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REPORT ON

TERRAIN EVALUATION
PROPOSED RESIDENTIAL SUBDIVISION
PART OF LOTS 29 AND 30
CONCESSION 3, FORMERLY THE CITY OF GLOUCESTER
NOW IN THE CITY OF OTTAWA, ONTARIO

Submitted to:

Emerald Creek Properties
R.R. #2
Ashton, ON
K0A 1B0

DISTRIBUTION:

4 copies - Emerald Creek Properties.
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April 2005

04-402

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1.0 INTRODUCTION

Morey Houle Chevrier Engineering Ltd. was retained by Emerald Creek Properties to conduct a terrain evaluation at the site of a proposed residential subdivision located on Part of Lots 29 and 30, Concession 3, in the City of Ottawa (formerly in the City of Gloucester), Ontario (see Key Plan, Figure 1).

The site consists of a 71.7 hectare (177 acre) parcel of land, which is to be developed as eighty-nine (89) residential lots with internal roadways. The residential lots have an average size of about 0.30 hectares (0.75 acres). The single family dwellings will be serviced by private wells and septic systems. The roadways have been constructed and geotechnical inspection and testing reports on the roadway construction have been provided in separate reports.

The land use in the area of the site is a mixture of undeveloped forested land and vacant open land. There are some scattered existing residential developments in the area of this site. The site is bounded on the north by vacant lands, on the east by Albion Road and scattered existing industrial development beyond, on the south by Mitch Owens Road and existing residential development beyond and on the west by mostly vacant land (see Site Plan, Figure 2).

Based on examination of published surficial geology maps, the site is indicated to be underlain by deposits of fine to medium sand as near shore deposits and glaciofluvial deposits. Based on examination of published bedrock geology maps, the surficial deposits have a thickness in the range of about 5 to 15 metres and are underlain by dolostone of the Oxford Formation.

The investigation of this site was carried out in stages. The results of the first stage of the investigation were provided in the report prepared by Morey Houle Chevrier Engineering Ltd. dated April 4, 2005. The results of the additional investigation work are incorporated into this report. It is understood that a hydrogeological investigation for the site has been carried out by Paterson Group Inc. The results of that study have been provided in a separate report.

The objectives of this terrain evaluation are as follows:

- To identify and characterize the shallow subsurface conditions as they relate to the design of septic sewage disposal systems under the Ontario Building Code (OBC).

- To provide geotechnical engineering guidelines relative to the proposed site development including those items requested by the City of Ottawa.

2.0 TERRAIN EVALUATION

2.1 Field Procedures

The field work for the terrain evaluation was carried out in stages. On March 17, 2005, eight (8) test pits, numbered 1 to 8 inclusive were excavated at the site using a track mounted excavator supplied and operated by Cavanagh Construction Ltd. The subsurface conditions encountered in the test pits were identified by visual and tactile examination of the materials exposed on the sides and bottom of the test pits and from the excavated materials. Where silty clay soils were encountered, vane shear strength testing was carried out on excavated samples of this material within the bucket of the excavator to estimate the undrained shear strength of this material. The groundwater conditions were observed and recorded in the open test pits during the relatively short period of time that the test pits were left open. The test pits were backfilled with the excavated materials and tamped with the bucket of the excavator during backfilling.

On April 6, 2005, three (3) boreholes numbered 1 to 3, inclusive were advanced at the site using a track mounted hollow stem auger drill rig owned and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of between about 4.4 and 5.2 metres. Standard penetration tests were carried out in the boreholes at regular depth intervals and samples of the soils encountered were recovered using drive open sampling equipment. In situ vane shear strength tests were carried out in the grey silty clay, where possible, to determine the undrained shear strength of this material.

The field work for both the test pits and boreholes were supervised throughout by a member of our engineering staff, who directed the excavating and drilling operations and logged the test pits and boreholes.

A description of the subsurface conditions encountered in each of the test pits and boreholes is provided in the Record of Test Pits and Borehole sheets following the text of this report.

The locations of the test pits and boreholes were determined by Morey Houle Chevrier Engineering Ltd. with reference to existing lot lines on a plan which was provided to us and were also measured using Global Positioning Equipment (GPS). The locations of the test pits and boreholes are shown on the attached Site Plan, Figure 2.

