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REPORT ON

SITE DEVELOPMENT CONSIDERATIONS GEOTECHNICAL STUDY AND SERVICING OPTIONS 5630 BOUNDARY ROAD OTTAWA, ONTARIO

Submitted to:

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February 2013

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at 5630 Boundary Road in Ottawa, Ontario and, based on the factual information obtained, provides general comments regrading the suitability of the site for development. This report is intended for site rezoning purposes only and, as such, specific design guidelines could be provided in a separate report as the development plans progress.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Site Description

The proposed development site is located at 5630 Boundary Road in Ottawa, Ontario (see Key Plan, Figure 1). The site is currently vacant; however, a north-south aligned gravel surfaced access roadway currently extends through the site. The southeast portion of the site is treed; the northwest portion of the site has been cleared. Imported fill material has been placed over portions of the site west of the existing gravel surfaced access roadway.

2.2 Previous Investigation

A previous geotechnical investigation was carried out at the site by St. Lawrence Testing and Inspection Co. Ltd. (SLTI) on April 7, 2010. The findings of the SLTI ground investigation are described in their report titled, "Property at County Roads 8 & 41, Edwards, ON, Geotechnical Subsurface Investigation, Report No. 10C75", dated April 27, 2010.

As part of the previous subsurface investigation carried out by SLTI, a total of four (4) boreholes were advanced at the site. The boreholes were advanced to depths ranging from about 3.7 to 22.6 metres below ground surface. Standard penetration tests were carried out in the upper 2.2 to 3.7 metres of the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter drive open sampler. Dynamic cone penetration testing was carried out in two (2) of the boreholes from about 2.2 metres to about 21.0 and 22.6 metres below ground surface. Vane shear testing was not carried out by SLTI within the silty clay layer. In addition, laboratory oedometer consolidation testing of the silty clay layer was not carried out.

The subsurface conditions encountered in the SLTI boreholes consisted of a surficial topsoil layer generally underlain by silty sand followed by sensitive silty clay deposits. At between about 18.3 and 18.9 metres below ground surface, the number of blows required to advance the cone during the dynamic cone penetration tests increased from 3 to between 20 and 113 blows per 0.3 metres of penetration. These results reflect a material having a dense to very dense relative density and/or the presence of cobbles/boulder obstructions.

2.3 Canadian Geotechnical Research Site No. 1

In March 1955, the National Research Council of Canada began measuring settlement of the new Accommodation Building constructed at HMCS Gloucester. The site, known as Canadian Geotechnical Research Site No. 1, is located just north of Mitch Owens Road approximately 7.4 kilometres west of the proposed commercial development. One of the objectives of this research was to obtain a long term performance record of a light structure constructed on a very compressible, near normally consolidated clay soil. The Accommodation Building was built in August 1954. The structure consisted of slab on grade construction with footings on the original ground. The finished floor elevation was about 1.4 metres higher than the original ground surface and the floor slab was supported on imported sand and gravel (Crawford and Bozozuk, 1990). The soil conditions at Geotechnical Research Site No. 1 are similar to those at this site.

Survey points were established around the exterior of the building and in a grid on the floor slab. The survey points were referenced to a benchmark set on bedrock. The initial survey was carried out on March 3, 1955. After 1 year, the slab had settled about 70 millimetres. Twenty years later the slab had a settled 300 millimetres, increasing to 330 millimetres after 33 years (Crawford and Bozozuk, 1990).

2.4 Review of Geology Maps

Published geology maps of the area indicate that the subsurface conditions are expected to consist of overburden deposits of sand underlain by marine deposits of silty clay. The bedrock is mapped as shale of the Carlsbad formation at depths of between 15 and 25 metres.

3.0 GEOTECHNICAL INVESTIGATION

3.1 Subsurface Investigation

The field work for this investigation was carried out on June 9 and 19, 2010. During that time, five (5) boreholes, numbered 10-1 to 10-5, inclusive, were advanced using a hollow stem auger drill rig supplied and operated by George Downing Estate Drilling of Ottawa, Ontario. The boreholes were advanced at the following locations:

- Two (2) boreholes, numbered 10-1 and 10-2, in the area of the proposed building to about 6.7 and 15.9 metres below ground surface for foundation design purposes, to assess the potential for liquefaction of the overburden deposits and to identify the seismic Site Class.
- One (1) borehole, numbered 10-3, in the area of the proposed septic bed to about 2.9 metres below ground surface.
- Two (2) boreholes, numbered 10-4 and 10-5, in the proposed parking area to 2.1 and 2.9 metres below ground surface for pavement design purposes.

Standard penetration tests were carried out in the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter split barrel sampler. In situ vane shear strength testing was carried out where possible in the clayey deposits to measure the undrained shear strength. Well screens were sealed in boreholes 10-1 and 10-2 to measure the groundwater levels.

The field work was supervised throughout by a member of our engineering staff, who located the boreholes, logged the samples and observed the in-situ testing. Following the field work, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content, grain size distribution, and Atterberg limits.

Five (5) relatively undisturbed Shelby tube samples of the softer portions of the clayey deposits were obtained from boreholes 10-1 and 10-2. The samples were recovered using a piston sampler supplied by George Downing Estate Drilling. Laboratory vane shear strength testing was carried out in selected Shelby tube samples. A Shelby tube sample recovered from borehole 10-1 at a depth of about 4.9 metres below ground surface was selected for oedometer consolidation testing, laboratory classification and laboratory vane shear strength testing.

