November 2011

REPORT ON

Preliminary Geotechnical Investigation
Proposed High Rise Development
96 Nepean Street
Ottawa, Ontario

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1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation carried out for a proposed high rise development to be located at 96 Nepean Street in Ottawa, Ontario.

The objective of this preliminary investigation was to determine the general soil, bedrock, and groundwater conditions at the site of the proposed development by means nine boreholes and, based on an interpretation of the factual information obtained, to provide preliminary geotechnical engineering guidelines on the design of the development, to identify significant geotechnical design challenges for the project, and to confirm that the proposed development is feasible from a geotechnical perspective.

It is understood that this geotechnical assessment is required as part of the ‘Site Plan Approval’ and “Re-zoning” process.

The guidelines provided in this report are intended solely for the preliminary planning of this development. Further investigation will be required before geotechnical input to the detailed design of this development can be provided.

This geotechnical investigation was carried out in conjunction with a Phase II Environmental Site Assessment, the results of which are reported in a Golder Associates report numbered 11-1121-0202-1000 titled “Phase I and II Environmental Site Assessment, 96 Nepean Street, Ottawa, Ontario” dated November 2011.

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

Consideration is being given to the design and construction of a high rise development to be located at 96 Nepean Street in Ottawa, Ontario (see Key Plan, Figure 1).

The following is known about the existing property:

- The site measures about 30 metres by 40 metres in plan area.
- The site is currently an asphalt surfaced parking lot, with a pay-booth at the northwest corner of the site.
- The site was formerly occupied by residential houses, which where demolished in the 1970’s.
- The site is bordered to the north by Nepean Street, to the west by an asphalt surfaced parking lot, to the south by an underground parking garage and a 10 storey building, and to the east by a three storey brick building.

Although preliminary in nature, current plans indicate that:

- The proposed development will occupy essentially the entire site.
- The proposed structure will be 27 storeys in height.
- The proposed structure will have 6 below grade parking levels.

Golder Associates has carried out several previous subsurface investigations within the area of the site. Based on the results of those previous investigations, the subsurface conditions on this site are expected to consist 4 to 5 metres of silty clay overlying glacial till, with the surface of the shale bedrock being at about 8 to 10 metres depth.

Published geologic maps indicate that bedrock in the vicinity of the site consists of shale of the Billings formation.

3.0 PROCEDURE

The field work for this investigation was carried out between September 7 and 25, 2011. At that time, nine boreholes (numbered 11-1 to 11-9, inclusive) were put down at the approximate locations shown on Figure 2. The boreholes were advanced using a truck-mounted hollow-stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario.

All of the boreholes were advanced to either auger or sampler refusal, which were encountered at depths ranging from approximately 10.0 to 11.8 metres below the existing ground surface. Within the boreholes, standard penetration tests were carried out and samples of the soils (and weathered bedrock) encountered were recovered using drive open sampling equipment. In situ vane testing was carried out in the silty clay in some of the boreholes to determine the undrained shear strength of this soil unit.

Upon encountering practical refusal to augering, borehole 11-2 was extended about 1.5 metres into bedrock using rotary diamond drilling equipment while retrieving NQ sized bedrock core, terminating at a depth of about 13.3 metres below the existing ground surface.

Monitoring wells were sealed into all of the boreholes to allow subsequent measurement of the groundwater levels and to allow for groundwater sampling (for the Phase II ESA).

The field work was supervised by experienced personnel from our staff who located the boreholes, directed the drilling operations, logged the boreholes and samples, took custody of the soil and bedrock samples retrieved, and directed the in situ testing. Samples of the soil and bedrock encountered within the boreholes were returned to our laboratory for examination by the project engineer.

The borehole locations were selected by Golder Associates and located in the field relative to existing site features. The ground surface elevation at the borehole locations was also determined by Golder Associates and was referenced to the top of the fire hydrant base located at the northeast corner of the site. This point was assigned a local datum elevation of 100.00 metres.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the boreholes are shown on the Record of Borehole Sheets in Appendix A.

In general, the subsurface conditions on this site consist of up to about 2.5 metres of fill material, overlying about 4 to 6 metres of sensitive silty clay, overlying glacial till, with the underlying shale bedrock surface varying from about 9 to 11 metres depth.

4.2 Fill Materials

Fill materials exist at the ground surface in all of the boreholes and are up to about 2.4 metres thick (this thickness was inferred in several of the boreholes). The fill material consists of asphaltic concrete overlying sand with variable amounts of gravel, silt, and brick.

4.3 Sensitive Silty Clay

The fill materials are underlain by a deposit of silty clay. The upper 0.7 to 1.5 metres of the silty clay have been weathered to grey brown crust. Standard penetration tests carried out within the weathered crust generally gave N values varying from 2 to 6 blows per 0.3 metres of penetration, indicating the weathered crust to have a stiff to very stiff consistency.

The silty clay below the depth of weathering is grey in colour. The grey silty clay was fully penetrated in all of the boreholes and extends to depths of about 5.6 to 7.9 metres below the existing ground surface. The results of in situ vane testing in the grey silty clay gave undrained shear strengths ranging from approximately 54 to 92 kilopascals, indicating a stiff consistency.

4.4 Glacial Till

The silty clay is underlain by a deposit of glacial till. The glacial till was fully penetrated in all of the boreholes and varies from approximately 1.1 to 4.5 metres in thickness (extending down to about 8.8 to 11.2 metres depth). The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand with a trace of clay and shale fragments.

Standard penetration tests carried out within the glacial till gave ‘N’ values ranging from 2 to greater than 50 blows per 0.3 metres of penetration, indicating a loose to very dense state of packing. However the higher ‘N’ values likely reflect the presence of cobbles and boulders, rather than the actual state of packing of the soil matrix. The deposit would more typically be considered to be compact.

4.5 Bedrock and Refusal

Practical refusal to augering or sampler advancement was encountered in all of the boreholes at depths varying from about 10.0 to 11.4 metres below the ground surface.

