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Geotechnical Engineering
Environmental Engineering
Hydrogeology
Geological Engineering
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September 26, 2012
File: PG2768-LET.01

Miriton
1775 Courtwood Crescent
Suite 203
Ottawa, Ontario
K2C 3J2

Attention: **Mr. Craig Ogden**

Subject: **Geotechnical Investigation**
Proposed Watermain Replacement
Carp Airport - Russ Bradley Road - Ottawa

Dear Sir,

Paterson Group (Paterson) was commissioned by Miriton to conduct a geotechnical investigation for the proposed watermain replacement for the existing hanger building located at the Carp Airport on Russ Bradley Road, in the City of Ottawa, Ontario. It is understood that an approximately 90 m section of watermain is to be placed running north from the existing hanger building. It is further understood that the hanger doors at the existing building are to be replaced and comments from a geotechnical perspective have been requested. Also, soil and groundwater samples were submitted for analytical testing to determine any environmental concerns, if any. The following letter report presents our findings and recommendations.

1.0 Field Investigation

The fieldwork for our investigation was conducted on August 29, 2012, and consisted of three (3) test pits excavated using a rubber tired backhoe along the subject alignment of the proposed watermain and three (3) test pits completed along the south side of the existing hanger building. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division.

The subsurface profile encountered at the test hole locations along the proposed watermain alignment consisted of a topsoil layer overlying a silty sand fill over a native, compact silty sand to sandy silt deposit. The subsurface profile at the test pit locations in the area of the proposed hanger door replacement consisted of a pavement structure underlain by topsoil and/or native silty sand. Reference should be made to the Soil Profile and Test Data sheets attached to the present letter report for specific details of the soil profile encountered at the test pit locations.

No groundwater infiltration was observed at the three (3) test pits (TP 1, TP 2 and TP 3) completed along the proposed watermain alignment. Based on soil colouring and consistency of the test pit sidewall soils, the long term groundwater level is anticipated between 1.7 to 2.3 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Groundwater infiltration was noted at TP 4, TP 5 and TP 6 between 2.2 and 2.4 m depth. The groundwater infiltration rate was noted to be low through the test pit sidewalls.

2.0 Geotechnical Assessment - Proposed Watermain Replacement

Based on observations at the test pit locations along the subject alignment of the proposed watermain replacement, it is anticipated that conventional excavation and service placement techniques are adequate for the proposed watermain replacement. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for bedding for water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's standard Proctor maximum dry density (SPMDD).

If suspected poor performing existing fill material is encountered at or below the proposed invert level, this material should be subexcavated to native soils and be backfilled with engineered fill. Engineered fill under service pipes should consist of OPSS Granular A (crushed stone) or Granular B Type II placed in maximum 300 mm thick layers and compacted to a minimum of 95% of the material's SPMDD. Alternatively, the acceptability of the fill could be reviewed by the geotechnical consultant once a sufficient area of the fill has been exposed.

Generally, it should be possible to re-use the moist, not wet, silty sand above the cover material if the excavation and filling operations are conducted in dry weather conditions. The trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to be used at all times to protect personnel working in with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Groundwater Control

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

The rate of flow of groundwater into the excavation through the overburden should be low for expected bedding depth. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

The soil subgrade will be affected by the presence of groundwater. It is recommended that the bedding be placed as soon as possible to reduce the disturbance to the subgrade due to construction traffic (equipment and workers).

3.0 Geotechnical Assessment - Proposed Hanger Door Replacement

Based on our field observations, it is anticipated that a conventional shallow footing placed over an undisturbed, compact silty sand bearing surface or over an engineered fill pad placed over a silty sand bearing surface is adequate for the proposed hanger door replacement.

Foundation Design

Footings founded on an undisturbed, compact brown silty sand bearing surface can be designed using the bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. Alternatively, footings could be placed on an engineered fill pad, consisting of an OPSS Granular A or Granular B Type II crushed stone placed in maximum 300 mm loose lifts and compacted to 98% of its SPMDD. Footings placed on an approved engineered pad placed over an undisturbed, compact silty sand bearing surface can be designed using the abovenoted bearing resistance values.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings designed using the bearing resistance value at SLS provided will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a soil bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil.

New Footings Adjacent to Existing Structure

As a general procedure, it is recommended that the proposed footings be founded at the same level as the existing footings. This accomplishes three objectives. First, the behaviour of the two structures at their connection will be similar due to the similar bearing medium. Second, there will be minimal stress added to the existing structure from the new structure. Third, the bearing of the new structure will likely not be influenced by any backfill material associated with the existing structure.

Protection of Footings Against Frost Action

Perimeter footings of heated structures should be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided. A minimum 2.1 m thick soil cover (or insulation equivalent) should be provided for other exterior unheated footings, such as those for isolated exterior piers.

4.0 Environmental Testing

Soil Sampling Protocol

Soil sampling protocols were followed using the MOE document titled “Guidance on Sampling and Analytical Methods for Use at Contaminated Sites in Ontario”, dated May 1996.

The soil samples were recovered using a stainless steel spaded shovel, using protective gloves (changed after each sample). The samples were placed into plastic bags. If significant contamination was encountered, the samples were placed into glass jars/vials. Sampling equipment was washed in soapy water after each sample was recovered to prevent cross contamination of the samples. Samples were stored in coolers to reduce analyte volatilization during transportation. Visual and olfactory observations of the soil samples noted no signs of contamination.

Soil Sample Headspace Analysis

Soil samples recovered at the time of sampling were placed immediately into airtight plastic bags with nominal headspace. All lumps of soil inside the bags were broken by hand, and the soil was allowed to come to room temperature prior to conducting the vapour survey. Allowing the samples to stabilize to room temperature ensures consistency of readings between samples.

To measure the soil vapours, the analyser probe is inserted into the nominal headspace above the soil sample. An RKI Eagle (gastech) with methane elimination and calibrated to hexane was used for this purpose. The sample is agitated/manipulated gently as the measurement is taken. The peak reading registered within the first 15 seconds is recorded as the vapour measurement.

The parts per million (ppm) scale is used to measure concentrations of hydrocarbon vapours that are too low to register on the Lower Explosive Limit (LEL) scale. The explosive point, 100% LEL, represents the leanest mixture which will burn (or explode) if ignited.

The combustible vapour readings were found to range from 0 to 15 ppm in all of the soil samples obtained. These readings are considered to be representative of background readings and not indicative of the presence of petroleum hydrocarbon contamination.

Groundwater Sampling Protocol

Groundwater samples were recovered from TP 4, TP 5 and TP 6 upon completion of the test pits. The samples were stored in bottles prepared by Paracel Laboratories and stored in a cooler to reduce analyte volatilization during transportation.

Soil and Groundwater Standards

The soil and groundwater standards for the subject property were obtained from Table 3 of the document entitled “Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act”, prepared by the Ontario Ministry of Environment (MOE), April 15, 2011. The MOE Table 3 Standards are based on the following considerations:

☐ Fine grained soil conditions.
☐ Surface soil and groundwater conditions.
☐ Non-potable groundwater situation.
☐ Commercial land use.

Paracel Laboratories (Paracel), of Ottawa, performed the laboratory analysis on the soil and groundwater samples submitted for analytical testing. Paracel is a member of the Standards Council of Canada/Canadian Association for Environmental Analytical Laboratories (SCC/CAEAL). Paracel is accredited and certified by SCC/CAEAL for specific tests registered with the association.

Four (4) soil samples and one (1) groundwater sample were submitted for petroleum hydrocarbon (PHC - F_1 to F_4), and benzene, toluene, ethylbenzene and xylenes (BTEX). The results of the analytical testing, and the selected MOE standards are presented in Table 1 and 2 on the following pages.

Table 1
Analytical Test Results - Soil
PHCs (Fractions 1 to 4) and BTEX

Parameter	MDL (µg/g)	Soil Samples (µg/g)				Table 1 Industrial / Commercial Property Use (µg/g)	Table 3 Industrial / Commercial Property Use (µg/g)
		TP2 - G2	TP3 - G2	TP5 - G1	TP6 - G2		
Benzene	0.02	nd	nd	nd	nd	0.02	0.4
Toluene	0.05	nd	nd	nd	nd	0.05	19
Ethylbenzene	0.05	nd	nd	nd	nd	0.2	78
Xylenes	0.05	nd	nd	nd	nd	0.05	30
F_1 (C_6-C_{10})	7.00	nd	nd	nd	nd	10	65
F_2 (C_{10}-C_{16})	4.00	nd	nd	nd	nd	10	250
F_3 (C_{16}-C_{34})	8.00	nd	nd	nd	nd	50	2500
F_4 (C_{34}-C_{50})	6.00	nd	nd	nd	nd	50	6600

Notes:
- ❑ MDL - Method Detection Limit
- ❑ nd - not detected above the MDL
- ❑ **Bold** - Value exceeds applicable MOE Table 3 Standard

The analytical test results of the soil samples submitted for petroleum hydrocarbon (F_1 to F_4), and benzene, toluene, ethylbenzene and xylenes (BTEX) recovered from the test holes are in compliance with the current MOE Table 1 and Table 3 commercial standards.

Table 2
Analytical Test Results - Groundwater
PHC (F_1-F_4) & BTEX

Parameter	MDL (µg/L)	Groundwater Sample	Table 1 Standards (µg/L)	Table 3 Standards (µg/L)
		TP4 - GW1		
Benzene	0.5	nd	0.5	430
Toluene	0.5	nd	0.5	2300
Ethylbenzene	0.5	nd	0.8	18000
Xylenes	0.5	nd	72	4200
F_1 (C_6-C_{10})	25.0	nd	420	750
F_2 (C_{10}-C_{16})	100.0	nd	150	150
F_3 (C_{16}-C_{34})	100.0	nd	500	500
F_4 (C_{34}-C_{50})	100.0	nd	500	500

Notes:
- ☐ MDL - Method Detection Limit
- ☐ nd - not detected above the MDL
- ☐ **Bold** - Value exceeds applicable MOE Table 3 Standard

The analytical test results of the groundwater sample submitted for petroleum hydrocarbon (F_1 to F_4), and benzene, toluene, ethylbenzene and xylenes (BTEX) recovered from TP 4 is in compliance with the current MOE Table 1 and Table 3 commercial standards.

5.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials used.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.

Upon request, a report confirming that these works have been conducted in general accordance with our recommendations could be issued following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

6.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Our recommendations should be reviewed when the project drawings and specifications are complete.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein, or by person(s) other than Miriton or their agents is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Best Regards,

Paterson Group Inc.

Richard Groniger, Technologist.

LICENSED PROFESSIONAL ENGINEER
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Attachments

☐ Soil Profile and Test Data sheets
☐ Analytical Test Results
☐ Figure 1 - Key Plan
☐ Drawing PG2768-1 - Test Hole Location Plan

Report Distribution

☐ Miriton (3 copies)
☐ Paterson Group (1 copy)

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Russ Bradley Road - Carp Airport
Ottawa, Ontario

DATUM TBM - Finished floor level of the existing aircraft hanger building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

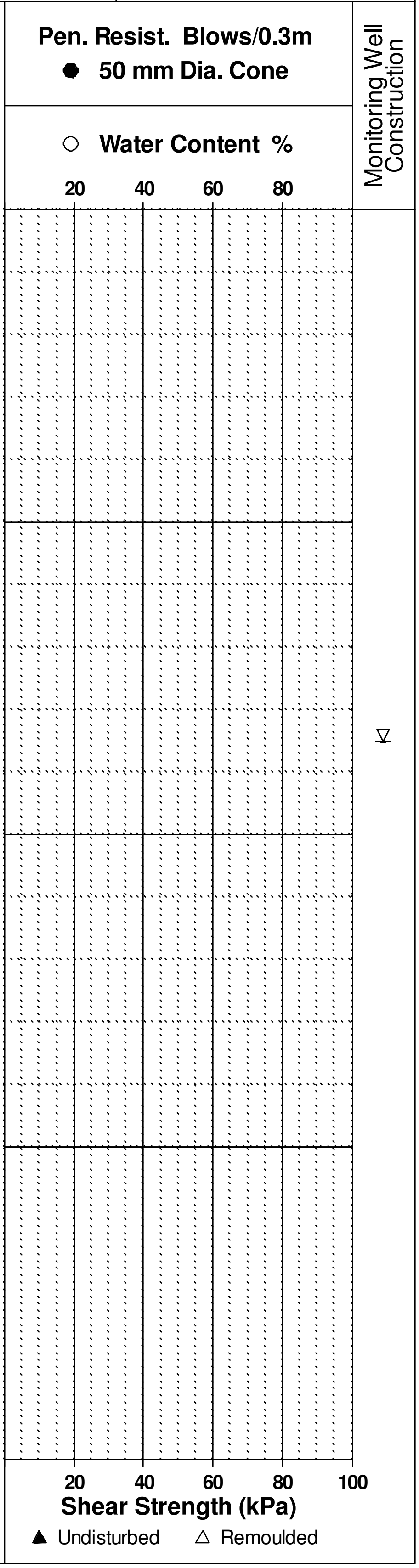
BORINGS BY Backhoe

DATE August 29, 2012

FILE NO. PG2768

HOLE NO. TP 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	99.62
25mm Topsoil							
		G	1				
FILL: Brown silty sand with crushed stone and gravel - topsoil with sand from 0.8 to 1.47m depth							
						1	98.62
1.47							
		G	2				
Compact, brown **SILTY FINE SAND/SANDY SILT**						2	97.62
		G	3				
3.00						3	96.62
End of Test Pit (GWL @ 1.7m depth based on field observations) (No groundwater infiltration observed at the time of excavation)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Russ Bradley Road - Carp Airport
Ottawa, Ontario

DATUM TBM - Finished floor level of the existing aircraft hanger building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Backhoe

DATE August 29, 2012

FILE NO. PG2768

HOLE NO. TP 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
GROUND SURFACE						0	99.18		
25mm Topsoil and organics **FILL:** Brown silty fine sand with gravel — 0.90									
FILL: Brown silty fine sand to sandy silt interbedded with topsoil, trace clay — 1.50		G	1			1	98.18		
Compact, brown **SILTY FINE SAND** with dark organic seams — 3.50						2	97.18		▽
		G	2						
		G	3			3	96.18		
End of Test Pit (GWL @ 2.3m depth based on field observations) (No groundwater infiltration observed at the time of excavation)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Russ Bradley Road - Carp Airport
Ottawa, Ontario

DATUM TBM - Finished floor level of the existing aircraft hanger building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

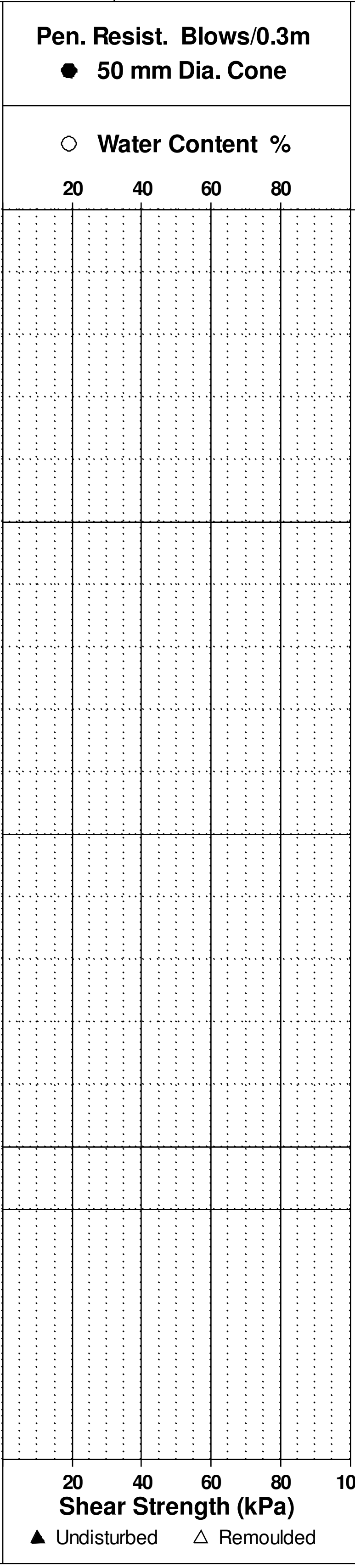
BORINGS BY Backhoe

DATE August 29, 2012

FILE NO. PG2768

HOLE NO. TP 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE						0	98.35	
TOPSOIL/ORGANICS 0.30								
		G	1					
Dense to compact, brown **SILTY FINE SAND** to **SANDY SILT** - trace sea shells at 1.4m depth						1	97.35	
1.70								
								∇
Compact, grey **SANDY SILT**						2	96.35	
		G	2					
						3	95.35	
3.20		G	3					
End of Test Pit (GWL @ 1.9m depth based on field observations) (No groundwater infiltration observed at the time of excavation)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Russ Bradley Road - Carp Airport
Ottawa, Ontario