The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 2. Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix A. The results of the soil classification testing from samples recovered from the boreholes are provided on the Record of Borehole sheets and on Figures A1 to A2 in Appendix A.

The borehole locations were determined relative to existing site features by Houle Chevrier Engineering Ltd. personnel. The borehole elevations were measured relative to the top of the culvert located on the subject property south of Mitch Owens Road. The elevation of this point was taken as 100.0 metres, local datum.

3.2 Site Reconnaissance

The boreholes were advanced on June 9 and 19, 2010. In order to visually assess the current conditions at the site (e.g., presence of imported fill material), a site reconnaissance was carried out by a member of our engineering staff on November 14, 2012. During that time, the surficial conditions at the site were observed and photographed.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the soil and groundwater conditions identified in the boreholes are given on the Record of Borehole sheets in Appendix A. The boreholes were advanced within the southwest portion of the site. It should be noted that fill material of unknown physical and chemical composition is present over portions of the site. It is possible that the subsurface conditions at other than the test locations may vary from the conditions encountered in the boreholes. As such, once the site plan has been finalized, we recommend that at least one (1) borehole or test pit be advanced in the area of any proposed buildings to confirm that the subsurface conditions encountered during the present borehole investigation are consistent with the subsurface conditions at the proposed building location. The requirement for an additional investigation should be assessed as the development plans progress.

The borehole 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 have been interpreted. The precision with which subsurface conditions are indicated depends on the method of drilling, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities in the area.

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 Houle Chevrier Engineering Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The following presents an overview of the subsurface conditions encountered during this investigation.

4.2 Fill Material

In borehole 10-5, fill material composed of dark brown silty sand with gravel and clay was encountered from ground surface. The fill material has a thickness of approximately 1.4 metres. A standard penetration test carried out in the fill material gave an N value of 4 blows per 0.3 metres of penetration, which reflects a very loose to loose relative density.

During our site reconnaissance on November 14, 2012, fill material was visually observed over the western portion of the site north of boreholes 10-1 and 10-3, and south of boreholes 10-2 and 10-4. Based on our visual assessment, the fill material has a thickness of about 0.5 to 1.0 metres. The physical and chemical composition of the fill material observed on November 14, 2012 is unknown.

4.3 Silty Sand

A deposit of brown silty sand, ranging in thickness from 0.4 to 1.1 metres, was encountered from ground surface in boreholes 10-1 to 10-4, inclusive, and below the fill material in borehole 10-5. Based on one standard penetration test conducted in borehole 10-5 the silty sand deposit has a loose relative density.

4.4 Silty Clay

Deposits of silty clay were encountered in all of the boreholes beneath the silty sand. The silty clay was not fully penetrated as part of this investigation; however, based on the results of the previous investigation by SLTI, the silty clay deposit is about 18 metres thick.

The upper part of the silty clay is weathered reddish grey brown. Standard penetration tests carried out in the weathered, silty clay gave N values ranging from 5 to 6 blows per 0.3 metres of penetration, which reflect a stiff to very stiff consistency. The weathered silty clay has a thickness of between about 0.6 to 1.4 metres and extends to depths of about 1.8 to 2.4 metres below ground surface (elevation 98.1 to 98.4 metres, local datum).

Below the weathered zone, the silty clay is grey in colour. The grey silty clay contains occasional silty sand and silt seams. Standard penetration tests carried out in the grey silty clay gave N values of "static weight of hammer (WH)" to 2 blows per 0.3 metres of penetration.

In situ vane shear strength tests carried out in the grey silty clay in boreholes 10-1 and 10-2 gave undrained shear strengths of 17 to 46 kilopascals, which indicate a soft to firm consistency. Between 2.6 and 11.6 metres below ground surface, the undrained shear strengths ranged between 17 and 29 kilopascals. Between 11.6 to 15.9 metres below ground surface, the undrained shear strengths steadily increased from 31 to 46 kilopascals. Laboratory vane shear strength tests carried out in the Shelby tube samples of the softest portions of the grey silty clay recovered from boreholes 10-1 and 10-2 gave undrained shear strength values of 17 to 33 kilopascals, which indicate a soft to firm consistency. A summary of the vane shear strength data obtained in the silty clay during this investigation is provided on Figure 3.

The results of one (1) laboratory oedometer consolidation test carried out on the relatively undisturbed thin walled Shelby tube piston sample of the grey silty clay recovered from borehole 10-1 gave the following results:

Borehole No.	Sample Depth, (metres)	Estimated Apparent Past Preconsolidation Pressure, P_c, (kilopascals)	Calculated Existing Vertical Effective Stress, P_o' (kilopascals)	Initial Void Ratio, e_o	Recompression Index, C_r	Compression Index, C_c
10-1	4.6 - 5.2	70	41	0.99	0.04	0.28

A graph of the variation in void ratio with applied stress from the consolidation tests carried out by Houle Chevrier Engineering Ltd. are provided on Figure A3 in Appendix A.

The results of an Atterberg limit test carried out on a sample of the grey silty clay from borehole 10-1 gave a liquid limit of 61 percent, a plastic limit of 25 percent and a corresponding plasticity index of 36 (see Figure A2). The water content of the sample tested was 92 percent, which is much greater than the measured liquid limit value. This testing indicates that the silty clay has high plasticity. A grain size distribution curve for a sample of the silty clay recovered from borehole 10-1 is provided on Figure A1.

4.5 Groundwater Levels

The groundwater level in the well screens installed in boreholes 10-1 and 10-2 were at 0.6 and 1.6 metres below ground surface (elevation 99.3 and 98.3 metres, local datum) on June 17, 2010, respectively. The groundwater conditions at the other borehole locations were not observed.

The groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

5.0 SUITABILITY FOR DEVELOPMENT

5.1 General

This site is underlain by deposits of sensitive silty clay, which have a reduced capacity to support loads imposed by grade raise fill material, pavement structures and foundations for buildings. Provided that the stresses imposed on the underlying silty clay from grade raise filling, foundations, and groundwater lowering are carefully controlled, the site is considered suitable for development from a geotechnical point of view.

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.

The following presents an overview of the possible constraints to development of the site from a geotechnical point of view.

5.2 Excavation

Excavation within silty sand above the groundwater level and within the silty clay above and below the groundwater level should not present any serious constraints. In contrast, excavation below the groundwater level within the silty sand deposit could present some constraints. There is potential for disturbance to the silty sand on the sides and bottom of the excavation and relatively flat side slopes may be required to prevent sloughing of material into the excavation. The groundwater inflow should be controlled throughout the excavation by pumping from sumps within the excavation. Notwithstanding, some disturbance and loosening of the subgrade materials could occur, and allowance should be made for subexcavation of any disturbed subgrade soil.

5.3 Site Grade Raise Restriction

This site is underlain by deposits of sensitive silty clay, which has a reduced capacity to support loads imposed by grade raise fill material, pavement structures and, to a lesser extent, the foundations for buildings. The placement of fill material on this site must therefore be carefully planned and controlled so that the stress imposed by the fill material does not result in excessive consolidation of the silty clay deposit. Concrete slabs, granular base materials, and pavement structures are considered grade raise filling. Groundwater lowering also results in a stress increase on the underlying sensitive silty clay deposit.

Based on the results of the subsurface investigation in conjunction with oedometer testing and empirical calculations correlating the undrained shear strength of the soft to firm silty clay to the preconsolidation pressure, the maximum thickness of any grade raise filling above the original ground surface (i.e. surface of the native soil) should be limited to 0.4 to 0.6 metres, depending on the type of grade raise fill material used.

The grade raise restriction for this site has been calculated in order to limit the total settlement of the foundations to about 50 to 75 millimetres in the long term. If excessive fill material currently exists over portions of the proposed site, consolidation of the silty clay below these areas may have been initiated. In these areas, the excessive fill material should be removed and the impacts on the underlying silty clay, if any, should be assessed by qualified geotechnical personnel.

Alternatives Where Grade Raise Restrictions Cannot be Met

In those areas where the grade raise restrictions cannot be achieved because of other civil engineering design considerations, the following options could be considered:

1) The use of relatively lightweight fill material, such as expanded polystyrene (EPS) blocks which is specifically manufactured for this purpose, could be used to raise the grade. EPS could also be used below any proposed foundations, slabs on grade, and pavement structures. The use of EPS for purposes such as this dates back at least 25 years and has been used locally on many residential projects as well as on the Hunt Club Bridge, Highway 416, and more recently, the sohowest Development at Eagleson Road and Fernbank Road in Ottawa, Ontario. The Ministry of Transportation of Ontario has also used EPS on many projects throughout Ontario. Within areas where EPS is used, there may be some design requirements for landscaping features.

2) Preloading of the site could be considered to allow for the majority of the primary consolidation settlement of the silty clay deposit to occur prior to development of the site. Preloading would require amount of earth fill or crushed stone to be imported to the site. It is recommended that preloading be carried out in conjunction with the use of wick drains to accelerate the rate of settlement. Monitoring of the settlement response of the silty clay deposit along with porewater pressure monitoring would be required to assess when the preloading can be stopped.

5.4 Foundations

Prior to construction of any concrete slabs on grade and foundations, all topsoil, organic material and fill material should be removed from within the proposed building area.

To minimize the stress imposed on the underlying silty clay, any spread footings should be founded at shallow depths and insulated using a layer of extruded polystyrene (for frost protection purposes). Depending on the footing loads, the spread footing sizes may be relatively large. For relatively high footing loads, conventional spread footings may not be feasible. For this case, a raft slab foundation or end bearing piles on sound bedrock could be considered.

The thickness of surcharge (fill, concrete slab on grade etc.) in the vicinity of the foundations should be carefully controlled to limit the stress increase on the softer, compressible grey silty clay to acceptable level such that foundation settlements will not be excessive. A grade raise of more than the allowable limits will require the use of EPS blocks to make up the difference where the depth of the grade raise fill exceeds these limits (if preloading is not carried out). The EPS should extend at least 2.4 metres around the entire perimeter of the foundation.

Although feasible, a basement is not ideally suited for this site from a geotechnical point of view since a conventional drained basement could result in excessive groundwater lowering and possible additional ground settlement due to consolidation of the grey silty clay. As such, any below grade space at this site should be waterproofed (as opposed to installing a drain at the base of the foundation walls). Furthermore, the excavation for a basement level will likely extend into the underlying soft to firm, grey silt clay resulting in difficult working conditions, possible disturbance to the silty clay deposits at subgrade level, and excavation side slopes of 3 horizontal to 1 vertical (Type 4 soil). Also, the foundation for a basement would be on or within the soft grey silty clay and therefore a reduced foundation bearing pressure would be required.

5.5 Seismic Site Classification

The native overburden deposits in the area of the proposed development are composed of silty sand followed by a thick deposit of silty clay. According to Table 4.1.8.4.A of the Ontario Building code (2006), Site Class E should be used for the design of the proposed building since the average undrained shear strength is less than 50 kilopascals. In our opinion, there is no potential for liquefaction of the overburden deposits at this site.