In most of the boreholes, the upper portion of the bedrock is highly weathered and the boreholes were advanced into the bedrock by up to an additional 0.5 to 2.4 metres before encountering practical refusal to augering or sampler advancement.

The depths to the bedrock surface are shown in the following table.

Borehole	Bedrock Depth at Borehole (m)
11-1	10.0^R
11-2	11.2
11-3	9.9
11-4	11.1^R
11-5	8.8
11-6	9.7
11-7	10.0
11-8	9.7
11-9	9.0^R

Note**:** R – Refusal to augering or sampler advancement.

The bedrock was cored in borehole 11-2 and consists of laminated to thinly bedded black shale. Published geological mapping indicates that this shale bedrock is of the Billings Formation.

4.6 Groundwater

Monitoring wells were installed in all of the boreholes. The results of the groundwater level measurements are provided in the following table.

Borehole Number	Geological Unit	Date of Measurement	Water Level Depth (m)
11-1	Glacial Till	September 23, 2011	8.1
11-2	Silty Clay	September 26, 2011	Dry
11-2	Glacial Till	September 26, 2011	8.3
11-3	Glacial Till	September 23, 2011	8.2
11-4	Glacial Till	September 26, 2011	8.1
11-5	Glacial Till	September 26, 2011	8.2
11-6	Glacial Till	September 26, 2011	Dry
11-7	Glacial Till	September 26, 2011	8.1
11-8	Glacial Till	September 26, 2011	8.3
11-9	Glacial Till / Bedrock	September 26, 2011	8.4

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

5.0 PROPOSED HIGH RISE DEVELOPMENT

5.1 General

This section of the report provides preliminary engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the borehole information and project requirements. It should be emphasized that the scope of this investigation is appropriate for preliminary design and site planning only. Additional investigation will be required at the detailed design stage before these guidelines can be confirmed.

5.2 Excavations

Preliminary plans indicate that the structure will have 6 basement levels which will extend to about 19.5 metres depth.

Considering that the excavation will likely need to extend a further 1.0 to 1.5 metres below the lowest basement floor level to accommodate the foundations and elevator pits, it is expected that the excavation will extend to about 21.5 to 22 metres depth.

The excavation will therefore extend through the fill materials, silty clay, glacial till, and into the underlying shale bedrock.

No unusual problems are anticipated with excavating in the overburden materials using conventional hydraulic excavating equipment, recognizing that large debris may be encountered within the fill materials (given that houses formerly occupied the site) and that large boulders should be expected within the glacial till.

The Occupational Health and Safety Act (OHSA) of Ontario indicates that side slopes in the overburden above the water table could be sloped at a minimum of 1 horizontal to 1 vertical (i.e. Type 3 soils). Steeper side slopes would require shoring to meet the requirements of the OHSA. Given the constraints by adjacent properties and roadways, it is expected that shoring of the overburden will be necessary. In general, there are three basic shoring methods that are commonly used in local practice: steel soldier piles and timber lagging, driven interlocking steel sheet piles and, less commonly, continuous concrete (secant pile or diaphragm) walls, each with appropriate lateral support. Additional guidelines on temporary shoring are provided in Section 5.3 of this report.

Some groundwater inflow into the excavation should be expected. However, considering the relatively low hydraulic conductivity of the silty clay and glacial till, it is expected that it should be possible to handle the groundwater inflow from these deposits by pumping from well filtered sumps in the floor of the excavation, using suitably sized pumps.

Based on previous experience with excavations within the Billings shale, groundwater inflows to excavations that extend into the bedrock should similarly be handled by pumping from within the excavation.

Bedrock removal will be required for basement and foundation construction. For shallow depths of excavation, it may be possible to remove the upper weathered portion of the bedrock, to about 0.5 to 1.0 metres depth (at least locally), using large hydraulic excavating equipment. Further bedrock removal could be accomplished using mechanical methods (such as hoe ramming), although this method may be slow and tedious. Excavations deep into the rock will require drill and blast procedures.

The upper 0.5 to 2.5 metres of the bedrock are highly weathered and will not likely stand vertically; it should therefore be planned to also shore the weathered zone of bedrock. However, it is considered that near vertical bedrock walls in the un-weathered shale bedrock will be feasible for the construction period. However some shotcreting and/or bolting may be needed. Blast induced damage to the bedrock must be avoided; otherwise (additional) rock reinforcement could be required. It should therefore be planned to either line drill the bedrock along the perimeter of the excavation at a close spacing in advance of blasting so that a clean bedrock face is formed, or to carry out perimeter drilling and pre-shearing of the excavation limits using controlled blasting.

The blasting should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be minimized. This will require blast designs by a specialist in this field. Due to the expected sensitivity of the adjacent buildings, the blasting operation will have to be carefully planned, closely controlled, and monitored throughout.

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

Excavation for the basement levels and foundations will result in exposure of the shale bedrock to air. The shale bedrock at this site has the potential to swell following exposure to oxygen, which could be damaging to the basement floor slab.

For the swelling 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. It is also possible for the products of the above reactions to attack the concrete in the foundations (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, such as by covering the shale with a mud slab of lean concrete.

Where shale is exposed on the sides of the excavation, the exposed shale should be shotcreted so that concrete covers the shale to the top-of-rock level. As previously discussed, this shotcreting will also likely be required to maintain vertical excavation walls within the shale.

Additional measures that would assist in limiting the risk of expansion of the shale bedrock at the subgrade level include:

- Providing a uniform subgrade level for the entire building such that no areas of higher bedrock are left in-place which would be vulnerable to drying.
- Not excavating sumps in the rock and/or pumping from the rock in such a manner as to lower the groundwater level into the rock, even temporarily.

A Permit-To-Take-Water (PTTW) from the Ministry of the Environment will likely need to be obtained for handling of groundwater inflow into the excavation. A PTTW is required if the daily groundwater pumping would exceed 50,000 Litres, which is likely the case. A hydrogeological assessment of the potential impacts of the temporary and permanent groundwater level lowering will need to be carried out; this study will also be required to support a PTTW application.