DATUM TBM - Finished floor level of the existing aircraft hanger building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Backhoe

DATE August 29, 2012

FILE NO. PG2768

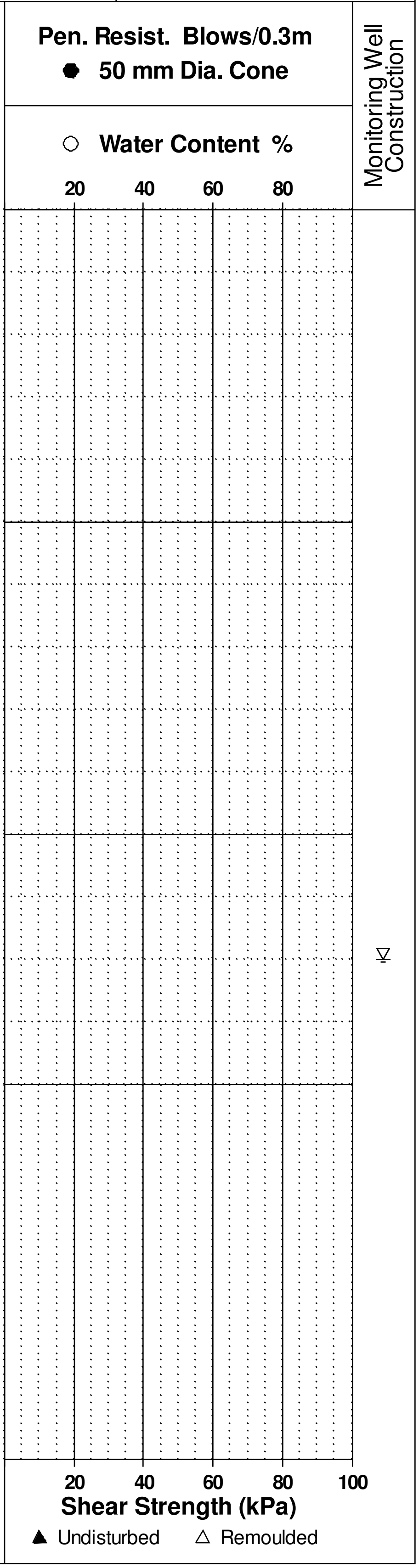
HOLE NO. TP 4

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE					0	100.18	
ASphaltic concrete 0.13							
FILL: Crushed stone 0.80							
TOPSOIL 0.90					1	99.18	
Compact, brown **SILTY FINE SAND** 1.40	G	1					
Compact, brown **SILTY FINE SAND** to **SANDY SILT** - trace clay from 1.9 to 2.2m depth 2.80	G G G	2 3 4			2	98.18	▽

End of Test Pit

Underside of footing at 1.4m depth. The footing was noted to be 170mm thick and extend 80mm beyond the foundation wall.

(GWL @ 2.4m depth based on field observations)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Russ Bradley Road - Carp Airport
Ottawa, Ontario

DATUM TBM - Finished floor level of the existing aircraft hanger building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Backhoe

DATE August 29, 2012

FILE NO. PG2768

HOLE NO. TP 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Monitoring Well Construction
GROUND SURFACE					0	99.99		
Asphaltic concrete 0.09								
FILL: Crushed stone 0.82								
FILL: Brown to black silty sand, trace topsoil 1.10	G	1			1	98.99		
Compact, brown **SILTY FINE SAND** to **SANDY SILT** - some sea shells by 1.5m depth - trace clay from 1.8 to 2.0m depth	G	2			2	97.99		∇
2.70	G	3						
End of Test Pit Underside of footing at 1.3m depth. The footing was noted to be 170mm thick and extend 100mm beyond the foundation wall. (GWL @ 2.4m depth based on field observations)								

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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REMARKS

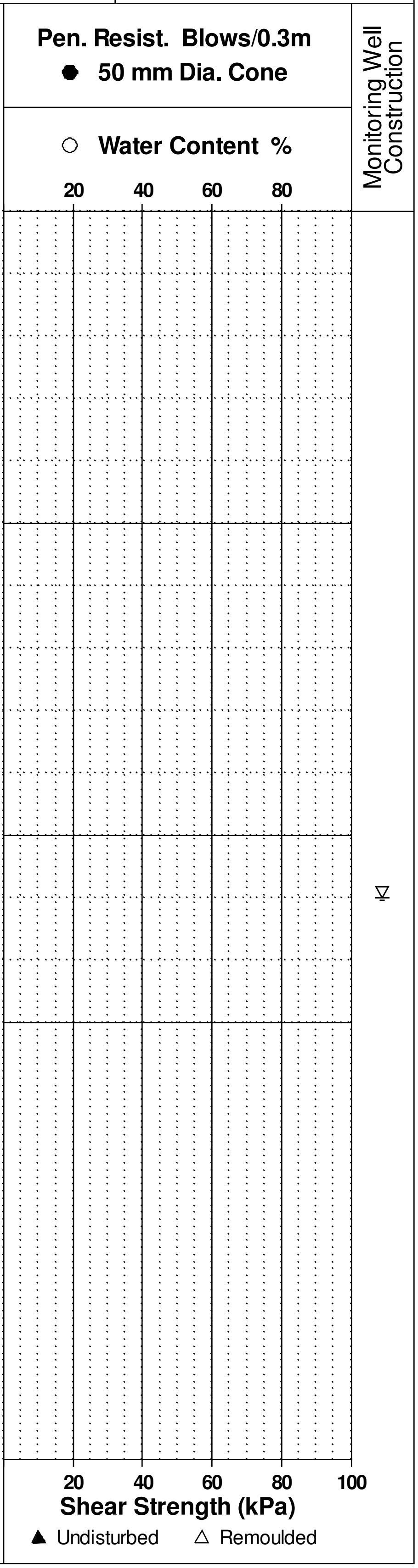
BORINGS BY Backhoe

DATE August 29, 2012

FILE NO. PG2768

HOLE NO. TP 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.00
Asphaltic concrete 0.10							
FILL: Crushed stone 0.75							
		G	1			1	99.00
Compact, brown **SILTY FINE SAND** to **SANDY SILT** - trace black sand from 0.75 to 0.9m depth		G	2				
						2	98.00
2.60							
End of Test Pit Underside of footing at 1.4m depth. The footing was noted to be 160mm thick and extend 100mm beyond the foundation wall. (GWL @ 2.2m depth based on field observations)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

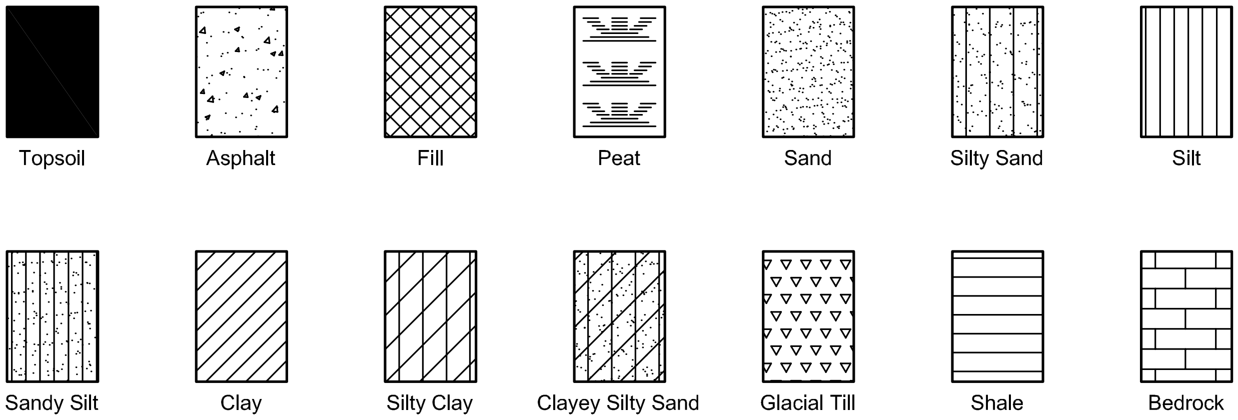
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

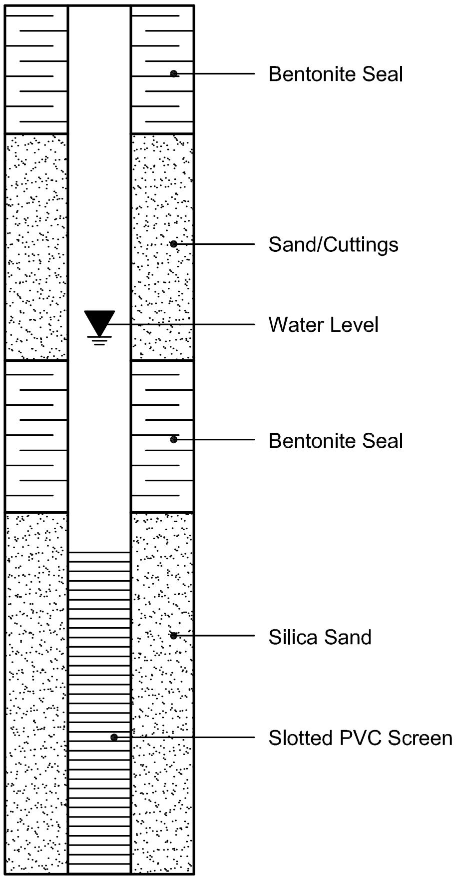
SYMBOLS AND TERMS (continued)

STRATA PLOT

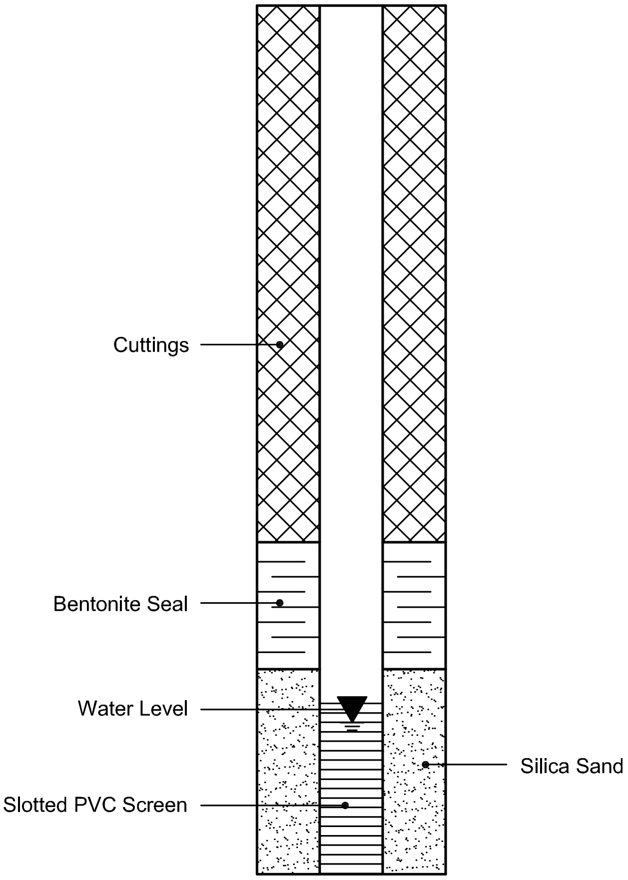


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 05-Sep-2012
Order Date:29-Aug-2012

Client: **Paterson Group Consulting Engineers**

Client PO: 12773

Project Description: PG2768

	Client ID:	TP4-GW1	-	-	-
	Sample Date:	29-Aug-12	-	-	-
	Sample ID:	1235172-01	-	-	-
	MDL/Units	Water	-	-	-
Volatiles					
Benzene	0.5 ug/L	<0.5	-	-	-
Ethylbenzene	0.5 ug/L	<0.5	-	-	-
Toluene	0.5 ug/L	<0.5	-	-	-
m,p-Xylenes	0.5 ug/L	<0.5	-	-	-
o-Xylene	0.5 ug/L	<0.5	-	-	-
Xylenes, total	0.5 ug/L	<0.5	-	-	-
Toluene-d8	Surrogate	104%	-	-	-
Hydrocarbons					
F1 PHCs (C6-C10)	25 ug/L	<25	-	-	-
F2 PHCs (C10-C16)	100 ug/L	<100	-	-	-
F3 PHCs (C16-C34)	100 ug/L	<100	-	-	-
F4 PHCs (C34-C50)	100 ug/L	<100	-	-	-
F1 + F2 PHCs	125 ug/L	<125	-	-	-
F3 + F4 PHCs	200 ug/L	<200	-	-	-

Certificate of Analysis

Report Date: 05-Sep-2012
Order Date:29-Aug-2012

Client: **Paterson Group Consulting Engineers**

Client PO: 12773

Project Description: PG2768

	Client ID:	TP2-G2	TP3-G2	TP5-G1	TP6-G2
	Sample Date:	29-Aug-12	29-Aug-12	29-Aug-12	29-Aug-12
	Sample ID:	1235171-01	1235171-02	1235171-03	1235171-04
	MDL/Units	Soil	Soil	Soil	Soil
Physical Characteristics					
% Solids	0.1 % by Wt.	87.1	83.1	87.4	82.3
Volatiles					
Benzene	0.02 ug/g dry	<0.02	<0.02	<0.02	<0.02
Ethylbenzene	0.05 ug/g dry	<0.05	<0.05	<0.05	<0.05
Toluene	0.05 ug/g dry	<0.05	<0.05	<0.05	<0.05
m,p-Xylenes	0.05 ug/g dry	<0.05	<0.05	<0.05	<0.05
o-Xylene	0.05 ug/g dry	<0.05	<0.05	<0.05	<0.05
Xylenes, total	0.05 ug/g dry	<0.05	<0.05	<0.05	<0.05
Toluene-d8	Surrogate	88.4%	87.8%	87.6%	88.4%
Hydrocarbons					
F1 PHCs (C6-C10)	7 ug/g dry	<7	<7	<7	<7
F2 PHCs (C10-C16)	4 ug/g dry	<4	<4	<4	<4
F3 PHCs (C16-C34)	8 ug/g dry	<8	<8	<8	<8
F4 PHCs (C34-C50)	6 ug/g dry	<6	<6	<6	<6