5.6 Access Roadways and Parking Areas

The thickness of grade raise fill within the access roadway and parking areas (including the pavement structure) should comply with the site grade raise restrictions. In those areas of the access roadways and parking areas where the grade raise restrictions cannot be achieved, considerations could be given to the use of EPS blocks and surcharge preloading as described in Section 5.3 of this report. Alternatively, consideration could also be given to exceeding the site grade raise restriction within the access roadway and parking areas provided some post construction total and differential settlement can be accommodated. The amount of settlement will depend on the thickness of the grade raise fill material; however, as shown from the Canadian Research Site No. 1, the settlement could be significant. It is noted that if the site grade raise restrictions are exceeded below the access roadways and parking areas, any adjacent building may be negatively impacted. Furthermore, damage to buried structures/services may occur in the transition zone between those areas where the site grade raise restrictions are adhered to and those areas where the site grade raise restrictions are exceeded.

Depending on vehicle loading, it is likely that the pavement structure for this site will exceed the site grade raise restrictions. Therefore, it may be necessary to use a light-duty pavement structure. For this case, regular maintenance should be anticipated. Alternatively, consideration could be given to subexcavating the underlying native soil to accommodate the pavement structure.

5.7 Site Servicing

5.7.1 Septic System

It is understood that any proposed development at the site will be serviced by a septic disposal system. The septic system design requirements will be depend on the proposed development plans at the site; however, a conventional septic system is considered feasible for this site. The native soils encountered during the investigation are considered to have a percolation rate (T-Time) which is not suitable for use in the construction of the proposed septic leaching bed. As such, the proposed system will be a fully raised Class IV system constructed with a sand mantle extending down gradient of the tile bed.

It is possible that the thickness of fill for the septic system leaching bed will exceed the grade raise restriction for the site. However, the pipes used for a septic leaching bed are relatively flexible and therefore can usually tolerate some differential settlement. Some re-leveling of pipes may be required over time due to the settlement of the leaching bed.

5.7.2 Drinking Water

It is understood that any proposed development at the site will be serviced by a drilled water well. A new drinking water well was drilled on the site on August 11, 2010, by Olympic Drilling Co. Ltd. A pump test was conducted on September 23, 2010, on the test well by Houle Chevrier Engineering Ltd. The results of a well evaluation carried out by Houle Chevrier Engineering Ltd., including well construction details, are provided in our report titled: "Hydrogeological Evaluation, Mitch Owens Road and Boundary Road, Ottawa, Ontario", dated November 11, 2010. The location of the existing well is provided on Figure 2.

Based on the results of a pumping test carried out in the drilled well, sufficient quantities of water are available from the water supply well for a typical commercial development.

Water samples were collected during the pumping test after three (3) and six (6) hours of pumping. The results of the testing on the water samples collected from the well indicate that the water is suitable for consumption with the exception of several Aesthetic Objectives (chloride, colour, hydrogen sulphide, sulphate, total dissolved solids, sodium, and iron) and Operational Guideline (hardness) exceedances. No health related parameters were exceeded in the water test results. The aesthetic objective exceedances and the operational guideline exceedance are considered to be reasonably treatable using conventional water softening equipment followed by reverse osmosis treatment at all points of consumption.

Therefore, the water quality available from the onsite water well is considered acceptable for development at the site as all exceedances are considered to be reasonably treatable using readily available treatment systems.

5.8 Effects of Trees

This site is underlain by deposits of sensitive silty clay, a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the sensitive silty clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations bearing on or above the silty clay. Therefore, no deciduous trees should be permitted closer to the buildings (or any ground supported structures which may be affected by settlement) than the ultimate height of the trees. For groups of trees or trees in rows, the separation distance should be increased to 1.5 times the ultimate height of the trees.

The effects of existing and future trees on any proposed buildings, services and other ground supported structures should be considered in the landscaping design.

5.9 Report Limitations

The details for the proposed construction were not available to us at the time of preparation of this report. As previously indicated, this report is intended for site rezoning purposes only and, as such, specific geotechnical design guidelines could be provided in a separate report as the development plans progress. In addition, it is recommended that at least one (1) borehole or test pit be advanced in the area of any proposed buildings to confirm that the subsurface conditions encountered during the present borehole investigation are consistent with the subsurface conditions at the proposed building location. The requirement for an additional investigation should be assessed as the development plans progress.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Yours truly,

