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REPORT ON

Geotechnical Investigation
Proposed Residential Development
Kellam Lands
Ottawa, Ontario

Submitted to:
Mary Jarvis
Urbandale Corporation
2193 Arch Street
Ottawa, Ontario
K1G 2H5

REPORT

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for a proposed residential development on the “Kellam Lands” which are located east of Bank Street, opposite Findlay Creek Drive, in Ottawa, Ontario.

The purpose of this subsurface investigation was to determine the general soil, bedrock and groundwater conditions across the site of the proposed development by means of a limited number of test pits and boreholes. Based on an interpretation of the factual information obtained, a general description of the subsurface conditions is presented. These interpreted subsurface conditions and available project details were used to provide geotechnical engineering guidelines on the design of the development, including construction considerations 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

Plans are being prepared for a proposed residential development on a parcel of land located east of Bank Street, opposite Findlay Creek Drive in Ottawa, Ontario (see Key Plan, Figure 1).

The following is known about the existing property and proposed development:

- The property is approximately rectangular in shape and measures about 1,000 by 300 metres in size.
- The site is bounded to the north and east by undeveloped land. The lands to the north are also proposed for a future residential development.
- The site is bounded to the south by further undeveloped land as well as an existing storm water management pond.
- An existing sanitary sewer is aligned along the north boundary of the site (along a corridor that is approximately in-line with the existing Findlay Creek Drive on the west side of Bank Street), which services lands further to the north (located north of Analdea Drive).
- An existing storm sewer crosses the east portion of the site (along an approximately north-south alignment), which connects that northern development to the storm water management pond.
- The east-central portion of the property, which is relatively lower in elevation compared to the extreme east and west portions, has been stripped of topsoil and engineered fill pads have been placed within future house areas. The west portion of the site, which is higher in elevation, is vegetated with some trees and scrub.
- The extreme east portion of the site, to the east of the storm sewer connection to the storm water management pond, has not been cleared of vegetation, but appears to have been previously filled (possibly during construction of the storm water management pond).
- The development plans for the site are not certain, but will likely consist of ground-oriented housing. Portions of the site have also been allocated for a commercial development (fronting on Bank Street), parks, and a school. However, these areas are not the focus of the current investigation.
- Based on the elevations of the existing sewers, it is understood that the founding levels will likely be in the range of elevation 90.5 to 91.0 metres. On the extreme west part of the site, the founding level could potentially be slightly higher, in the order of elevation 91.5 metres.

Golder Associates carried out the previous geotechnical investigation for the sanitary sewer which extends along the north side of the site. The results of that investigation were provided in the following report:

- “Geotechnical Investigation, Proposed Trunk Sewers, Sundance Village Development, Ottawa, Ontario” dated December 2010 (report number 10-1121-0014).

The results of that investigation indicate that the subsurface conditions along the sewer alignment (and therefore along the north part of the Kellam Lands site) generally consist of between about 2.5 and 6 metres of silt, sand, clayey silt, and glacial till overlying bedrock. Beneath the west section of the sewer alignment, the bedrock consists of dolomitic limestone. Beneath the east section, the bedrock consists of shale. This difference is consistent with the published geologic mapping, which indicates the Gloucester Fault to cross the west portion of this site. To the west of the fault, the bedrock is mapped as being dolomitic limestone of the Oxford Formation. To the east of the fault, the bedrock is mapped as being shale of the Carlsbad Formation.

Based on the previous investigation, the overburden soils above the glacial till along the western portion of the sewer/site primarily consist of more granular soils (silt and sand over glacial till). Over the east part, more cohesive soils, consisting of clayey silt, were encountered over the glacial till.

3.0 PROCEDURE

The field work for this investigation was carried out in two stages.

Between September 13 and 17, 2013, nineteen test pits (numbered 13-1 to 13-10, 13-201 to 202, and 13-301 to 13-306) were excavated across the site. On October 16 and 17, 2013, five boreholes (numbered 13-9A, 13-11, 13-12, 13-13A and 13-14A) were drilled at the site.

The approximate locations of the test pits and boreholes are shown on Figure 2.

The test pits were advanced using a track mounted excavator owned and operated by R.W. Tomlinson. The test pits were advanced to depths ranging from about 1.5 to 5.7 metres below the existing ground surface. The soils exposed on the sides of the test pits were classified by visual and tactile examination. Chunk samples were obtained from the major soil strata encountered in the test pits. The groundwater seepage conditions were observed in the open test pits. The test pits were loosely backfilled upon completion of excavating and sampling.

The boreholes were advanced using a track mounted hollow stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. The boreholes were advanced to depths ranging from about 5.9 to 8.2 metres below the existing ground surface. Within the boreholes, standard penetration tests were carried out at regular intervals of depth. Samples of the soils encountered were recovered using drive-open sampling equipment. Standpipe piezometers were sealed into three of the boreholes to allow for subsequent measurement of the groundwater level.

The field work was supervised by experienced personnel from our staff who located the boreholes and test pits; directed the drilling, excavating, and in situ testing operations; logged the boreholes, test pits, and samples; and took custody of the samples retrieved.

On completion of the drilling and test pitting operations, samples of the soils encountered were transported to our laboratory for examination by the project engineer and for laboratory testing. The laboratory testing program consisted of grain size determinations.

Soil samples from borehole 13-14A and test pit 13-4 were submitted to Exova Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The borehole and test pit locations were selected by Golder Associates. The locations were subsequently marked at the site, and the ground surface elevation surveyed, by Annis O'Sullivan Vollebekk Ltd. (AOV). A few of the test pit and borehole locations had to be adjusted from the surveyed locations based on site conditions. Where that was the case, the locations and elevations were referenced relative to the pickets installed by AOV.

The groundwater levels in the standpipe piezometers were measured on October 23, 2013.

4.0 SUBSURFACE CONDITIONS

4.1 General

The following information on the subsurface conditions on this site is provided in this report:

- The results of the test pits from the current investigation are provided in Table 1 – Record of Test Pits, following the text of this report.
- The results of the boreholes from the current investigation are provided on the Record of Borehole sheets in Appendix A.
- The results of the laboratory grain size distribution tests are given on Figures 3 and 4.
- The results of the basic chemical analyses on two samples of soil, one from each of test pit 13-4 and borehole 13-14A, are provided in Appendix B.
- The borehole and test pit records along with relevant laboratory test results from previous Golder report 10-1121-0014 are provided in Appendix C.

In general, the subsurface conditions at this site consist of topsoil, over silt and sand, over glacial till, over bedrock. Cohesive soil deposits, consisting primarily of firm to stiff clayey silt, are present (discontinuously) on the east side of the site. The bedrock surface exists at a depth of about 1.5 and 5.7 metres below the ground surface on the west side of the site and slopes downward toward the east (to below the depth investigated on the east side of the site). Random fill materials are present on the east side of the site. Engineered fill pads have also recently been constructed within the central portion of the site.

The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes and test pits. The following discussion is based primarily on the subsurface information collected from the present investigation. The results of the previous investigation along the north side of the site (Golder report 10-1121-0014) are referenced only in regards to selected details.

4.2 Fill Material and Topsoil

Heterogeneous fill materials were encountered on the east part of the site, at test pits 13-10, 13-201, 13-202 to 13-203, 13-301, 13-302, 13-305 and 13-306, as well as at all of the boreholes with the exception of 13-13A. The fill consists of varying amounts of silty sand, gravel, clay, and topsoil, and the layer ranges in thickness from 0.80 to 2.60 metres. A thin layer of wood chips, with a thickness of 150 millimetres, was encountered at surface at borehole 13-13A.

Granular/engineered fill materials, associated with the site preparation, were encountered at test pits 13-7 and 13-9, where they are about 2.0 and 1.5 metres thick, respectively.

Topsoil was encountered at ground surface in test pits 13-1 to 13-4, on the western portion of the site (i.e., the portion which hasn’t be stripped or filled), where it ranges in thickness of about 150 to 400 millimetres.

A buried topsoil layer was also encountered below the fill across much of the east portion of the site. Where present, the buried topsoil was encountered at depths ranging from 1.2 to 2.6 metres below ground surface, and was generally up to about 300 millimetres thick.

4.3 Silty Sand to Sandy Silt, Silt, and Sand

A deposit of silt and sand was encountered beneath the topsoil and/or fill across almost the entire site, with the exception of the northwest part of the site (i.e., except at test pits 13-2, 13-3, 13-4, and 13-6, where the glacial till deposit underlies the topsoil directly).

This stratum extends to depths of between 1.0 and at least 7.62 metres below the existing ground surface, with the deposit generally being thicker (and extending deeper) to the east.

The deposit generally consists of silt, sandy silt, and silty sand. However, shallower deposits of sand / sand and gravel were encountered at several locations. More significantly, a deposit of water-bearing and relatively “clean” sand was encountered below 4.6 metres depth in borehole 13-11. The deposit extends to at least the depth investigated, of about 8.2 metres.

The results of grain size distribution testing on selected samples of these deposits are provided on Figures 3 and 4. The results of previous testing (from Golder report 10-1121-0014) are also provided in Appendix C.

The results of standard penetration testing carried out within the silt, sandy silt, and silty sand gave ‘N’ values ranging from ‘weight of hammer’ to 20 blows per 0.3 metres of penetration, indicating a very loose to compact state of packing. Standard penetration testing carried out within the cleaner sand layers in boreholes 13-9A and 13-11 gave ‘N’ values ranging from ‘weight of hammer’ to 19 blows per 0.3 metres of penetration, also indicating a very loose to compact state of packing.

4.4 Silty Clay and Clayey Silt

Grey silty clay and clayey silt were encountered at test pit 13-301 and borehole 13-12, which are located in close proximity to each other in the southeastern portion of the site. These deposits are located below depths of about 2.6 and 4.6 metres at the two locations, respectively.

The deposit was fully penetrated by borehole 13-12, where it is about 2.4 metres thick.

Standard penetration testing carried out within the clayey silt gave ‘N’ values ranging from 2 to 4 blows per 0.3 metres of penetration indicating a probably firm to stiff consistency.

Similar deposits of clayey silt were encountered by the previous investigation (Golder report 10-1121-0014) along the east side of the north site boundary.

4.5 Glacial Till

A deposit of glacial till was generally encountered beneath all of the shallower deposits described above. Based on the recovered samples, as well as general experience with the local glacial till deposits, the deposit is considered to consist of a heterogeneous mixture of gravel, cobbles, and boulders in a silty sand matrix.

The glacial till was encountered at shallow depths, directly beneath the topsoil, within the western portion of the site. On the eastern portion, the deposit is present at greater depth (encountered at about 7 metres depth in borehole 13-12).Many of the test pits and boreholes on the east portion of the site did not in fact reach/encounter the deposit by their termination depths, including borehole 13-14A which was advanced to about 7.6 metres depth at the extreme southeast corner of the site.

Where fully penetrated by the test pits on the west side of the site, the glacial till overlies the bedrock.

4.6 Refusal and Bedrock

In general, the bedrock appears to slope downward toward the east.

The bedrock surface was encountered in test pits 13-1 to 13-6 and 13-8, all located on the west side of the site, at depths ranging between 1.5 and 5.7 metres below ground surface.

A summary of the depths and elevations of the bedrock surface, as well as the ground surface elevations at the test pit locations, is provided in the following table. Also included in this table are the data for the relevant boreholes and test pits previously advanced along the north side of the site (Golder report 10-1121-0014).

Test Pit / Borehole Number	Ground Surface Elevation (m)	Bedrock Surface / Refusal Depth (m)	Bedrock Surface / Refusal Elevation (m)	Remarks
13-1	91.50	3.00	88.50	Bedrock exposed in test pit
13-2	93.21	3.40	89.81	Bedrock exposed in test pit
13-3	91.04	1.50	89.54	Bedrock exposed in test pit
13-4	94.33	4.70	89.63	Bedrock exposed in test pit
13-5	90.91	4.20	86.71	Bedrock exposed in test pit
13-6	90.48	5.70	84.78	Bedrock exposed in test pit
13-8	89.02	4.40	84.62	Bedrock exposed in test pit
10-1	93.66	4.62	89.04	Cored
10-2	90.45	4.85	85.60	Bedrock sampled and weathered
10-101	94.24	2.49	91.75	Refusal
10-102	94.30	2.57	91.73	Refusal
10-103	93.49	2.13	91.36	Bedrock sampled and weathered
10-104	92.04	3.20	88.84	Weathered
10-105	92.51	4.72	87.79	Weathered
10-106	90.13	4.95	85.18	Refusal
10-107	89.62	4.80	84.82	Weathered
10-108	89.54	4.57	84.97	Bedrock sampled and weathered
10-109	89.60	5.84	83.76	Refusal
10-A	-	2.20	-	Bedrock exposed in test pit
10-B	-	2.70	-	Bedrock exposed in test pit
10-C	-	4.50	-	Bedrock exposed in test pit
10-D	-	1.90	-	Bedrock exposed in test pit

All of the boreholes advanced for the *current* investigation on site were terminated prior to reaching refusal/bedrock.

The results of additional previous laboratory testing on the bedrock (from Golder report 10-1121-0014), including compressive strength testing and 'whole rock' analyses, are provided in Appendix C.

4.7 Groundwater

The groundwater seepage conditions were observed in the test pits during the short time that they remained open. Groundwater seepage was only observed in three of the test pits (numbers 13-2, 13-3, and 13-8), where it occurred at depths ranging from about 1.4 to 4.4 metres below the existing ground surface. No seepage was observed in the remaining test pits. Groundwater seepage was also observed in three test pits during a previous investigation (Golder Report 10-1121-0014) along the north side of the site, at depths ranging from 1.0 to 3.8 metres below the existing ground surface.