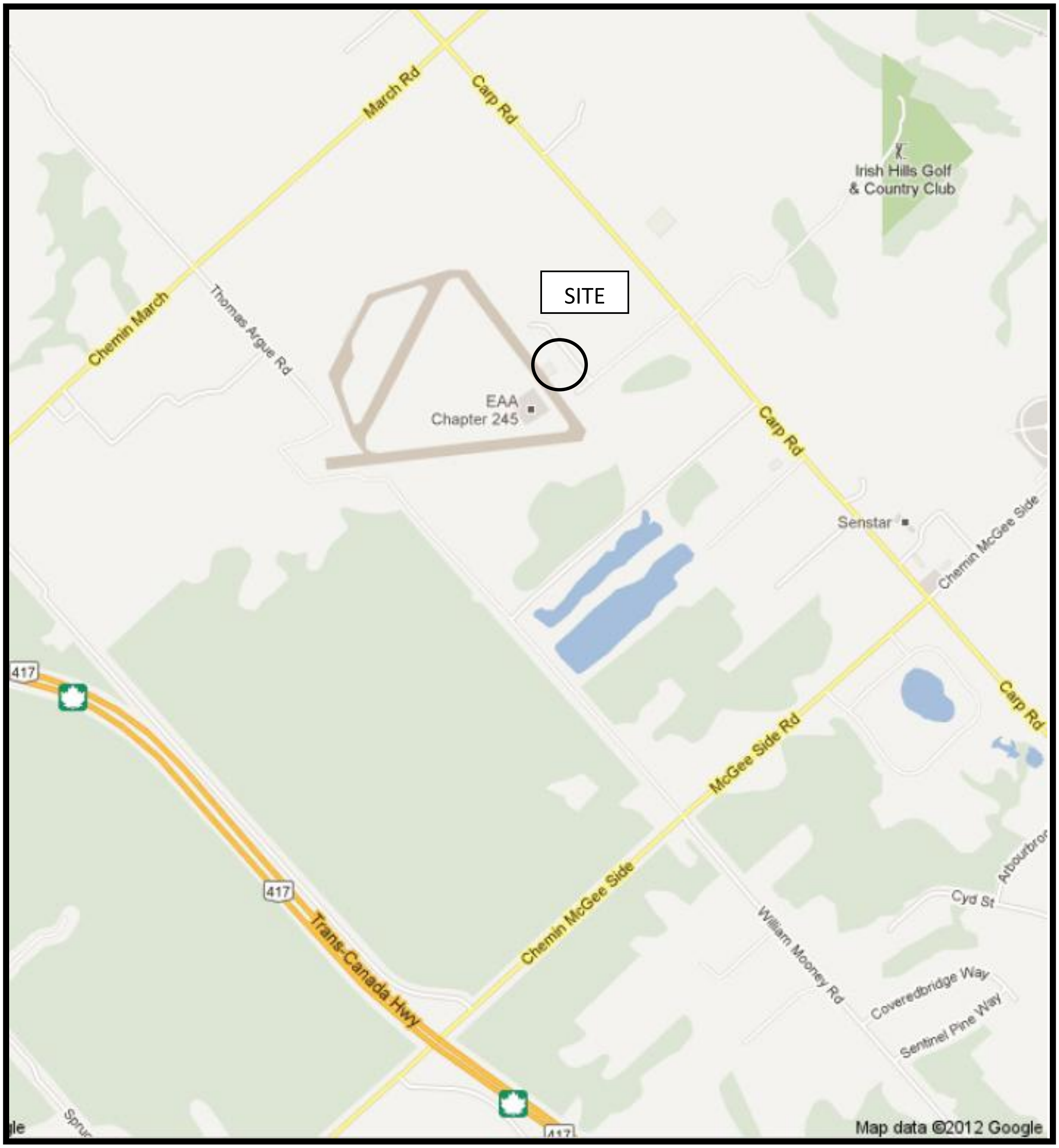
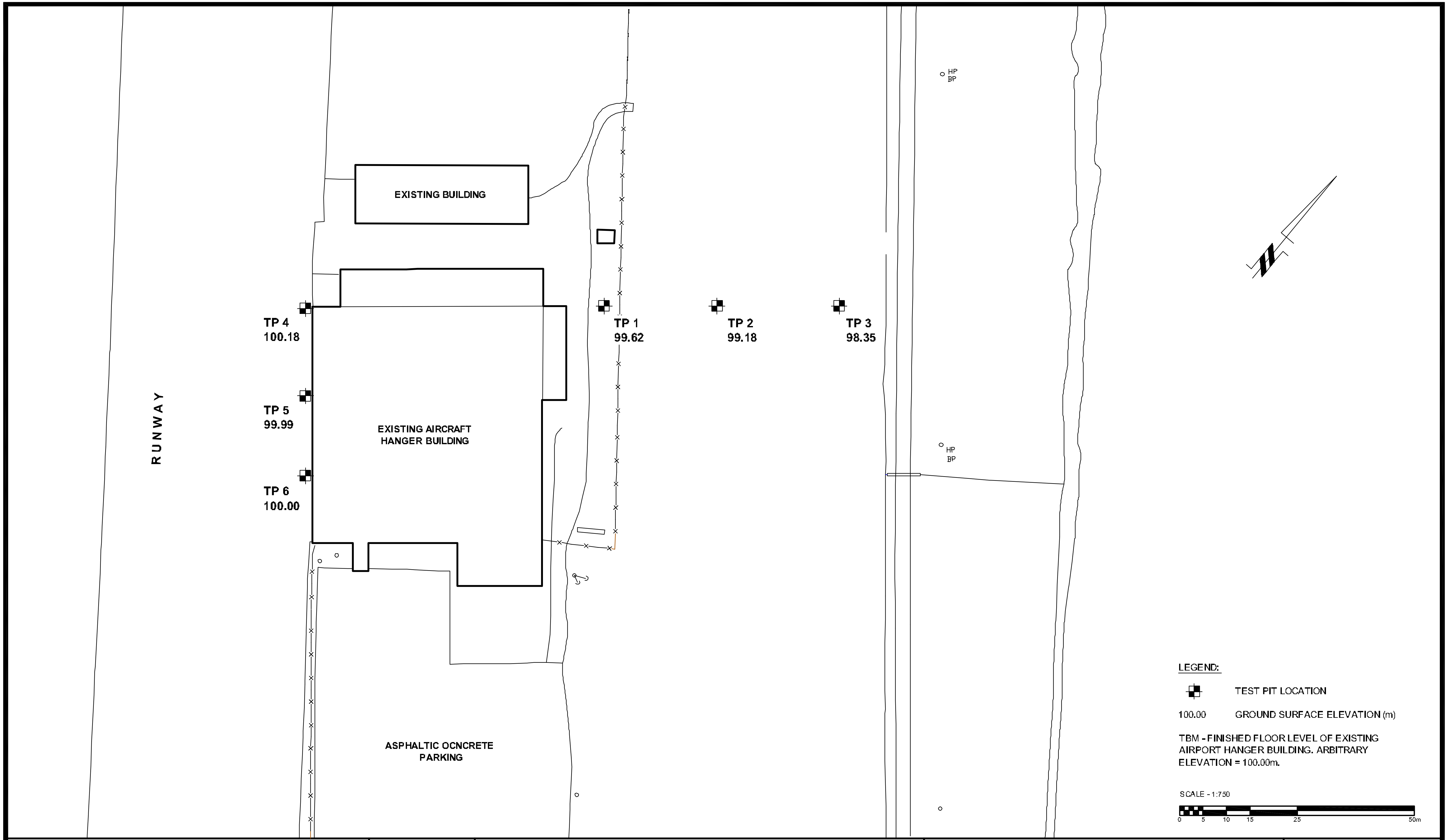


FIGURE 1

KEY PLAN

patersongroup



paterson group
consulting engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

Scale: 1:750
Des.: RG
Dwn: MPG
Chkd: DG

MIRITON

GEOTECHNICAL INVESTIGATION
PROPOSED WATERMAIN REPLACEMENT
RUSS BRADLEY ROAD - CARP AIRPORT

OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No. PG2768-1
Report No.: PG2768-1
Date: 09/2012

11x17

GEOTECHNICAL INVESTIGATION – FINAL REPORT
RCMP – CARP AIRPORT
NEW WATER RESERVOIR, PUMP CHAMBER, AND WATERMAIN
CARP, ONTARIO

Date : **September 8, 2010** Ref. : **T020587-A1**

FONDEX

179 Colonnade Road, Suite 400, Ottawa, Ontario K2E 7J4 Tel.: (613) 727-0895 Fax (613) 727-0581

Reference No.: T020587-A1

September 8, 2010
(Revised from original draft of June 12, 2009)

Mr. Robert Warner
Project Manager
SNC Lavalin-O&M
900 Lady Ellen Place
Ottawa, Ontario
K1Z 5M2

Re: Geotechnical Investigation – Final Report
RCMP – Carp Airport
New Water Reservoir, Pump Chamber, and Watermain
Carp, Ontario

Dear Mr. Warner,

Please accept the attached report as the Final Geotechnical Report for the above captioned project. This report was completed and issued earlier under draft form under **Fondex Ontario Limited** (a member of **Inspec-Sol Inc.**). This report contains the comments and recommendations that were used for the design of the structure.

We trust that this information meets with your approval. Please do not hesitate to contact us, should any questions arise.

Yours very truly,

FONDEX ONTARIO LIMITED

Joseph B. Bennett, P. Eng
Vice-President

JBB/vl

Enclosures:

Dist: Mr. Robert Warner –email - (robert.warner@snclavalinom.com) Mail (4)
Mr. Bruce Sinclair – email - (bruce.sinlcair@snclavalinom.com)
Mr. Lee Jablonsky- email - (ljablonski@jlrichards.ca)

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1.0 INTRODUCTION

Fondex Ontario Limited (a member of the **Inspec-Sol Inc**.) was authorized to carry out a geotechnical study at the site of a proposed water storage tank to the north of the existing hanger building at the Carp Airport. Mr. William Borghese of Public Works and Government Services Canada, acting as the representative of the client, authorized the fieldwork at the time. The management of the project was subsequently taken over by SNC-Lavalin/Profac (SNC) and authorization for work was being provided by Mr Robert Warner of SNC.

The original proposal was to construct a water storage tank on the east side of the existing hanger. The tank was to be 15 m x 15 m in size, and 3 m tall, with a smaller pump chamber attached, and a watermain connecting the reservoir to the main hanger building. The hanger building is an existing building and requires this tank for fire suppression supply. During the investigation, the location and the size of the reservoir was changed to become an 11 m x 5 m x 3 m tank set at a depth of approximately 1 m into the ground and covered with up to 1 m of fill material. This means that surrounding the tank would be an embankment approximately 4.3 m high, sloping out from the edges of the tank at a slope of 3H:1V. The pump chamber structure is now proposed to be near the original location of the reservoir.

The site is located on the north eastern portion of the property known as the Carp Airport. The airport is on the west side of Carp Road, and currently is used by small aircraft. The hanger in question is currently being utilized as a training facility by the Royal Canadian Mounted Police.

The site of the proposed water reservoir is currently vacant, and located at the edge of a gravel parking area, with a majority of the tank resting in the landscaped area.

A Site Location Plan, Dwg. No. T020587-A1-1, which shows the location of the site relative to the Town of Carp, Ontario, is attached.

The purpose of the investigation was to evaluate the subsoil stratigraphy in the area of the proposed reservoir, pump chamber and associated watermain in order to provide recommendations concerning bearing capacity, foundation design, groundwater conditions and other pertinent subsoil conditions that may affect construction. The investigation also included the determination of the seismic site class by means of multi-channel analysis of surface waves (MASW), as well as performing slug tests to provide hydraulic data.

2.0 FIELDWORK

The site work consisted of three phases: subsurface investigation by means of drilling of boreholes; MASW shear wave velocity testing; and hydraulic testing.

The initial boreholes were advanced at two (2) locations; one in the proposed location of the pump chamber structure (BH-2); and the other in the proposed location of the reservoir (BH-1).

The boreholes were advanced to 8.2 m and 6.7 m, respectively. The completed boreholes had ground surface elevations recorded relative to the benchmark, which is described as the floor slab of the garage of the hanger, which has an elevation of 113.60 m according the provided drawings.

The locations of the boreholes are shown on the enclosed Dwg. No. T020587-A1-2. A description of the stratigraphy encountered at each test location, is presented on the Borehole Logs, Enclosures No. 1 to No. 3.

The boreholes were carried out by means of a truck mounted drill rig adapted for soil sampling. The holes were advanced with a continuous flight auger. Samples were recovered at regular intervals with a split spoon sampler driven with an energy of 470 kJ using a falling weight. The number of drops with the falling weight to drive the sampler 0.3 m is recorded and shown on the borehole logs as "N" values.

Once the holes were completed, monitoring wells were installed in the boreholes, for the purposes of slug testing. The monitoring well installed in borehole BH-1 was placed within native soils (i.e. below the granular surface fill soils), while the monitoring well installed in borehole BH-2 was placed in the granular fill soils.

Field work for the initial phase of drilling was conducted on December 12, 2008, under the supervision of **Fondex** technical staff.

The fieldwork for the MASW testing was conducted on December 8, 2008. At the time of the MASW survey, the ground surface was snow covered, and moderately frozen.

The test was carried out using a 24 channel seismograph (Geometrics Geode 24 console #3389), twenty-four 1.5 Hz geophones, and one 24 take-out cable with 5 m spacing. The field data was collected using an IBM T60 laptop computer and Geometrics Single Geode OS controller version 9.14.0.0 software.

Two lines of twenty-four geophones spaced at 1.5 m were used to collect seismic data. The vibration generated by a tracked excavator running at an angle to the geophone array, was used as the seismic source. In order to obtain a two-dimensional profile (2-D) the complete set-up was rolled four (4) times with an interval of 1.5 m. Three (3) sets of data were collected at each array location without vertical stacking. For each survey, the ground vibration was recorded for 16 seconds at a sampling rate of one sample per 1 ms. The results from the MASW testing, as well as the approximate location of the MASW survey lines are shown on the attached drawing in Appendix B.

Hydraulic testing (single-well recovery tests or slug tests) were conducted on the monitoring wells placed in boreholes BH-1 and BH-2, to establish the anticipated infiltration of groundwater into the proposed excavation. The test was conducted by quickly inserting a small PVC cylinder to the well, after which the fall in the water level was measured with an electronic water level meter. Once the water level returned to its original level, the PVC cylinder was quickly removed, and the rise in the water level was recorded. The results of the hydraulic testing allowed for an estimate of the corresponding hydraulic conductivity of the soil.

As a result of the decision to raise the grade around the reservoir structure, **Fondex** recommended that an additional borehole be advanced in the new location of the proposed reservoir that would include collecting shear strength, split spoon samples, as well as undisturbed thin-walled samples to conduct consolidation testing. This borehole is described as borehole BH-3, and was located in the centre of the proposed new location of the reservoir (approximately 23 m northeast of the hanger structure). The boreholes were carried out by means of a track mounted drill rig adapted for soil sampling, and was advanced with a continuous flight auger. Samples were collected to a depth of 29.6 m, and a penetration cone was extended to a total depth of 33.2 m below the ground surface.

3.0 SITE CONDITIONS AND SUBSOIL CONDITIONS

The area is generally flat, sloping off to a storm water management ditch to the east. Surface elevations at the boreholes range within 0.1 m. Approximately half of the proposed tank area is part of the parking area and is covered with compacted granular soil. The remainder of the area is covered with grass and topsoil.

East of the main hanger is a driveway/parking area that is covered with compacted granular material. The location of borehole BH-2 was within this compacted granular fill, which extends to a depth of 0.5 m. Below the crushed granular material in BH-2 is a layer of compact brown silty sand. This material is generally of a medium grain and is probably a fill material used to bring the driveway area to the final grade.

In boreholes BH-1 and BH-3, there was a surface layer of topsoil and organic material. In borehole BH-1, this layer was a 0.13 m skim, whereas in borehole BH-3, this layer extended to a depth of 1.0 m.

