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REPORT ON

GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL DEVELOPMENT MCLEAN LANDING

MERRICKVILLE, ONTARIO

Submitted to:

David McManus Engineering Consultants Ltd.
30 Camelot Drive, Unit 400
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November 22, 2006 06-1120-338

David McManus Engineering Consultants Ltd.
30 Camelot Drive, unit 400
Nepean, Ontario
K2G 5W6

Attention: Mr. H. Sandanayake

RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
MCLEAN LANDING
MERRICKVILLE, ONTARIO

Dear Sir:

Please find attached our report on the geotechnical investigation for the proposed residential development to be located on lands at the northwest quadrant of St. Lawrence Street and County Road 16 near Merrickville, Ontario.

We trust that this report is sufficient for your present requirements. If you have any questions concerning this report or if we can be of further service to you on this project, please call us.

Yours truly,

GOLDER ASSOCIATES LTD.

J.A.Stephenson, B.Eng., M.A.Sc.

G.S. Webb, P.Eng.
Principal

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the site of a proposed residential development to be located on lands northwest of St. Lawrence Street and County Road 16 near Merrickville, Ontario. The proposed development consists of ground oriented wood frame residential structures with basements. The purpose of the subsurface investigation was to determine the general subsurface and groundwater conditions across the site by means of a limited number of boreholes and test pits and, based on an interpretation of the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could affect design and development 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 residential subdivision to be located on lands northwest of St. Lawrence Street and County Road 16 near Merrickville, Ontario (see Key Plan, in Figure 1).

The property measures approximately 190 metres by 360 metres in plan dimension, and is bounded to the south by County Road 16 and to the east by St. Lawrence Street.

The site is generally flat lying but slopes gradually from northwest to southeast. In the lowest areas of the site, water was observed to be ponding on the ground surface. The site is presently undeveloped and most of the area of the proposed development has been recently cleared of trees. However, the west side of the site and property boundary remains tree-covered. The existing trees are of moderate size and are not particularly dense.

The property is proposed for development with a conventional subdivision consisting of fifty-five (55) single family homes. A storm water management pond is proposed to be located at the approximate centre of the south side of the development (County Road 16).

Published geologic maps indicate that the bedrock in this area is relatively shallow and consists of Dolostone of the Oxford formation.

3.0 PROCEDURE

The field work for this investigation was carried out between November 1 and 3, 2006.

On November 1, 2006 eleven test pits (numbered 06-113 to 06-125) were excavated at the approximate locations shown on the Site Plan, Figure 2. The test pits were excavated using a rubber tire back-hoe supplied and operated by D. McConnell Construction of Merrickville, Ontario (on contract to R.W. Tomlinson Limited of Ottawa, Ontario). The test pits were excavated to depths ranging from approximately 0.2 metres to 1.3 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 and the test pits were loosely backfilled upon completion of excavating and sampling.

On November 2 and 3, 2006 two boreholes (numbered 06-1 and 06-2) were put down at the approximate locations shown on the Site Plan, Figure 2. The borings were advanced using portable drilling equipment supplied and operated by O.G.S. Inc. of Almonte, Ontario. The boreholes were advanced to depths ranging from approximately 3.8 metres to 4.5 metres below the existing ground surface.

Standard penetration tests were carried out where possible in the boreholes and samples of the soils encountered were recovered using drive open sampling equipment.

In each of the boreholes bedrock was proven for depths of between 3.8 and 4.5 metres by rotary core drilling in NWT size. The core obtained was sequentially packed into proper core boxes.

The subsurface conditions encountered in the test pits are shown on the Record of Test Pits, Table 1, which follows the text of this report. The conditions encountered in the boreholes are shown in the Record of and Borehole Sheets in Appendix A.

A monitoring well was sealed into borehole 06-1 to measure the in situ permeability characteristics of the bedrock and groundwater level. A standpipe was sealed into borehole 06-2, to allow subsequent measurement of the groundwater level.

The test pit and borehole locations were selected by Golder Associates Ltd., and located in the field by survey personnel for David McManus Engineering Consultants Ltd.

The field work was supervised and/or carried out by a member of our engineering staff, who directed the drilling and excavation operations, logged the boreholes and test pits, directed the in situ testing and took custody of the samples retrieved. The soil and rock samples obtained during the field work were brought to our laboratory for further examination by the project engineer.