2.2 General

As previously indicated, the soil and groundwater conditions encountered in the test pits and boreholes are described in the Record of Test Pit and Borehole sheets following the text of this report. The logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and may have been interpreted. Subsurface conditions at other than the test pit/borehole locations may vary from the conditions encountered in the test pits/boreholes. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and Morey Houle Chevrier Engineering Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

2.3 Soil and Groundwater Conditions

The following presents an overview of the subsurface conditions encountered in the test pits and boreholes advanced for this investigation.

2.3.1 Topsoil

A layer of topsoil was encountered from ground surface in all of the boreholes and test pits except for test pit 7. The topsoil has a thickness ranging from about 0.2 to 0.4 metres. At the location of test pit 7 a layer of former topsoil was encountered below a layer of fill material. The former topsoil layer has a thickness of about 0.15 metres.

2.3.2 Fill Material

Test pit 7 encountered a deposit of fill material from ground surface. The fill material consists of blast rock with some organic material and has a thickness of about 0.4 metres.

2.3.3 Sand and Silty Sand

Deposits of fine to medium sand and silty sand were encountered below the topsoil in all eight (8) test pits and the boreholes. The thickness of the sand and silty sand deposits ranges from about 0.7 to 1.9 metres. At test pit 7 and borehole 3, the fine to medium sand is underlain by fine to coarse sand, containing some shells. Test pit 7 was terminated in the fine to coarse sand at a depth of 3.9 metres.

2.3.4 Clayey Silt

A layer of clayey silt with trace gravel was encountered in test pit 3 at a depth of about 1.3 metres below ground surface. At the test pit location the clayey silt layer has a thickness of about 0.2 metres.

2.3.5 Sensitive Silty Clay

Deposits of sensitive, grey brown to grey silty clay were encountered below the sand and silty sand deposits in each test pit, with the exception of test pit 7 and in all of the boreholes. At test pits 4 and 8, and boreholes 1 and 2, the upper 0.6 to 1.0 metres of the silty clay deposit is weathered grey brown. Test pits 1 to 6, inclusive and test pit 8 were terminated within silty clay at depths ranging from about 2.8 to 4.5 metres below ground surface. Due to groundwater inflow and caving of the sides of the test pits, in situ vane shear tests on the silty clay deposit could not be carried out. Instead, vane shear strength tests were carried out on excavated samples of the material within the bucket of the excavator to estimate the undrained shear strength of the silty clay deposits in test pits 2, 5, 6 and 8. Based on this testing, the undrained shear strengths range from 23 to 58 kilopascals, which reflect a variable, soft to stiff consistency. The low (i.e. soft) test results could be due to disturbance of the material caused by the excavator.

In situ vane shear strength tests were carried out within the grey silty clay in boreholes 1 and 3 and gave undrained shear strength values ranging from 34 to 67 kilopascals. These tests indicate that the grey silty clay has a firm to stiff consistency. The remoulded vane shear test values ranged from 6 to about 17 kilopascals. The low remoulded vane shear values reflect the highly sensitive nature of the silty clay deposit.

2.3.6 Groundwater

Groundwater inflow to the test pit ranged from about 0.8 to 2.1 metres below ground surface on March 17, 2005. Water was ponded at the ground surface at the borehole locations on April 6, 2005.

It is pointed out that the recorded groundwater levels were observed in the open test pits during the relatively short period of time that the test pits were left open and, therefore, may not represent stabilized conditions.

It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

2.4 Class IV Septic Sewage Disposal Systems

This section discusses the results of the terrain evaluation as they relate to the feasibility of installing Class IV septic sewage disposal systems within the proposed residential subdivision.

2.4.1 Septic Envelopes

The septic system envelope area (septic envelope) represents the area on a lot set aside for the construction of the leaching bed and is for the leaching bed only. It does not include that area required for the septic tank or the isolation/separation distances required by the Ontario Building Code (OBC).

The size of the septic envelope is a function of the percolation time (T-time) of the native soil in the vicinity of the septic envelope or the fill used for the construction of a septic bed and the daily effluent loading to the septic bed. The test pits indicate that the shallow subsurface at this site is characterized primarily by deposits composed of fine to medium sand and silty sand. Based on our experience, the native sand deposits can be expected to have a T-time of about 6 to 8 minutes per centimetre.