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Principal

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Feb 28/13
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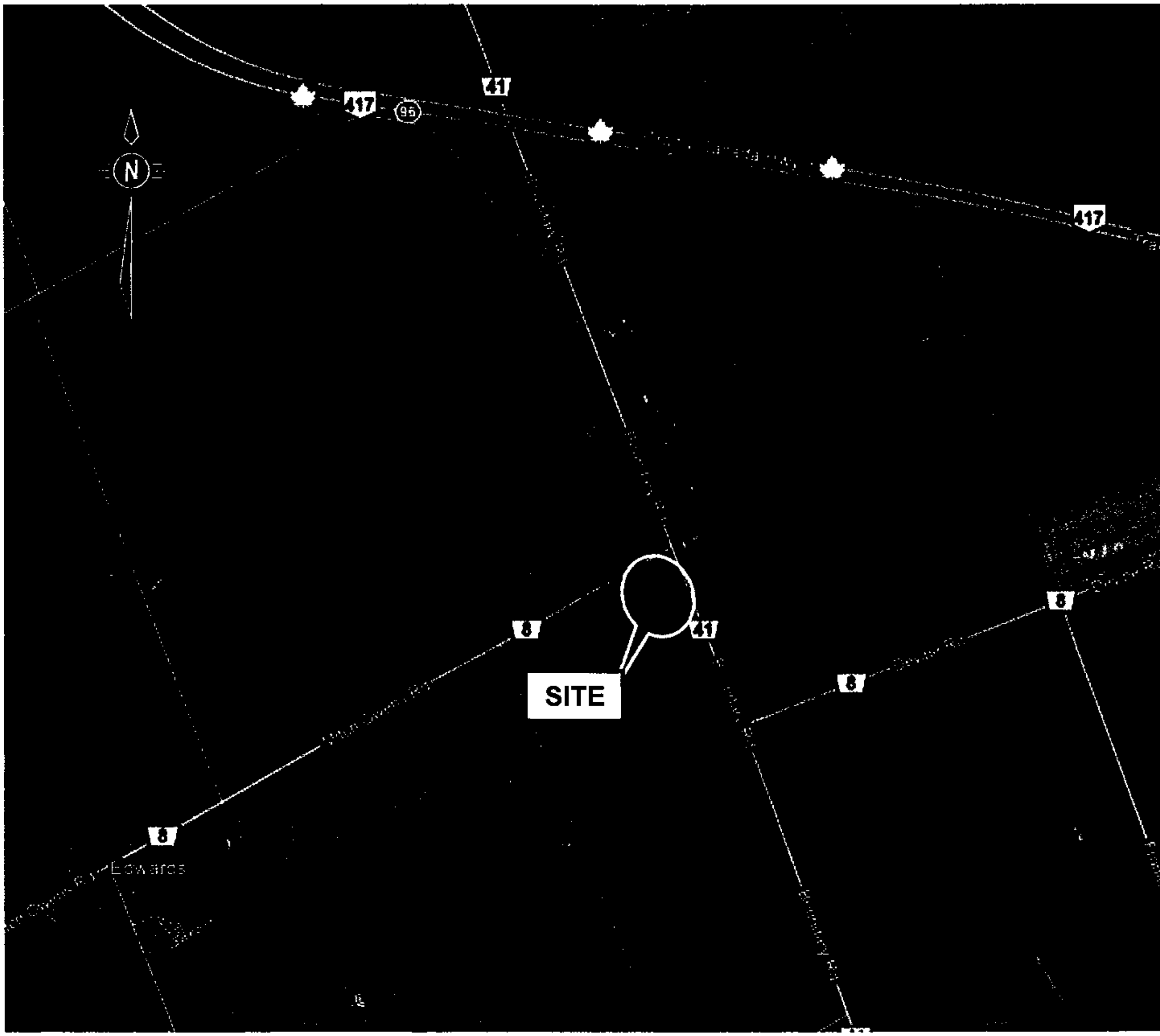
PROVINCE OF ONTAR...

References

Crawford, C.B., and Bozozuk, M. 1990. Thirty years of secondary consolidation in sensitive marine clay. Canadian Geotechnical Journal, 27: 315-319.

KEY PLAN	FIGURE 1



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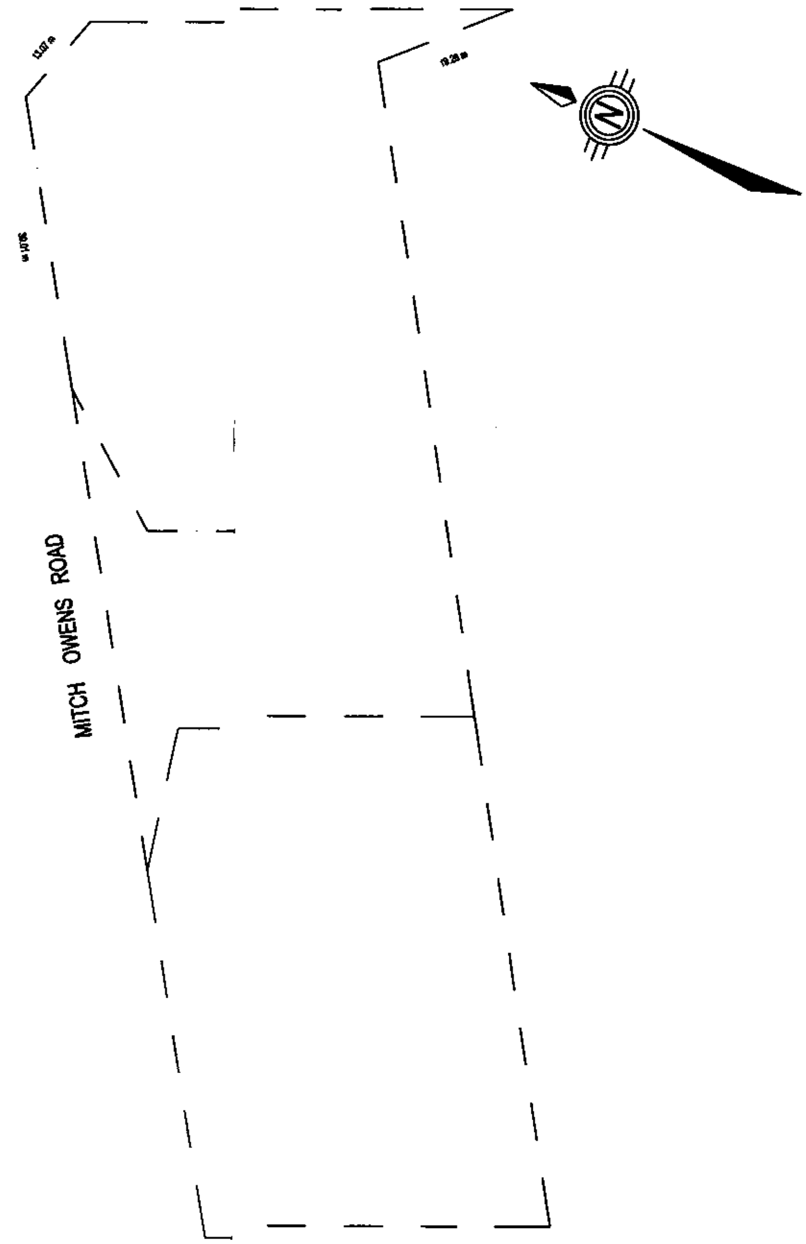