The groundwater levels in the monitoring well and standpipe were measured on November 8, 2006. At that time, a rising head test to determine the in situ permeability characteristics of the bedrock was performed in the monitoring well installed in borehole 06-1.

One sample of soil from test pit 06-125 was submitted to Accutest Laboratories Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of this testing are provided in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the test pits are shown on the Record of Test Pits, Table 1. The subsurface conditions encountered in the boreholes are shown on the Record of Borehole sheets in Appendix A. The results of the basic chemical analysis on the soil sample from test pit 06-125 are provided in Appendix B.

In general the subsurface conditions on this site consist of topsoil and glacial till overlying weathered bedrock. The following sections provide a summary of the subsurface conditions encountered across the site.

4.2 Topsoil and Sand

A layer of topsoil was encountered at ground surface in both of the boreholes and almost all of the test pits. The topsoil ranges in thickness from 150 to 370 millimetres.

In test pit 06-117 brown fine sand was encountered at ground surface. The sand was 300 millimetres thick.

4.3 Glacial Till

A deposit of glacial till of was encountered beneath the topsoil in test pits 06-119, 06-124 and 06-125. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sand or silty sand. The glacial till ranges in thickness from 0.15 to 1.05 metres, and extends to elevations between 115.1 and 114.5 metres where encountered. The deposit of glacial till is limited to the south central portion of the site and was for the most part quite thin, with the exception of the area around test pit 06-125 where it is about 1 metre thick.

4.4 Bedrock

Beneath the topsoil and glacial till, weathered bedrock was encountered in all of the test pits and boreholes.

In the test pits the upper portion of bedrock was able to be excavated with a rubber tire back-hoe. The test pits were excavated into the bedrock surface to depths ranging from 0.2 to 0.7 metres below existing ground surface. However in test pit 06-123 the bedrock was too hard to be excavated with a back-hoe and in test pit 06-125, the test pit was terminated when moderately weathered bedrock was encountered beneath the glacial till.

Both boreholes 06-1 and 06-2 were advanced into bedrock to depths ranging from about 3.8 to 4.5 metres below existing ground surface (i.e., elevation 113.4 and 113.03 metres) by coring using NWT size coring equipment.

In borehole 06-1 the upper 0.3 metres of bedrock had been weathered and in borehole 06-2 the upper 1.3 metres of bedrock was fractured and weathered. The Rock Quality Designation (RQD) within the weathered bedrock was zero indicating a poor quality rock. The bedrock underlying the weathered zone is fresh dolomitic limestone. The Rock Quality Designation values of the fresh bedrock ranged widely from 29 to 88 percent indicating a poor to good quality rock. In general the RQD values increase significantly with depth.

A monitoring well was installed in borehole 06-1 and a rising head permeability test was conducted in this monitoring well. The test result was interpreted using the Hvorslev method and indicate hydraulic conductivity of about 4.4 x 10^{-4} centimetres per second.

4.5 Groundwater

In test pit 06-114 the water level was at ground surface, however the remaining test pits were dry upon completion.

The groundwater levels in borehole 06-1 and 06-2 were measured on November 8, 2006. At that time the water levels were found to be 2.9 and 3.0 metres below ground surface respectively (elevations 112.9 to113.9 metres).

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 RESIDENTIAL DEVELOPMENT

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of this project based on our interpretation of the borehole information and project requirements, and is subject to the limitations in the "Important Information and Limitations of This Report" attachment which follows the text of this report.

5.2 Site Grading

The subsurface conditions on this site generally consist of a thin layer of topsoil, followed by discontinuous glacial till and limestone bedrock.

For these subsurface conditions, no restrictions apply to the thickness of grade raise fill which may be placed on the site, from a foundation design perspective.

5.3 Foundations

It is considered that the proposed houses can be founded on spread footings on or within the glacial till or bedrock.

Footings founded within the glacial till may be designed using a maximum allowable bearing pressure at serviceability limit states (SLS) of 150 kilopascals and a factored bearing resistance at ultimate limit states (ULS) of 250 kPa. Footings founded on fractured bedrock may be sized using a maximum allowable bearing pressure of 500 kPa and footings founded on the fresh bedrock may be sized using a maximum allowable bearing pressure of 1000 kPa.

