0.05 | mS/cm | | 0.37 | 0.22 |
| | Resistivity | 1 | ohm-cm | | 2700 | 4540 |
| | SO4 | 0.01 | % | | <0.01 | <0.01 |

Guideline = * = **Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).

Results relate only to the parameters tested on the samples submitted.

Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

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For more information, visit golder.com

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| Europe | + 44 1628 851851 |
| North America | + 1 800 275 3281 |
| South America | + 56 2 2616 2000 |

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