Below the fill soils in borehole BH-2 and below the topsoil in borehole BH-1, the soil is a layer of grayish brown silty sand. In borehole BH-1, there is trace organic material in the upper 1 m. This material become wet and compact by a depth of 1.5 m, and had traces of oxidation.

Below depths of 3 m. the soil becomes a Clayey Silt, trace to some Sand with fine sand seams and pockets. It is grey in colour, wet, and of a firm consistency. The natural moisture content of this material increases with depth, from 20 to 40 %. The liquid limit and plasticity indexes of this layer are summarized in Table I.

TABLE I

ATTERBERG LIMIT PARAMETERS FOR SELECTED SOIL SAMPLES

Sample Location	Natural Moisture Content	Liquid Limit	Plasticity Index
BH-3 SS-4	21.0	23.2	Non-plastic
BH-3 SS-7	29.7	24.8	17.5
BH-3 SS-11	30.4	28.0	12.6
BH-3 SS-15	35.4	29.0	14.0
BH-3 ST-21	40.9	40.0	22.0

Below a depth of 10 m, the soil becomes a Silty Clay, trace Sand. This material is grey in colour, wet, and of a firm consistency. The natural moisture content of this material ranges from 35 % to 50 %.

The bedrock depth was not encountered in the investigation depth, and therefore exists beyond a depth of 33.2 m.

4.0 GROUNDWATER

Based on the water level readings taken on December 29th, 2008, the water levels in boreholes BH-1 and BH-2 are less than 1 m below the existing grade at EL. 112.36 m and 112.46 m, respectively.

The groundwater levels will fluctuate seasonally and will be at the highest level during the spring thaw. It is suggested that for design purposes the high water table be considered to be near the ground surface.

The details of the hydraulic analyses performed at the site may be found in the attached Appendix A.

Based on the hydraulic analyses, the following are the estimated infiltration rates based on excavation depth.

TABLE II
ANTICIPATED INFILTRATION INTO RESERVOIR EXCAVATION

Depth of Excavation	Anticipated Infiltration Rate (at Steady State Conditions) in L/day
1.2 m (~ EL. 111.9 m)	Approx 11,000 to 15,000
1.8 m (~ EL. 112.5 m)	Approx 18,000 to 20,000
2.4 m (~ EL. 110.7 m)	Approx 25,000 to 30,000
3.0 m (~ EL. 110.1 m)	Approx 30,000 to 35,000
3.6 m (~ EL. 109.5 m)	Approx 35,000 to 40,000

TABLE III
ANTICIPATED INFILTRATION INTO WATERMAIN EXCAVATION

Depth of Excavation	Anticipated Infiltration Rate (at Steady State Conditions) in L/day
1.2 m (~ EL. 111.9 m)	Approx 6,900 to 10,000
1.8 m (~ EL. 112.5 m)	Approx 10,900 to 15,000
2.4 m (~ EL. 110.7 m)	Approx 16,100 to 18,000

These estimations are based on the following assumptions:

- The data collected represent homogenous and stable conditions across the site,
- The excavation has achieved steady state flow. Until it is brought to steady state, groundwater flows can be 2 to 3 times the rate listed in the Tables above, and
- Precipitation rates are less than 0.02 m/day.

5.0 GEOTECHNICAL COMMENTS AND RECOMMENDATIONS

At the proposed location of the water reservoir, there are two major geotechnical concerns:

- **Settlement**: Because of the proposed grade raise, the settlement of the foundation under the loading of the reservoir and surrounding embankment are anticipated to exceed the ordinary design standard of 25 mm.
- **Liquefaction**: There is also potential at this site that the silts and sands present a potential for liquefaction in the event of a seismic occurrence.

5.1 Pump Chamber and Water Reservoir

5.1.1 Foundations – Bearing Depth, Pressures & Settlements

5.1.1.1 Environmental Impact on Foundation Design

The foundation for the pump chamber is anticipated to be a conventional strip footing, and will be approximately 20 m to the south of the reservoir. The proposed foundation system for the reservoir is expected to be a structurally reinforced raft foundation.

Based on the values provided in Table II above, the deeper the excavation for the reservoir extends, the greater the water infiltration will be. If the reservoir is to be installed at depths greater than 1 m the base of the excavation will be well below the water table, in running sand, and will require the pumping of infiltrating water with a positive dewatering system to ensure a stable base for construction. A wellpoint system or deep wells are two examples of positive dewatering systems that by be contemplated. Based on the hydrological testing, it is anticipated that the infiltration rate at steady state conditions may range from 2300 L/day to 34,250 L/day, depending on the depth of the excavation. However it should be noted that the water pumped to bring the excavation to a steady state, or during significant precipitation events or wet seasons, may exceed the limit of 50,000 L/day, and Permits to Take Water from the Ontario Ministry of Environment may be required.

It is understood therefore that the design elevation of the foundations for both the pump chamber and the reservoir will be at approximately 0.8 m below the existing grade.

5.1.1.2 Pump Chamber Foundations

The strip footings for the proposed Pump Chamber building will be founded below any surface organics and will rest on the silty sands at a depth of approximately 0.8 m. The recommended bearing value of the clay at Serviceability Limit States (SLS) is 40 kPa. Ultimate Limit States (ULS), the bearing value is 250 kPa.

The estimated total settlement of the pump chamber footings designed using the recommended SLS bearings is less than 25 mm. The differential settlement between adjacent column footings is not expected to exceed 12 mm.

To resist differential settlements under liquefaction conditions (as described in Section 5.1.3 of this report), the foundation for the pump chamber may also be designed as rigid raft foundation. Placed at a depth of 0.8 m below the ground, under the SLS conditions described above, the estimated total and differential settlements of the pump chamber foundation are less than 25 mm and 12 mm respectively (under regular, non-liquefaction conditions).

5.1.1.3 Reservoir Foundations

The raft foundation that will support the reservoir is proposed to be placed at a depth of approximately 0.8 m, and the reservoir will be covered with approximately 1.0 m of soil. This embankment over the tank will taper at a grade of 3H:1V away from the edges of the tank. Therefore the increased load on the underlying soils is potentially rather high, and the increased surcharge as a result of the backfill placed against the proposed reservoir walls (i.e. the grade raise) will induce settlements as a result of the deep silty clay soils.

Our consolidation analysis has indicated that the pre-consolidation pressures acting on the tested depth (15 m) are in the order of 220 kPa. Since the calculated effective stress at that depth is 140 kPa. Since the increased applied load due to the embankment will be less than 80 kPa, we remain on the elastic portion of the stress-strain consolidation curve.

If a well-graded sand material is utilized to construct the embankments around the reservoir, these settlements are anticipated to be on the order of 30-50 mm across the entire embankment area, however may range from 70 to 90 mm at the edges of the tank, where the applied load will be at its greatest.

We recommend that the structurally supported raft foundation be designed as a rigid structure, thereby reducing the impact of differential settlement.

These settlements may be reduced by decreasing the total loading on the underlying soils. This may be done by using lightweight fill soils rather than traditional granular fill, or by increasing the depth at which the bottom of the tank rests. The lightweight fill option would decrease the load applied to the soil from the embankment, and therefore would reduce the settlements. Alternatively, lowering the elevation of the base of the reservoir would reduce the elevation above ground surface of the embankment over the tank, thereby reducing the applied load and the associated settlements. It is understood that this option would not be preferable however due to the high water table and the potential cost of treating infiltrating groundwater, as outlined above.

An alternative, but long term method of dealing with the settlement due to fill lading is to preload the Site with earth fill. This process could take up to three years would involve a settlement monitoring program to determine when the settlement has reached an acceptable level for construction.

5.1.1.4 General Foundation Comments

Once the soil has been removed to design foundation level, and all organic soils have been removed, the underlying soil should be compacted with vibratory compaction equipment to ensure that any disturbed soil has been properly consolidated.

Footings set at varying levels and/or constructed adjacent to utility trenches should be constructed such that the higher footings are set at a level below an imaginary line constructed 10H:7V from the base of the lower excavation.

The soil beneath the design foundation level of the pump chamber and the reservoir should be compacted with vibratory compaction equipment to ensure that any disturbed soil has been properly consolidated.

The equivalent frost cover requirement for foundations in the Ottawa area for unheated structures is 1.8 m. However, given that the preference of the Client that soil excavation should be minimized, the foundations of the reservoir may be placed with less soil cover if they are provided with adequate insulation, as is detailed in Dwg. No. T020587-A1-3.

It is recommended that the completed structural drawings be reviewed by **Fondex** for compliance with the report recommendations.

5.1.2 Seismic Classifications

The foundations must be structurally designed to resist a minimum lateral earthquake force as defined by the Ontario Building Code (OBC 2006). In order to accurately determine the site classification for seismic site response, the average shear wave velocity was determined using the MASW procedure.

The average shear wave velocity was determined to be 187.9 m/s, which places the Site as borderline between Site Classes D and E for seismic site response based upon the measured average shear wave velocity of the soils in the area. The results for the Average Shear Wave Velocities based on the tested lines, along with the corresponding graphical representations and the approximate location of the survey lines may be found in Appendix B.

It was noted in our original draft however, that due to the high water table and the relatively loose nature of the native soils, it was considered possible that soils could become liquefiable in case of a severe seismic event.

As part of our expanded mandate, additional information was gathered from the deep borehole (BH-3). Analysis of the risk of liquefaction was conducted by means of MASW test results, SPT testing, and analysis of Atterberg Limits. This analysis has confirmed that there is a risk of liquefaction of the soils in the area.

According to information provided by JL Richards on June 30th, 2009, the fundamental period of vibration for the structures is 0.3 seconds. Our interpretation of the NBC 2005 or OBC 2006 is that for structures having fundamental periods of vibration less than or equal to 0.5 sec, then site specific evaluations are not required to determine spectral accelerations for liquefiable soils. Rather, the Site Class may be determined in accordance with Sec 3.5.2, and the corresponding values of Fa and Fv determined from Tables 3.3-1 and 3.3-2.

Therefore, by following this interpretation, the shear wave velocities from the MASW testing indicate the site is marginal for Site Class D and we therefore recommended Seismic Site Class for design purposes is E and then the values as provided in Tables in the code may be used to determine the structural loading on the tank.

It is important to note that the recommendation for Site Class E is for seismic design purpose only. The Client and the designers must be aware of the risk of liquefaction and its associated settlement during an earthquake event, which is discussed further in the following section.

5.1.3 Liquefaction

The process known as liquefaction occurs under certain conditions during seismic events when the shear stresses build up in the soil under undrained conditions, which increases the pore water pressure of the soil. This increased pore pressure has two major consequences: a loss of bearing capacity and increased settlements. The increased pore pressures push the soil particle apart, which reduces the contact pressure between them and thereby effectively reduces the strength of the soil. This can lead to bearing capacity failures in structures founded on these soils. The change in the shear stresses within the soil during the earthquake also leads to densifying the soil due to the liquefaction event, which can lead to potentially large surface settlements.

The analysis of the data from the site (including grain size analyses, Atterberg Limit testing, MASW testing and analyses, and SPT testing) indicates that there is a risk that under a design earthquake magnitude of 6.1, the sandy silt and clayey silt soils (located within the top 10 m of the overburden) would be come liquefiable. Analysis of deeper soils (i.e. below 10 m) indicates that the silty clay soils are not at risk of liquefaction. Based on our analysis, the settlement of the soils during a liquefaction event could potentially exceed 0.3 m.

If the reservoir structure is designed as a rigid box, and the slab for the pump chamber structure is designed as a structural slab, then the structures themselves would likely survive such an event, however any pipe connections to the structures would likely be broken due to differential settlement. This breakage may be a result of differential settlement between the edge of the tank and the pipe, or by differential settlement across the area of the tank itself.

5.2 Floor Slabs

The floor slab of the pump chamber is expected to be a roughly the existing ground level. Floor slabs may be designed as conventional slab-on-grade. The native inorganic soils to the east of the parking area must be exposed by topsoil stripping methods. Following stripping of existing surface organics and deleterious fill soils, the subgrade should be proof-rolled to ensure the exposed subgrade is consistent and has no local anomalies or soft areas.

It is recommended that the base slab be poured on a minimum of 200 mm of compacted Granular ‘A’ material, as per the Ontario Provincial Standards and Specification (OPSS).

It is not anticipated that grades will need to be raised to accommodate the foundations of the reservoir. However, if any grade raises below the footings are contemplated, or if there is a requirement for fill below the slab, the fill material should be treated as Engineered Fill.

To be considered Engineered Fill, the fill operations must satisfy the following criteria:

- Engineered Fill must be placed under continuous supervision by field technicians under the direct supervision of the Geotechnical Engineer. Prior to placing any Engineered Fill, all unsuitable fill materials must be removed, the subgrade must be investigated for old buried fill or deleterious material, the subgrade must be proof-rolled, and the subgrade elevations must be surveyed;

- Prior to the placement of Engineered Fill, the source or borrow areas for the Engineered Fill must be evaluated for its suitability. Samples of proposed fill material must be provided to the Geotechnical Engineer and tested in the geotechnical laboratory for Standard Proctor Maximum Dry Density (SPMDD) and grain size, prior to approval of the material for use as Engineered Fill. The Engineered Fill must consist of environmentally suitable, free of organics and other deleterious material (building debris such as wood, bricks, metal, and the like), well graded, compactable, and of suitable moisture content so that it is within -2 to +0.5% of the Optimum Moisture as determined by the Standard Proctor Test. Material meeting the grading of OPSS Granular B Type 'I' will fit these requirements;

- The Engineered Fill must be placed in maximum loose lift thicknesses of 200 to 300 mm. Each lift of Engineered Fill must be compacted with a heavy roller, to 100 percent SPMDD; and

- Field density tests must be taken by field technicians under the supervision of a Geotechnical Engineer, on each lift of the Engineered Fill. Any Engineered Fill, which is tested and found to not meet the specifications, shall be either removed or reworked and retested.

All fill operations below the proposed footings or floor area of the proposed building should be considered to be Engineered Fill areas.

Soils should be handled, transported and disposed of in an environmentally suitable manner, meeting current environmental legislation.

If the reservoir is to be located below the perched water table, the tank should be designed to resist uplift forces when it is empty.

5.3 Watermain Installation

The depth to which the watermain will be installed will be dependent on the depth at which the reservoir is set. However, it is understood that proposed depth in an ideal situation, the pipe would be located at a depth of 2.4 m below the surface grade.

According to Table III above, if the watermain is to be installed at this depth, the base of the excavation will be well below the water table, in running sand, and will require the pumping of infiltrating water with a positive dewatering system to ensure a stable base for construction. A wellpoint system or deep wells are two examples of positive dewatering systems that by be contemplated. Based on our hydrogeological testing, we anticipate that the infiltration rate at steady state conditions at this depth will be approximately 16,000 to 20,000 L/day. However it should be noted that to bring the excavation to steady state conditions or during significant precipitation events or wet seasons, the limit of 50,000 L/day may be exceeded, and Permits to Take Water may be required.

Of greater concern is the possibility that the groundwater and the soils in the area are contaminated, and will require appropriate disposal and/or treatment. As the details of this treatment are not a part of the mandate of this report, it is assumed for the purposes of this report that any groundwater that infiltrates into excavations will be contaminated, and the Client is advised to carry out further investigation to properly assess this issue.

Given the cost of treating the infiltrating groundwater, it will be preferable to install the watermain at higher elevations. If the base of the watermain is located at a depth of 1.0 m, frost protection may be provided in the form of a layer of high density foam insulation 75 mm thick placed on top of the soil cover over the watermain. This insulating layer must also extend out a minimum of 1.4 m to either side of the watermain.

As the insulation will prevent frost penetration in the area of the watermain, it is probable that there will be some differential frost heave between the backfilled soils over the watermain and the surrounding soils. Frost Tapers are recommended to minimize the impact of differential frost heave on the road surface. However, given that the driveway surface is gravel, the effects of differential frost heave may not be as noticeable and annual maintenance would be easier than it would for an asphaltic concrete pavement.