For structures where the underside of footing level is above the existing ground surface, all topsoil or organic material should be removed to expose the underlying undisturbed soil or bedrock. Engineered fill consisting of OPSS Granular B Type II or well broken broadly graded blast rock up to 300 millimetres in size should then be placed to raise the exposed subgrade level up to the proposed underside of footing level. The engineered fill should be sized to accommodate the spread of load out from the edge of the footing at an angle of 1 horizontal to 1 vertical. The engineered fill should be compacted to at least 95 percent of the standard Proctor dry density using suitable vibratory compaction equipment, in lifts of 300 millimetres. The allowable bearing pressure for spread footings on engineered fill is 150 kilopascals.

The post construction total and differential settlements of footings sized using the above maximum allowable bearing pressures should be less than about 25 and 15 millimetres respectively, provided that the soil at or below founding level is not disturbed during construction.

In order to avoid the distress created from differential settlement, structures should be either founded entirely on soil or on bedrock, but not both. Where both bedrock and native soil exist at founding level in the footprint of a given foundation, the rock should be excavated out an additional 0.3 metres over the entire area. The rock subgrade should then be brought up to the underside of the footing level using OPSS Granular A or Granular B type II, uncompacted. Another option would be remove the existing soil to bedrock level.

Where boulders project above founding level in the footing areas or where they have been loosened by the excavation process, they should be removed along with any surrounding disturbed soil and the void filled with concrete.

Excavation for the foundations may require drilling, blasting and excavation of bedrock in order to reach founding grade. This work should be carried out with care in order to avoid overblast and/or blast induced facturing within the bedrock below founding level. However, it is recognized that with the rock type at this site, some overblast is inevitable. The following procedures should be used in preparing the foundation areas of those houses on bedrock where overbreak has been created by blasting. Following blasting and excavation to founding level, test pits are to dug in the foundation areas, inspected and:

1. When the test pits indicate that there are 0.6 metres of overbreak, or less, the overbreak may be left in place, and, upon inspection:

    - If the surface of the blasted rock is tight and does not contain appreciable open voids, the footing can be poured directly on the rock surface.
    - If the rock surface contains significant voids, 150 millimetres of OPSS Granular B Type II is to be placed over the rock surface to chink the voids and level the bearing surface. The granular B is to be compacted to at least 95 percent of the standard Proctor value using suitable vibratory compaction equipment.

2. When the test pits indicate that there are more than 0.6 metres of overbreak, all but the lower 0.6 metres of overbreak is to be excavated from the structure area.

    - If the excavated blast rock is well graded, the over excavation can be brought to grade using the excavated rock. The blast rock fill should be placed in loose lifts of 0.6 metres maximum thickness and compacted with multiple passes of a self propelled vibratory roller. If the surface of the blast rock fill contains open voids they shall be chinked with well compacted Granular B as described previously.

- If the rock excavated from the structure area is not well fractured, the excavation should be raised to design founding level using Granular B Type II. This fill is to be placed in maximum 300 millimetre thick lifts and compacted to al least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.4 Basement and Garage Floor Slabs

In preparation for the construction of the 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 crushed clear stone to form the base of the basement floor slabs.

To prevent hydrostatic pressure build up beneath the basement floor slabs, it is suggested that the granular base for the floor slabs be positively drained.

The backfill material inside the garages should consist of acceptable earth borrow or select subgrade material, be placed in maximum 300 millimetre thick lifts and be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment. The granular base for the garage floor slab should consist of at least 150 millimetres of Granular A compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

5.5 Frost Protection

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

5.6 Basement Walls and Foundation Wall Backfill

The soils at this site although of limited quantity 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 either 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 wall backfill should be provided by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, 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.

Basement walls backfilled with granular material and effectively drained as described above should be designed to resist lateral earth pressures as described in Part 9 of the Ontario Building Code, for structures which conform to Part 9 of this code.

5.7 Basement Excavations

If excavations are made for basements they will be primarily be made into bedrock or in a few areas glacial till.

No unusual problems are anticipated in excavating the glacial where encountered using conventional hydraulic excavating equipment although some significant cobble and boulder removal should be expected.