For the standpipe piezometers installed in three of the boreholes on the east side of the site, a summary of the depths and elevations of the groundwater level measurements is provided in the following table. Also included in this table are the data for the relevant boreholes previously advanced along the north side of the site (Golder report 10-1121-0014).

Borehole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Date of Observation
BH 13-9A	90.38	0.65	89.73	October 23, 2013
BH 13-11	91.33	2.73	88.60	October 23, 2013
BH 13-14A	91.66	2.15	89.51	October 23, 2013
10-1 deep	93.66	3.17	90.49	September 28, 2010
10-1 shallow	93.66	3.72	89.94	February 16, 2010
10-2	90.45	0.04	90.41	March 29, 2010
10-103	93.49	0.17	93.32[1]	September 28, 2010
10-108 deep	89.54	0.06	89.48	September 28, 2010
10-108 shallow	89.54	0.70	88.84	September 28, 2010

Notes: [1] The measured groundwater level in borehole 10-103 is much higher than recorded elsewhere on the site but it consistent with groundwater levels measured in boreholes located on the property to the north of this site.

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.

Based on the measured groundwater levels as well as visual observations of the site conditions (e.g., surface ponding), the groundwater level is noted to be at relatively shallow depth (i.e., near the native ground surface level) beneath the central and east portions of the site (i.e., beneath those portions of the site where the ground surface level is lower, the silts and sands are thicker, and the bedrock surface is deeper).

5.0 DISCUSSION

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the test hole information and project requirements, and is subject to the limitations in the “Important Information and Limitations of This Report” which follows the text but forms an integral part of this report.

5.2 Site Grading

In general, the subsurface conditions at this site consist of topsoil, over silt and sand, over glacial till, over bedrock. Cohesive soil deposits, consisting primarily of firm to stiff clayey silt, are present on the east side of the site. The bedrock surface exists at a depth of about 1.5 and 5.7 metres below the ground surface on the west side of the site and slopes downward toward the east. Random fill materials are present on the east side of the site. Engineered fill has also recently been placed within the central portion of the site.

It is understood that a detailed grading design is not currently available for this site. However, preliminary grading information indicates that significant filling is required for the central and east (i.e., low) portions of the site. The footing levels will in some locations likely be up to about 1.5 metres (or possibly more) above the native ground surface level.

From a foundation design perspective, no practical restrictions apply to the thickness of grade raise fill that may be placed within the proposed residential development area. However, the feasibility of grade raises in excess of 4 metres, if proposed for portions of this site, should be reviewed.

With regards to the site grading, it should also be noted that the silt and sand deposits are relatively permeable and the groundwater levels within the central and east portions of the site are relatively shallow. Based on the anticipated founding elevations, excavations for basement construction should not need to extend below the groundwater level in these deposits. However, excavations for the installation of the site services in these areas which extend below the groundwater level could encounter problematic groundwater inflows. Therefore, there would be some advantage to limiting the required depth of excavation since the groundwater management requirements (and costs) increase with excavation depth below the groundwater level.

The grading should also ideally be selected so as to avoid or limit the bedrock excavation on the west side of the site.

As a more general comment, for predictable performance of the structures, roadways, and site services, preparation for filling of the site should include stripping the existing topsoil. The topsoil is not suitable as general fill and should be stockpiled separately for re-use in landscaping applications only.

On the east side of the site, the buried layer of topsoil encountered at some testing locations (beneath existing fill materials) will also need to be removed beneath building foundation areas, and from beneath grade-sensitive services (such as the storm and sanitary sewers). However, the need to remove the buried topsoil layer beneath future pavement areas will depend on the pavement subgrade level in comparison to the elevation of the topsoil layer. Further discussion is provided in Section 5.9.

5.3 Foundations

With the exception of the topsoil, the native soils and bedrock on this site are considered suitable for the support of conventional wood frame houses and townhouse blocks on spread footing foundations. For design purposes, the allowable bearing pressures for spread footings may be taken as 75 kilopascals for the silt and sand deposits, the clayey silt, and the glacial till provided these soils have not been disturbed by groundwater inflow or construction traffic. For footings founded on or within bedrock, an allowable pressure of 250 kilopascals may be used. These maximum allowable bearing pressures would be applicable for strip footings up to 1 metre in width and pad footings up to 2 metres in size.

Based on these allowable bearing pressure values, the house footings may be sized in accordance with Part 9 of the Ontario Building Code.

The post-construction total and differential settlements of footings supported on soil and sized using the above maximum allowable bearing pressures should be less than 25 and 15 millimetres, respectively, provided that the soil at or below founding level is not disturbed before or during construction. Suitable control of the groundwater inflow is required if such disturbance is to be avoided.

The glacial till overburden materials on this west side of this site contain cobbles and boulders. Any cobbles or boulders in footing areas which are loosened by the excavation process should be removed (and not pushed back into place) and the cavity filled with lean concrete. Otherwise, recompression of the disturbed soils could lead to larger than expected post-construction settlements.

Where the subgrade at footing level changes from bedrock to overburden, differential settlement could result at this transition due to the different settlement properties of these materials. To limit the magnitude of the differential settlement, transition details (such as placing additional reinforcing steel in the foundation walls, or removing additional bedrock to provide a more gradual transition) may be required. The details will need to be developed on a case-by-case basis, and the structural engineering consultant will need to be involved in the development of those measures. Wherever possible, it is recommended that individual units all be founded on the same medium, i.e., all soil or all bedrock.

It is expected that having footings bearing directly on bedrock would only be the case on the western part of the site. It is further expected that the bedrock in this area consists entirely of dolomitic limestone (and not shale). However, the exact location of the transition from dolomitic limestone to shale is not known, and will only be determined on a house-by-house basis at the time of construction.

The shale bedrock in the area of this site has the potential to expand (swell) following exposure to oxygen. This process involves a series of chemical reactions, some of which are purely chemical and others of which are at least catalyzed by micro-organisms. For these reactions to occur there must be both water and oxygen available. An increase in the ground temperature, such as due to the heat from the basement area, is also considered to promote the above reactions. Heaving of the shale could damage the foundations, basement floor slabs, and superstructures. It is also possible for the products of the above reactions to attack the concrete (i.e., sulphate attack).

To prevent expansion of the shale and/or reaction with the concrete, the shale must be protected from exposure to oxygen both in the long term as well as temporarily during construction. Therefore, if/where shale bedrock is encountered, the bedrock will need to be protected/covered with a mud slab of lean concrete.

The concrete mud slab should be made with sulphate resistant cement (Type HS or equivalent). Construction planning should ensure the shale is not left exposed and uncovered overnight.

In addition, the houses should be designed so that a uniform subgrade level will be provided for the entire house such that no areas of higher bedrock are left in-place which would be vulnerable to drying (i.e., stepped foundations or walk-outs should be avoided).

At some locations on the property, and based on some preliminary grading design information, it is understood that the inorganic subgrade elevation will be lower than the underside of footing elevation. At these locations, the subgrade will need to be raised to the footing elevation using compacted engineered fill. A broadly graded and relatively ‘clean’ granular soil has already been used to construct some engineered fill pads on this site, which is acceptable. Those materials were placed under essentially continuous compaction control/inspection and were compacted to at least 98 percent of their standard Proctor maximum dry density. More generally, and where additional engineered fill is required, Ontario Provincial Standard Specification (OPSS) Granular B Type II would be suitable, provided it is placed in maximum 300 millimetre thick lifts, and compacted to 95 percent of the material’s standard Proctor maximum dry density using suitable vibratory compaction equipment.

The engineered fill should be placed to occupy the full zone of influence/support of the building foundations, which is considered to extend out and down from the edge of the perimeter footings at a slope of 1 horizontal to 1 vertical. The native topsoil layer as well as any random (i.e., non-select) fill material should also be removed from within these limits.

Although significant settlements are not expected, there is the potential for slightly elevated settlements if/where particularly high grade raises are required (e.g., in excess of about 3 metres) and where the clayey silt deposit is present. The tolerance of the house foundations to accept those settlements could be increased by providing nominal amount of reinforcing steel in the top and bottom of the foundation walls.

There may be portions of the site where the shallow silty sand deposits will be exposed at footing/subgrade level. Prior to construction of footings or the placement of engineered fill within these areas, the surface of the native sandy material should be proof-rolled to provide surficial densification of any loose or disturbed material.

Since these shallow sandy deposits, wherever present, are typically loose, they could be potentially liquefiable in an earthquake (i.e., potentially subject to temporary strength loss and post-earthquake settlements). That potential issue is not, however, considered relevant to the house design because:

- The expected long term groundwater level will generally be below these soils, such that they will be above the water level and therefore non-liquefiable.
- The potential post-earthquake differential settlements would be relatively small in relation to the expected collapse potential of a house (and the objective of earthquake-resistant design is only to avoid collapse and to provide for safe exit).
- The proof rolling of the sandy subgrade soils, as specified above, would densify any such soils in the immediate area of the footings and therefore the directly supporting soils would be non-liquefiable.

5.4 Basement Excavations

Where the design founding level is below the native ground surface level, excavations for the house basements and the construction of the foundations will be made through topsoil, layered silts and sands, clayey silt, and glacial till. In some areas, bedrock excavation may also be required, which could be the case within the western portion of the site, in the vicinity of test pits 13-1 to 13-6, inclusive.

No unusual problems are anticipated with excavating the overburden using conventional hydraulic excavating equipment, recognizing that large boulders may be encountered in the glacial till. Boulders larger than 0.3 metres in size should be removed from the excavation side slopes, for worker safety.

Bedrock removal on the west part of the site, for the shallow depths of excavation that are anticipated, could be accomplished using mechanical methods (such as hoe ramming). Deeper excavations into the rock will likely require drill and blast procedures (but are not anticipated to be necessary on this site). Near vertical trench walls in the bedrock should stand unsupported for the construction period, at least for the shallow anticipated depths.

On the west/high portion of the site, excavations deeper than about 1.5 metres may extend below the groundwater level. However, based on the observed rate of seepage in the test pits, and given the relatively shallow expected depths of basement excavation, the rate of groundwater inflow is expected to be limited.

On the central and east portions of the site, and based on the anticipated founding elevations, excavation below the groundwater level in the sand and silt deposits (which could require significant groundwater control) is not anticipated to be required. This assessment should, however, be confirmed once a grading plan is available.

Provided the basement excavations are made above the groundwater level, excavation side slopes should be stable in the short term at 1 horizontal to 1 vertical. Flatter side slopes would be required below the groundwater level (i.e., Type 4 soils).

As previously discussed in Section 5.3, to prevent expansion of the shale (where present) and/or reaction with the concrete, the shale must be protected from exposure to oxygen both in the long term as well as temporarily during construction. When exposed during construction, the shale must be covered as soon as practical following exposure with a 50 millimetre thick concrete mud slab.

The concrete mud slab should be made with sulphate resistant cement (Type HS or equivalent). Construction planning should ensure the shale is not left exposed and uncovered overnight.

5.5 Basement Floor Slabs

In preparation for the construction of basement floor slabs, all loose, wet, and disturbed material should be removed from beneath the floor slab. Provision should be made for at least 200 millimetres of 19 millimetre clear crushed stone to form the base of the floor slab. The underslab fill should be compacted to at least 95 percent of its standard Proctor maximum dry density using suitable vibratory compaction equipment.

To prevent hydrostatic pressure build up beneath the floor slab, it is suggested that the granular base for the floor slab be drained. This could be achieved by providing a hydraulic link between the underfloor fill and the exterior drainage system.

Where the footing level is below the natural groundwater level, and the subgrade consists of the native sand and silt, there would be the potential for the groundwater inflow into the underslab drainage system to lead to loss of ground and settlement of structures, due to the loss of soil particles from the subgrade soils into the voids in the underslab clear stone. Therefore, where that is the case, the clear stone should be separated from the subgrade with a Class II non-woven geotextile, in accordance with OPSS 1860, having a Filtration Opening Size (FOS) not exceeding 100 microns. However, where the footings (and overall structure) are constructed on at least 200 millimetres of engineered fill (or on bedrock), the geotextile would not be required.

5.6 Frost Protection

The native soils at this site are frost susceptible. For frost protection purposes, all exterior footings or interior footings in unheated areas should be provided with a minimum of 1.5 metres of earth cover. Isolated, 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.

5.7 Basement Walls and Wall Backfill

The soils at this site are highly frost susceptible and should not be used as backfill directly against exterior, unheated or well insulated foundation elements. To avoid problems with frost adhesion and heaving, these foundation elements should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I or, alternatively, a bond break such as the Platon system sheeting could be placed against the foundation walls.

Drainage of the basement wall backfill should be provided by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit. Conventional damp proofing of the basement walls is appropriate with the above design approach.

Should the foundations be designed in accordance with Part 4 of the Ontario Building Code, further guidelines on the foundation wall design will need to be provided.

5.8 Site Servicing

Excavations for the installation of site services will be through the overburden soils and, at least on some portions of the site, into bedrock. Based on the observed groundwater conditions in the open test pits and in the standpipes, it is expected that many of these excavations will be below the groundwater level.

Significant groundwater inflow should be expected from the dolomitic limestone bedrock which is present on the west portion of the site, and also from the sandier portions of the overburden. Lesser groundwater inflow is expected from the silt, glacial till, and shale bedrock.