5.3 Excavation Shoring

It is expected that the excavation will encompass essentially the full limits of the property and therefore vertical (or near vertical) excavation walls will be required. The contractor should be responsible for the detailed design of the shoring. However, this section of the report provides some general guidelines on possible concepts for the shoring, to be used by the designers for assessing the costs of the shoring as well as possible impacts of the shoring design and construction on the design of the superstructure and site works. These guidelines can also be used to evaluate, at the design stage, the potential for impacts of this shoring on the adjacent properties.

The shoring method(s) chosen to support the excavation sides must take into account the soil stratigraphy, the groundwater conditions, the potential ground movements associated with the excavation and construction of the shoring system, and their impact on adjacent structures and utilities. In general, there are three basic shoring methods that are used in the Ottawa area: steel soldier piles and timber lagging, driven steel sheet piles and, less commonly used, continuous concrete (secant pile or diaphragm) walls, each with appropriate lateral support. Soldier piles and lagging is suitable where the objective is to maintain an essentially vertical excavation wall and the movements above and behind the wall need only be sufficiently limited that relatively flexible features (such as roadways) will not be adversely affected. Where foundations lie within the zone of influence of the shoring (such as adjacent to the south and east limits of the site), the shoring deflections need to be greatly limited and interlocking steel sheet piling with pre-stressed tie backs is often used. Secant pileor diaphragm walls would be appropriate where heavily loaded foundations exist beside the shoring, or where groundwater inflow needs to be controlled. Underpinning of the existing foundations could also be required/justified, if the settlements due to shoring movements would be unacceptable and/or if the loads on the adjacent foundations are large.

For all of the above systems, some form of lateral support to the wall is required for excavation depths greater than about 3 or 4 metres, which will be the case for this site. Lateral restraint could be provided by means of tie-backs consisting of grouted bedrock anchors. However, the use of rock anchor tie-backs would require the permission of the adjacent property owners (including the City, who owns the adjacent roadways) since the anchors would be installed beneath their properties. The presence of utilities beneath the adjacent streets or piles beneath the existing buildings which could interfere with the tie-backs should also be considered. Alternatively, interior struts can be considered, connected either to the opposite side of the excavation (if not too distant) or to raker piles and/or footings within the excavation. However internal struts could interfere with the construction of the foundations and superstructure.

For this site, it is envisioned that steel soldier piles and timber lagging shoring would be used along the northern (Nepean Street) and western (parking lot) limits of the site where the excavation will be adjacent to the existing roadway and parking lot. For excavations where existing buildings are present immediately beside (or close to) the excavation, such as the southern and eastern limits of the site, rigid steel sheet pile shoring with prestressed tie-backs will likely be needed. However, even with proper shoring design and construction practices, it may not be practical to entirely avoid impacts to the nearest structures. In particular, underpinning of the structures located adjacent to the east and south sides of the site may be required. One option may be to drive piles adjacent to the wall of the structure and bracket the existing foundations onto those piles. Continuous concrete shoring (such as a diaphragm wall) would also be an option, for the sides adjacent to these existing structures, and would greatly mitigate the potential for foundation movements, but would also be much more expensive. However the shoring and underpinning options will require further evaluation at the detailed design stage. Further details on the foundations of the existing structures will be required.

Adjacent to the overburden shoring, some unavoidable inward horizontal deformation and vertical settlement of the adjacent ground will occur as a result of excavation, installation of shoring, deflection of the ground support system (including bending of the walls, compression of the struts and/or extension of the tie-backs) as well as deformation of the soil/rock in which the toes of the walls are embedded. The ground movements could affect the performance of buildings, surface structures or underground utilities adjacent to the excavation.

The structures that are apparently most at risk of being impacted by the shoring ground movements are the three storey brick building located east of the of the site and the underground parking garage located at the south limit of the site. Even with proper shoring design and construction practices, it may not be practical to entirely avoid impacts to these structures without first underpinning them.

A preconstruction survey of all of these structures should be carried out prior to commencement of the excavation.

5.4 Foundations

In general, the subsurface conditions on this site consist of up to about 2.5 metres of fill material, overlying about 4 to 6 metres of sensitive silty clay, overlying glacial till, with the underlying shale bedrock surface varying from about 9 to 11 metres depth.

Based on preliminary plans, it is expected that the foundations for this structure will be at about 21.5 to 22 metres depth below the existing ground surface, which will be within shale bedrock.

The boreholes for the current investigation did not penetrate to the likely founding depth. However, as a preliminary guideline, footings on or within un-weathered shale bedrock can likely be sized using an Ultimate Limit States (ULS) factored bearing resistance of about 1 to 2 Megapascals. However, if the rock below founding level can be proven to be free of seams and fractures, an Ultimate Limit States (ULS) factored bearing resistance of about 3 to 4 Megapascals can be used for design. For footings bearing on or within bedrock, Serviceability Limit States (SLS) generally do not govern the design since the stresses required to induce 25 millimetres of movement (the typical SLS criteria) exceed those at ULS. Accordingly the post construction settlement of structural elements which derive their support from footings bearing on bedrock should be negligible.

If the hydrogeological study indicates that the permanent groundwater level lowering could be an issue with regards to surrounding ground settlements due to the sensitive and compressible clay soils which exist within the expected zone of influence of the groundwater level lowering, then the structure will require a water-tight raft slab foundation. The above bearing resistance values are also applicable for the design of a raft-slab foundation.

5.5 Foundation Seismic Design

The seismic design provisions of the 2006 Ontario Building Code depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock beneath the founding level. For this proposed development, the bearing stratum, and 30 metres below the bearing level, will be shale bedrock. Based on previous experience within Billings Formation shale, this site will likely meet the requirements of Site Class A. However, site specific shear wave velocity testing will have to be carried out to confirm this Site Class, as per the 2006 Ontario Building Code.

5.6 Impacts on Adjacent Developments

Impacts on surrounding structures could result from ground movements around the excavation shoring, groundwater level lowering, and heaving of surrounding shale.

The shoring and underpinning requirements and the potential impacts on surrounding structures due to ground movements are discussed in Section 5.3 of this report.