The glacial till at this site would generally be classified as Type 3 soils in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects (OHSA) and therefore, if open cut side slopes were considered, they would need to be cut back at an inclination no steeper than 1 horizontal to 1 vertical (1H:1V).

If a shallow amount of bedrock removal, removal could possibly be accomplished using mechanical methods (such as hoe ramming). If the excavated bedrock is fractured and loose, it should be sloped at 1 horizontal to 1 vertical. If more significant excavation in the bedrock is required the excavation could be accomplished by blasting. Near-vertical excavation side slopes should be feasible in the fresh bedrock.

Some groundwater inflow into the excavation should be expected below about elevation 113 metres. It is expected that it should be possible for the contractor to handle the groundwater inflow by pumping from well filtered sumps in the floor of the excavation, using suitably sized pumps.

5.8 Site Servicing

Excavations for the installation of site services will be through glacial till and bedrock.

No unusual problems are anticipated in trenching in the glacial till using conventional hydraulic excavating equipment although significant boulder removal from the glacial till should be expected. Side slopes should be stable at 1 horizontal to 1 vertical.

Excavation into bedrock may be achieved by mechanical methods such as hoe ramming, for excavations of shallow depth. However, if deeper excavations are required drill and blast procedures should be used. If the excavated bedrock is fractured and loose, it should be sloped at 1 horizontal to 1 vertical. Near vertical trench walls in the fresh bedrock should stand unsupported for the construction period.

Some groundwater inflow into the trenches should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps established in the floor of the excavations.

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 does occur, it may be necessary to place a sub-bedding layer consisting of compacted OPSS Granular B Type II beneath the Granular A or to thicken the Granular A bedding. 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 sandy soils on the trench walls could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

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 should generally be possible to re-use the glacial till and excavated bedrock as trench backfill. Where glacial till is to be used for trench backfill, boulders which are larger than 300 millimetres in size should be removed prior to its use. Excavated bedrock may be used as trench backfill provided it is broadly graded and has no particles larger than 300 millimetres in size. Where the trench will be covered with hard surfaced areas, the type of material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

Impervious dykes or cut-offs should be constructed in the service trenches near the street connection just inside the property to reduce groundwater lowering at the site due to the "French drain" effect of the granular bedding and surround for the service pipes. It is important that these barriers extend from trench wall to trench wall and that they fully penetrate the granular materials to the trench bottom. The dykes should be at least 1.5 metres wide. The native glacial till would be suitable for construction of the dykes.

5.9 Pavement Design and Construction Considerations

5.9.1 Hot Mix Asphaltic Concrete

HL3 or Superpave 12.5 (Level B) surface course and HL8 or Superpave 19.0 (Level B) base course asphaltic concrete may be used on this project (OPSS 310).

5.9.2 Asphalt Cement

The asphaltic concrete used on this project should be made with PG 58-34 asphalt cement on all lifts.

5.9.3 Granular Base and Subbase

The granular base and subbase for new construction should consist of Granular A and Granular B Type II, respectively (OPSS 1010).

5.9.4 Compaction

Compaction of the granular base, subbase and grade raise fill should be carried out in accordance with OPSS 501.08 Method A.

5.9.5 Pavement Structure

In preparation for pavement construction, all topsoil, disturbed, or otherwise deleterious materials should be removed from the roadway areas.

Pavement areas requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material (OPSS 1010). These materials should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

The surface of the pavement subgrade should be crowned to promote drainage of the roadway granular structure into ditching. In urban sections 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.

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

The pavement structure for collector roadways should consist of:

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

Alternatively bedrock shatter can be used to replace some of the subbase structure, in which case a layer of Granular B Type II material is required to fill any voids in the surface of the bedrock shatter and reduce the loss of fine material.

The asphaltic concrete should be compacted to at least 97 percent of Marshall Density.

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

HL3 or Superpave 12.5 - 50 millimetres

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

HL3 or Superpave 12.5 - 40 millimetres
HL8 or Superpave 19.0 - 50 millimetres

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

5.10 Stormwater Management Pond

It is understood that a stormwater management pond is planned for the subdivision as indicated on the site plan (Figure 2). It is also understood that the details of the design for the stormwater management pond are not known at this time. This section addresses some preliminary geotechnical considerations relating to the design of a stormwater management pond. However, at the detailed design stage of the pond a review of the geotechnical design issues should be carried out.