Based on previous investigation work completed in the area of this site, including the investigation for the sanitary sewer along the north site boundary, the dolomitic limestone is expected to have a hydraulic conductivity in the range of 10^{-3} to 10^{-1} centimetres per second, which is very high. Therefore, significant groundwater inflows are expected for excavations extending into this bedrock formation. The flow will be primarily from the upper several metres, where the bedrock is typically quite fractured. Therefore, where excavations are expected to extend into the dolomitic limestone bedrock, the pumping requirements will be significant. Pre-pumping from sumps in the bedrock for a period of a few weeks might be a feasible method to lowering the groundwater in advance of excavation. This method of groundwater control was required and successfully used on other nearby sites.

The rate of groundwater inflow from the sandier overburden materials will likely also be significant, resulting in possible disturbance of the excavation subgrade and potential instability of the excavation side slopes. Based on past experience on adjacent sites, some pre-drainage of the sandier overburden will likely be required, but which may also occur in conjunction with pre-drainage of the bedrock. The drainage could also be carried out by constructing several sumps and pre-pumping from the sandier overburden carried out in advance of excavation.

It should be noted that an apparently very permeable sand deposit was encountered in the east portion of the site, at borehole 13-11. As indicated on Figure 4, the 'fines' content of this deposit is no more than about 10 percent. Excavation below the groundwater level within this deposit could result in significant groundwater inflow, loss of ground, disturbance of the trench subgrade, and instability of the excavation side slopes. Post-construction settlement of the sewers might occur if the subgrade becomes disturbed due to excessive groundwater inflow during construction. Active groundwater level lowering is therefore likely to be required if excavation bellow the groundwater level, and into the sand deposit, is required in this area. It would in that case be recommended to lower the groundwater level to at least 0.5 metres below the planned bottom of excavation level, in advance of excavation. The installation of active eductor wells and/or well points to achieve the groundwater lowering in the sandy soils might be necessary and is considered an appropriate methodology. The design of the temporary groundwater control system should, however, be entirely the responsibility of the contractor. Consideration should be given at the time of tender to carrying out a test excavation in this area, in the presence of the bidders, so that the actual excavating conditions and rate of groundwater inflow can be assessed.

The hydraulic conductivities of the silty soils, glacial till and shale bedrock are expected to be in the range of 10^{-6} to 10^{-4} centimetres per second. Groundwater inflow into the trenches in these materials could initially be significant, but should diminish with time and continued pumping, and it should generally be possible to handle the groundwater inflow by pumping from well filtered sumps and using suitably sized and multiple pumps within the excavations.

Where the trench will be entirely within the glacial till but with the surface of the underlying bedrock at only shallow depth below the trench floor, there could be a risk basal heaving of the trench floor; basal heaving occurs where the weight of the soil cover is less than the piezometric pressure in the underlying bedrock. Such basal heaving could result in disturbance of the pipe subgrade. However, the groundwater control operations for the westerly sections of sewer, which will be installed in bedrock, will involve pumping from the dolomitic limestone bedrock, and the zone of influence of that pumping may extend beneath the adjacent sections of pipe as well. If that is the case, and if the rate of pumping is sufficient, it is possible that this pumping could sufficiently lower the groundwater level in the bedrock such that basal heaving would not occur.

The actual rate of groundwater inflow to the trenches will depend on many factors including the contractor's schedule and rate of excavation, the size of excavation, and the time of year at which the excavation is made. The expected level of pumping would require that a Category 3 Permit-To-Take-Water (PTTW) be obtained from the Provincial Ministry of the Environment (MOE).

As discussed above, significant volumes of water will be pumped from the excavations. Water pumped from the excavations will likely be discharged (possibly via ditches) to the storm water management pond which is located south of this site, north of Blais Road. The dewatering or excavation contractor should be made responsible for obtaining the necessary permits for discharge and ensuring compliance with the applicable sewer use by-law.

Excavations within the layered sand, sandy silt, silty sand, and silt, and glacial till below the water table should be carried out within a protective trench box. The stand up time for exposed side slopes will be extremely short and the subgrade will be disturbed if left exposed for any length of time. Construction of the site services should be planned to be carried out in short sections which can be fully completed in a minimal amount of time.

The contractor should prepare a groundwater management plan for review and approval.

Bedrock removal could be accomplished using mechanical methods (such as hoe ramming), at least for shallow depths of excavation. Deeper excavations will likely require drill and blast procedures. Near vertical trench walls in the bedrock should stand unsupported for the construction period, at least for moderate depths (i.e., less than about 3 metres).

It should also be noted that the bedrock surface elevation and quality may be very irregular in the area of the fault that defines the transition between the dolomitic limestone and the shale.

Blasting should be controlled to limit the peak particle velocities at all adjacent structures or services (e.g., the existing storm sewer north of the site) such that blast induced damage will be avoided. Blast designs should be prepared by a specialist in this field.

A pre-blast survey should be carried out of all the surrounding structures and utilities.

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 submission 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 and services are suggested.

Frequency Range (Hz)	**Vibration Limits (mm/sec)**
< 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 operations be carried out both in the ground adjacent to the closest structures/utilities and within the structures/utilities themselves.

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs, it may be necessary to place a sub-bedding layer consisting of 300 millimetres of compacted OPSS Granular B Type II beneath the Granular A. 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 standard Proctor maximum dry density. 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 or surrounding soil could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

On the east side of the site, where existing fill materials are present and overlie a buried topsoil layer, the fill materials and topsoil should be subexcavated beneath grade-sensitive pipes and be replaced with compacted engineered fill. That fill could consist of OPSS Granular B Type II, placed and compacted as described above.

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 standard Proctor maximum dry density.

It is should be generally acceptable to re-use the excavated overburden soils as trench backfill. However, some of the overburden materials (such as the sandy silts) may be too wet to compact. Where that is the case, the wet materials should be wasted (and drier materials imported) or these materials should be placed only in the lower portions of the trench, recognizing that some future settlement of the roadways may occur and some significant padding of the roadways may be required prior to final paving. In that case, it would also be prudent to delay final paving for as long as practical.

Well fractured or well broken bedrock will be acceptable as backfill within the lower portions of the service trenches in areas where the excavation is in rock. The rock fill, however, should only be placed from at least 300 millimetres above the pipes to minimize damage due to impact or point loading. The rock fill should be limited to a maximum of 300 millimetres in size.

In areas where the trench will be covered with hard surfaced materials, the type of material placed within the frost zone (between finished grade and two 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 standard Proctor maximum dry density. It should be noted that some of the excavated materials will be quite wet and difficult to compact. These materials would best be placed in the lower portions of the trenches to minimize the post-construction settlements of the backfill.

5.9 Pavement Design

In preparation for pavement construction, all topsoil and deleterious material (i.e., those fill material containing organic material) should be removed from all pavement areas.

The existing fill materials on the east side of the site, and the underlying topsoil layer, can potentially remain in-place beneath future pavements, depending on the subgrade profile level. As a preliminary guideline, and based on the relatively limited apparent organic content of the topsoil, it is considered that the presence of the buried topsoil layer should not have a measurably detrimental impact on the pavement performance provided it will be located at least 1 metre below subgrade level (i.e., provided it is separated from the pavement structure by at least a 1 metre thickness of subgrade fill). However, the subgrade surface, where composed of fill material, should be heavily proof-rolled with a vibratory roller, to compact the fill and to identify soft areas requiring subexcavation and replacement with more suitable fill. Some limited post-construction settlement of the pavement surface should, however, be expected if the fill material is left in-place (since it is not feasible to compact the full thickness of the fill material solely by means of proof rolling the subgrade surface). However, the level of post-construction settlement is not expected to exceed what is normally expected for residential roadways due to trench backfill compression. This issue should, however, be reviewed once the design roadway grades are known.

Sections requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material. These materials should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of their standard Proctor maximum dry density using suitable compaction equipment.

Transitions from bedrock to earth subgrade (if this condition is encountered) should be carried out in accordance with the OPSD 205 series. The transition depth “t” should be taken as 1.8 metres.

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

The required pavement structure for the roadways will depend upon the quality of the backfill in the service trenches. Within at least the central (i.e., lowest) portion of the site, the shallow subgrade soils are generally wet of the optimum for compaction. It should therefore be expected that the subgrade in at least these areas will need to be covered with a suitable woven geotextile.

The pavement structure for local roads should be:

Pavement Component	Thickness (mm)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	375

The pavement structure for collector roadways should be:

Pavement Component	Thickness (mm)
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 310, 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 pavement design should be based on a Traffic Category of Level B on local roads and Level C on collector roads. The asphalt cement should be PG 58-34.

In regards to the above pavement structure for the local roads, it should be noted that the 50 millimetres of asphaltic concrete base course would provide sufficient structural support and would therefore be adequate for the initial periods of roadway service. However, the 90 millimetres of asphaltic concrete is often specified for the local roadways based on the typical construction sequence which would require a ‘final’ surface course placement following substantial completion of the house construction.

In addition, if a similar paving sequence is proposed for the *collector* roads, with an additional course being required upon substantial completion of site development, then a thicker overall asphaltic concrete layer would be required (to allow for three lifts), since two initial lifts will likely be required to support the construction traffic. Alternatively, a thicker base course could be provided, to support the construction traffic. For example, one 80 millimetre thick base course could be provided during the construction phase and a 40 millimetre surface course provided at the substantial completion. Further guidelines for both options can be provided, if required.

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. Given that the roadway subgrade in some locations could consist of relatively wet trench backfill, it should be planned to include a significant contingency for such works.

5.10 Corrosion and Cement Type

Two soil samples, one each from test pit 13-4 and borehole 13-14A, were submitted to Exova Ltd. for chemical analysis related to potential corrosion of exposed buried ferrous steel and potential sulphate attack on buried concrete elements. The results of this testing are provided in Appendix B.

The results indicate a relatively elevated potential for corrosion of exposed ferrous metal (particularly for the data for test pit 13-4), which should be considered in the design of substructures.

The results also indicate that concrete made with Type GU Portland cement should be acceptable for substructures. However, the results of previous testing (Golder report 10-1121-0014) indicated an elevated sulphate concentration for one area along the sanitary sewer alignment, such that concrete made with Type HS cement should be used for substructures. It is understood, however, that concrete sewer pipes and manholes are generally made with sufficient chemical resistance additives in the concrete to provide the necessary sulphate resistance. This understanding should, however, be confirmed with the sewer pipe and manhole supplier.

5.11 Pools, Decks and Additions

5.11.1 Above Ground and In Ground Pools

No special geotechnical considerations are necessary for the installation of in-ground or above ground pools.

5.11.2 Decks

There are no special geotechnical considerations for decks on this site.

5.11.3 Additions

Any proposed addition to a house (regardless of size) will require a geotechnical assessment. Written approval from a geotechnical engineer should be required by the City of Ottawa prior to the building permit being issued.

5.12 Re-Use of Shale Bedrock

As previously discussed, the shale bedrock (if encountered) has the potential to swell once exposed to air (i.e., once allowed to dry); this swelling could be detrimental to the performance of overlying grade dependent structures. Therefore, given the potential swelling nature of the shale bedrock, this material should not be used for roadway subgrade fill, garage backfill, or foundation wall backfill, unless the swelling potential and characteristics are assessed (by means of laboratory testing) and found acceptable.

5.13 Trees

When trees draw water from clayey soils, the soil can experience shrinkage which can result in settlement of adjacent structures.

The soils at this site are generally non-clayey in nature, in which case no restrictions would apply to the planting of trees adjacent to proposed structures.

Some clayey silt was encountered beneath the extreme east part of the site, however, this soil is considered to be have a low shrinkage-potential, and the grading will also likely be such that the deposit would be quite deep and therefore below the expected depth of root penetration. In this regard, restrictions on the planting of trees (from a geotechnical perspective) are also not expected to be necessary in this area. However, this assessment should be reviewed once the site grading is known.

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 view point.

The test pits were loosely backfilled upon completion of excavating and therefore constitute zones of disturbance. The locations were selected to be outside of foundation areas. However, should the development layout change such that the test pits will be located within the areas of influence/support of future buildings, then those test pits will need to be repaired at the time of construction.

At the time of the writing of this report, only conceptual details for the proposed development were available. Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

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 will either be destroyed during construction or can be more economically abandoned as part of the construction contract. If that is not the case or is not considered feasible, abandonment of the monitoring wells can be carried out separately.

7.0 CLOSURE

We trust that this report contains sufficient information for your present purposes. If you have any questions regarding this report, or if we can be of further service to you on this project, please contact the undersigned.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
M. I. CUNNINGHAM
PROVINCE OF ONTARIO

Mike Cunningham, P.Eng
Associate

KE/MIC/bg

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Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

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.