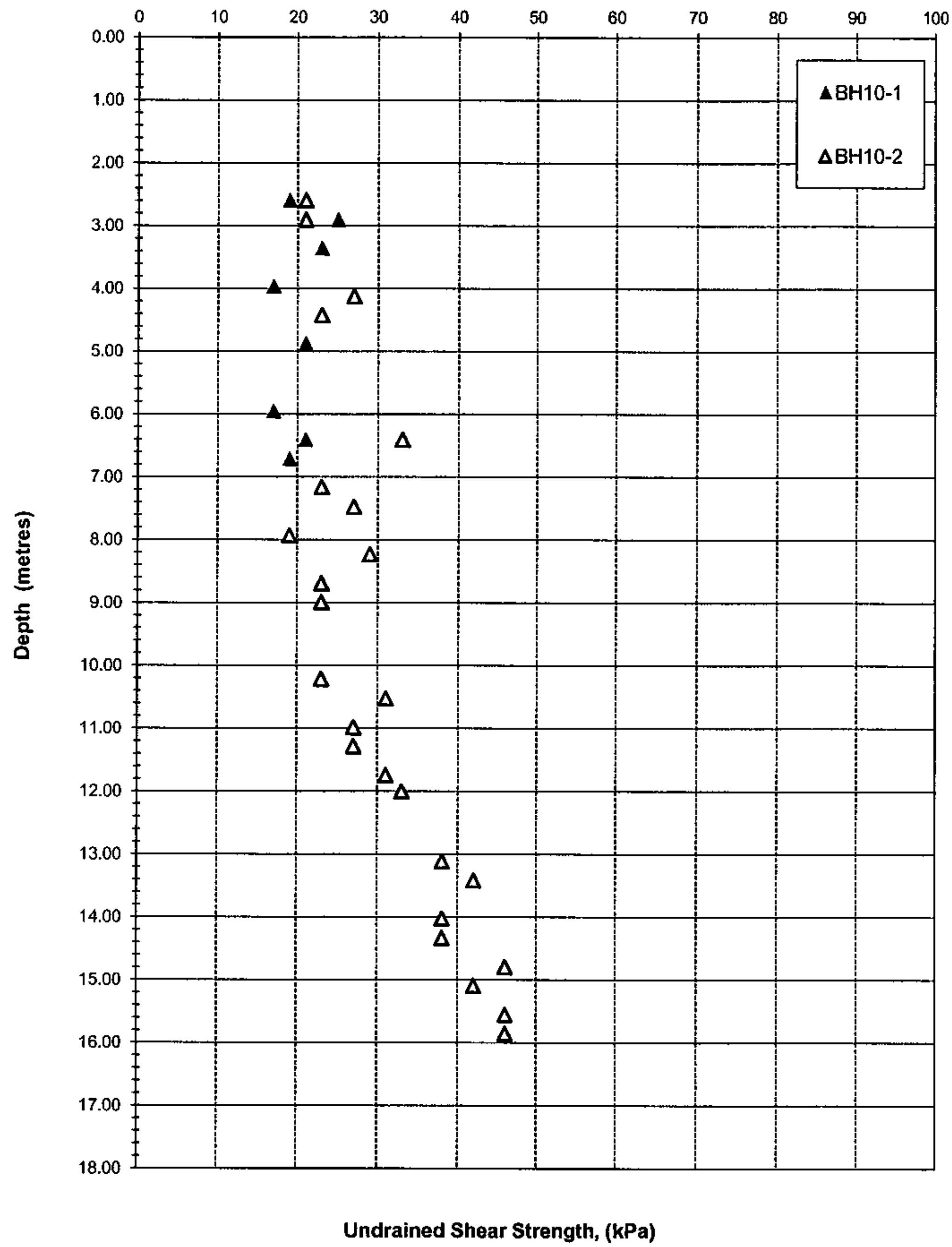
NE	Location MITCH OWENS ROAD AND BOUNDARY ROAD OTTAWA, ON	Revision 0
pproved by .C.H	Project No. 12-626	Scale 1:1000
Title	BOREHOLE LOCATION PLAN	
Date	February 2012	FIGURE 2

LEGEND

BH 1 APPROXIMATE BOREHOL INVESTIGATION BY HOUL JUNE 2010

TW 1 APPROXIMATE TEST WEL

Summary of Vane Strengths vs Depth	FIGURE 3



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APPENDIX A

LIST OF ABBREVIATIONS AND SYMBOLS
RECORD OF BOREHOLE SHEETS
FIGURES A1 to A3

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° 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 10-1

PROJECT: 10-203
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: June 9, 2010

SHEET 1 OF 1
DATUM: Local
SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	99.88					
	Brown SILTY SAND						
	Stiff, reddish brown SILTY CLAY	99.43 0.45					
1			1	50 D.O.	6		
2	Soft, grey SILTY CLAY - occasional grey silty sand seams throughout	98.05 1.83	2	50 D.O.	2		
4			3	T P	PH	C, MH	Bentonite Seal
5			4	T.P.	PH		2" Diameter slotted well screen,
6.71	End of borehole	93.17 6.71					Groundwater level is 0.62 metres below orginal ground surface. Sampled on June 17, 2010.