Borehole 06-1 and test pit 06-123, which are located in close proximity to the proposed pond location encountered bedrock at about 0.2 metres below existing ground surface. In borehole 06-1, the upper 0.3 metres of bedrock was weathered but the underlying bedrock consisted of fresh grey dolomitic limestone. The groundwater level in borehole 06-1 was 2.9 metres below ground surface (elevation 112.9 metres). A rising head insitu permeability test was performed and the test result was interpreted using the Hvorslev method and indicated hydraulic conductivity of about 4.4×10^{-4} centimetres per second.

It is understood that the stormwater management ponds is to be located on the bedrock surface. Any hydraulic connection between the stormwater and the bedrock will result in infiltration of storm water into the bedrock with possible negative impacts on the bedrock aquifer. If such hydraulic connection is to be avoided, the pond will require a low permeability lining. Excavation for the pond will be through topsoil and possibly a thin layer of glacial till. No unusual problems are anticipated in trenching in the overburden using conventional hydraulic excavating equipment, although some boulder excavation should be anticipated within the glacial till. Temporary side slopes should be stable in the short term at 1 horizontal to 1 vertical. Wherever workers will enter the pond excavation, boulders should be removed from the excavation side slopes.

Pond side slopes of 4 horizontal to 1 vertical are acceptable.

If the pond floor is to be provided with a durable surface upon which maintenance vehicles can travel, a structure consisting of 500 millimetres of compacted OPSS Granular B Type II is considered suitable.

5.11 Corrosion

One sample of soil from test pit 06-125 was submitted to Accutest Laboratories Ltd. for chemical analysis related to potential corrosion of exposed buried steel and concrete elements (corrosion and sulphate attack). The results of this testing are provided in Appendix B. The results indicate that concrete made with Type 10 Portland cement should be acceptable for substructures. The results also indicate a mild potential for corrosion of exposed ferrous metal.

6.0 ADDITIONAL CONSIDERATIONS

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

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

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

GOLDER ASSOCIATES LTD.

J.A.Stephenson, M.A.Sc, EIT.

G.S.Webb, P.Eng.
Principal

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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. 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 can not 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 upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. 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 can not 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 can not 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.