TABLE 1

RECORD OF TEST PITS

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 13-1	0.0 – 0.4	Black TOPSOIL
(91.50 m)	0.4 – 1.5	Grey brown SANDY SILT
	1.5 – 3.0	Grey SILTY SAND, trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	3.0	Refusal on BEDROCK
		Test Pit dry upon completion
		<u>Sample</u> <u>Depth (m)</u> 1 0.4 – 1.50
TP 13-2	0.0 – 0.2	Black TOPSOIL with roots
(93.21 m)	0.2 – 1.15	Brown SILTY SAND trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	1.15 – 3.4	Grey SILTY SAND, trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	3.4	Refusal on BEDROCK
		Water Seepage at 3.4 metres
		<u>Sample</u> <u>Depth (m)</u> 1 0.2 – 1.15
TP 13-3	0.0 – 0.15	Black TOPSOIL with roots
(91.04 m)	0.15 – 1.40	Brown SILTY SAND trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	1.1 – 1.5	Grey SILTY SAND, trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	1.5	Refusal on BEDROCK
		Water Seepage at 1.4 metres
		<u>Sample</u> <u>Depth (m)</u> 1 0.15 – 1.4

TABLE 1

RECORD OF TEST PITS

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 13-4	0.0 – 0.15	Black TOPSOIL
(94.33 m)	0.15 – 2.7	Brown SILTY SAND trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	2.7 – 4.7	Grey SILTY SAND, trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	4.7	Refusal on BEDROCK
		Test Pit dry upon completion
		<u>Sample</u> <u>Depth (m)</u> 1 0.15 – 2.70 2 2.70 – 4.70
TP 13-5	0.0 – 1.0	Grey brown SANDY SILT
(90.91 m)	1.0 – 4.2	Grey SANDY SILT, trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	4.2	Refusal on BEDROCK
		Test Pit dry upon completion
		<u>Sample</u> <u>Depth (m)</u> 1 1.0 – 4.20
TP 13-6	0.0 – 1.0	Brown SAND and GRAVEL
(90.48 m)	1.0 – 5.7	Grey SANDY SILT, trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	5.7	Refusal on BEDROCK
		Test Pit dry upon completion
		<u>Sample</u> <u>Depth (m)</u> 1 1.0 – 5.7

TABLE 1

RECORD OF TEST PITS

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 13-7	0.0 – 2.0	Sand and gravel (FILL)
(92.69 m)	2.0 – 3.9	Grey SILTY SAND
	3.9 – 5.25	Grey SILTY SAND, trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	5.25	End of Test Pit
TP 13-8	0.0 – 1.5	Grey brown SANDY SILT
(89.02 m)	1.5 – 3.8	Grey SANDY SILT
	3.8 – 4.40	Grey SANDY SILT, trace gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	4.40	Refusal on BEDROCK
		Water Seepage at 4.35 metres
		Sample Depth (m) 1 0.0 – 1.5 2 1.5 – 3.80
TP 13-9	0.0 – 1.5	Sand, gravel and crushed stone (FILL)
(90.56 m)	1.5 – 2.8	Brown SILTY SAND to SAND SILT
	2.8 – 4.75	Grey SILTY SAND to SANDY SILT
	4.75	End of Test Pit
TP 13-10	0.0 – 1.2	Grey brown silty sand (FILL)
(90.25 m)	1.2 – 1.55	Black TOPSOIL with roots
	1.55 – 2.85	Brown SILTY SAND to SAND SILT
	2.85 – 5.65	Grey SILTY SAND to SANDY SILT
	5.65	End of Test Pit

TABLE 1

RECORD OF TEST PITS

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 13-201	0.0 – 0.8	Grey silty sand, trace topsoil (FILL)
(91.60 m)	0.8 – 2.2	Brown SAND, some gravel
	2.2 – 3.0	Grey brown SANDY SILT
	3.0	End of Test Pit
		Test Pit dry upon completion
		Sample Depth (m)
		1 2.2 – 3.0
TP 13-202	0.0 – 2.4	Clay, sand, gravel and till intermixed (FILL)
(91.40 m)	2.4 – 2.6	Black TOPSOIL with roots
	2.6 – 3.6	Grey brown SANDY SILTY, some clay
	3.6	End of Test Pit
		Test Pit dry upon completion
		Sample Depth (m)
		1 2.6 – 3.6
TP 13-203	0.0 – 1.5	Silty sand, some clay (FILL)
(90.30 m)	1.5 – 1.7	Black TOPSOIL with roots
	1.7 – 2.5	Grey brown SAND SILT
	2.5	End of Test Pit
		Test Pit dry upon completion

TABLE 1

RECORD OF TEST PITS

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 13-301	0.0 – 2.3	Clay and till (FILL)
	2.3 – 2.6	Black TOPSOIL with roots
	2.6 – 3.0	Grey SILTY CLAY
	3.0	End of Test Pit
		Test Pit dry upon completion
		Sample Depth (m) 1 2.6 – 3.0
TP 13-302	0.0 – 1.9	Silty sand, clay and till (FILL)
	1.9 – 2.2	Black TOPSOIL with roots
	2.2 – 2.8	Grey brown SANDY SILT
	2.8	End of Test Pit
		Test Pit dry upon completion
		Sample Depth (m) 1 2.2 – 2.8
TP 13-303	0.0 – 2.3	Brown medium grained SAND, some gravel, trace organics
	2.3 – 3.3	Grey SANDY SILT
	3.3	End of Test Pit
		Test Pit dry upon completion

TABLE 1

RECORD OF TEST PITS

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 13-304	0.0 – 2.3	Brown medium grained SAND, some gravel, trace organics
	2.3 – 2.9	Grey brown SANDY SILT
	2.9	End of Test Pit
		Test Pit dry upon completion
		<u>Sample</u> <u>Depth (m)</u> 1 0.0 – 2.3 2 2.3 – 2.9
TP 13-305	0.0 – 2.3	Grey silty sand and clay (FILL)
	2.3 – 2.6	Black TOPSOIL with roots
	2.6 – 3.2	Grey brown SANDY SILT
	3.2	End of Test Pit
		Test Pit dry upon completion
		<u>Sample</u> <u>Depth (m)</u> 1 2.6 – 3.2
TP 13-306	0.0 – 2.6	Grey silty sand (FILL)
	2.6 – 2.9	Black TOPSOIL with roots
	2.9 – 3.6	Grey brown SANDY SILT, trace clay
	3.6	End of Test Pit
		Test Pit dry upon completion
		<u>Sample</u> <u>Depth (m)</u> 1 2.9 – 3.6

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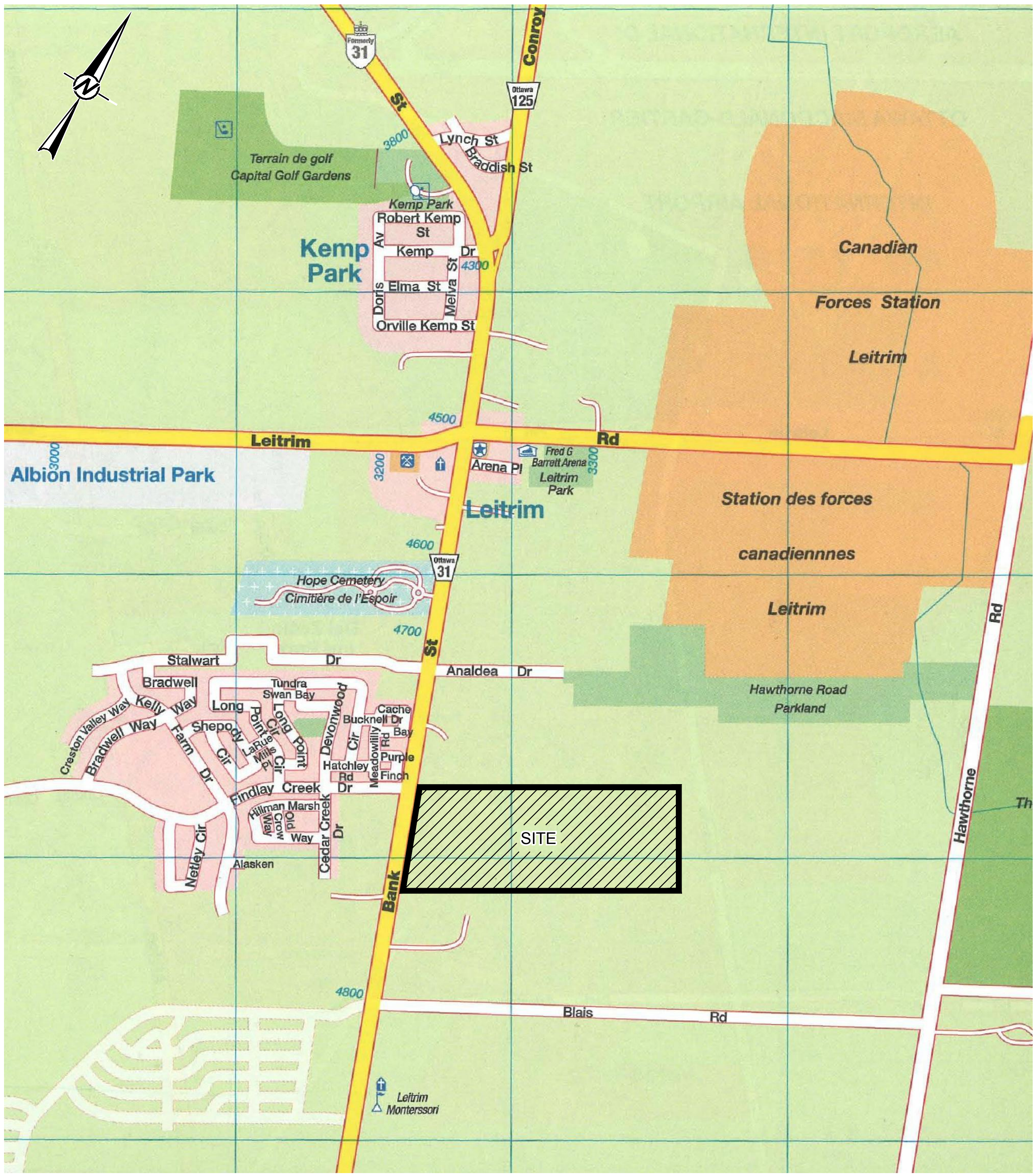
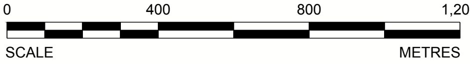


NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1121-0286



△						
REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT

GEOTECHNICAL INVESTIGATION
KELLAM LANDS
OTTAWA, ONTARIO

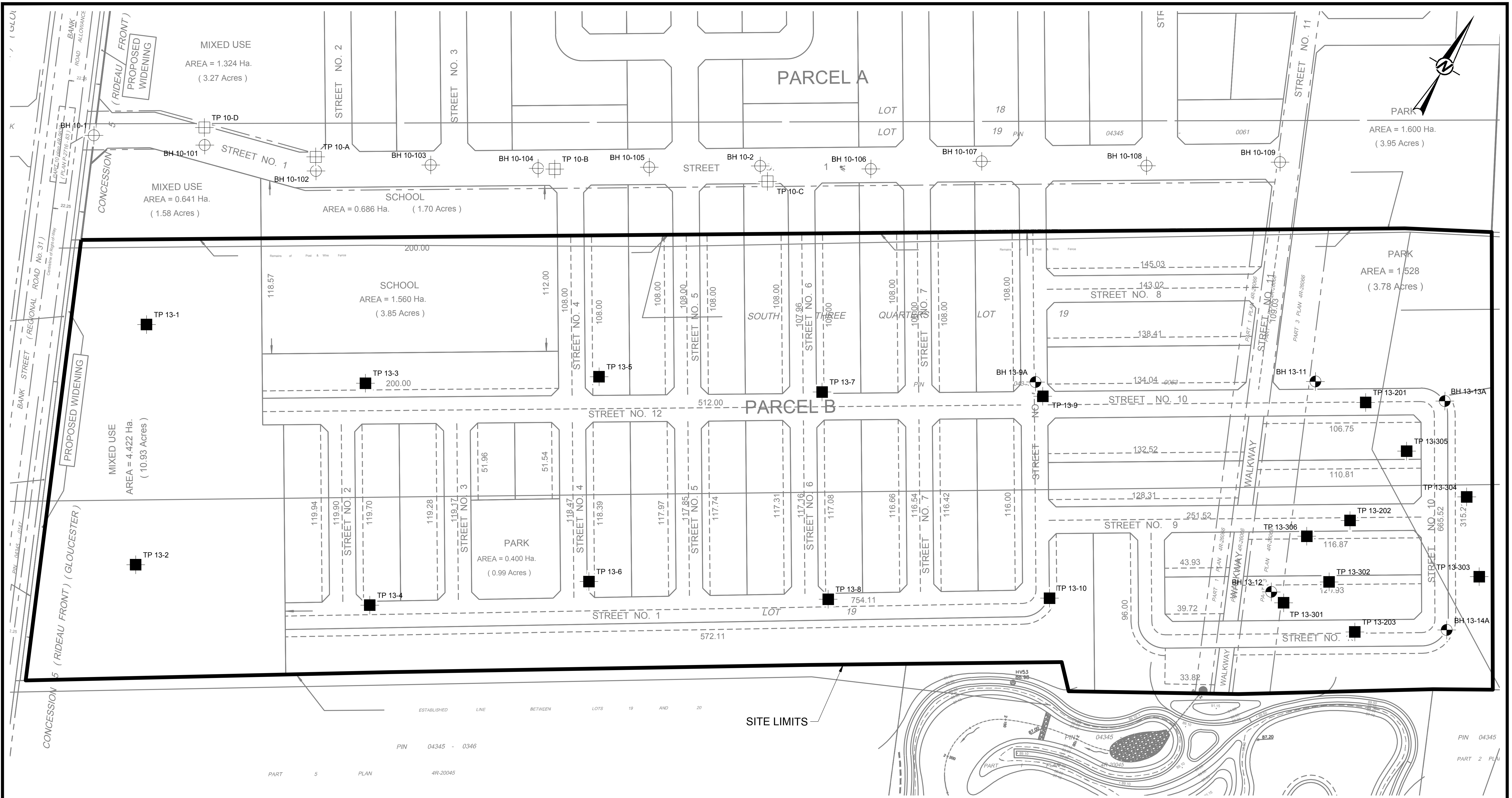
TITLE

KEY PLAN

Golder Associates

PROJECT No.		12-1121-0286	FILE No.	1211210286-1000-01
DESIGN			SCALE	AS SHOWN
CADD	JM	2013-10-10	FIGURE	
CHECK	MIC	2013-12-19		1
REVIEW	MIC	2013-12-19		