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

DEPTH SCALE
1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.
CHECKED: Jc

BOREHOLE RECORD GINT LOGS 10-203.GPJ HCE DATA TEMPLATE.GDT 2/2/11

PROJECT: 10-203

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: June 9, 2010

RECORD OF BOREHOLE 10-2

SHEET 1 OF 1

DATUM: Local

SPT HAMMER: 63.5 kg; drop 0.76 m

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	99.89
Brown SILTY SAND	99.48 / 0.41
Stiff, reddish brown SILTY CLAY	98.06 / 1.83
Soft to firm, Grey SILTY CLAY - occasional silty sand seams throughout	87.85 / 12.04
Firm, dark grey SILTY CLAY	84.04 / 15.85
End of borehole	

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 D.O.	5
2	50 D.O.	2
3	T.P.	PH
4	50 D.O.	1
5	50 D.O.	1
6	T.P.	PH
7	T.P.	PH
8	50 D.O.	WH

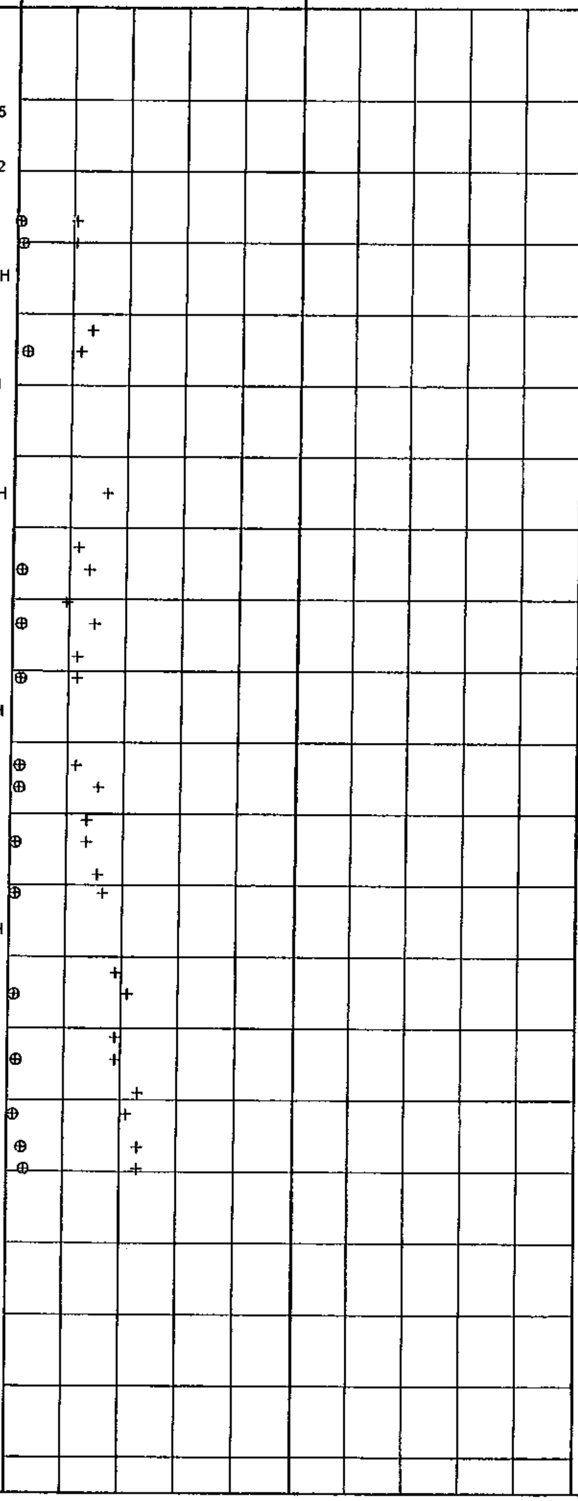
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

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

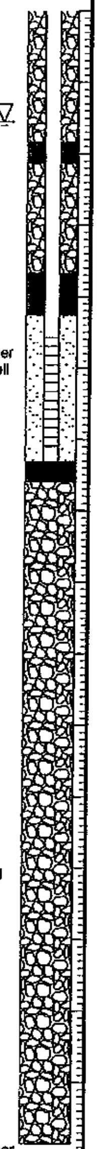
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-7} 10^{-6} 10^{-5} 10^{-4}

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

ADDITIONAL LAB. TESTING



PIEZOMETER OR STANDPIPE INSTALLATION



Bentonite seal

2" Diameter slotted well screen

Native Soil

Groundwater level is 1.55 metres below orginal ground surface. Sampled on June 17, 2010.