In accordance with the OBC, for soils with a T time between 1 and 20 minutes per centimetre, the maximum permissible loading rate is 10 litres per square metre per day (L/m^2/day). The OBC also requires that the upper 0.25 metres of soil (soil mantle) underlying the leaching bed and extending at least 15 metres beyond the outer distribution pipes in any direction in which the effluent entering the soil or leaching bed will migrate horizontally, have a T time of not less than 75 percent of the percolation time of the unsaturated soil or leaching bed fill. In view of the percolation time of the native sand at this site, a sand mantle will likely not be required.

As a conservative approach to calculating the maximum expected septic system envelope required to service a single family dwelling at this site, a septic system envelope size was calculated using a daily sewage flow of 3500 litres. A daily design flow of 3500 litres per day is suitable for a five bedroom dwelling with 300 square metres (m^2) of finished area and 40 fixture units. The septic envelope area required for a daily sewage flow of 3500 litres and a loading rate of 10 l/m^2/day for the native sand is approximately 350 m^2, or about 10 percent of the area of the proposed lots. This septic system envelope should be readily accommodated on the lot sizes that are proposed for this subdivision (i.e. approximately 0.3 hectares).

Prior to establishing the actual septic envelope (leaching bed) location on any particular lot, test holes should be excavated to determine the actual subsurface conditions in the area of the proposed leaching bed. The design and construction of individual septic disposal systems on the proposed lots should be carried out in accordance with the requirements in the OBC.

2.4.2 Leaching Bed Design Considerations

The design of septic leaching beds involves a combination of a number of interrelated factors, including the volume of effluent discharged to the system, properties of the soil materials used in the construction of the leaching bed, length of distribution lines in the leaching bed and the subsurface conditions in the area of the leaching bed. The construction of individual septic disposal systems within the proposed residential subdivision should be carried out in accordance with the requirements in the OBC.

Within most of the site, the design must ensure that the bottom of the absorption trenches is at least 0.9 metres above silty sand and silty clay soils, and at least 0.9 metres above the seasonally high groundwater table. Based on the soil and groundwater conditions encountered in the test pits, it is expected that the septic leaching beds could be constructed in-ground or partially raised and comply with the required separation distance between the underside of the absorption trench and the seasonally high groundwater table and/or low permeability soils.

Any imported sand for the leaching beds should have a percolation time (T time) of between 4 and 8 minutes per centimetre.

The OBC requires that the upper 0.25 metres of unsaturated soil (soil mantle) underlying the leaching bed and extending at least 15 metres beyond the outer distribution pipes in any direction that effluent may migrate have a percolation time between 1 and 50 minutes per centimetre for Class IV leaching beds. The OBC also specifies that where a leaching bed is constructed in unsaturated soil having a percolation time of greater than 15 minutes per centimetre, any fill used in the construction of the leaching bed must have a percolation time not less than 75 percent of the percolation time of the unsaturated soil. The thickness of unsaturated soil in the downgradient direction from the leaching bed should be investigated on a lot-specific basis to determine whether the 0.25 metre unsaturated depth requirement can be met with native soils. Based on the test pits that were advanced at the site, it is considered that an imported sand mantle will not likely be required.

2.4.3 Tertiary Septic Systems

Approved septic disposal systems that meet the OBC requirements for tertiary treatment could also be considered for this development. The disposal beds for tertiary treatment systems require a smaller area than those for conventional Class IV septic systems. Furthermore, the required separation distance between the underside of the crushed stone layer in the disposal bed and low permeability soils or the seasonally high groundwater table is 0.6 metres (compared to 0.9 metres for conventional septic systems).

3.0 GEOTECHNICAL CONSIDERATIONS

3.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the test pit/borehole information, and the project requirements. It is stressed that the information in the following sections is provided for the guidance of the designers and is intended for this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from off site sources are outside the terms of reference for this report and have not been investigated or addressed.