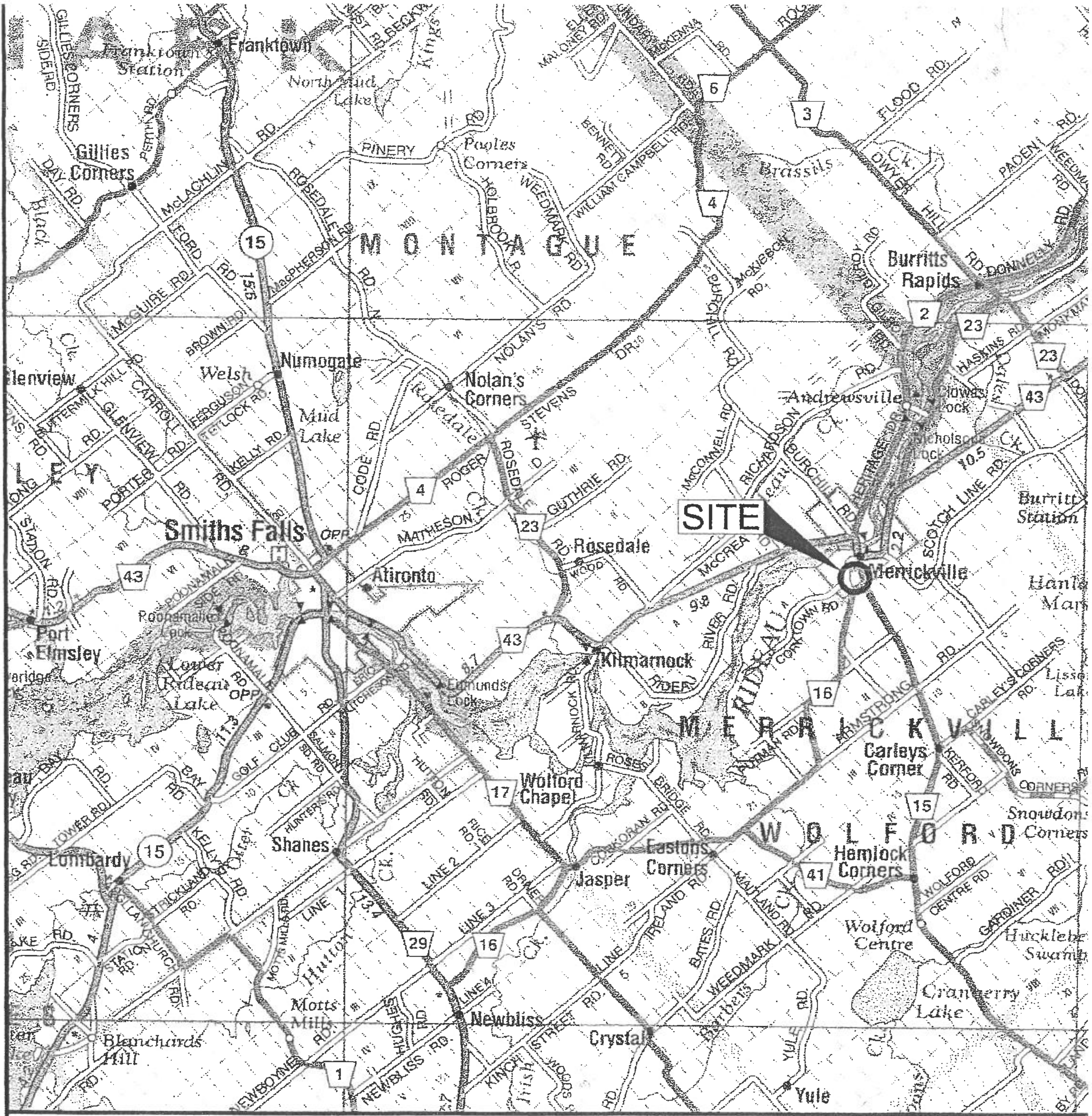
During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and 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.

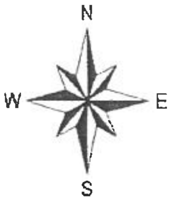
Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided 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.

TABLE 1
RECORD OF TEST PITS

Test Pit Number (Local Elevation m)	Depth (metres)	Description
TP-113	0.00 – 0.35	Brown sand (TOPSOIL)
(114.51)	0.35 – 0.58	Highly Weathered BEDROCK
		Note: Test pit dry upon completion
TP-114	0.00 – 0.15	Dark brown silty sand (TOPSOIL)
(115.51)	0.15 – 0.20	Highly weathered BEDROCK
		Note: Water entering test pit from ground surface (area of pooled water)
TP-115	0.00 – 0.15	Dark brown sandy silt (TOPSOIL)
(116.31)	0.15 – 0.67	Brown sand and gravel with cobbles, completely weathered BEDROCK
	0.67	Highly weathered BEDROCK
		Note: Test pit dry upon completion
TP-117	0.00 – 0.3	Dark brown fine SAND
(115.53)	0.3 – 0.63	Highly weathered BEDROCK
	0.63	Moderately Weathered BEDROCK
		Note: Test pit dry upon completion
TP-118	0.00 – 0.22	Dark brown sandy silt with root matter (TOPSOIL)
(115.61)	0.22 – 0.42	Highly weathered BEDROCK
	0.42	Moderately Weathered BEDROCK
		Note: Test pit dry upon completion
TP-119	0.00 – 0.28	Dark brown sandy silt (TOPSOIL)
(115.35)	0.28 – 0.43	Brown sand, some gravel and cobbles (GLACIAL TILL)
	0.43 – 0.71	Highly weathered BEDROCK
		Note: Test pit dry upon completion





SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT

Golder Associates
Ottawa, Ontario

SCALE	1:200,000
DATE	NOV. 2006
DESIGN	
CADD	N.B.H.S.
CHECK	J.A.S.
REVIEW	G.S.W.