Pipe bedding should consist of materials meeting OPSS Granular 'A' requirements. The bedding should have a minimum thickness of 150 mm below the pipe and 300 mm above and adjacent to the pipe and should comply with the OPSS. The bedding and cover materials should be compacted to a minimum of 95 percent of its SPMDD.

5.4 *Excavation & Dewatering*

All excavations should be completed and maintained in accordance with the Occupational Health and Safety Act (OHSA) requirements. The following recommendations for excavations should be considered to be a supplement to, not a replacement of, the OHSA requirements.

The amount of groundwater seepage is largely dependent on the depth to which the reservoir and the watermain are set in the ground. If both are set at approximately 1 m below the present grades, groundwater seepage is not expected to be a major concern and normal construction is anticipated.

At greater depths (i.e. more than 1 m below present grades) infiltrating water from the water table will be a concern, and will have to be controlled by means of pumping from sumps or a wellpoint dewatering system. Given that this water may be contaminated, this may require pumping into a collection tank or some other form of water capture for treatment, prior to discharge. The cost of hauling away and treating contaminated groundwater is approximately $0.25/L, excluding the cost of groundwater collection measures, chemical analysis of the collected groundwater, permit requirements, etc. Also, as stated previously, there may be a need for a Permit to Take Water from the Ontario Ministry of Environment.

Greater volumes of seepage may be encountered if excavations are conducted during wet seasonal periods.

The native silty sand overburden soils encountered in the boreholes considered to be Type 3 soils, but are Type 4 soils below the water table, as defined by the OHSA Regulations for Construction

5.5 *Earth Pressures on Backfilled Walls*

The earth pressure on backfilled walls can be estimated using an earth pressure coefficient K_a=0.35 and a hydrostatic pressure distribution. The earth pressure diagram for the design of the tank walls is provided on Dwg. No. T020587-A1-4. Generally, the additional lateral force due to earthquake loading can be compensated for by using the "at rest" earth pressure coefficient K_o=0.5.

5.6 *Backfill/Permanent Drainage*

Reservoir wall backfill should be in accordance with the OBC 2006 requirements and should include free draining backfill as discussed later.

An exterior drainage is recommended if the foundations of the tank are to be set at depths more than 1 m below present grades. However, additional investigation and assessment (both for chemical and design purposes) is recommended if the Client chooses this option. Generally, the drainage system should consist of a perforated tile surrounded by clear stone and wrapped with geofabric. The drainage system should be connected to a frost-free outlet. Given the potential for this groundwater to be contaminated the outlet should abide by current Ministry of Environment and City of Ottawa regulations. The alternative is to design the structure as a "tank" and account for hydrostatic pressure and to delete the perimeter drainage.

The backfill placed against foundation walls should be free draining materials, such as the OPSS Granular 'B' Type I, pit run gravel or better.

Foundation backfill should be placed and compacted as outlined below:

- Free-draining backfill should be used for the outside of the foundation wall;
- Backfill should be placed and compacted in uniform lift thickness compatible with the selected construction equipment, but not thicker than 200 mm;
- Backfill should not be placed in a frozen condition, or placed on a frozen subgrade;
- For backfill that would underlie paved areas, sidewalks or slabs-on-grade, each lift should be uniformly compacted to at least 98 percent of its SPMDD;
- For backfill on the building exterior that would underlie landscaped areas, each lift should be uniformly compacted to at least 95 percent of its SPMDD; and
- Exterior grades should be sloped away from the foundation wall, and roof drainage downspouts should be placed so that water flows away from the foundation wall.

Any fill used to raise the grade beneath floor slabs or foundations should be compacted to 100 percent of its SPMDD.

Bedding for service pipes should conform to type and dimension with local municipal requirements. Clear stone is not recommended as a bedding material. Sand cover is recommended to be placed on top of pipes with a minimum cover of 300 mm.

The trench backfill should be able to re-use the excavated soils subject to the soils being at suitable moisture content for compaction. If work is done in wet seasonal conditions then the excavated material should be stockpiled and protected from moisture infiltration and/or wet material will require time to drain to achieve suitable moisture content.

5.7 Construction Field Review

The recommendations provided in this report are based on an adequate level of construction monitoring being conducted during construction of the proposed facilities. Due to the nature of the proposed development, an adequate level of construction monitoring is considered to be as follows:

- Prior to construction of footings, the exposed foundation subgrade should be examined by a geotechnical engineer or a qualified technologist acting under the supervision of a geotechnical engineer, to assess whether the subgrade conditions correspond to those encountered in the test pits, and the recommendations provided in this report have been implemented.
- A qualified technologist acting under the supervision of a geotechnical engineer should monitor placement of engineered fill underlying footings and floor slabs.
- Backfilling operations should be conducted in the presence of a qualified technologist to ensure that proper material is employed and specified compaction is achieved.
- Qualified personnel should conduct testing of concrete.

6.0 LIMITATIONS OF THE INVESTIGATION

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to **Fondex** at the time of preparation. No portion of this report may be used as a separate entity, it is written to be read in its entirety. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete, or if the proposed construction should differ from that mentioned in this report.

It is also important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments are based on the results obtained at the test locations only. It is, therefore, assumed that these results are representative of the subsoil conditions across the site. Should any conditions at the site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations.

FONDEX ONTARIO LIMITED

JBBennett

REGISTERED PROFESSIONAL ENGINEER
J. B. BENNETT
PROVINCE OF ONTARIO

Joseph B. Bennett, P. Eng.
Vice-President

JBB/vl

Enclosures:

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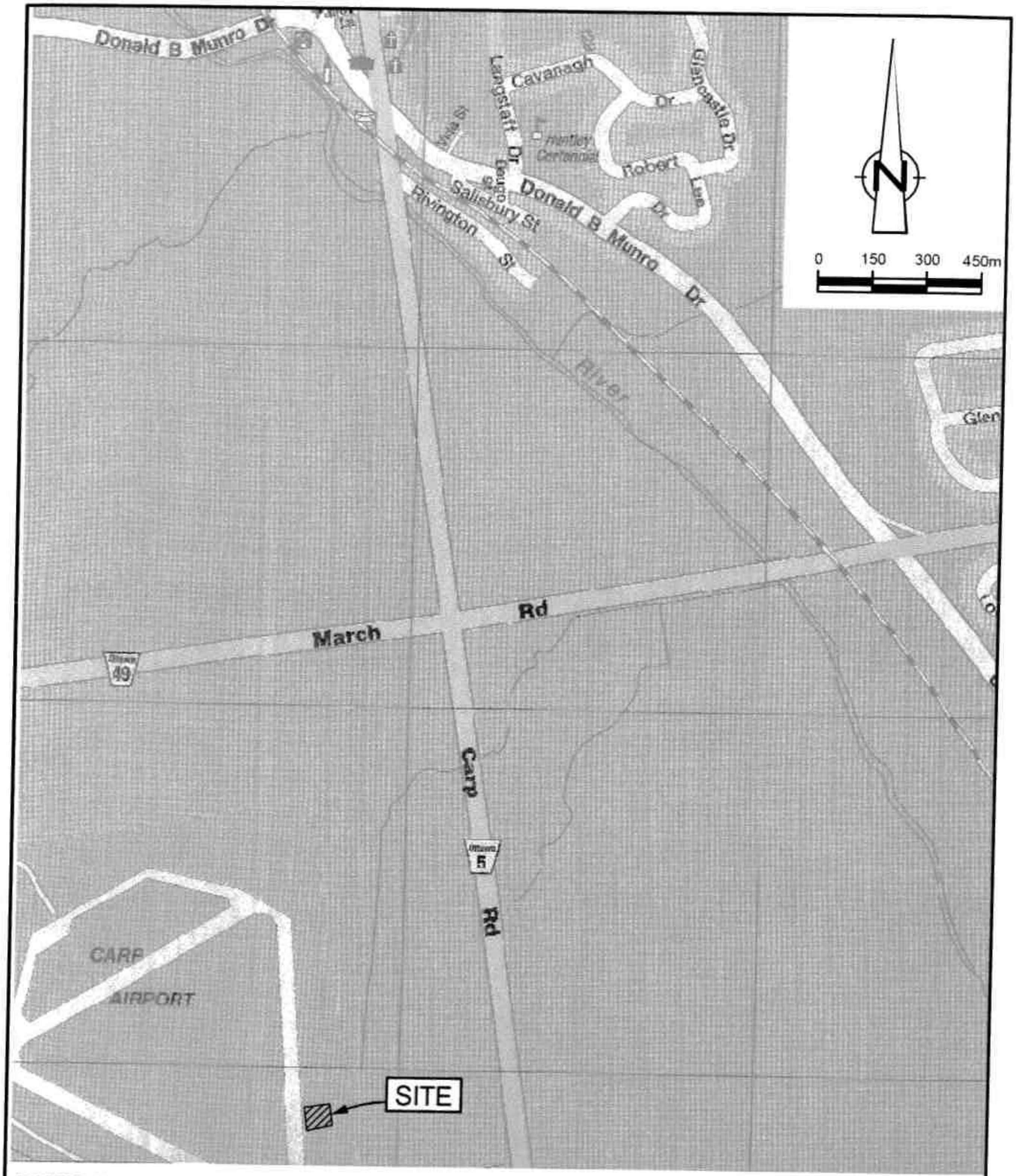
Mr. Lee Jablonsky- email - (ljablonski@jlrichards.ca)