The temporary and permanent groundwater level lowering could be an issue with regards to surrounding ground settlements due to the sensitive and compressible clay soils which exist within the expected zone of influence of the groundwater level lowering. Such soils are expected to exist to the south and east of the site. At the detailed design stage, a separate hydrogeological study will need to be undertaken to evaluate the potential impacts of the proposed development on the groundwater levels in the vicinity of the site. In particular, the study will have to focus on the potential for groundwater lowering which could cause ground settlements of adjacent structures and utilities that are supported on the silty clay deposits to the south and east.

If the hydrogeological study shows that *permanent* groundwater level lowering will result in unacceptable settlements of the ground and adjacent structures, then water-tight construction could be required for the lower levels of this development (i.e., below the groundwater level). Similarly, If the hydrogeological study shows that the *temporary* groundwater level lowering required for construction (which could be up to 9 to 12 months in duration) will result in unacceptable settlements of the ground and adjacent structures, then a groundwater recharge system may need to be implemented during construction, and/or pre-excavation of the bedrock might be necessary.

Regardless, at the detailed design stage, a separate hydrogeological study will need to be undertaken to evaluate the groundwater pumping requirements and the potential impacts of the proposed development on the hydrogeological conditions in the vicinity of the site. The hydrogeological study will also be required to support an application for a Permit-To-Take-Water from the Ministry of Environment, if more than 50,000 litres of water per day is expected to be pumped from the excavation (which is likely the case).

As discussed in Section 5.2, the shale bedrock at this site and beneath surrounding structures has the potential to swell if exposed to oxygen. That swelling would not likely heave the *foundations* of the adjacent structures, but could heave floor slabs, if located just above the bedrock. Therefore, as a preliminary guideline, where shale is exposed on the sides of the excavation, the exposed shale should be shotcreted so that concrete covers the shale to the top-of-rock level.

5.7 General Foundation Wall Construction Guidelines

Basement walls may be poured directly against bedrock, shoring, and/or formwork.

The details of the drainage system (if required/permitted) will need to be confirmed once the hydrogeological assessment has been completed and the impact of the potential water level lowering is known. The following guidelines are provided on the basis that water-tight construction will not be required.

Where the basement walls will be poured directly against the bedrock and shoring, vertical drainage such as Miradrain or Deltadrain must be installed on the face of the excavation to provide the necessary drainage.

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

Both the wall backfill and/or the Miradrain should be connected to a perimeter drain at the base of the excavation which is connected to a sump pump.

Conventional damp proofing of the basement walls is appropriate with the above design approach. For concrete walls poured against bedrock or shoring, damp proofing using an interior treatment such as Crystal Lok is suggested.

If, however, water-tight construction is shown to be necessary, additional guidelines will need to be provided.

5.8 Frost Protection

All perimeter and exterior foundation elements or interior foundation elements in unheated areas should be provided with a minimum of 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover.

It is expected that these requirements will be satisfied due to the deep founding level required to accommodate the below grade parking, and assuming that the parking garage will be heated.

5.9 Environmental Considerations

This geotechnical investigation was carried out in conjunction with a Phase I and II Environmental Site Assessment, the results of which are reported in a Golder Associates' report numbered 11-1121-0202-1000 titled "Phase I and II Environmental Site Assessment, 96 Nepean Street, Ottawa, Ontario" dated November 2011. The Phase I and II Environmental Site Assessment report should be read in conjunction with this report.

In brief, the results of the environmental investigation indicate that the glacial till, upper portion of the bedrock, and the groundwater have hydrocarbon impacts. Considering that there was no apparent contamination observed in the silty clay soils, and that all the silty clay samples analysed reported hydrocarbon parameters below detection limits, it is interpreted that the source of contamination is not likely on the property. Rather, the source would be somewhere off-site and the impacts have travelled down to the water table and then spread via groundwater movement.

Based on preliminary plans, the foundations for this structure will be at about 21.5 to 22 metres depth below the existing ground surface. Based on this founding depth, all of the soil will be removed from the site and the excavation will extend some 10 to 12 metres into the underlying shale bedrock. Therefore, to construct the basement of the structure, there will be a quantity of hydrocarbon contaminated soil, and possibly upper bedrock, requiring landfill disposal; as well, there will be impacted groundwater to be managed and treated on-site for discharge to the City sewer.

As discussed above, the source of contamination is likely off-site and the contamination likely spread to this site via groundwater flow. Although all of the "on-site" contamination will be removed during construction, there is a potential that if the structure is constructed using a drained foundation system, contaminated groundwater could be drawn to the site and ultimately into the building's drainage system. This condition may not be acceptable to the City. Consideration may need to be given to constructing the lower portion of the basement to be water-tight so that off-site contamination is not drawn to this site.

6.0 ADDITIONAL CONSIDERATIONS

The factual information and guidelines provided in this report are suitable for planning and preliminary design of this development only. Additional investigation and geotechnical design input will need to be provided at the detailed design stage.

The additional investigation required for the detailed design of the structure should include:

- Advancing boreholes to at least 5 metres below the underside of the proposed founding level so that the quality and strength of the bedrock can be determined.
- Carrying out a large scale pump test so that the zone of groundwater level drawdown can be determined so that the potential for groundwater lowering to cause ground settlements of adjacent structures can be assessed.

We trust that this report is sufficient for your present requirements. If you have any questions concerning this report, please call us.

Yours truly,

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, Claridge Homes. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

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.