N:\Active\2012\1121 - Geotechnical\12-1121-0286 Urbandale Kellum Lands Ottawa\Spatial IM\CAD\PRODUCTION\Phase 1000\1211210286-1000-01.dwg | Layout: ANSI_A_FIGURE_PORTRAIT | Modified: jmcconnell 12/19/2013 4:14 PM | Plotted: jmcconnell 12/19/2013



LEGEND

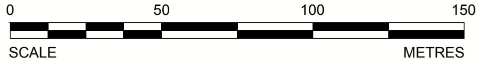
■	APPROXIMATE TEST PIT LOCATION IN PLAN, CURRENT INVESTIGATION
◐	APPROXIMATE BOREHOLE LOCATION IN PLAN, CURRENT INVESTIGATION
⊞	APPROXIMATE TEST PIT LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 10-1121-0014
⊕	APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTGATION BY GOLDER ASSOCIATES LTD., REPORT No. 10-1121-0014

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1121-0286

REFERENCE

BASE PLAN SUPPLIED IN ELECTRONIC FORMAT BY URBANDALE



REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW
△						

PROJECT

GEOTECHNICAL INVESTIGATION
KELLAM LANDS
OTTAWA, ONTARIO

TITLE

SITE PLAN

Golder Associates

PROJECT No.		12-1121-0286	FILE No.	1211210286-1000-02
DESIGN			SCALE	AS SHOWN
CADD	JM	2013-10-22	FIGURE	
CHECK	MIC	2013-12-19		2
REVIEW	MIC	2013-12-19		

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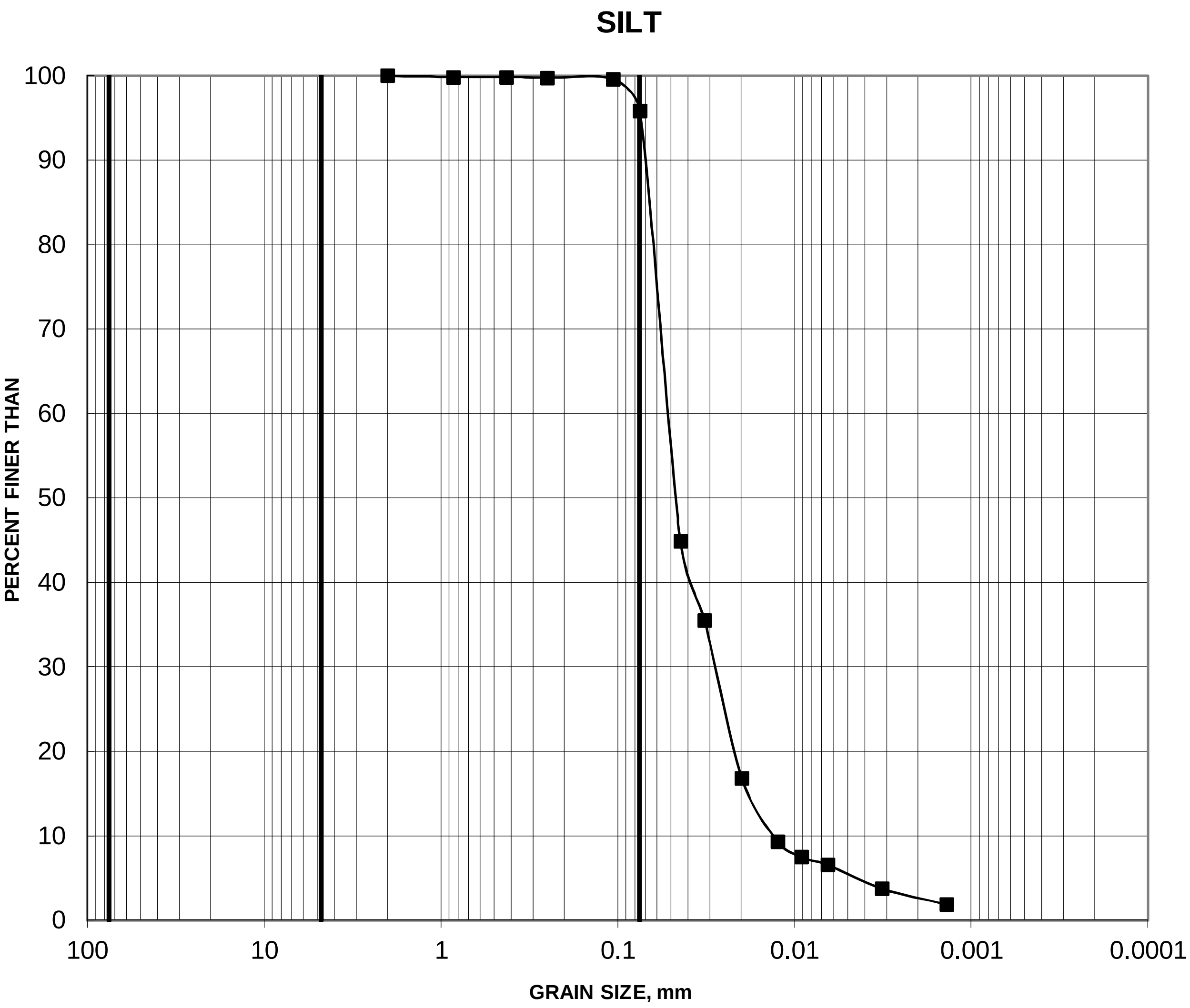
GRAIN SIZE DISTRIBUTION	FIGURE 3



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

Borehole	Sample	Depth (m)
■ 13-12	5	3.81-4.42

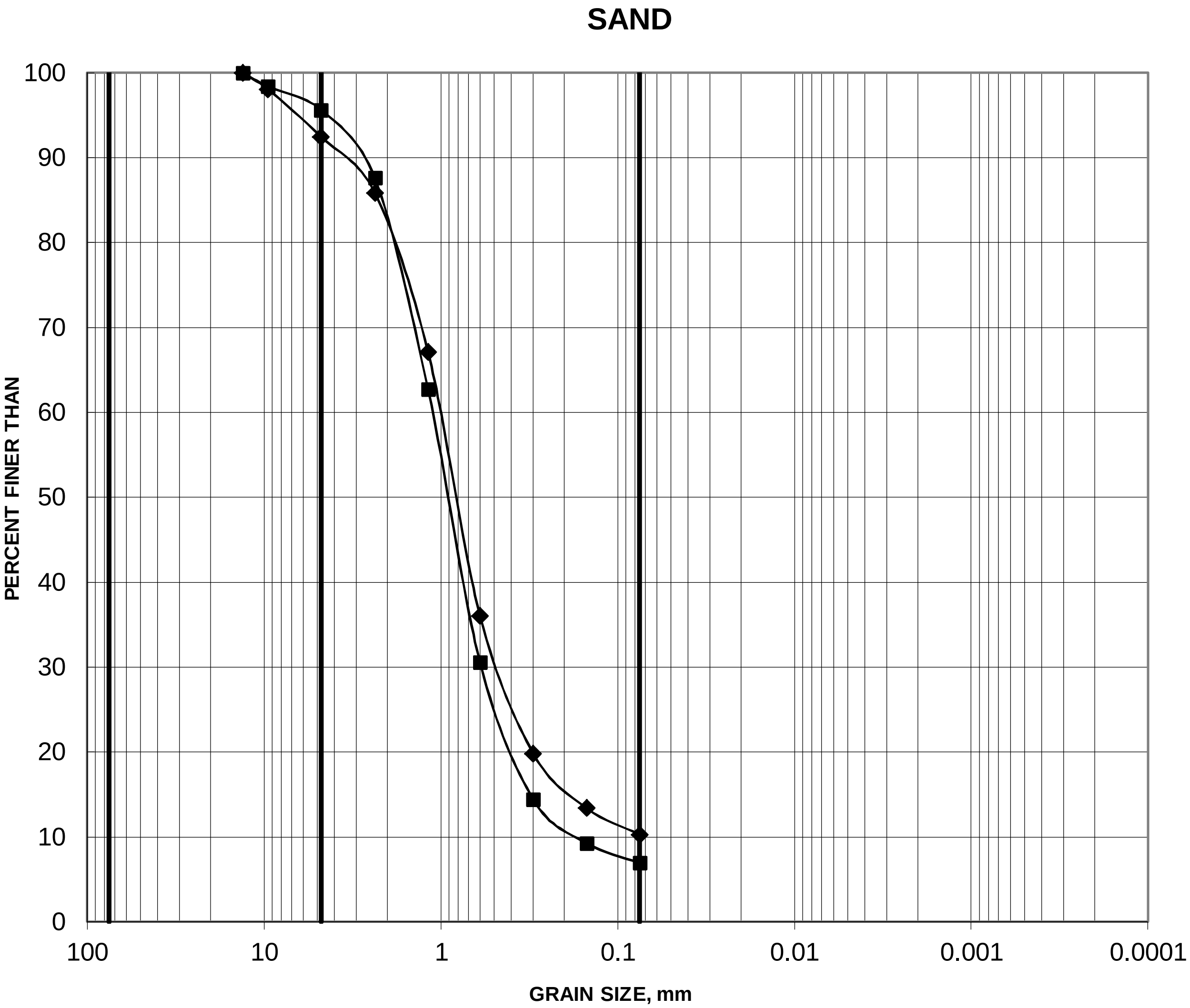
Project: 12-1121-0286

Golder Associates

Created by: CW
Checked by: CNM

GRAIN SIZE DISTRIBUTION

FIGURE 4



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

Borehole	Sample	Depth (m)
■ 13-11	7	5.34-5.95
◆ 13-11	9	6.86-7.47

Created by: CW
Checked by: CNM

Project: 12-1121-0286

Golder Associates

APPENDIX A

List of Abbreviations and Symbols
Record of Borehole Sheets

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures, and in the text of the report are as follows:

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

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 an uncased 50 mm (2 in.) diameter, 60^0 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

Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60^0 conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	**C_u or S_u**	
	kPa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	Water content
w_p or PL	Plastic limited
w_1 or LL	Liquid limit
C	Consolidaiton (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
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	Field vane test (LV-laboratory vane test)
γ	Unit weight

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

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
FOS	factor of safety
V	volume
W	weight

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 vertical effective overburden stress
$\sigma_1\sigma_2\sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3) / 3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_1 or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity Index = $(w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_c	consistency index = $(w_1 - w) / I_p$
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 (overconsolidated range)
C_s	swelling index
C_α	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	overconsolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

Notes:

[1] $\tau = c' + \sigma' \tan \phi'$

[2] shear strength = (compressive strength) / 2

PROJECT: 12-1121-0286

RECORD OF BOREHOLE: BH 13-9A

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct 16, 2013

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 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m				
		GROUND SURFACE		90.38							
0 1	Rotary Drill 200mm Diam (Hollow Stem)	Brown sand and gravel (FILL)		0.00							▽
2		Loose grey brown SANDY SILT		88.96 1.42	1 2	50 DO 50 DO	6 4				Native Backfill
3 4		Very loose to loose grey fine SAND, trace to some silt		87.64 2.74	3 4	50 DO 50 DO	WH 8				
5		Compact to dense grey SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)		86.11 4.27	5 6	50 DO 50 DO	31 22				Bentonite Seal Granular Filter Standpipe Cave
6 7 8 9 10		End of Borehole		84.44 5.94							WL in Standpipe at Elev. 89.73 m on Oct. 23, 2013

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: MIC

PROJECT: 12-1121-0286

RECORD OF BOREHOLE: BH 13-11

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct 16, 17, 2013

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES: 0 – 10

BORING METHOD: Rotary Drill, 200mm Diam (Hollow Stem)

SOIL PROFILE

DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	91.33
Brown silty fine sand (FILL)	0.00
Grey fine sand, some gravel (FILL)	90.64 0.69
Loose to compact brown SILTY fine SAND	90.26 1.07
Compact grey SILT, trace sand	88.28 3.05
Compact grey medium to coarse SAND, trace gravel, trace silt	86.76 4.57
End of Borehole	83.10 8.23

SAMPLES

NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	50 DO	15	
2	50 DO	4	
3	50 DO	5	
4	50 DO	27	
5	50 DO	10	
6	50 DO	10	
7	50 DO	19	M
8	50 DO	16	
9	50 DO	18	M
10	50 DO	14	

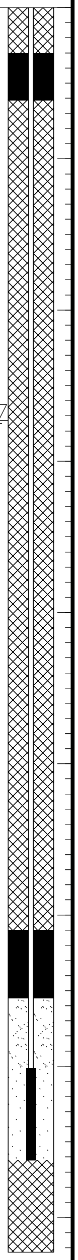
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ ; 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢ ○W ⊣ Wl ; 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION



Native Backfill

Bentonite Seal

Native Backfill

Bentonite Seal

Granular Filter

Standpipe

Cave

WL in Standpipe at Elev. 88.60 m on Oct. 23, 2013

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: MIC

PROJECT: 12-1121-0286

RECORD OF BOREHOLE: BH 13-12

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct 17, 2013

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl : 10 20 30 40

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	91.86					
Power Auger 200mm Diam (Hollow Stem)	Compact brown silty fine sand (FILL)	0.00	1	50 DO	29		
			2	50 DO	13		
	Loose grey SANDY SILT with organic matter	89.57 2.29	3	50 DO	6		
	Compact brown SILT, trace sand	88.96 2.90	4	50 DO	11		
	Loose grey SILT, trace sand	88.05 3.81	5	50 DO	9	MH	
	Firm to stiff grey CLAYEY SILT, some sand	87.29 4.57	6	50 DO	2		▽
			7	50 DO	3		
			8	50 DO	4		
	Compact grey SILTY SAND, trace gravel (GLACIAL TILL)	84.85 7.01	9	50 DO	12		
	End of Borehole	84.24 7.62					WL in open hole at Elev. 86.98 m upon completion of drilling

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: MIC

PROJECT: 12-1121-0286

RECORD OF BOREHOLE: BH 13-13A

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct 17, 2013

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 10 20 30 40

ADDITIONAL LAB. TESTING

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	90.13				
0	Power Auger 200mm Diam (Hollow Stem)	Wood chips (FILL)	0.00				
		Loose brown SILTY fine SAND	0.15				
1				1	50 DO	8	
		Compact grey brown SANDY SILT	88.76 1.37				
2				2	50 DO	20	
		Very loose to compact grey SILT, trace very fine sand, trace clay	87.84 2.29				
				3	50 DO	11	
3				4	50 DO	7	
4				5	50 DO	WH	∇
5				6	50 DO	4	
6				7	50 DO	6	
		End of Borehole	84.03 6.10				
7							WL in open hole at Elev. 86.17 m upon completion of drilling
8							
9							
10							

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: MIC

PROJECT: 12-1121-0286

RECORD OF BOREHOLE: BH 13-14A

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Oct 17, 2013

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

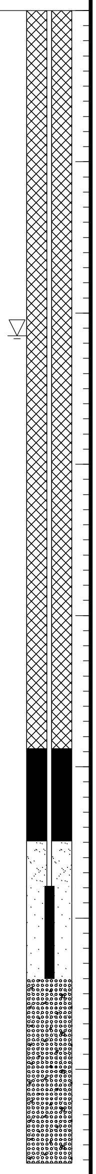
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE: DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m) | SAMPLES: NUMBER, TYPE, BLOWS/0.30m | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80 | SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○: 20 40 60 80 | HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3} | WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl: 10 20 30 40 | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

BORING METHOD: Power Auger, 200mm Diam (Hollow Stem)

DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	91.66
Compact grey brown silty fine sand, trace gravel (FILL)	0.00
Loose to compact brown SILT, some sand, trace clay	89.37 2.29
Very loose grey SILT, trace very fine sand, trace clay	85.87 5.79
End of Borehole	84.04 7.62

NUMBER	TYPE	BLOWS/0.30m
1	50 DO	13
2	50 DO	10
3	50 DO	15
4	50 DO	10
5	50 DO	6
6	50 DO	9
7	50 DO	1
8	50 DO	WH
9	50 DO	WH



WL in Standpipe at Elev. 89.51 m on Oct. 23, 2013

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: MIC

APPENDIX B

Results of Basic Chemical Analysis
Exova Laboratories Report Number 1323486

Certificate of Analysis

Client: Golder Associates Ltd. (Ottawa)
32 Steacie Drive
Kanata, ON
K2K 2A9
Attention: Mr. Mike Cunningham
PO#:
Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1323486
Date Submitted: 2013-10-23
Date Reported: 2013-10-30
Project: 12-1121-0286
COC #: 778651

				Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1067348 Soil 2013-10-13 TP-D 13-4 Sample 2	1067349 Soil 2013-10-17 BH 13-14A Sample 6
Group	**Analyte**	**MRL**	**Units**	**Guideline**		
Agri. - Soil	Electrical Conductivity	0.05	mS/cm		0.42	0.09
	pH	2.0			7.4	8.2
General Chemistry	Cl	0.002	%		<0.002	<0.002
	Resistivity	1	ohm-cm		2380	11100
	SO4	0.01	%		0.05	<0.01

Guideline = *** = Guideline Exceedence**
** = Analysis completed at 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

APPENDIX C

Borehole and Test Pit Records
Selected Laboratory Test Results
Previous Investigation by Golder Associates
Report 10-1121-0014

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-1

SHEET 1 OF 3

LOCATION: See Site Plan

BORING DATE: Jan. 26 & Sept. 16-17, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

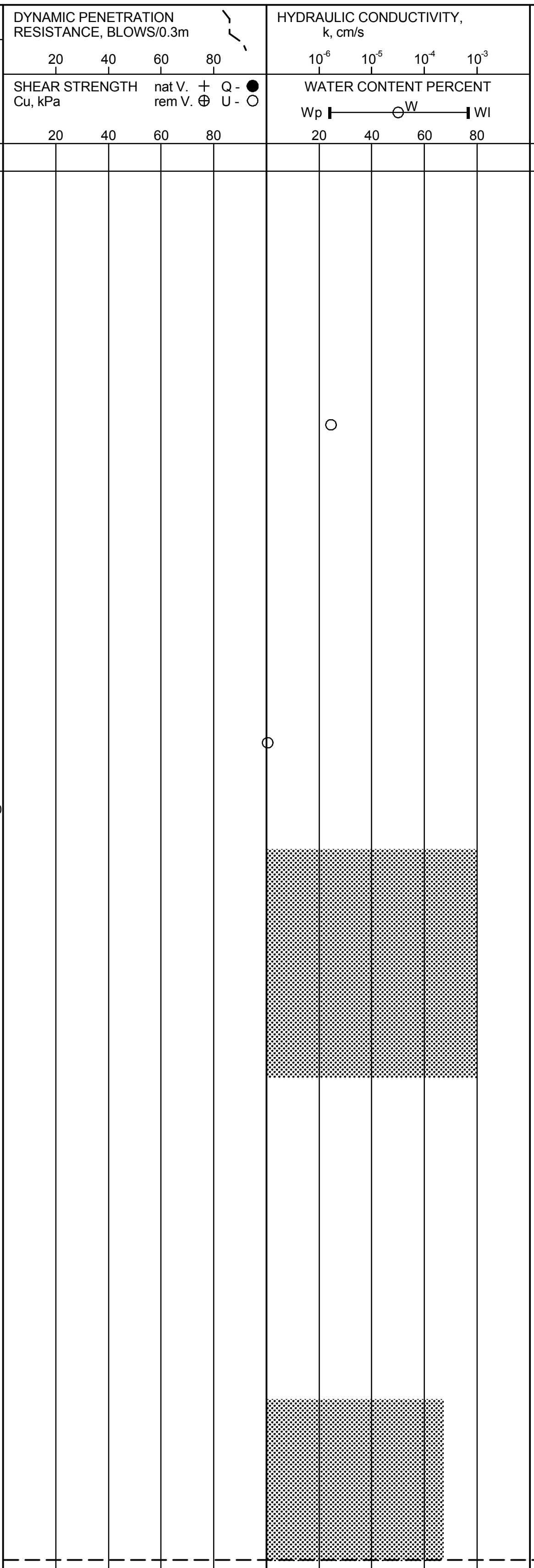
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

SOIL PROFILE

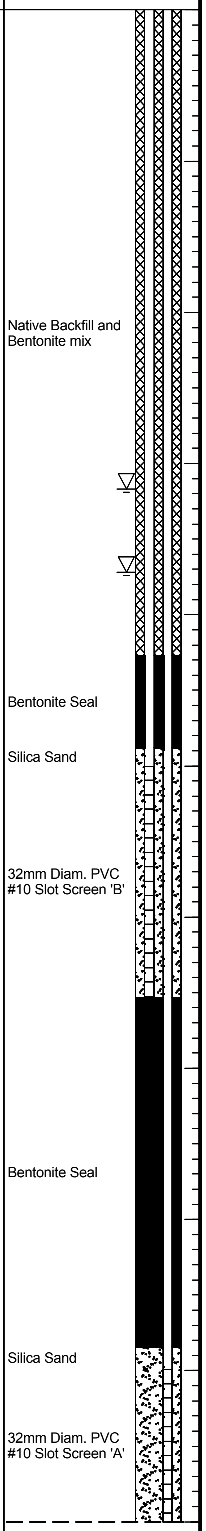
BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)
	GROUND SURFACE	93.66
Power Auger 200mm Diam. (Hollow Stem)	Black sandy silt, with organic matter (TOPSOIL)	0.00
	Loose to compact brown SILT, some sand, trace clay	93.30 0.36
	Compact to dense grey SANDY SILT to SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)	91.37 2.29
Rotary Drill NQ Core	Fresh thinly bedded grey DOLOMITIC LIMESTONE BEDROCK, with black shale interbeds	89.04 4.62

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	50 DO	8	
2	50 DO	10	MH
3	50 DO	49	
4	50 DO	25	
5	50 DO	16	MH
6	50 DO	>100	
C1	NQ RC	DD	
C2	NQ RC	DD	
C3	NQ RC	DD	
C4	NQ RC	DD	



PIEZOMETER OR STANDPIPE INSTALLATION



DEPTH SCALE METRES: 0 – 10

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-1

SHEET 2 OF 3

LOCATION: See Site Plan

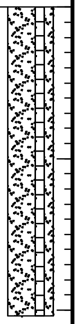
BORING DATE: Jan. 26 & Sept. 16-17, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		— CONTINUED FROM PREVIOUS PAGE —									
10	Rotary Drill NQ Core	Fresh thinly bedded grey DOLOMITIC LIMESTONE BEDROCK, with black shale interbeds			C4	NQ RC	DD				
11					C5	NQ RC	DD				32mm Diam. PVC #10 Slot Screen 'A'
12		End of Borehole Note: Probable void or mud seam encountered between 8.1m and 8.7m depth.		81.62 12.04							W.L. in screen 'A' at Elev. 90.49m on Sept. 28, 2010
13											W.L. in screen 'B' at Elev. 89.94m on Feb. 16, 2010
14											
15											
16											
17											
18											
19											
20											



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF DRILLHOLE: 10-1

SHEET 3 OF 3

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Jan. 26 & Sept. 16-17, 2010

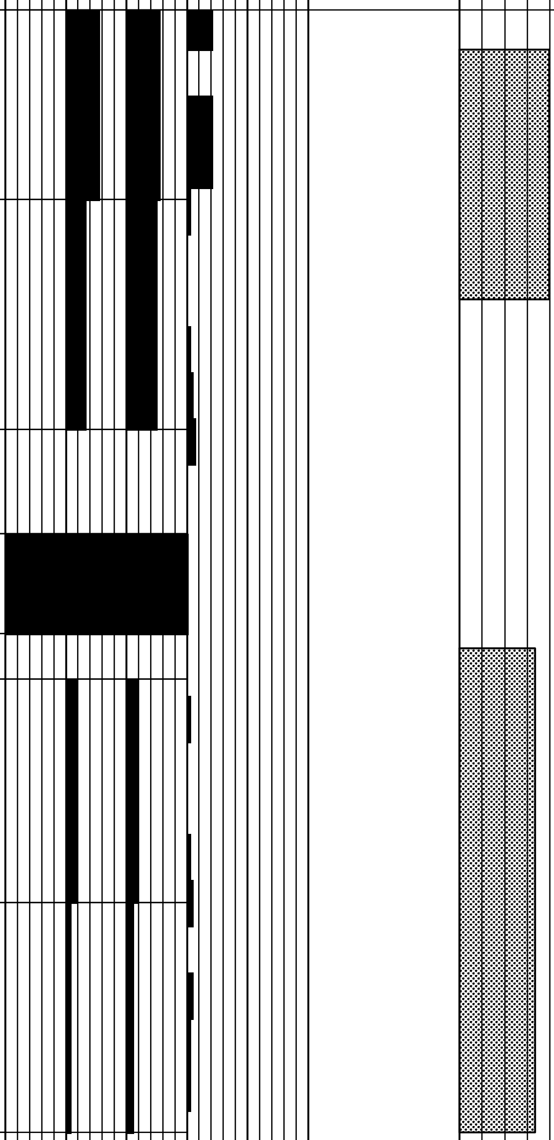
DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.3 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	DIAMETRAL POINT LOAD INDEX (MPa) 2 4 6	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		89.04												
	Rotary Drill NQ Core	Fresh thinly bedded grey DOLOMITIC LIMESTONE BEDROCK, with black shale interbeds		4.62	1											Bentonite Seal
5																Silica Sand
																32mm Diam. PVC #10 Slot Screen 'B'
6					2											
7																
					3A											Bentonite Seal
8					3B											
					3C											Silica Sand
9					4											
10																32mm Diam. PVC #10 Slot Screen 'A'
11					5											
12				81.62 12.04												
		End of Borehole Note: Probable void or mud seam encountered between 8.1m and 8.7m depth.														W.L. in screen 'A' at Elev. 90.49m on Sept. 28, 2010
13																W.L. in screen 'B' at Elev. 89.94m on Feb. 16, 2010
14																