3.2 Residential Dwellings

3.2.1 Foundation Design

In general, the native sand, silty sand and silty clay deposits are considered suitable for the support of residential structures on conventional spread footing foundations. The sand and silty sand deposits below the groundwater level may become disturbed during excavation and may not provide suitable support. To avoid problems associated with the construction of foundations in sandy soils below the groundwater level and to reduce long term groundwater inflow into the sump pits for the houses, it is suggested that, where possible, the underside of the footings be planned to be at least 0.15 metres above the seasonally high groundwater level. Alternatively, the groundwater could be lowered in advance of excavation by means of ditches/drainage swales. It is pointed out that the lowering of the groundwater on this site by means of ditches/swales could take some time, depending on the spacing of the ditches.

The excavations for the foundations should be taken through any surficial fill, topsoil, organic soils, or otherwise deleterious material to expose undisturbed native soil.

The allowable bearing pressure for foundations depends on a number of interrelated factors, including the relative density of the sand deposits, the undrained shear strength of the underlying grey silty clay, the amount of grade raise fill around and below the proposed house, the depth of the grey silty clay below the underside of the spread footing, the amount of groundwater level lowering that may occur as a result of development, the size and type (exterior or interior) of spread footing, etc.

For preliminary planning purposes, spread footings which are founded on or within the native sand deposits above the groundwater level could be sized using an allowable bearing pressure of 90 kilopascals. This is based on the following:

- Footings founded at 1.0 to 1.5 metres depth;
- Maximum width of footings, 0.5 metres;
- Depth of groundwater, 2.0 metres (i.e. lowered by about 1 metre as a result of development)
- Maximum exterior grade raise around the foundations, 2.0 metres.

The above is considered to be reasonable parameters for the development of this site. The allowable bearing pressure is provided for preliminary design purposes only. The subsurface conditions could vary beyond the test locations. As such, site specific geotechnical investigations should be carried out on a lot by lot basis by a geotechnical engineer. There may be grade raise restrictions required in other areas of the site due to the presence of localized deposits of silty clay having a soft consistency. This in turn could affect the house foundation design, site grading and septic system design. For example, septic systems with tertiary treatment combined with a small area bed may be required in areas where grade raise restrictions are required for foundation design purposes.

In any areas where proposed founding level is above the level of the native, inorganic deposits or where subexcavation of disturbed material is required below proposed founding level, imported granular material (engineered fill) should be used. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.3 metres beyond the outside edges of the footings and then down and out from this point at 1 horizontal to 1 vertical, or flatter. The excavations for the residential dwellings should be sized to accommodate this fill placement. The granular materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density.

Currently, OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A and B Type II materials. Since the source of recycled material cannot be determined, it is suggested that any granular materials used below founding level be composed of virgin material only.

All exterior footings and those in any unheated parts of the structures should be provided with at least 1.5 metres of earth cover for frost protection purposes. If 1.5 metres of earth cover for foundations is not practicable, a combination of earth cover and polystyrene insulation could be considered.

Groundwater inflow from the native soils into basement excavations during construction should be handled by pumping from sumps within the excavations on an as required basis.

3.2.2 Basement Foundation Wall Backfill and Drainage

In accordance with Section 9 of the Ontario Building Code, the following alternatives could be considered for drainage of the basement foundation walls:

- Damp proof the exterior of the foundation walls and backfill the walls with free draining, non-frost susceptible sand or sand and gravel, such as that meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type I. OR

- Install an approved proprietary drainage material (such as System Platon) on the exterior of the foundation walls and backfill the walls with native material or imported soil.

A perforated drain should be installed around the basement area at the level of the bottom of the footings. The drain should outlet to a sump from which the water is pumped or should drain by gravity to a suitable outlet.

3.2.3 Garage Foundation Backfill

To avoid adfreeze between the unheated garage foundation walls and the wall backfill, the interior and exterior of the garage foundation walls should be backfilled with free draining, non-frost susceptible sand or sand and gravel such as that meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type I. The sand backfill within the garage should be compacted in maximum 300 millimetres thick lifts to at least 95 percent of the standard Proctor dry density value using suitable vibratory compaction equipment. Alternatively, suitable water sluicing methods would be acceptable.