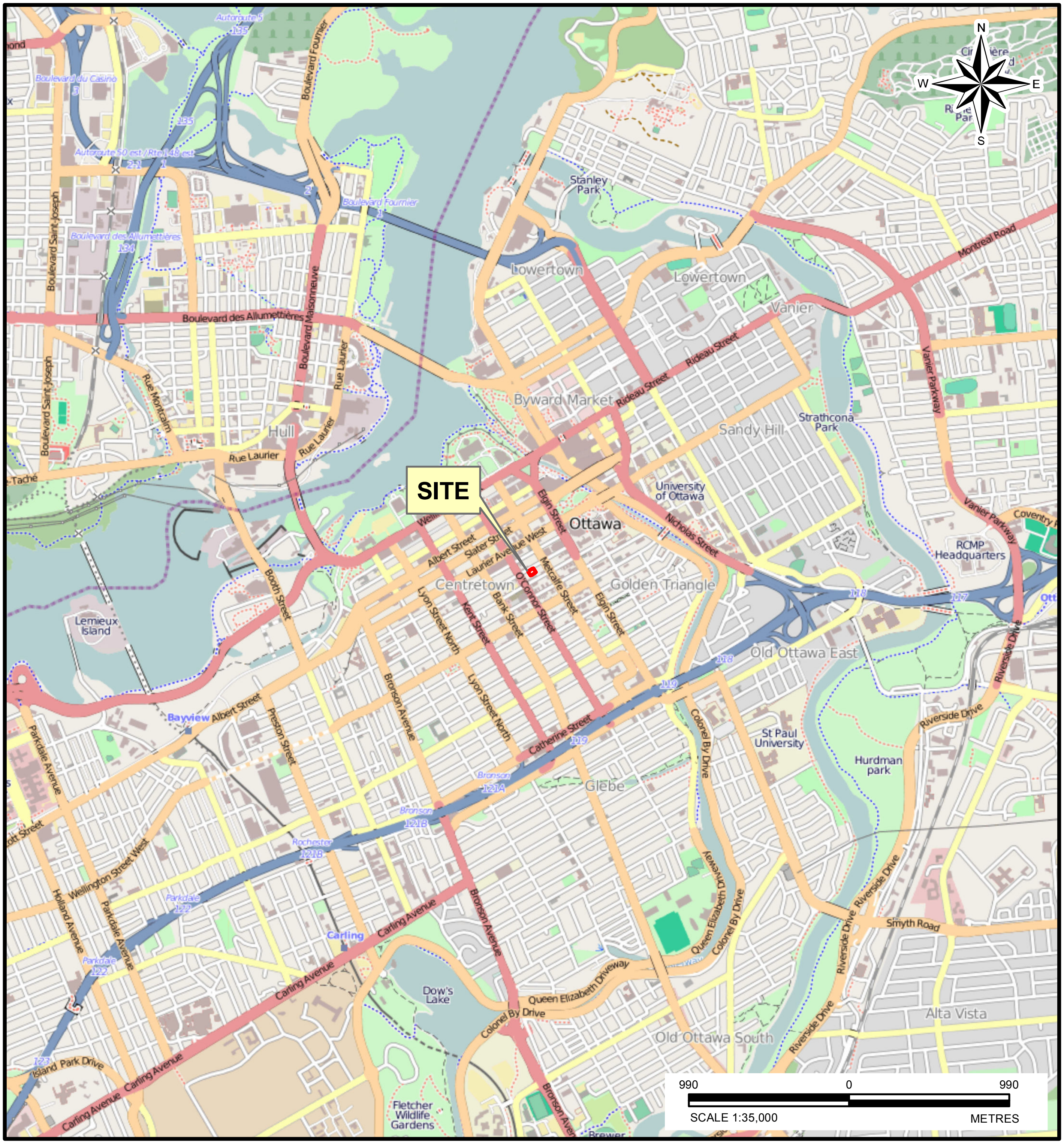
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater 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.



NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 11-1121-0202-3000

REFERENCE

DATA PROVIDED BY ESRI CANADA ARCGIS ONLINE 2011
DATUM: NAD 83, COORDINATE SYSTEM: UTM ZONE 18

Golder Associates
Ottawa, Ontario

PROJECT No.	11-1121-0202-3000		
SCALE	1:35,000	REV.	0

DATE	Nov. 2011
DESIGN	BM
GIS	BJ
CHECK	TMS
REVIEW	MIC

TITLE

KEY PLAN

PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION
96 NEPEAN STREET, OTTAWA, ONTARIO

FIGURE 1

Path: N:\Active\2011\1121 - Geotechnical\11-1121-0202 Claridge Nepean St Ottawa\Spatial_IM\MXD\1111210202-3000-01.mxd

LEGEND

⊕ APPROXIMATE LOCATION OF BOREHOLE

☐ APPROXIMATE STUDY BOUNDARY

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 11-1121-0202-3000.

REFERENCE

DATA PROVIDED BY ESRI CANADA, 2011
DATUM: NAD 83, COORDINATE SYSTEM: UTM ZONE 18

Golder Associates
Ottawa, Ontario

PROJECT No.	11-1121-0202-3000
SCALE	1:600
REV.	0
DATE	Nov. 2011
DESIGN	AC
GIS	BJ
CHECK	TMS
REVIEW	MIC

TITLE

SITE PLAN

PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION
96 NEPEAN STREET, OTTAWA, ONTARIO

FIGURE 2

Path: N:\Active\2011\1121 - Geotechnical\11-1121-0202 Claridge Nepean St Ottawa\Spatial_IM\MXD\1111210202-3000-02.mxd

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	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual Tube sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

DD- Diamond Drilling

Dynamic Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive Uncased a 50 mm (2 in.) diameter, 60^0 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

Peizo-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 (PWP) 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	plastic limited
w_1	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 (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	modified Proctor compaction test
SPC	standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane 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
F	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 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 ($D_R= p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation

*	Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_1	liquid limit
w_p	plastic limit
I_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p\tau_r$	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
$c_{u,}s_u$	undrained shear strength (ϕ=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. τ=c'σ' tan |'
2. Shear strength=(Compressive strength)/2