FILE No 061120338-1000-01.dwg

PROJECT No 06-1120-338 REV. 0

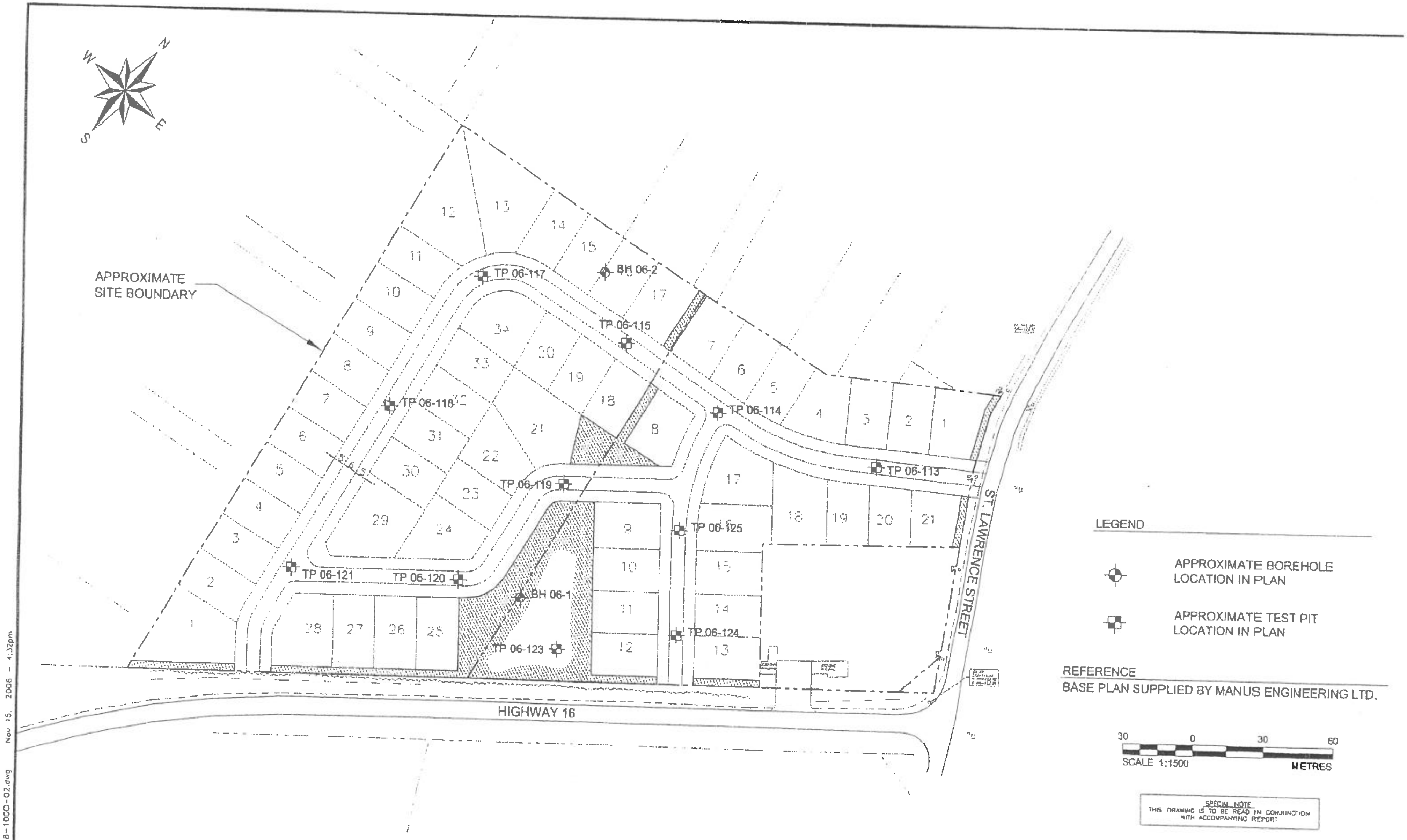
TITLE

KEY PLAN

GEOTECHNICAL INVESTIGATION - McLEAN LANDING,
MERRICKVILLE, ONTARIO

FIGURE 1

Drawing file: 061120338-1000-01.dwg Nov 08, 2006 - 2:57pm



Golder Associates
Ottawa, Ontario

SCALE	1:1500	TITLE	
DATE	NOV.2006		
DESIGN			**SITE PLAN**
CADD	N.B.H.S.		
CHECK	J.A.S.		
REVIEW	G.S.W.	GEOTECHNICAL INVESTIGATION - McLEAN LANDING, MERRICKVILLE, ONTARIO	FIGURE 2

FILE No. 061120338-1000-02.dwg
PROJECT No. 06-1120-338 REV. 0

Drawing file: 061120338-1000-02.dwg Nov 15, 2006 - 4:32pm

APPENDIX A

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

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.)