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-2

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Jan. 26-27, 2010

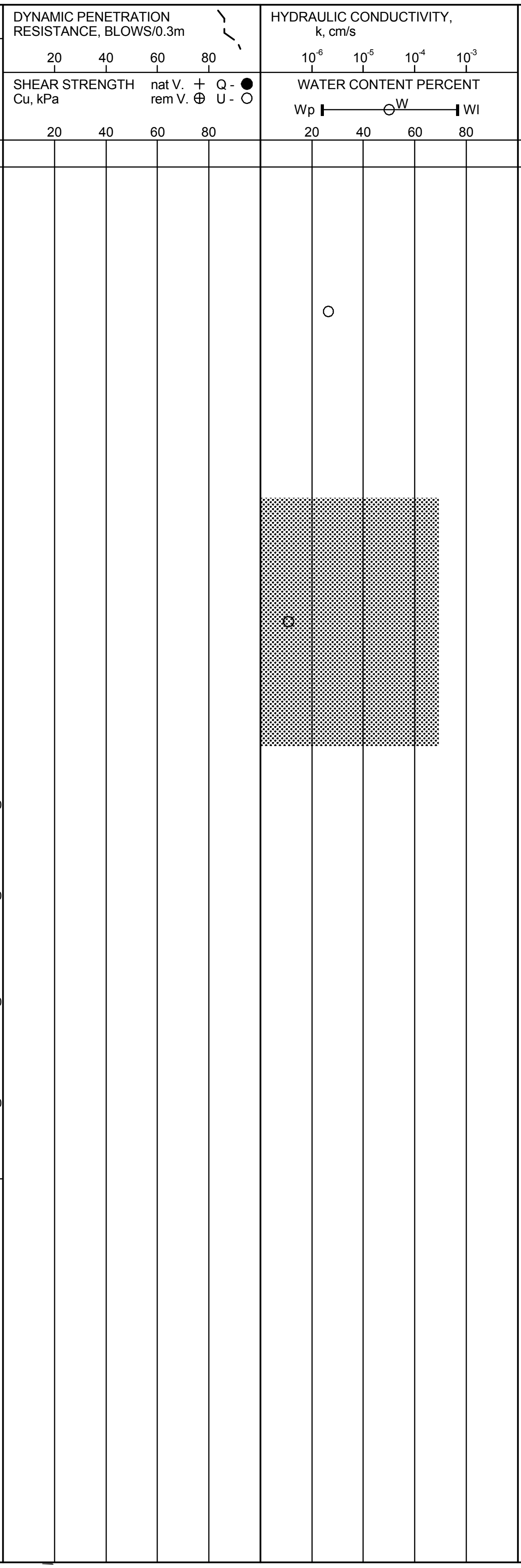
DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

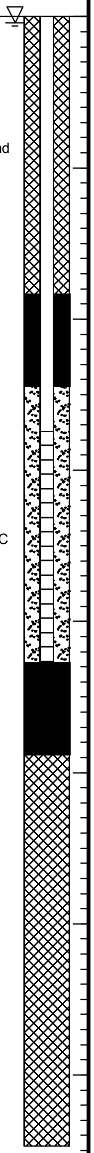
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger 200mm Diam. (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
GROUND SURFACE	90.45				
Black sandy silt, with organic matter (TOPSOIL)	0.00				
Loose grey brown SILT, some sand, trace clay	90.15 0.30	1	50 DO	4	
Compact brown fine SAND, trace silt	88.93 1.52	2	50 DO	15	
Compact grey fine SAND, some silt	88.16 2.29	3	50 DO	15	
Compact to very dense grey SILTY SAND, some gravel and shale fragments, trace clay, with cobbles and boulders (GLACIAL TILL)	87.40 3.05	4 5 6	50 DO 50 DO 50 DO	13 18 >100	MH
Highly weathered to weathered black SHALE BEDROCK	85.60 4.85	7 8 9	50 DO 50 DO 50 DO	>100 >100 >100	
End of Borehole	82.98 7.47				



PIEZOMETER OR STANDPIPE INSTALLATION



- Native Backfill and Bentonite mix
- Bentonite Seal
- Silica Sand
- 32mm Diam. PVC #10 Slot Screen
- Bentonite Seal
- Caved Material

W.L. in screen at Elev. 90.41m on Mar. 29, 2010

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-101

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Sept. 17, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		94.24							
0	Power Auger 200mm Diam. (Hollow Stem)	Black sandy silt, with organic matter (TOPSOIL)		0.00							
		Brown SANDY SILT		93.96 0.28	1	GRAB					
1											
		SILTY SAND, with cobbles and boulders (GLACIAL TILL)		92.79 1.45							
2											
		End of Borehole Auger Refusal Note: Soil stratigraphy inferred from limited sampling		91.75 2.49							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-102

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Sept. 17, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		94.30							
0	Power Auger 200mm Diam. (Hollow Stem)	Dark brown sandy silt, with organic matter (TOPSOIL)		0.00							
		Dense brown SANDY SILT, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)		94.00 0.30	1	GRAB					
1											
2					2	50 DO	33				
		End of Borehole Auger Refusal		91.73 2.57							
3		Note: Soil stratigraphy inferred from limited sampling									Borehole dry upon completion of drilling
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-103

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Sept. 17, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m
		GROUND SURFACE	93.49			
0	Power Auger 200mm Diam. (Hollow Stem)	Black sandy silt, with organic matter (TOPSOIL)	0.00			
		Compact to very dense SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)	93.11 0.38			
1				1	50 DO	13
				2	50 DO	>50
2		Highly weathered to weathered black SHALE BEDROCK	91.36 2.13	3	50 DO	>50
3			90.49	4	50 DO	>50
		End of Borehole Auger Refusal	3.00			
4						
5						
6						
7						
8						
9						
10						

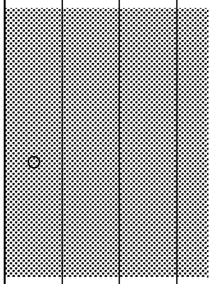
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○: 20 40 60 80

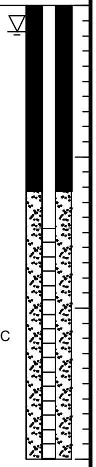
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ├──○W──┤ Wl: 20 40 60 80

ADDITIONAL LAB. TESTING



PIEZOMETER OR STANDPIPE INSTALLATION



Bentonite Seal

Silica Sand

50mm Diam. PVC
#10 Slot Screen

W.L. in screen at
Elev. 93.32m on
Sept. 28, 2010

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-104

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Sept. 20, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		92.04							
0	Power Auger 200mm Diam. (Hollow Stem)	Black sandy silt, with organic matter (TOPSOIL)		0.00 91.74							
		Grey brown CLAYEY SILT		0.30 91.38							▽
1		Loose brown fine to medium SAND, trace silt		0.66 90.36	1	50 DO	3				
		Compact grey fine SAND, trace silt		1.68 89.97	2	50 DO	24				
2		Grey SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)		2.07 88.84							
3		Probable highly weathered to weathered Shale Bedrock		3.20 87.93							
4		End of Borehole Auger Refusal Note: Soil stratigraphy inferred from limited sampling		4.11							W.L. in borehole at 0.6m depth below ground surface upon completion of drilling
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

MIS-BHS 001 1011210014-1000.GPJ GAL-MIS.GDT 12/8/10 JM

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-105

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Sept. 20, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		92.51							
0	Power Auger 200mm Diam. (Hollow Stem)	Black sandy silt, with organic matter (TOPSOIL)		0.00							
1		Compact brown fine to medium SAND, some gravel, trace silt		92.15 0.36	1	50 DO	24				
2 3 4		Brown to dark grey SANDY SILT, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)		90.99 1.52	2	50 DO	28				∇
5		Probable highly weathered to weathered Shale Bedrock		87.79 4.72							
6 7 8 9 10		End of Borehole Auger Refusal Note: Soil stratigraphy inferred from limited sampling		87.18 5.33							W.L. in borehole at 3.1m depth below ground surface upon completion of drilling

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-106

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Sept. 20, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		90.13							
0	Power Auger 200mm Diam. (Hollow Stem)	Black sandy silt, with organic matter (TOPSOIL)		0.00							
		Stiff grey brown CLAYEY SILT, trace sand		89.77 0.36	1	50 DO	2				
		Compact grey SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)		87.54 2.59	2	50 DO	21				▽
5		End of Borehole Auger Refusal Note: Soil stratigraphy inferred from limited sampling		85.18 4.95							W.L. in borehole at 2.6m depth below ground surface upon completion of drilling

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

MIS-BHS 001 1011210014-1000.GPJ GAL-MIS.GDT 12/8/10 JM

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-107

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Sept. 20, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	89.62				
0	Power Auger 200mm Diam. (Hollow Stem)	Black silty clay, with organic matter (TOPSOIL)	0.00				
		Brown to grey brown CLAYEY SILT, trace sand	89.26 0.36	1	GRAB		▽
2				2	50 DO	5	
		Stiff grey CLAYEY SILT	87.33 2.29	3	50 DO	3	
3		Grey SILTY SAND, with cobbles and boulders (GLACIAL TILL)	86.75 2.87				
5		Probable highly weathered to weathered Shale Bedrock	84.82 4.80				
		End of Borehole Auger Refusal Note: Soil stratigraphy inferred from limited sampling	4.93				W.L. in borehole at 0.6m depth below ground surface upon completion of drilling

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-108

SHEET 1 OF 1

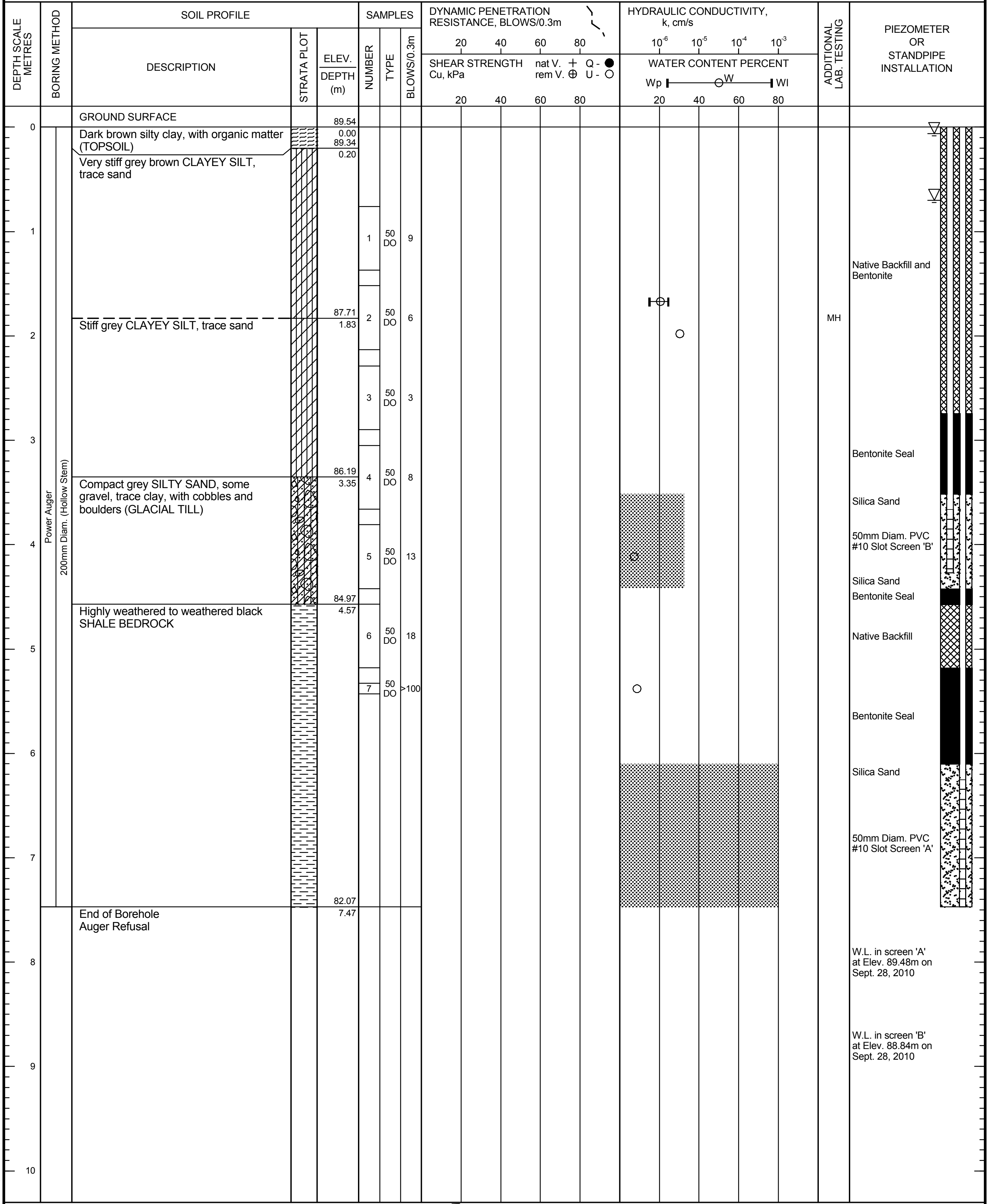
LOCATION: See Site Plan

BORING DATE: Sept. 21-22, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm



BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	89.54					
Power Auger 200mm Diam. (Hollow Stem)	Dark brown silty clay, with organic matter (TOPSOIL)	0.00					
	Very stiff grey brown CLAYEY SILT, trace sand	89.34 0.20	1	50 DO	9		Native Backfill and Bentonite
	Stiff grey CLAYEY SILT, trace sand	87.71 1.83	2	50 DO	6	MH	
			3	50 DO	3		Bentonite Seal
	Compact grey SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)	86.19 3.35	4	50 DO	8		Silica Sand
			5	50 DO	13		50mm Diam. PVC #10 Slot Screen 'B' Silica Sand Bentonite Seal
	Highly weathered to weathered black SHALE BEDROCK	84.97 4.57	6	50 DO	18		Native Backfill
			7	50 DO	>100		Bentonite Seal Silica Sand 50mm Diam. PVC #10 Slot Screen 'A'
	End of Borehole Auger Refusal	82.07 7.47					

W.L. in screen 'A' at Elev. 89.48m on Sept. 28, 2010

W.L. in screen 'B' at Elev. 88.84m on Sept. 28, 2010

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 10-1121-0014

RECORD OF BOREHOLE: 10-109

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Sept. 23, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		89.60							
0	Power Auger 200mm Diam. (Hollow Stem)	Black sandy silt, with organic matter (TOPSOIL)		0.00							
		Very stiff to stiff grey brown to grey CLAYEY SILT, occasional sand seam, trace gravel		89.37 0.23	1	GRAB					
1											
2					2	50 DO	5				
3		SILTY SAND, with cobbles and boulders (GLACIAL TILL)		86.45 3.15							
4											
5											
6		End of Borehole Auger Refusal Note: Soil stratigraphy inferred from limited sampling		83.76 5.84							
7											
8											
9											
10											

DEPTH SCALE

1 : 50

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LOGGED: D.G.