PROJECT: 11-1121-0202

RECORD OF BOREHOLE: 11-1

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: September 7, 2011

DATUM: Local

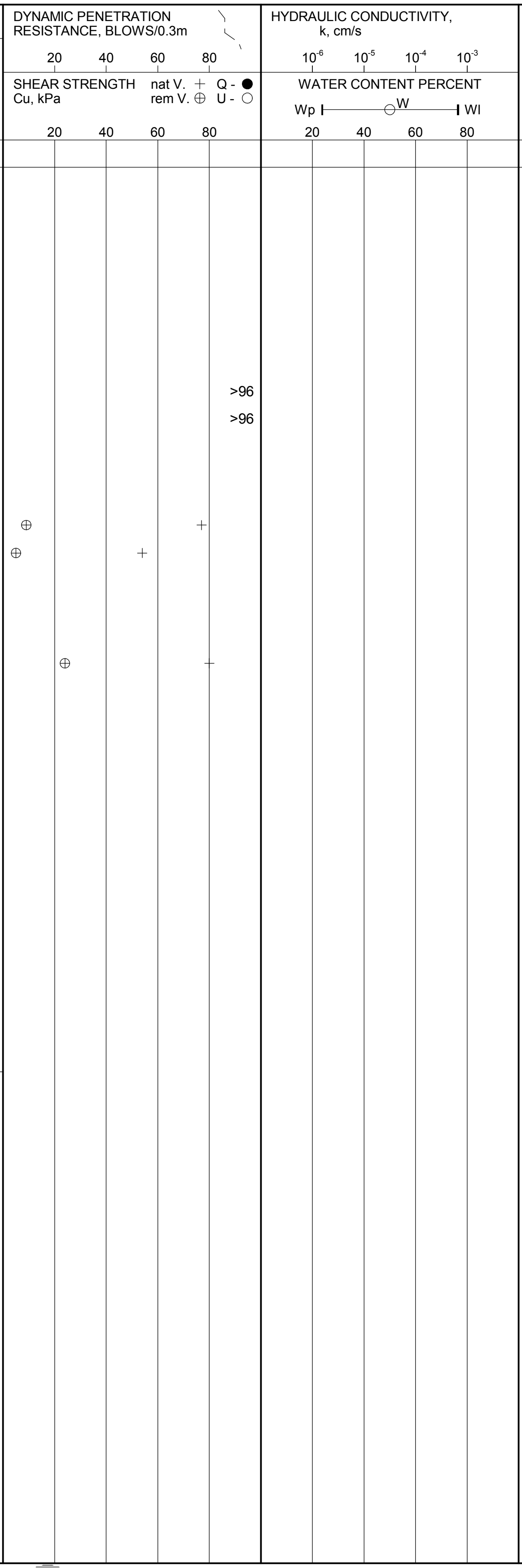
SAMPLER HAMMER, 64kg; DROP, 760mm

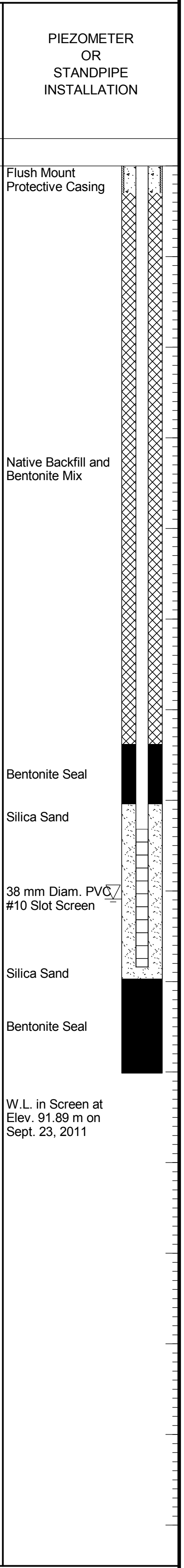
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	99.98
ASPHALTIC CONCRETE	0.08
Compact brown sand, some gravel, with brick (FILL)	98.46 1.52
Very stiff grey brown SILTY CLAY (Weathered Crust)	96.93 3.05
Stiff grey SILTY CLAY	94.42 5.56
Very loose dark brown to black SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)	93.12 6.86
Loose to dense black SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)	89.97 10.01
End of Borehole Auger Refusal	

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	10
2	50 DO	5
3	50 DO	
4	50 DO	2
5	50 DO	WH
6	50 DO	2
7	50 DO	11
8	50 DO	9
9	50 DO	32
10	50 DO	44
11	50 DO	>50