3.2.4 Effects of Trees on Foundations

Sensitive silty clay was encountered in all of the test pits and boreholes, with the exception of test pit 7. The silty clay is known to be susceptible to shrinkage with reductions in moisture content. Research published by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada indicates that the shrinkage induced near trees of high water demand in the Ottawa area can result in settlement of nearby buildings founded on shallow (conventional) foundations. This previous work showed that deciduous trees, such as maple, ash, willow, poplar, etc., can cause settlement related distress when they are located within a horizontal distance equivalent to the height of the tree from a foundation wall; for multiple trees, the zone of influence of the trees to the house increases to about 1.5 times the trees' height. The volume of soil affected by trees increases continuously as they grow, and settlement problems may only appear after a number of years, when the trees mature and the water requirements of the trees exceed available supplies.

The future effects of trees on the houses should be considered in landscaping the lots.

3.3 Slopes

There are no areas of slope stability concern on this site. In general, slopes which are created by the placement of fill should be sloped at 3 horizontal to 1 vertical, or flatter for maintenance purposes.

3.4 Culverts

The installation of culverts for driveways should in accordance with City of Ottawa standards. The culverts should be provided with a minimum 150 millimetre thick bedding layer of crushed stone meeting OPSS requirements for Granular A. Cover material for the culverts could consist of OPSS Granular A or Granular B Type Type II.

3.5 Pavement Structure

As stated earlier, the internal roadways within this subdivision have been constructed. The roadway design is as follows:

80 millimetres of hot mix asphaltic concrete
(40 millimetres of HL3 over 40 millimetres of HL8)
150 millimetres of OPSS Granular A base, over
500 millimetres of OPSS Granular B Subbase, over
native, undisturbed sandy soil

This pavement structure is considered adequate for this site.

3.6 Grade Raise Fill

Fill material for site grading purposes should consist of environmentally clean, inorganic material, such as sand, sand silt mixtures, or silty clay. Compaction of fill material which is used for lot grading purposes is not considered necessary, however to minimize future settlement of the fill materials, nominal compaction using spreading equipment such as small bulldozers is suggested.

3.7 Site Dewatering

As stated in Section 3.2.1, Foundations, it is expected that the development of this site will result in a gradual lowering of the groundwater levels to a depth of about 2.0 metres below the present ground surface. Further lowering of groundwater levels is not anticipated, or considered necessary.

3.8 Pools

There are no geotechnical constraints for the installation of in-ground or above ground swimming pools at this site. In-ground pools should be located above the groundwater level or designed to resist the applicable hydrostatic pressures caused by the groundwater.

4.0 CONSTRUCTION CONSIDERATIONS AND OBSERVATIONS

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed development do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

The native soils at this site will be sensitive to construction operations at and below the groundwater level and from ponded water and frost. The construction operations should therefore be carried out in a manner that will prevent disturbance of the subgrade surfaces.

Footing surfaces and any engineered fill areas for the residences should be inspected by qualified geotechnical personnel to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of granular materials beneath the foundations should be inspected to ensure that the materials used conform to the grading and compaction specifications.

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

Yours truly,

MOREY HOULE CHEVRIER ENGINEERING LTD.

ORIGINAL SIGNED BY

B. D. Wiebe, P.Eng.

ORIGINAL SIGNED BY

A. C. Houle, P.Eng.
Principal

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
MS	manual sample
RC	rock core
ST	slotted tube
TO	thin-walled open Shelby tube
TP	thin-walled piston Shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance, N

The number of blows by a 63.5 kg hammer dropped 760 millimetres required to drive a 50 mm drive open sampler for a distance of 300 mm. For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

Dynamic Penetration Resistance

The number of blows by a 63.5 kg hammer dropped 760 mm to drive a 50 mm diameter, 60^{o} cone attached to 'A' size drill rods for a distance of 300 mm.