CHECKED: ________

MIS-BHS 001 1011210014-1000.GPJ GAL-MIS.GDT 12/8/10 JM

TABLE 1

RECORD OF TEST PITS

Test Pit Number (Elevation)	Depth (metres)	Description
10-A	0.00 – 0.30	TOPSOIL
	0.30 – 2.20	Brown SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	2.20	Refusal on grey DOLOMITIC LIMESTONE BEDROCK
		Note 1: Test pit excavated just north of borehole 10-102. Note 2: Boulders and cobbles were encountered within the glacial till (maximum boulder dimension: about 0.7 metres x 1.5 metres). Note 3: Water seepage at about 1.7 metres depth. Note 4: A test pit was also excavated just to the south of borehole 10-102. Refusal encountered at about 1.6 metres depth on a probable large boulder.
10-B	0.00 – 0.30	TOPSOIL
	0.30 – 0.55	Brown CLAYEY SILT, some sand
	0.55 – 2.20	Brown fine to coarse SAND, trace to some silt
	2.20 – 2.70	Grey SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	2.70	Refusal on black weathered SHALE BEDROCK
		Note 1: Test pit excavated just east of borehole 10-104. Note 2: Water seepage at about 1.0 metres depth.

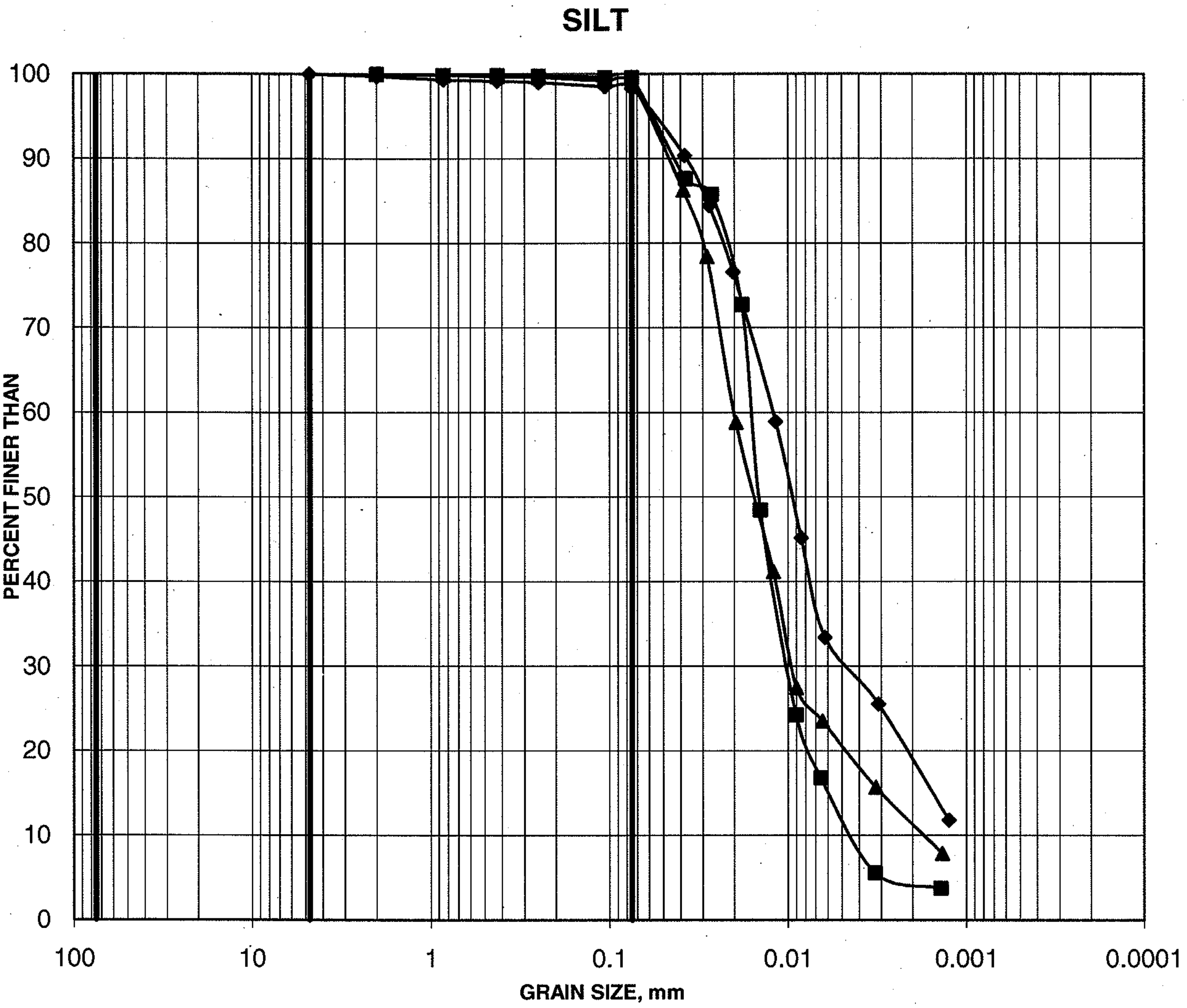
TABLE 1

RECORD OF TEST PITS

Test Pit Number (Elevation)	Depth (metres)	Description
10-C	0.00 – 0.05	TOPSOIL
	0.05 – 0.50	Brown SILT, trace to some sand
	0.50 – 3.20	Grey brown medium to coarse SAND, with gravel and cobbles
	3.20 – 4.50	Grey SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	4.50	Refusal on probable weathered SHALE BEDROCK
		Note 1: Test pit excavated just south of borehole 10-2. Note 2: Unable to excavate further due to water inflow and considerable sloughing of excavation side walls. Note 3: Water inflow at about 3.8 metres depth.
10-D	0.00 – 0.30	TOPSOIL
	0.30 – 1.70	Brown SILTY SAND
	1.70 – 1.90	Brown SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)
	1.90	Refusal on grey DOLOMITIC LIMESTONE BEDROCK
		Note 1: Test pit excavated just north of borehole 10-101. Note 2: Test pit dry upon completion

GRAIN SIZE DISTRIBUTION

FIGURE 3



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

Borehole	Sample	Depth (m)
■ 10-1	2	1.52-2.13
▲ 10-3	4	3.05-3.66
◆ 10-108	2B	1.83-2.13

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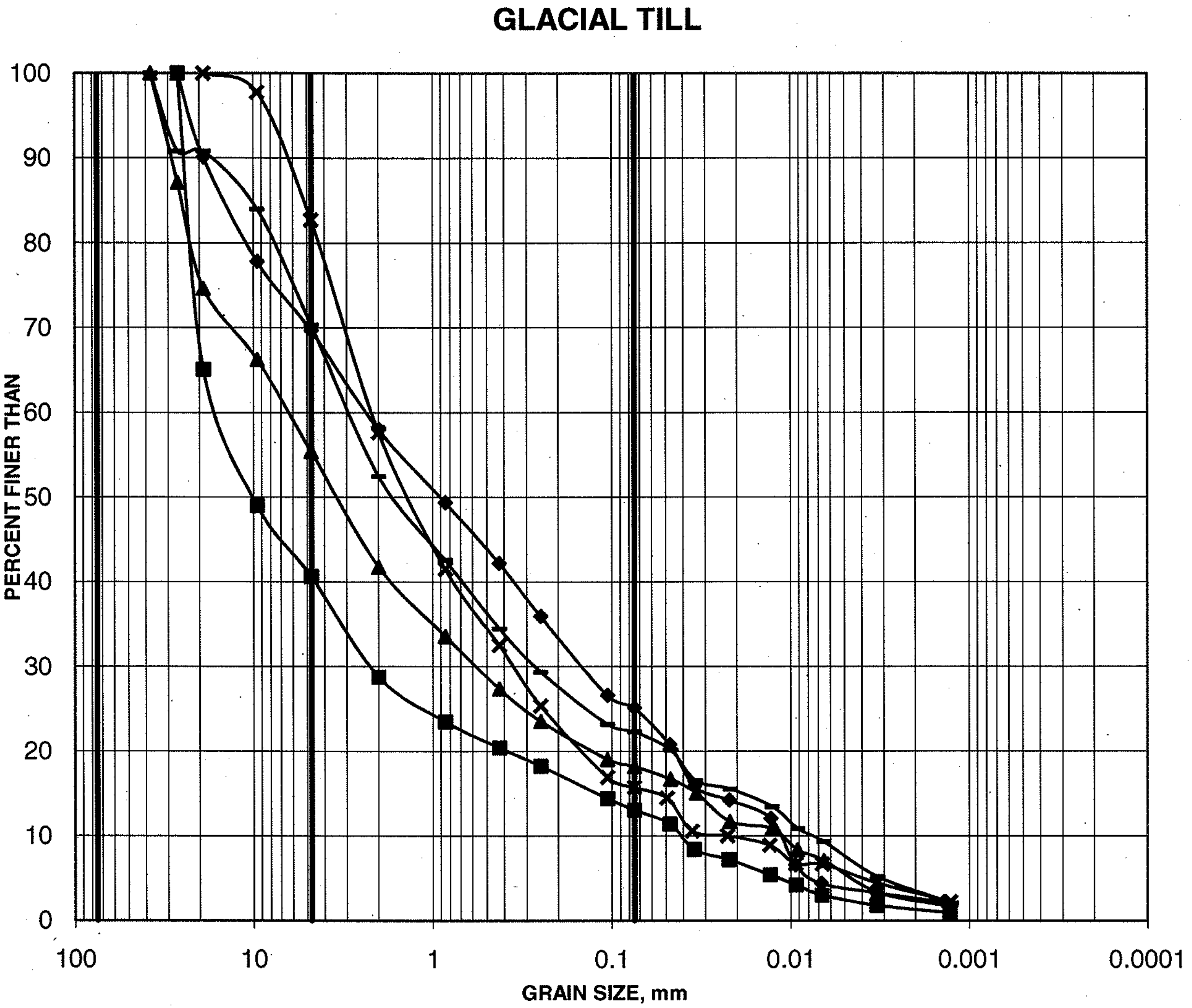
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE 4



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

Borehole	Sample	Depth (m)
■ 10-1	5	3.81-4.42
× 10-2	4	3.05-3.66
▲ 10-3	7	5.34-5.95
— 10-4	4	3.05-3.66
◆ 10-113	2	1.52-2.13

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Created by: MI
Checked by: CNM

Golder Associates Ltd.
32 Steacie Drive
Kanata, Ontario
K2K 2A9

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UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE

Project: Leitrim Sewer

Project No.: 10-1121-0014

Date: October 19, 2010

Location(s):

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m³)	Compressive Strength (MPa)
10-1	9.09-9.19	Oct 19/10	NQ	47.5	2760	166.0

REMARKS : - Compressive Strength Corrected for L/D Ratio.
- Cores tested in vertical direction.

TESTING WAS CARRIED OUT IN GENERAL ACCORDANCE WITH ASTM D7012 - Method C

SIGNED:

C.N.Mangione P.Eng.

SGS

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Golder Associates
Attn : Kim Lesage

32 Steacie Drive, Kanata
Canada, K2K 2A9
Phone: 613 592 9600, Fax:613 592-9601

Thursday, October 21, 2010

Date Rec. : 07 October 2010
LR Report : **CA02364-OCT10**
Client Ref : Ref No 10-1121-0014

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	SiO2 %	Al2O3 %	Fe2O3 %	MgO %	CaO %	Na2O %	K2O %	TiO2 %	P2O5 %	MnO %	Cr2O3 %	V2O5 %	LOI %	Sum %
1: BH10-1 Dolomitic Lomestone	12.8	3.36	1.36	17.0	25.0	0.04	1.59	0.11	0.04	0.06	0.01	< 0.01	38.3	99.6

Control Quality Analysis

Debbie Waldon

Debbie Waldon
Project Coordinator,
Minerals Services, Analytical

OnLine LIMS

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Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Ltd.
32 Steacie Drive
Kanata, Ontario, K2K 2A9
Canada
T: +1 (613) 592 9600

Golder Associates