W.L. in Screen at Elev. 91.89 m on Sept. 23, 2011

DEPTH SCALE

1 : 75

LOGGED: JMC

CHECKED: CK/TMS

MIS-BHS 001 1111210202.GPJ GAL-MIS.GDT 11/18/11 JEM/PG

PROJECT: 11-1121-0202

RECORD OF BOREHOLE: 11-2

SHEET 1 OF 2

LOCATION: See Site Plan

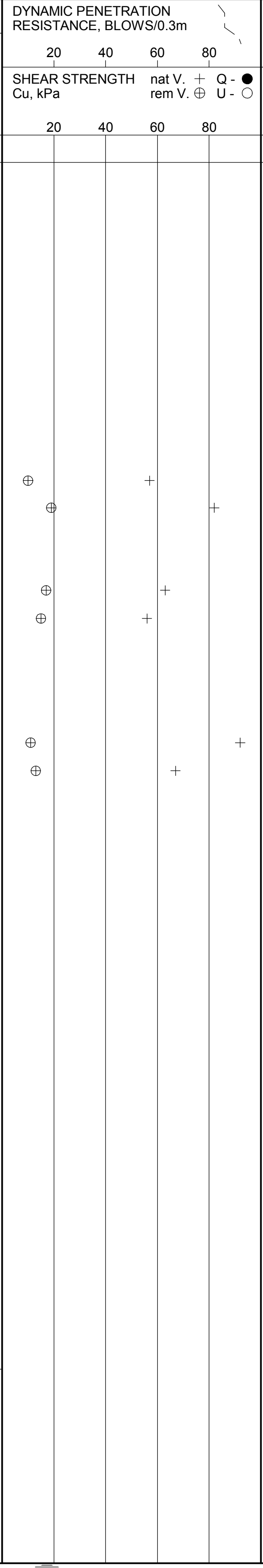
BORING DATE: September 7, 2011

DATUM: Local

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
	GROUND SURFACE	99.73			
Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE				
	Compact brown sand, some gravel, trace silt, with brick (FILL)	0.10	1	50 DO	11
	Very stiff grey brown SILTY CLAY (Weathered Crust)	98.02 1.71	2	50 DO	3
	Stiff grey SILTY CLAY	97.32 2.41	3	50 DO	2
			4	50 DO	1
			5	50 DO	WH
	Stiff grey SILTY CLAY, some sand, trace gravel	92.87 6.86	6	50 DO	2
	Loose to compact dark brown to black SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)	92.41 7.32	7	50 DO	4
			8	50 DO	10
			9	50 DO	7
	Compact to dense dark brown to black SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)	89.83 9.90	10	50 DO	29
			11	50 DO	36
	Highly weathered, thinly laminated to thinly bedded, black SHALE BEDROCK	88.58 11.15			
Rotary Drill NQ Core	Fresh, thinly laminated to thinly bedded, black SHALE BEDROCK	87.94 11.79	C1	NQ RC	DD
			C2	NQ RC	DD
	End of Borehole	86.42 13.31			



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

Native Backfill and Bentonite Mix

Bentonite Seal

Silica Sand

38 mm Diam. PVC #10 Slot Screen 'B'

Bentonite Seal

Silica Sand

38 mm Diam. PVC #10 Slot Screen 'A'

Bentonite Seal

W.L. in Screen at Elev. 91.42 m on Sept. 26, 2011

Screen 'B' dry on Sept. 26, 2011

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: PH

CHECKED: CK/TMS

PROJECT: 11-1121-0202

RECORD OF BOREHOLE: 11-3

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: September 20, 2011

DATUM: Local

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

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	100.04				
Power Auger 200 mm Diam. (Hollow Stem)	Inferred Fill Material	0.00				Flushmount Casing Bentonite Seal
	Very stiff to stiff grey brown to grey SILTY CLAY (Weathered Crust)	97.75 2.29	1	50 DO	6	
	Stiff grey SILTY CLAY, trace gravel, with sand seams	96.99 3.05	2	50 DO	2	Native Backfill
			3	50 DO	2	
			4	50 DO	3	
			5	50 DO	2	
			6	50 DO	3	Bentonite Seal
	Compact dark brown to black SILTY SAND, some gravel, trace clay, with cobbles, boulders, and shale fragments (GLACIAL TILL)	93.03 7.01	7	50 DO	17	Silica Sand
			8	50 DO	15	▽
			9	50 DO	13	32 mm Diam. PVC #10 Slot Screen
			10	50 DO	24	
	Highly weathered SHALE BEDROCK	90.13 9.91	11	50 DO	>62	Silica Sand
	End of Borehole Auger Refusal	89.68 10.36				W.L. in Screen at Elev. 91.91 m on Sept. 23, 2011

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: PH

CHECKED: CK/TMS

PROJECT: 11-1121-0202

RECORD OF BOREHOLE: 11-4

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: September 20, 2011

DATUM: Local

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

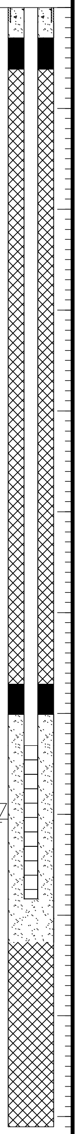
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

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	SAMPLES BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	100.05				
0	Power Auger 200 mm Diam. (Hollow Stem)	Inferred Fill Material	0.00				Flushmount Casing Bentonite Seal
2		Very stiff to stiff grey brown to grey SILTY CLAY (Weathered Crust)	97.76 2.29	1	50 DO	5	
3		Stiff to firm grey SILTY CLAY, trace gravel, with sand seams	97.00 3.05	2	50 DO	2	Native Backfill
				3	50 DO	WH	
				4	50 DO	2	
				5	50 DO	WH	
				6	50 DO	3	
7		Loose to very dense dark brown to black SANDY SILT, some gravel, trace clay, with cobbles, boulders, and shale fragments (GLACIAL TILL)	93.42 6.63	7	50 DO	4	Bentonite Seal Silica Sand
				8	50 DO	8	32 mm Diam. PVC #10 Slot Screen
				9	50 DO	17	Silica Sand
				10	50 DO	11	Caved Material
				11	50 DO	15	
				12	50 DO	>61	
11		End of Borehole Sampler Refusal	88.95 11.10				W.L. in Screen at Elev. 92.00 m on Sept. 26, 2011



DEPTH SCALE

1 : 75

LOGGED: PH

CHECKED: CK/TMS

PROJECT: 11-1121-0202

RECORD OF BOREHOLE: 11-5

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: September 25, 2011

DATUM: Local

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

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	100.09				
Power Auger 200 mm Diam. (Hollow Stem)	Inferred Fill Material	0.00				Flushmount Casing
	Very stiff to stiff grey brown to grey SILTY CLAY (Weathered Crust)	97.80 2.29	1	50 DO	2	
	Stiff to firm grey SILTY CLAY, trace gravel, with sand seams	97.04 3.05	2	50 DO	2	
			3	50 DO	WH	Native Backfill
			4	50 DO	WH	
			5	50 DO	WH	
	Compact dark brown to black SILTY SAND, some gravel, trace clay, with cobbles, boulders, and shale fragments (GLACIAL TILL)	93.72 6.37	6	50 DO	2	
			7	50 DO	17	Bentonite Seal Silica Sand
			8	50 DO	14	32 mm Diam. PVC #10 Slot Screen
	Highly Weathered SHALE BEDROCK	91.25 8.84	9	50 DO	27	Silica Sand
			10	50 DO	25	Bentonite Seal
			11	50 DO	33	
	End of Borehole Auger Refusal	89.67 10.42				W.L. in Screen at Elev. 91.91 m on Sept. 26, 2011