DRAWINGS

SITE LOCATION MAP
BOREHOLE LOCATION PLAN
SUGGESTED FOOTING INSULATION DETAIL
EARTH PRESSURE ON BACKFILLED WALLS

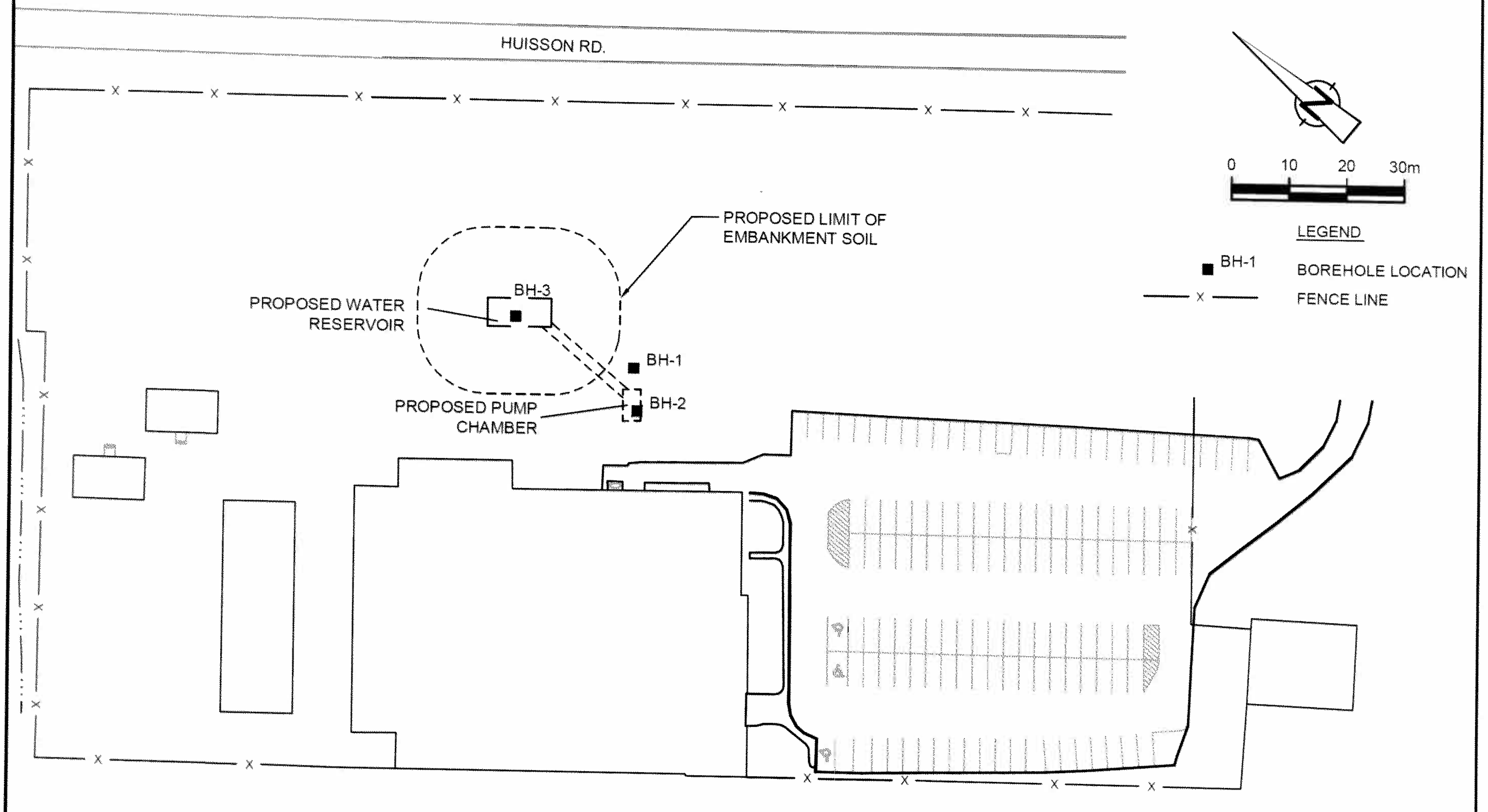


SOURCE: MAPART OTTAWA

SITE LOCATION MAP
GEOTECHNICAL INVESTIGATION
PROPOSED WATER RESERVOIR - CARP AIRPORT
CARP, ONTARIO
T020587-A1-1

FONDEX

T020587-A1(001)GN-OT001 FEB 24/2009

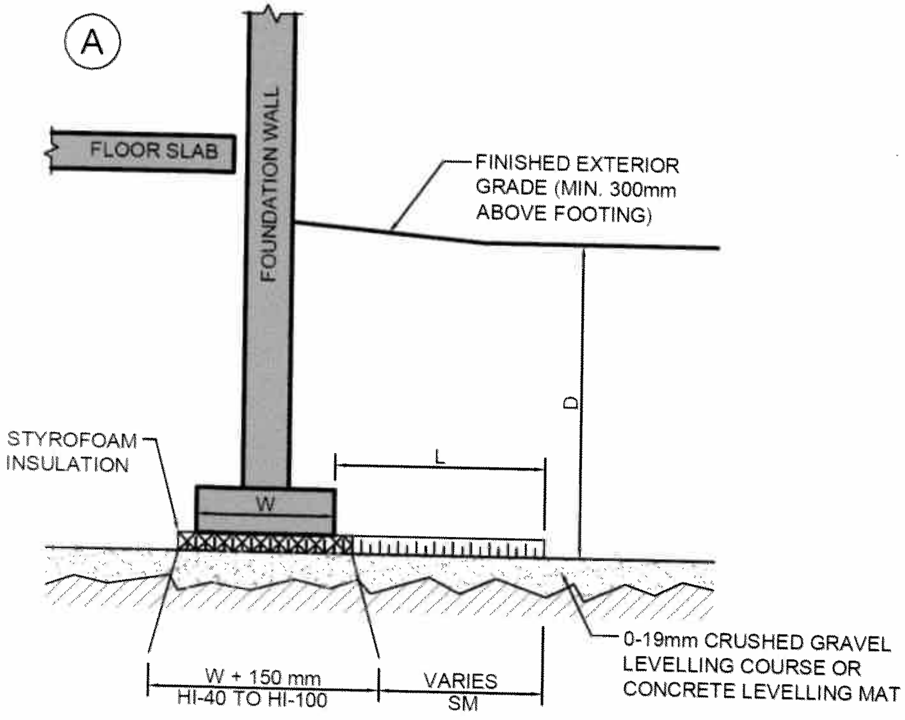


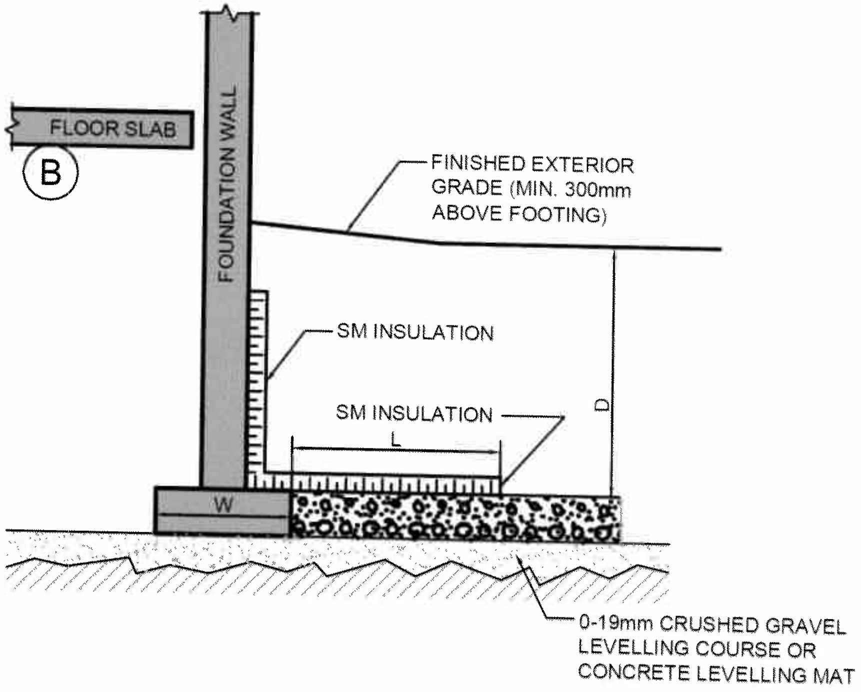
FONDEX

BORHEOLE LOCATION PLAN
GEOTECHNICAL INVESTIGATION
PROPOSED WATER RESERVOIR - CARP AIRPORT
CARP, ONTARIO

T020587-A1-2

T020587-A1(001)GN-OT002 JUN 08/2009





NOTES:
1) INSULATION JOINTS TO BE LAPPED.
2) FOR DESIGN BEARING PRESSURES ≤ 100 kPa USE DOW HI-40.
FOR DESIGN BEARING PRESSURES ≤ 150 kPa USE DOW HI-60.
FOR DESIGN BEARING PRESSURES < 250 kPa USE DOW HI-100.
3) FOR D < 550mm USE 75mm THICKNESS OF INSULATION.
FOR D > 550mm USE 50mm THICKNESS OF INSULATION.
4) REQUIRED EXTENSION L = (1500 - D)mm.
5) CONCRETE LEVELLING MAT MAY BE DELETED IF SITE CONDITIONS ARE FOUND TO BE FAVOURABLE AT THE TIME OF CONSTRUCTION.
6) ALTERNATIVE FOOTING INSULATION DETAILS SHOULD BE REVIEWED BY FONDEX.

WHERE
W = FOOTING WIDTH
D = DEPTH OF EARTH COVER MEASURED FROM BOTTOM OF INSULATION. A MINIMUM OF 350mm IS RECOMMENDED.
L = LENGTH OF INSULATION EXTENDING FROM OUTSIDE FOOTING EDGE.

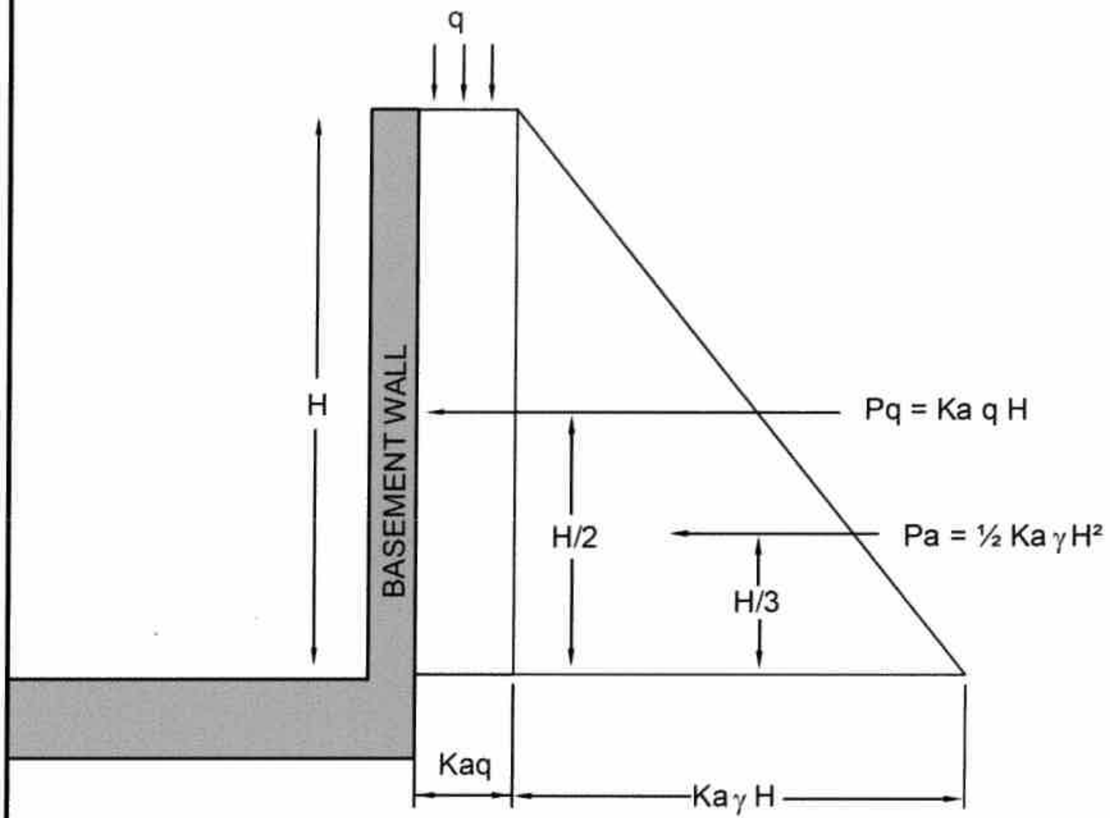
FONDEX

SUGGESTED FOOTING INSULATION DETAIL
GEOTECHNICAL INVESTIGATION
PROPOSED WATER RESERVOIR - CARP AIRPORT
CARP, ONTARIO
T020587-A1-3

T020587-A1(001)GN-OT003 FEB 24/2009

SKETCH OF EARTH PRESSURE ON BASEMENT WALL

THE EARTH PRESSURE ACTING ON RETAINING WALLS CAN BE CALCULATED USING THE FOLLOWING EARTH PRESSURE DIAGRAM.



ASSUME

H = HEIGHT OF RETAINED SOIL (m).

K_a = 0.35 WHERE SOME SLIGHT MOVEMENT IS PERMISSIBLE.

K_a = 0.5 FOR SEISMIC LOADING.

γ = 20 kN/m³ FOR FREE DRAINING GRANNULAR BACKFILL.

q = THE VALUE OF ANY SURCHARGE LOADING WHICH MAY ACT ON THE GROUND SURFACE ADJACENT TO THE WALL.

WITH THE ABOVE DIAGRAM, IT HAS BEEN ASSUMED THAT AN EFFECTIVE DRAINAGE SYSTEM HAS BEEN INSTALLED, AND THERE IS NO BUILD UP OF HYDROSTATIC PRESSURE.

EARTH PRESSURE ON BACKFILLED WALLS
GEOTECHNICAL INVESTIGATION
PROPOSED WATER RESERVOIR - CARP AIRPORT
CARP, ONTARIO
T020587-A1-4

ENCLOSURES

BOREHOLE LOGS

REFERENCE No.: T020587-A1

ENCLOSURE No.: 1

BOREHOLE No.: BH-1

ELEVATION: 113.07 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: PWGSC

PROJECT: Geotechnical Investigation - Carp Air Hanger Development

LOCATION: Carp, Ontario

DESCRIBED BY: A.Solomon

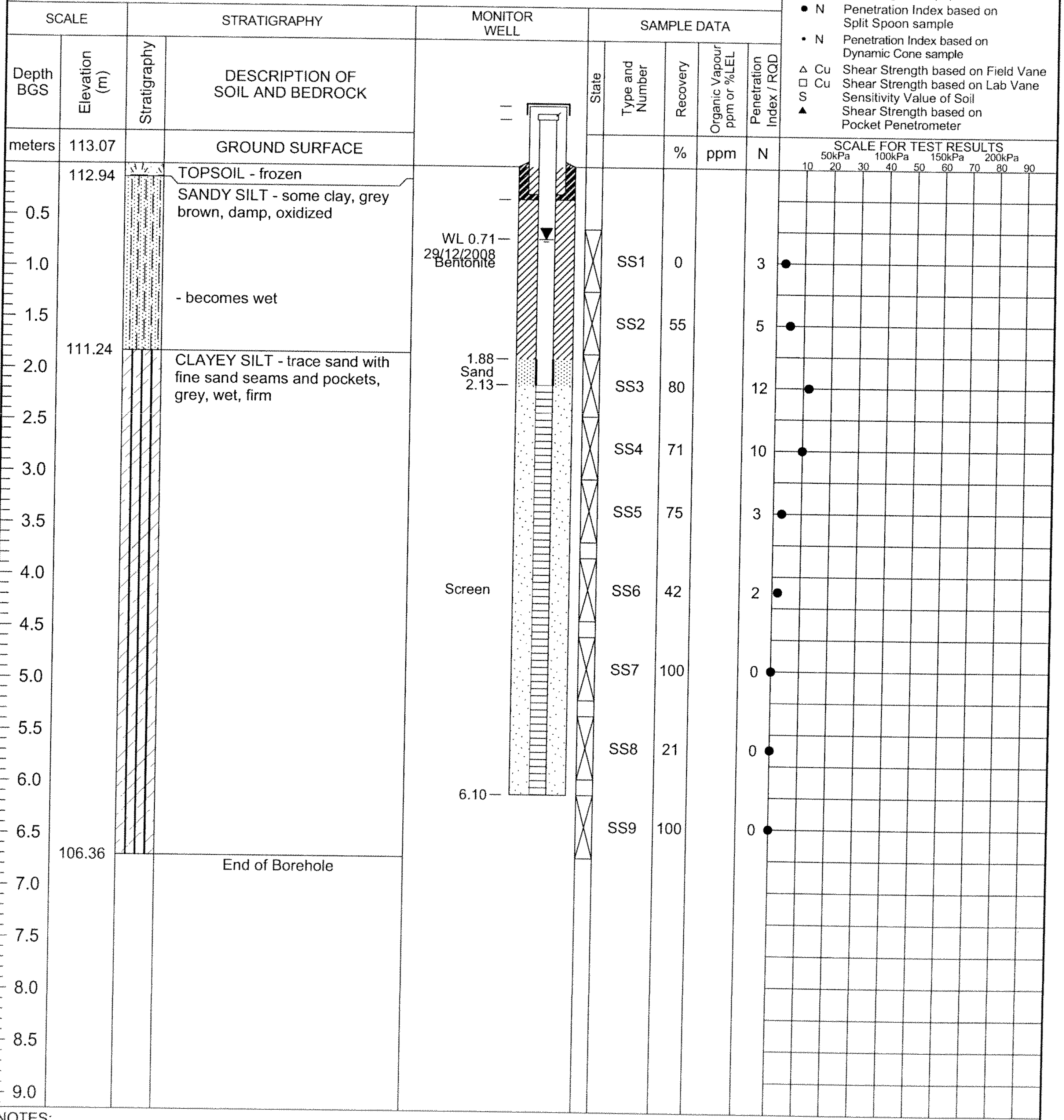
CHECKED BY: J.Bennett

DATE (START): December 12, 2008

DATE (FINISH): December 12, 2008

LEGEND

☒	SS	Split Spoon
▨	ST	Shelby Tube
▮	RC	Rock Core
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer



Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK
113.07	GROUND SURFACE
112.94	TOPSOIL - frozen
	SANDY SILT - some clay, grey brown, damp, oxidized - becomes wet
111.24	CLAYEY SILT - trace sand with fine sand seams and pockets, grey, wet, firm
106.36	End of Borehole

State	Type and Number	Recovery %	Organic Vapour ppm or %LEL ppm	Penetration Index / RQD N
☒	SS1	0		3
☒	SS2	55		5
☒	SS3	80		12
☒	SS4	71		10
☒	SS5	75		3
☒	SS6	42		2
☒	SS7	100		0
☒	SS8	21		0
☒	SS9	100		0

NOTES:
N-Value of 0 represents spoon advancement under weight of hammer.

REFERENCE No.: T020587-A1

ENCLOSURE No.: 2

FONDEX

BOREHOLE No.: BH-2

ELEVATION: 113.20 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: PWGSC

PROJECT: Geotechnical Investigation - Carp Air Hanger Development

LOCATION: Carp, Ontario

DESCRIBED BY: A.Solomon

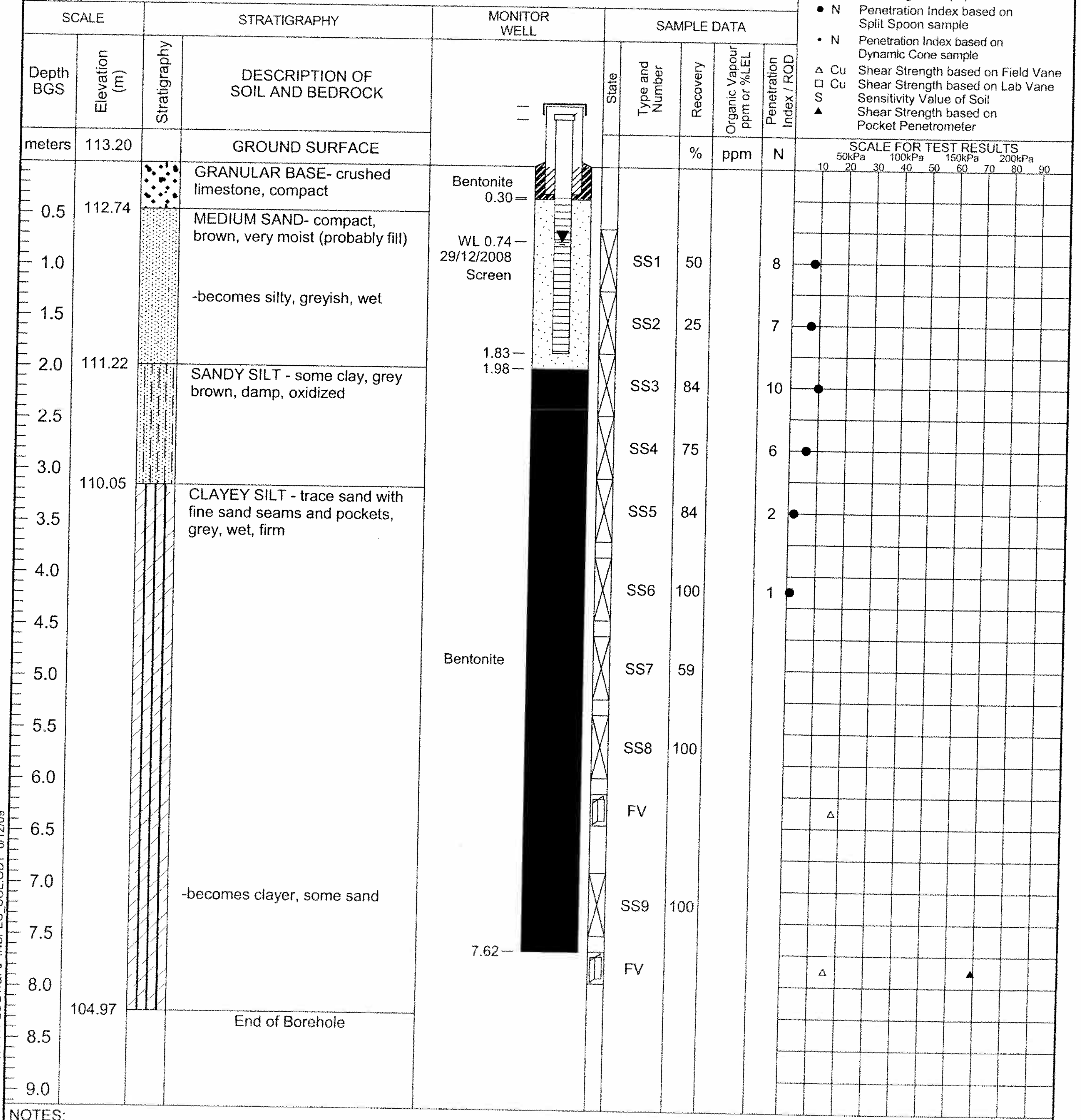
CHECKED BY: J.Bennett

DATE (START): December 12, 2008

DATE (FINISH): December 12, 2008

LEGEND

- ☒ SS Split Spoon
- ▨ ST Shelby Tube
- ▯ RC Rock Core
- ▼ Water Level
- ○ Water content (%)
- ├─┤ Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer



Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK
113.20	GROUND SURFACE
	GRANULAR BASE- crushed limestone, compact
112.74	MEDIUM SAND- compact, brown, very moist (probably fill) -becomes silty, greyish, wet
111.22	SANDY SILT - some clay, grey brown, damp, oxidized
110.05	CLAYEY SILT - trace sand with fine sand seams and pockets, grey, wet, firm -becomes clayer, some sand
104.97	End of Borehole

MONITOR WELL: Bentonite 0.30; WL 0.74 29/12/2008; Screen; 1.83; 1.98; Bentonite; 7.62

Type and Number	Recovery %	N
SS1	50	8
SS2	25	7
SS3	84	10
SS4	75	6
SS5	84	2
SS6	100	1
SS7	59	
SS8	100	
FV		
SS9	100	
FV		

NOTES:
N-Value of 0 represents spoon advancement under weight of hammer.

BOREHOLE LOG T020587-A1-LOG1.GPJ INSPEC_SOL.GDT 6/12/09

REFERENCE No.: T020587-A1

ENCLOSURE No.:

FONDEX

BOREHOLE No.: BH3

ELEVATION: 112.99 m

BOREHOLE LOG

Page: 1 of 4

CLIENT: PWGSC

PROJECT: Geotechnical Investigation

LOCATION: Carp Airport - RCMP Hanger

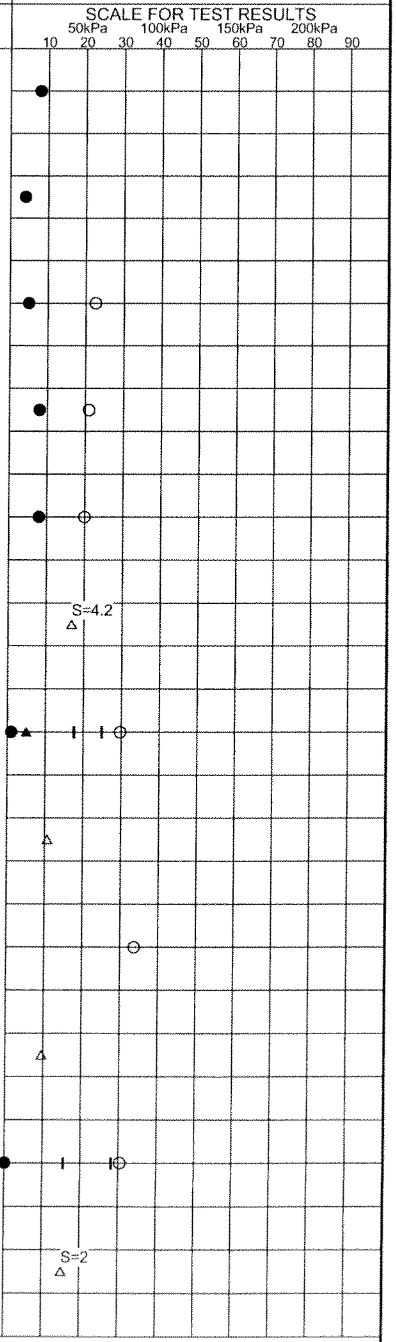
DESCRIBED BY: S. Dunstan CHECKED BY: J. Bennett

DATE (START): April 30, 2008 DATE (FINISH): May 1, 2008

LEGEND

☒	SS	Split Spoon
▨	ST	Shelby Tube
▯	RC	Rock Core
▼		Water Level
○		Water content (%)
⊢⊣		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY	SAMPLE DATA			
Depth BGS	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK	Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD
meters	112.99	GROUND SURFACE		%	ppm	N
		TOPSOIL (POSSIBLY FILL)	SS1	17		8
0.5						
1.0	111.95	SANDY SILT - some clay, grey, damp	SS2	67		4
1.5						
2.0		-becoming moist	SS3	67		5
2.5		- becoming wet	SS4	54		8
3.0	109.87	CLAYEY SILT, trace sand with fine sand seams and pockets, grey, wet, firm	SS5	50		8
3.5						
4.0			FV6			
4.5						
5.0			SS7	100		1
5.5			FV8			
6.0						
6.5			GS9	83		
7.0			FV10			
7.5						
8.0			SS11	100		0
8.5						
9.0			FV12	100		



NOTES:
N-Value of 0 represents spoon advancement under weight of hammer.
SS4 Atterburg Limits: Liquid Limit = 23.2%, No-Plastic

REFERENCE No.: T020587-A1

ENCLOSURE No.:

FONDEX

BOREHOLE No.: BH3

ELEVATION: 112.99 m

BOREHOLE LOG

Page: 2 of 4

CLIENT: PWGSC

PROJECT: Geotechnical Investigation

LOCATION: Carp Airport - RCMP Hanger

DESCRIBED BY: S. Dunstan

CHECKED BY: J. Bennett

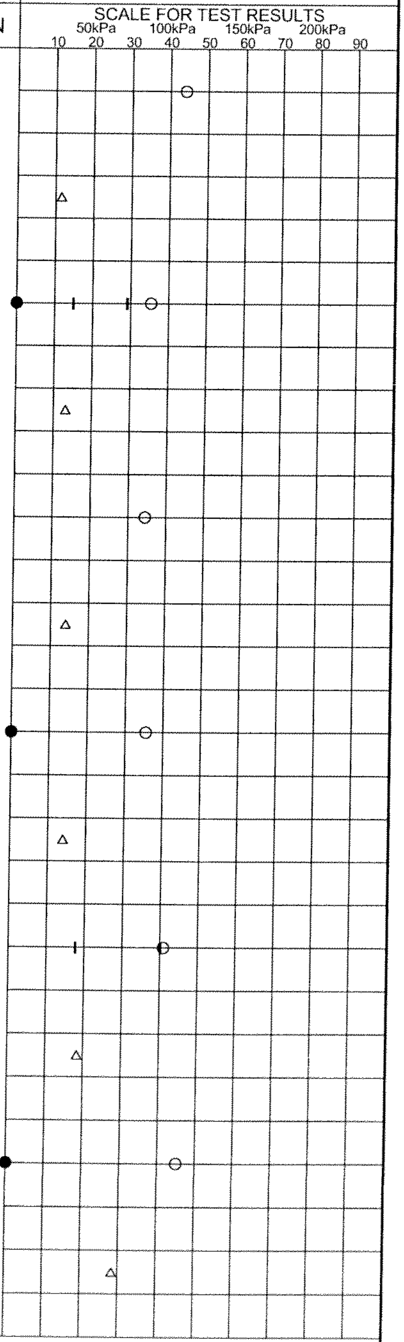
DATE (START): April 30, 2008

DATE (FINISH): May 1, 2008

LEGEND

☒ SS	Split Spoon
▨ ST	Shelby Tube
▮ RC	Rock Core
▼	Water Level
○	Water content (%)
├─┤	Atterberg limits (%)
● N	Penetration Index based on Split Spoon sample
• N	Penetration Index based on Dynamic Cone sample
△ Cu	Shear Strength based on Field Vane
□ Cu	Shear Strength based on Lab Vane
S	Sensitivity Value of Soil
▲	Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY		SAMPLE DATA			
Depth BGS	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD
meters	112.99	GROUND SURFACE			%	ppm	N
9.5				ST13			
10.0	102.99	SILTY CLAY- trace sand with fine sand seams and pockets, grey, wt, firm		FV14			
11.0				SS15	100		0
11.5				FV16	100		
12.5				ST17			
13.0				FV18	100		
14.0				SS19	100		0
14.5				FV20	100		
15.5				ST21			
16.0				FV22	100		
17.0				SS23	0		0
18.0				FV24	100		



NOTES:

N-Value of 0 represents spoon advancement under weight of hammer.

SS4 Atterburg Limits: Liquid Limit = 23.2%, No-Plastic

REFERENCE No.: T020587-A1

ENCLOSURE No.:

BOREHOLE No.: BH3

ELEVATION: 112.99 m

BOREHOLE LOG

Page: 3 of 4

CLIENT: PWGSC

PROJECT: Geotechnical Investigation

LOCATION: Carp Airport - RCMP Hanger

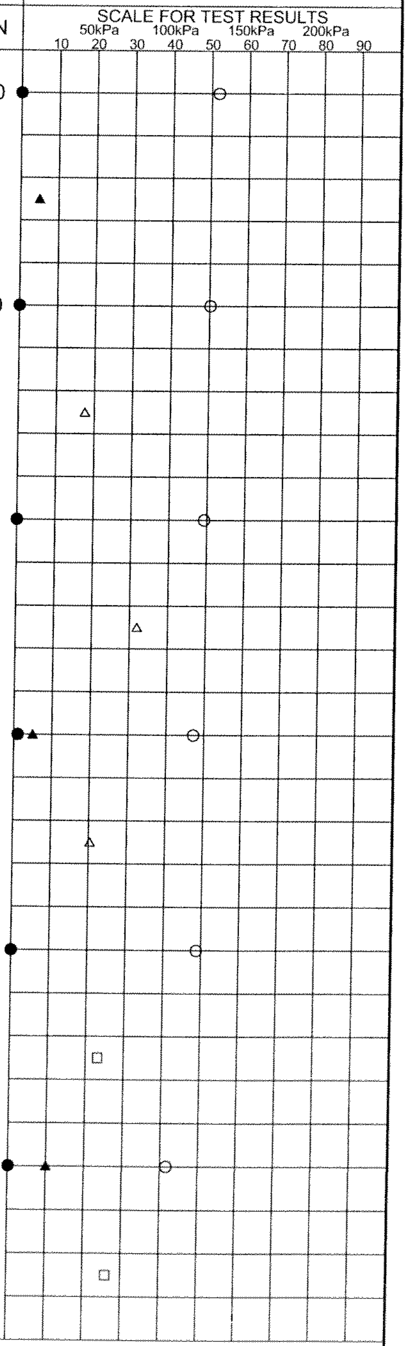
DESCRIBED BY: S. Dunstan CHECKED BY: J. Bennett

DATE (START): April 30, 2008 DATE (FINISH): May 1, 2008

LEGEND

☒	SS	Split Spoon
▨	ST	Shelby Tube
▮	RC	Rock Core
▼		Water Level
○		Water content (%)
⊢⊣		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
	S	Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY		SAMPLE DATA				
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD
meters	112.99		GROUND SURFACE			%	ppm	N
18.5								
				☒	SS25	100		0
19.0								
					FV26			
19.5								
20.0								
				☒	SS27	100		0
20.5								
					FV28			
21.0								
21.5								
				☒	SS29	100		0
22.0								
					FV30			
22.5								
23.0								
				☒	SS31	100		1
23.5								
					FV32			
24.0								
24.5								
				☒	SS33	100		0
25.0								
				■	ST34			
25.5								
26.0								
				☒	SS35	100		0
26.5								
27.0								
				■	ST36			



NOTES:
N-Value of 0 represents spoon advancement under weight of hammer.
SS4 Atterburg Limits: Liquid Limit = 23.2%, No-Plastic

REFERENCE No.: T020587-A1

ENCLOSURE No.:

FONDEX

BOREHOLE No.: BH3

ELEVATION: 112.99 m

BOREHOLE LOG

Page: 4 of 4

CLIENT: PWGSC

PROJECT: Geotechnical Investigation

LOCATION: Carp Airport - RCMP Hanger

DESCRIBED BY: S. Dunstan

CHECKED BY: J. Bennett

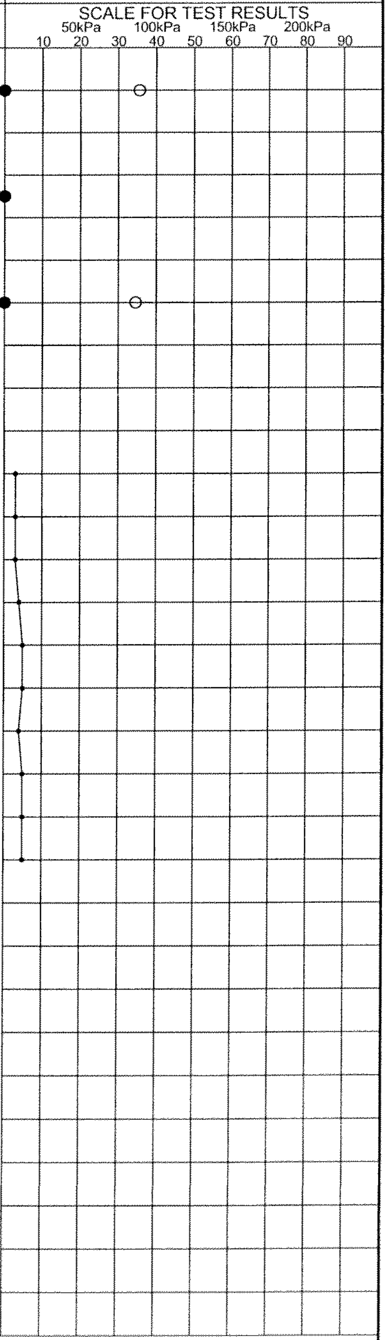
DATE (START): April 30, 2008

DATE (FINISH): May 1, 2008

LEGEND

☒ SS	Split Spoon
▨ ST	Shelby Tube
RC	Rock Core
▼	Water Level
○	Water content (%)
├─┤	Atterberg limits (%)
● N	Penetration Index based on Split Spoon sample
• N	Penetration Index based on Dynamic Cone sample
△ Cu	Shear Strength based on Field Vane
□ Cu	Shear Strength based on Lab Vane
S	Sensitivity Value of Soil
▲	Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY		SAMPLE DATA				
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD
meters	112.99		GROUND SURFACE			%	ppm	N
27.5		▨		☒	SS37	16		0
28.0		▨						
28.5		▨		☒	SS38	100		0
29.0		▨		☒	SS39	100		0
29.5	83.42		END OF BOREHOLE					
30.0								
30.5								
31.0								
31.5								
32.0								
32.5								
33.0								
33.5								
34.0								
34.5								
35.0								
35.5								
36.0								
36.5								



NOTES:
N-Value of 0 represents spoon advancement under weight of hammer.
SS4 Atterburg Limits: Liquid Limit = 23.2%, No-Plastic

APPENDICES

APPENDIX A

HYDRAULIC ANALYSES

APPENDIX A – HYDRAULIC TESTING

1.0 SLUG TESTING RESULTS

Slug testing of monitor wells advanced into the area of the proposed construction revealed

- hydraulic conductivity = k = 2.3×10^{-7} m/s in the native silt with sand/clay soils (BH 1).
    - This value is assumed to be representative of the native soils below the surficial granular soils.
- hydraulic conductivity = k = 6.7×10^{-7} m/s in granular surficial soils (BH 2).
    - This value is assumed to be representative of the surficial granular soils.