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	**$C_{u2}S_u$**	
	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_l	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_l	liquid limit
w_p	plastic limit
I_p	plasticity Index=(w_l-w_p)
w_s	shrinkage limit
I_L	liquidity index=(w-w_p)/I_p
I_c	consistency index=(w_l-w)/I_p
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index-(e_{max}-e)/(e_{max}-e_{min}) (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (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. $\tau=c'\sigma'$ tan |'

2. Shear strength=(Compressive strength)/2

PROJECT: 06-1120-338

RECORD OF BOREHOLE: BH 06-1

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: 2 November 2006

DATUM: Local

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760m

BORING METHOD: Portable Rotary Drill, N RC

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	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	PIEZOMETER OR STANDPIPE INSTALLATION
Ground Surface	115.85							
Dark brown sandy silt (TOPSOIL)	115.70 0.15	1	50 DO	1				Native Backfill
Weathered grey DOLOMITIC LIMESTONE BEDROCK	115.40 0.45	2	N RC	DD	92	56	0	
Fresh grey DOLOMITIC LIMESTONE BEDROCK with occasional horizontal seams, white calcite infilling and black shale partings		3	N RC	DD	90.5	83	78	Bentonite Seal
		4	N RC	DD	97	97	63	Silica Sand
		5	N RC	DD	100	98	73	▽ 32mm Diam. PVC #10 Slot Screen
	111.38	6	N RC	DD	100	100	88	
End of Borehole	4.47							Water level in well screen at elev. 112.92m on November 8, 2006.

*** Note : Blow counts corrected to standard hammer energy ***

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: J.A.S.
CHECKED: J.A.S.

PROJECT. 06-1120-338

RECORD OF BOREHOLE: BH 06-2

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: 3 November 2006

DATUM: Local

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 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)			
		Ground Surface		116.84									
0	Portable Rotary Drill N RC	Dark brown sandy silt with gravel and cobbles (TOPSOIL)		116.64 0.20	1	50 DO	6						
		Grey fractured and broken highly weathered DOLOMITIC LIMESTONE BEDROCK with sand and gravel infilling			2	N RC	DD	63	38	0			
1					3	N RC	DD	62	23	0			Bentonite Seal
					4	N RC	DD	57	0	0			
		Fresh grey DOLOMITIC LIMESTONE BEDROCK		115.32 1.52	5	N RC	DD	56	0	0			
2					6	N RC	DD	90	90	48			
					7	N RC	DD	89	89	56			Silica Sand
3				113.03	8	N RC	DD	92	92	29			∇ 20mm Diam. PVC #10 Slot Screen
		End of Borehole		3.81									Water level in standpipe at elev. 113.85m on November 8, 2006.
4													
		*** Note : Blow counts corrected to standard hammer energy.***											
5													
6													
7													
8													
9													
10													

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: J.A.S.

CHECKED J.A.S.

APPENDIX B

RESULTS OF CHEMICAL ANALYSIS
ACCUTEST LABORATORIES LTD. REPORT NO.2624218

ACCUTEST LABORATORIES LTD

REPORT OF ANALYSIS

Client: Golder Associates Ltd
32 Steacie Drive
Kanata, ON
K2K 2A9
Attention: Ian Alfonso

Report Number: 2624218
Date: 2006-11-21
Date Submitted: 2006-11-14

Project: 06-1120-339

P.O. Number:
Matrix: Soil

		LAB ID:	505374					GUIDELINE		
		Sample Date:	2006-11-01							
		Sample ID:	T.P 06-125 Soil 260-1300mm							
PARAMETER	**UNITS**	**MDL**						TYPE	LIMIT	UNITS
Chloride	%	0.001	0.002							
Electrical Conductivity	mS/cm	0.01	0.07							
pH			8.0							
Sulphate	%	0.01	0.01							

MDL = Method Detection Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

Comment:

APPROVAL:
Lorna Wilson
Agriculture Lab Supervisor