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: PH

CHECKED: CK/TMS

PROJECT: 11-1121-0202

RECORD OF BOREHOLE: 11-6

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: September 22, 2011

DATUM: Local

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

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

SOIL PROFILE DESCRIPTION	ELEV.	DEPTH (m)
GROUND SURFACE	100.20	
Inferred Fill Material		0.00
Very stiff to stiff grey brown to grey SILTY CLAY (Weathered Crust)	97.91	2.29
Stiff to firm grey SILTY CLAY, trace gravel, with sand seams	97.15	3.05
Loose to compact dark brown to black SANDY SILT, some gravel, trace clay, with cobbles, boulders, and shale fragments (GLACIAL TILL)	93.49	6.71
Highly weathered SHALE BEDROCK	90.51	9.69
End of Borehole Sampler Refusal	89.43	10.77

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	3
2	50 DO	2
3	50 DO	1
4	50 DO	WH
5	50 DO	5
6	50 DO	39
7	50 DO	11
8	50 DO	10
9	50 DO	7
10	50 DO	16
11	50 DO	34
12	50 DO	>50

PIEZOMETER OR STANDPIPE INSTALLATION

Flushmount Casing
Bentonite Seal
Native Backfill
Bentonite Seal
Silica Sand
32 mm Diam. PVC #10 Slot Screen
Silica Sand
Bentonite Seal
Monitoring Well dry on Sept. 26, 2011

DEPTH SCALE
1 : 75

LOGGED: PH
CHECKED: CK/TMS

PROJECT: 11-1121-0202

RECORD OF BOREHOLE: 11-7

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: September 21, 2011

DATUM: Local

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES: 0 – 15

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

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}

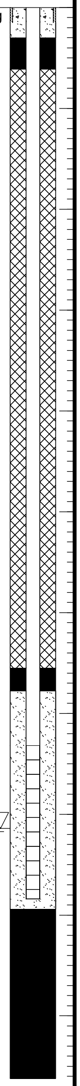
WATER CONTENT PERCENT: Wp ⊢ ○W ⊣ Wl ; 20 40 60 80

ADDITIONAL LAB. TESTING

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	100.10
Inferred Fill Material	0.00
Very stiff to stiff grey brown to grey SILTY CLAY (Weathered Crust)	97.81 2.29
Stiff to firm grey SILTY CLAY, trace gravel, with sand seams	97.05 3.05
Compact dark brown to black SANDY SILT, some gravel, trace clay, with cobbles, boulders, and shale fragments (GLACIAL TILL)	92.88 7.22
Highly weathered SHALE BEDROCK	90.10 10.00
End of Borehole Auger Refusal	89.48 10.62

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	3
2	50 DO	2
3	50 DO	WH
4	50 DO	WH
5	50 DO	WH
6	50 DO	5
7	50 DO	WH
8	50 DO	10
9	50 DO	11
10	50 DO	27
11	50 DO	32

PIEZOMETER OR STANDPIPE INSTALLATION



Flushmount Casing

Bentonite Seal

Native Backfill

Bentonite Seal

Silica Sand

32 mm Diam. PVC #10 Slot Screen

Silica Sand

Bentonite Seal

W.L. in Screen at Elev. 91.96 m on Sept. 26, 2011

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: PH

CHECKED: CK/TMS

PROJECT: 11-1121-0202

RECORD OF BOREHOLE: 11-8

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: September 23, 2011

DATUM: Local

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		100.29							
0 1 2	Power Auger 200 mm Diam. (Hollow Stem)	Inferred Fill Material		0.00							Flushmount Casing Bentonite Seal
		Sand, some gravel, trace silt (FILL)		98.00 2.38							
3		Stiff grey brown SILTY CLAY (Weathered Crust)			1	50 DO	WH				
4 5 6 7		Stiff to firm grey SILTY CLAY, trace gravel, with sand seams		97.24 3.05	2 3 4 5 6	50 DO 50 DO 50 DO 50 DO 50 DO	WH WH WH WH WH				Native Backfill Bentonite Seal
8 9		Loose to compact dark brown to black SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)		93.22 7.07	7 8 9 10	50 DO 50 DO 50 DO 50 DO	7 13 7 4				Silica Sand 32 mm Diam. PVC #10 Slot Screen ▽ Silica Sand
10		Highly weathered SHALE BEDROCK		90.61 9.68	11 12	50 DO 50 DO	14 >50				Bentonite Seal
11 12 13 14 15		End of Borehole Sampler Refusal		89.38 10.91							W.L. in Screen at Elev. 91.99 m on Sept. 26, 2011

MIS-BHS 001 1111210202.GPJ GAL-MIS.GDT 11/18/11 JEM/PG

DEPTH SCALE

1 : 75

LOGGED: PH

CHECKED: CK/TMS

PROJECT: 11-1121-0202

RECORD OF BOREHOLE: 11-9

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: September 22, 2011

DATUM: Local

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES: 0 – 15

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem) (0.00 – 11.43 m)

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
GROUND SURFACE	100.31			
Inferred Fill Material	0.00			
Very stiff to stiff grey brown to grey SILTY CLAY (Weathered Crust)	98.02 2.29	1	50 DO	2
Stiff grey SILTY CLAY, trace gravel, with sand seams	97.19 3.12	2	50 DO	WH
		3	50 DO	WH
		4	50 DO	2
		5	50 DO	WH
		6	50 DO	PH
		7	50 DO	PH
		8	50 DO	PH
Compact dark brown to black SILTY SAND, some gravel, trace clay, with cobbles and boulders (GLACIAL TILL)	92.39 7.92	9	50 DO	PH
		10	50 DO	16
Highly weathered SHALE BEDROCK	91.32 8.99	11	50 DO	17
		12	50 DO	18
		13	50 DO	28
End of Borehole Auger Refusal	88.88 11.43			

PIEZOMETER OR STANDPIPE INSTALLATION

Flushmount Casing

Bentonite Seal

Native Backfill

Bentonite Seal

Silica Sand

32 mm Diam. PVC #10 Slot Screen

Silica Sand

Bentonite Seal

Silica Sand

W.L. in Screen at Elev. 91.94 m on Sept. 26, 2011

DEPTH SCALE

1 : 75

LOGGED: PH

CHECKED: CK/TMS

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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