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drill rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	sieve and hydrometer analysis
U	unconfined compression test
Q	undrained triaxial test
V	field vane, undisturbed and remoulded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	over 100

LIST OF COMMON SYMBOLS

c_u	undrained shear strength
e	void ratio
C_c	compression index
c_v	coefficient of consolidation
k	coefficient of permeability
I_p	plasticity index
n	porosity
u	pore pressure
w	moisture content
w_L	liquid limit
w_P	plastic limit
ϕ^1	effective angle of friction
γ	unit weight of soil
γ^1	unit weight of submerged soil
σ	normal stress

RECORD OF BOREHOLE 1

PROJECT: 04-402
LOCATION: Refer to Site Plan, Figure 2
BORING DATE: April 6, 2005

SHEET 1 OF 1
DATUM:
SPT HAMMER: 63.6 kg; drop 0.76 m

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^{-7} 10^{-6} 10^{-5} 10^{-4}) / WATER CONTENT, PERCENT Wp — W — Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger, 200 mm Diameter Hollow Stem	Ground Surface									
		TOPSOIL									
		Grey brown fine to medium SAND, trace silt		0.30							
1					1	50 DO	4				
		Grey brown SILTY CLAY (weathered crust)		1.30							
2					2	50 DO	6				
		Firm to stiff grey SILTY CLAY		2.29	3	50 DO	1				
3											
4											
5		Stiff grey CLAYEY SILT		4.88							
		End of borehole		5.18							Ponded water at borehole location on April 6, 2005.
6											

DEPTH SCALE
1 to 30

Houle Chevrier Engineering Ltd.

LOGGED: BW
CHECKED:

RECORD OF BOREHOLE 2

PROJECT: 04-402

LOCATION: Refer to Site Plan, Figure 2

BORING DATE: April 6, 2005

SHEET 1 OF 1

DATUM:

SPT HAMMER: 63.6 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	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^{-7} 10^{-6} 10^{-5} 10^{-4}); WATER CONTENT, PERCENT Wp —○W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface									
		TOPSOIL									
		Grey brown fine to medium SAND, trace silt		0.20							
1					1	50 DO	7				
		Grey brown SILTY CLAY (weathered crust)		1.52							
2					2	50 DO	9				
		Stiff to very stiff grey SILTY CLAY		2.29							
3					3	50 DO	15				
		Stiff to very stiff grey CLAYEY SILT		3.05							
		Grey SILTY SAND/SANDY SILT		3.35	4	50 DO	5				
4					5	50 DO	31				
		End of borehole		4.42							Ponded water at borehole location on April 6, 2005.
5											
6											

DEPTH SCALE

1 to 30

Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED:

RECORD OF BOREHOLE 3

PROJECT: 04-402

LOCATION: Refer to Site Plan, Figure 2

BORING DATE: April 6, 2005

SHEET 1 OF 1

DATUM:

SPT HAMMER: 63.6 kg; drop 0.76 m

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa (rem. V ⊕ / nat. V +)	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface						
	TOPSOIL						
	Grey brown fine to medium SAND, trace silt	0.20					
1			1	50 DO	11		
			2	50 DO	12		
2	Grey fine to coarse SAND, trace silt, some shells	1.98					
			3	50 DO	PH		
3	Firm to stiff grey SILTY CLAY	3.05					
			4	50 DO	1		
4							
5	End of borehole	5.18					Ponded water near borehole location on April 6, 2005.
6							

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^{-7} 10^{-6} 10^{-5} 10^{-4}

WATER CONTENT, PERCENT: Wp W Wl: 20 40 60 80

ADDITIONAL LAB. TESTING

DEPTH SCALE

1 to 30

Houle Chevrier Engineering Ltd.

LOGGED: BW

CHECKED:

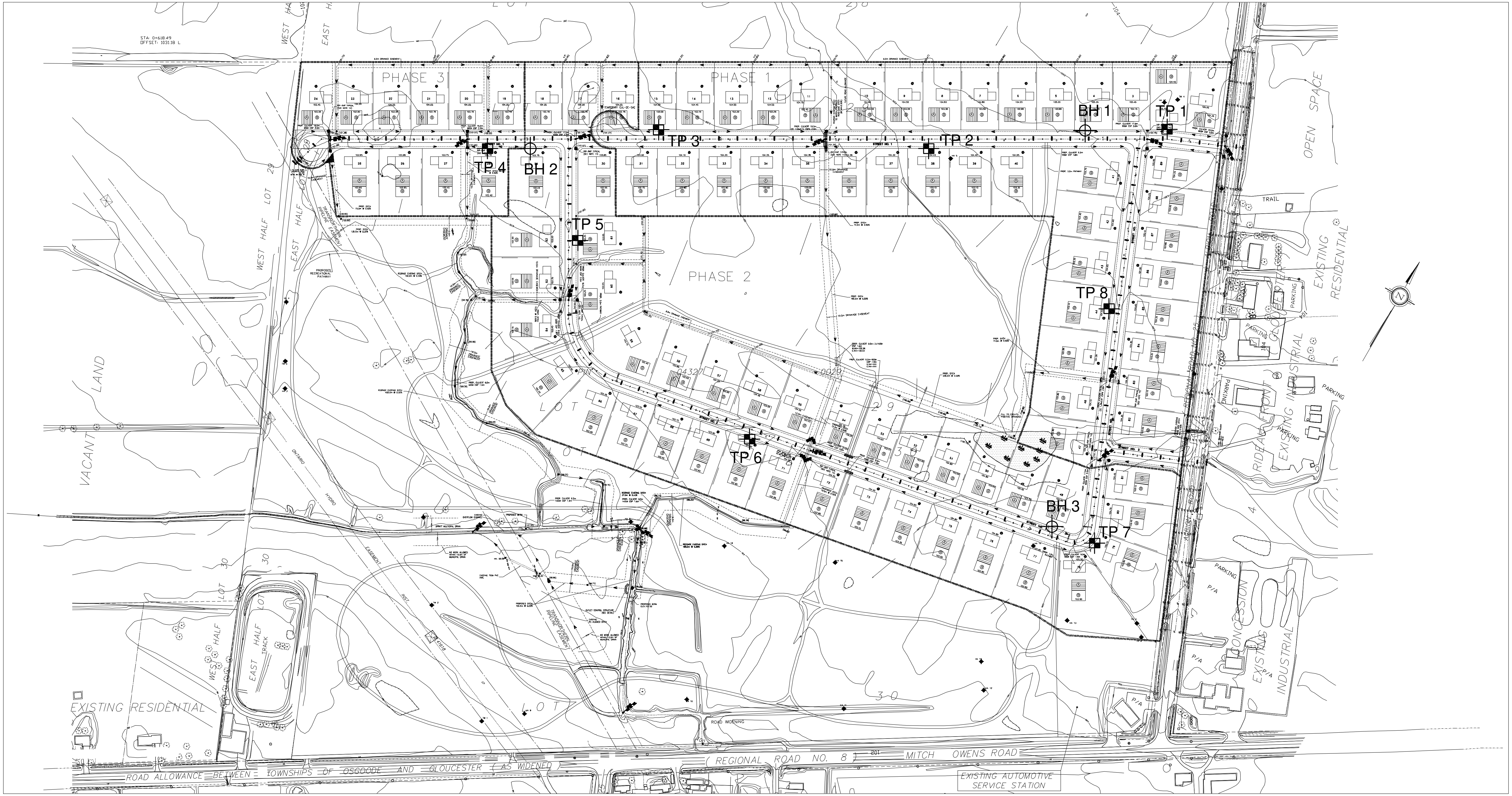
KEY PLAN	FIGURE 1

SCALE
1:250, 000

MOREY
HOULE
CHEVRIER
ENGINEERING LTD.

Date: April 2005

Project: 04-402



LEGEND

TP # APPROXIMATE TEST PIT LOCATION IN PLAN, PRESENT INVESTIGATION BY MOREY HOULE CHEVRIER ENGINEERING LTD.

BH # APPROXIMATE BOREHOLE LOCATION IN PLAN, PRESENT INVESTIGATION BY MOREY HOULE CHEVRIER ENGINEERING LTD.

REFERENCE: PLAN PREPARED USING SITE PLAN PROVIDED BY STANTEC CONSULTING LTD.

Client	Location	Revision
EMERALD CREEK PROPERTIES	ALBION RD., OTTAWA, ON	0

Drawn by	Approved by	Project No.	Scale
M.R.	C.H.	04-402	1 : 4000

MOREY HOULE CHEVRIER
ENGINEERING LTD

Sheet Title: SITE PLAN

Date: APRIL 4, 2005

FIGURE 2