DEPTH SCALE 1 to 100

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED: JC

PROJECT: 10-203

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: June 10, 2010

RECORD OF BOREHOLE 10-3

SHEET 1 OF 1

DATUM: Local

SPT HAMMER: 63.5 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger, 200 mm Diameter Hollow Stem	Ground Surface	100.16				
		Brown SILTY SAND					
1		Stiff, reddish brown SILTY CLAY	99.09 1.07	1	50 D.O.	9	
2		Soft to firm, grey SILTY CLAY, trace silt seams	98.27 1.89	2	50 D.O.	3	
				3	50 D.O.	WH	
3		End of borehole	97.26 2.90				Groundwater conditions not observed
4							
5							
6							
7							
8							
9							
10							

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

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

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-7} 10^{-6} 10^{-5} 10^{-4}

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

ADDITIONAL LAB. TESTING

STRATA PLOT

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED: fe

PROJECT: 10-203

RECORD OF BOREHOLE 10-4

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Local

BORING DATE: June 10, 2010

SPT HAMMER: 63.5 kg; drop 0.76 m

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger, 200 mm Diameter Hollow Stem	Ground Surface	100.04				
		Brown SILTY SAND					
1		Stiff, reddish brown SILTY CLAY, trace silt seams	98.97 / 1.07	1	50 D.O.	6	
		Soft to firm, Grey SILTY CLAY, trace silt seams	98.36 / 1.68	2	50 D.O.	2	
2		End of borehole	97.91 / 2.13				Groundwater conditions not observed

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

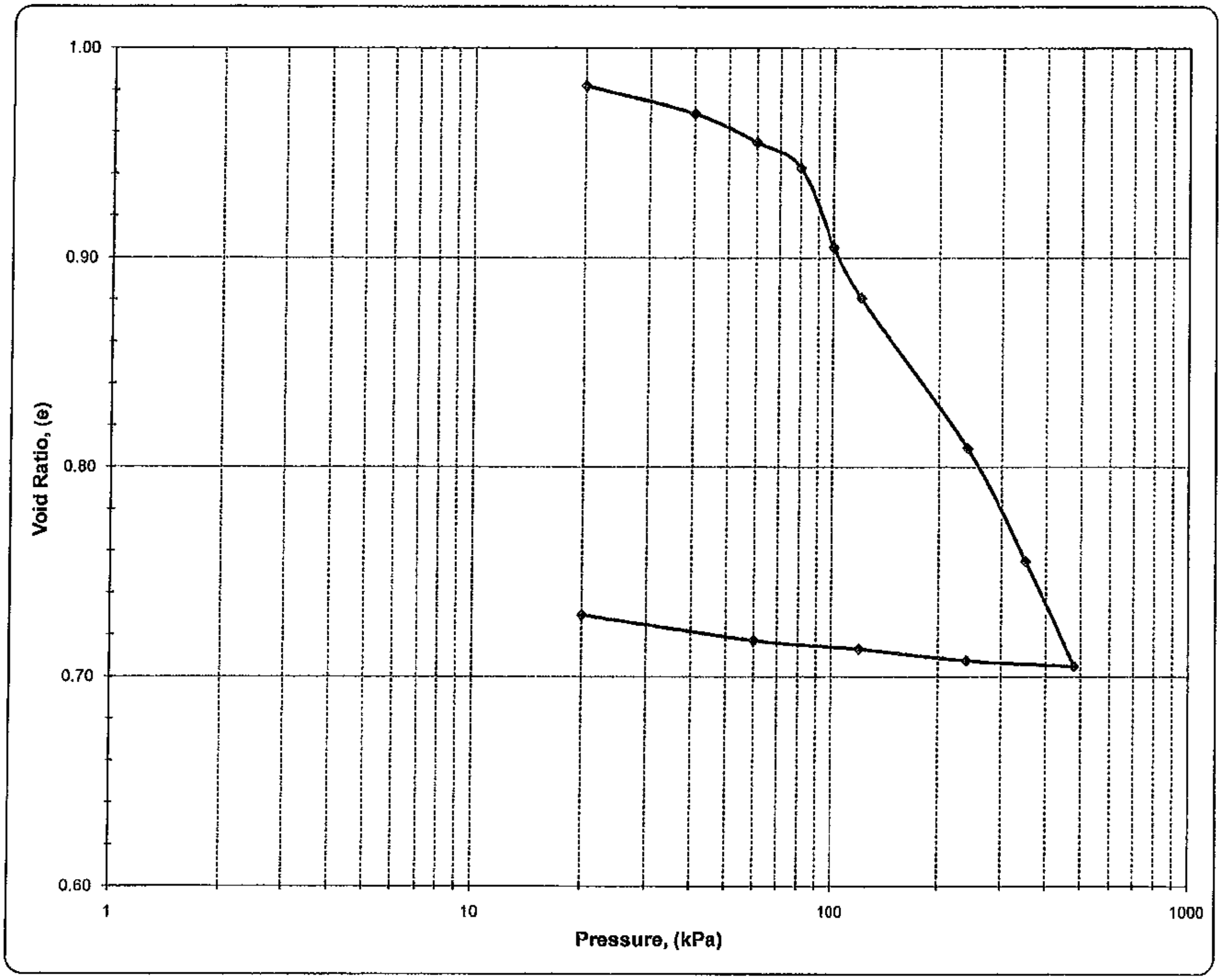
DEPTH SCALE 1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED: JC

CONSOLIDATION TEST RESULTS	FIGURE A3



Borehole	Sample	Depth (m)
BH 10-1	3	4.57 - 5.18

Determined Properties:

W	92
W_L	61
W_p	25

Test Results:

C_r	0.04	
C_c	0.28	
σ'_p	70	kPa

Houle Chevrier Engineering

Date: February 2011
Project: 10-203