The depth at which the reservoir will be placed is not yet known, so hydraulic calculations were made for different excavation depths

2.0 FLOW CALCULATIONS

2.1 Radius of Influence – Reservoir Excavation

For a partially penetrating well with gravity flow from a circular source, the radius of influence (R) may be approximated from the equations:

$$R = r_w + R_{29}$$

and

$$R_{height} = C(H-h_w)(k)^{0.5}$$

Where: R_{height} = radius of influence

C = 3 for artesian and gravity flows to a well

H = groundwater thickness in unconfined aquifer before pumping (ft)

h_w = groundwater thickness in unconfined aquifer after pumping(ft)

k = hydraulic conductivity of rock mass (expressed in $x10^{-4}$cm/s)

r_w = radius of well (ft)

EXAMPLE:. for 12' deep excavation *(3.6 m)*

$R_{12'} = C(H-h_w)(k)^{0.5}$

$= (3)(17.7'-8')(0.2)^{0.5}$

= 13.0' = 4.0 m

$$\begin{aligned} \text{and } R &= r_w + R_{12} \\ &= (31') + (13.0') \\ &= 44.0' = 13.4 \text{ m} \end{aligned}$$

For the purposes of calculations, based on an hydraulic conductivity of $k = 2\text{x}10^{-7}$ m/s, then **R = radius of influence = 13.4 m** from centre of excavation.

2.2 Groundwater Flow into Reservoir Excavation

For a partially penetrating well with gravity flow from a circular source, the flow may be approximated from the equation:

$$Q_w = \frac{\pi k [(H-s)^2 - t^2] [1 + (0.30 + 10r_w/H) \sin(1.8s/H)]}{\ln(R/r_w)}$$

where Q_w = flow
H = groundwater thickness in unconfined aquifer before pumping (ft)
h_w = groundwater thickness in unconfined aquifer after pumping(ft)
s = aquifer thickness below well base (ft)
t = water thickness in well after pumping (ft)
r_w = radius of well (ft)
k = hydraulic conductivity (ft/day)
R = radius of influence (ft)

EXAMPLE:. for 12' deep excavation *(3.6 m)*

Partially solving this equation for flow (Q_w),

$$Q_w = \frac{\pi k [(H-s)^2 - t^2] [1 + (0.30 + 10r_w/H) \sin(1.8s/H)]}{\ln(R/r_w)}$$

$$= \frac{\pi (k) [(17.7ft-8ft)^2-(0ft)^2] [1 + (0.30 + (10)(31ft)/(17.7ft)] \sin[(1.8)(8ft)/(17.7ft)]}{\ln[(R)/(31ft)]}$$

$$= \frac{(k)\, \pi\, [94.1]\, [18.81]\, [0.7267]}{\ln[R/31\text{ft}]}$$

$$Q_w = \frac{(k)\, 4040.95}{\ln[R/31\text{ft}]} \; \frac{\text{ft}^3}{\text{day}}$$

For the purposes of calculations, the conservative values will be used of
k = hydraulic conductivity = $0.2\text{x}10^{-4}$ cm/s or $5.7\text{x}10^{-2}$ft/day
R = radius of influence = 13.4 m = 44.1 ft

$$Q_w = \frac{(k)\ 38903}{\ln[R/31]}\ \frac{ft^3}{day} = \frac{(0.057)(4040.95)}{\ln(44.1/31)} = 672\ ft^3/day = 19{,}025\ L/day$$

Applying a standard safety factor (F) of 1.5
$Q = (F)(Q_w)$
$= (1.5)(19{,}025)$
$= 28{,}537$ L/day

Including the influence of precipitation (assumed to be less than 0.02 m/day)
$Q = (Q_{with\ (F)})(A_{exc})(d_{ppt})$

where $Q_{with(F)}$ = flow with applied Safety Factor
A_{exc} = excavation surface area
d_{ppt} = depth of precipitation for 24 hr period

EXAMPLE:. for 12' deep excavation *(3.6 m)*
$Q = (Q_{with\ (F)})(A_{exc})(d_{ppt})$
$= (28{,}537\ L/day)(281\ m^2)(0.02\ m)$
$= 34{,}158$ L/day

Based on these calculations, the anticipated groundwater infiltration into the excavation for the RESERVOIR were calculated and presented in Table 1 of the main report.

2.3 Radius of Influence – Reservoir Excavation

$$R_{height}=C(H-h_w)(k)^{0.5}$$

Where: R_{height} = radius of influence
C = 2 for a line of wells
H = groundwater thickness in unconfined aquifer before pumping (ft)
h_w = groundwater thickness in unconfined aquifer after pumping(ft)
k = hydraulic conductivity of rock mass (expressed in $\text{x}10^{-4}$cm/s)
r_w = radius of well (ft)

EXAMPLE:. for 8' deep excavation *(2.4 m)*

$R_{12'} = C(H-h_w)(k)^{0.5}$

$= (2)(17.7'-12')(0.2)^{0.5}$

$= 5.1' = 1.6$ m

2.4 *Groundwater Flow into Trench Excavation*

For a groundwater that flows into one side of a slot, the flow may be approximated from the equation:

$$Q_w = \frac{(0.73 + (0.27))(H- h_0)/H)(k)(X)\ [H^2 - h_0^2]}{2L}$$

where Q_w = flow

H = vertical distance base of static level to base of aquifer (ft)

h_0 = vertical distance base of trench to base of aquifer (ft)

k = hydraulic conductivity (ft/day)

X = horizontal length of trench (ft)

L = lesser of horizontal distance of trench wall to source or R_{height} (ft)

EXAMPLE:. for 8' deep excavation *(2.4 m)*

$$Q_w = \frac{(0.73 + (0.27))(17.7\,ft- 12\,ft)/H)(k)(X)\ [17.7\,ft^2 - 12\,ft^2]}{2(2\,ft)}$$

Q_w = 4,719 L/day

Applying a standard safety factor (F) of 1.5

$Q = (F)(Q_w)$

$= (1.65)(4,719$ L/day)

$= 7,800$ L/day

Assuming groundwater infiltration will be from both sides of excavation

$Q = 15,600$ L/day

Including the influence of precipitation (assumed to be less than 0.02 m/day)

$Q = (Q_{with\ (F)})(A_{exc})(d_{ppt})$

where $Q_{with(F)}$ = flow with applied Safety Factor

A_{exc} = excavation surface area

d_{ppt} = depth of precipitation for 24 hr period

EXAMPLE:. for 12' deep excavation *(3.6 m)*

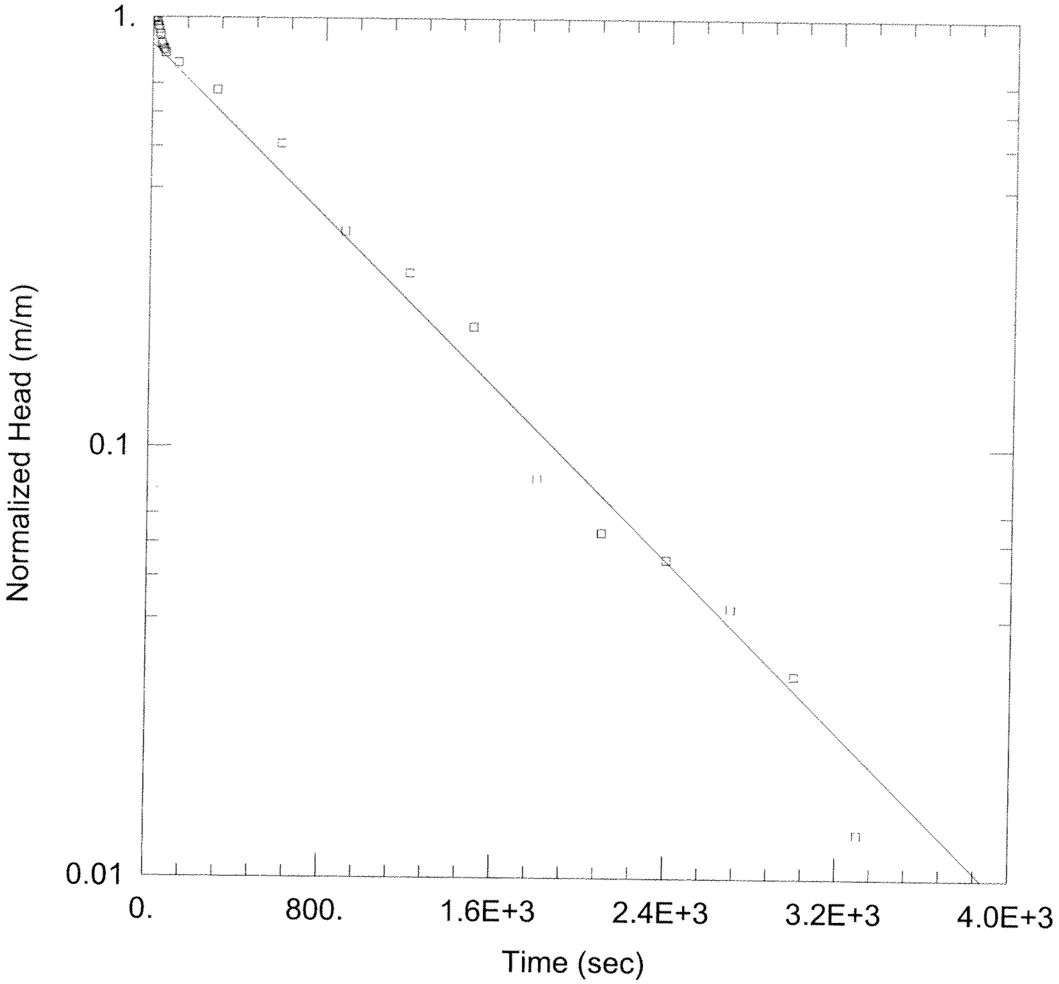
Q $= (Q_{with\ (F)})(A_{exc})(d_{ppt})$
$= (15{,}600\ L/day)(23.7\ m^2)(0.02\ m)$
$= 16{,}046\ L/day$

Based on these calculations, the anticipated groundwater infiltration into the excavation for the WATERMAIN were calculated and presented in Table 2 of the main report.

3.0 ASSUMPTIONS

In the calculation of the dewatering activities, we have made the following assumptions:

- the excavation for the reservoir is to be 15 m x 15 m, and its effect is similar to a circular excavation
- the source is circular
- conditions are homogeneous
- the excavation has achieved steady state flow
- the slug test data is representative of the actual site conditions
- precipitation events are no greater than 0.02 m/day



BH1-RISING HEAD

Data Set: U:\klindsay\T020587\BH1-RH.aqt
Date: 02/09/09 Time: 15:05:58

PROJECT INFORMATION

Company: Conestoga Rovers & Associates
Client: Carp Airport
Project: T020587
Location: Carp, Ontario
Test Well: BH1
Test Date: 12/28/08

AQUIFER DATA

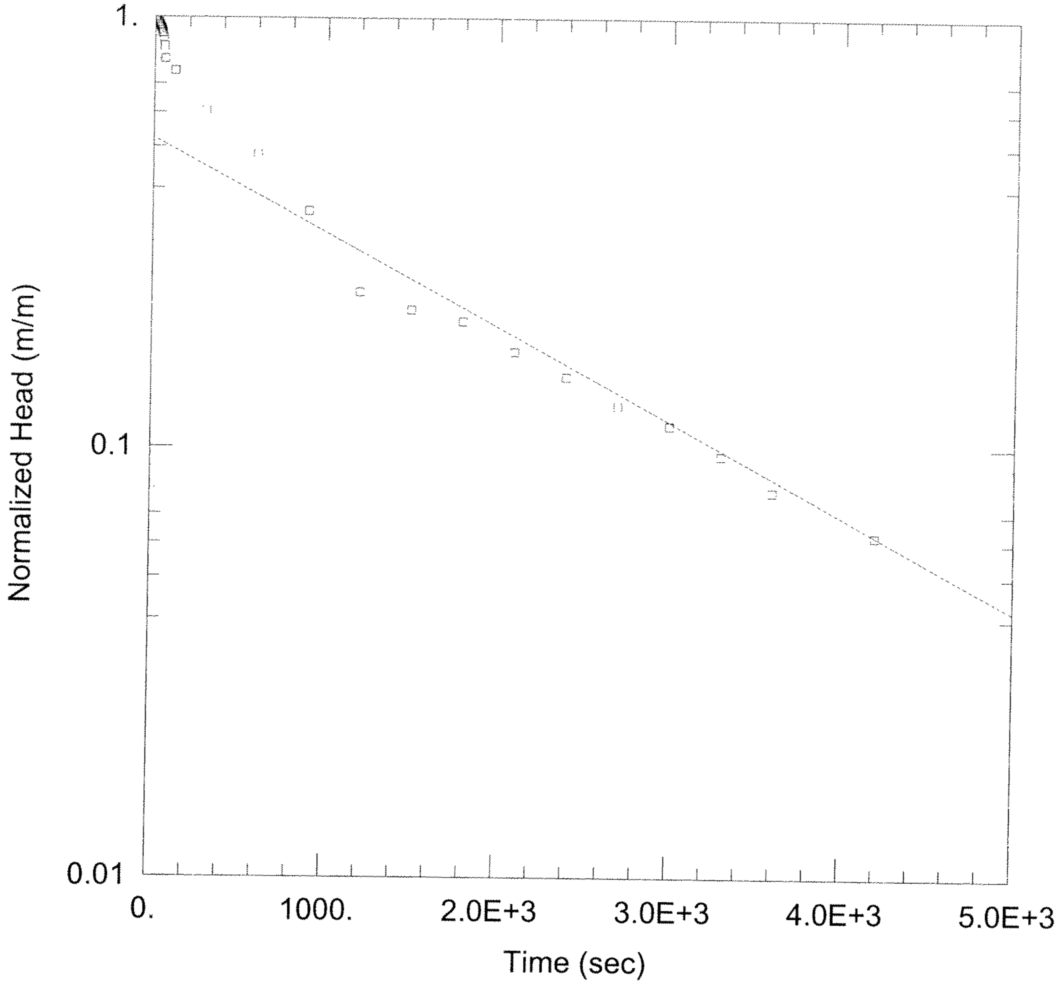
Saturated Thickness: 5.285 m
Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (BH1-RH)

Initial Displacement: 0.236 m
Total Well Penetration Depth: 5.285 m
Casing Radius: 0.025 m
Static Water Column Height: 5.285 m
Screen Length: 3.97 m
Well Radius: 0.075 m
Gravel Pack Porosity: 0.

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice
K = 2.922E-5 cm/sec
y0 = 0.2049 m



BH1-FALLING HEAD

Data Set: U:\klindsay\T020587\BH1-FH.aqt
Date: 02/09/09 Time: 15:03:02

PROJECT INFORMATION

Company: Conestoga Rovers & Associates
Client: Carp Airport
Project: T020587
Location: Carp, Ontario
Test Well: BH1
Test Date: 12/28/08

AQUIFER DATA

Saturated Thickness: 5.285 m
Anisotropy Ratio (Kz/Kr): 1.

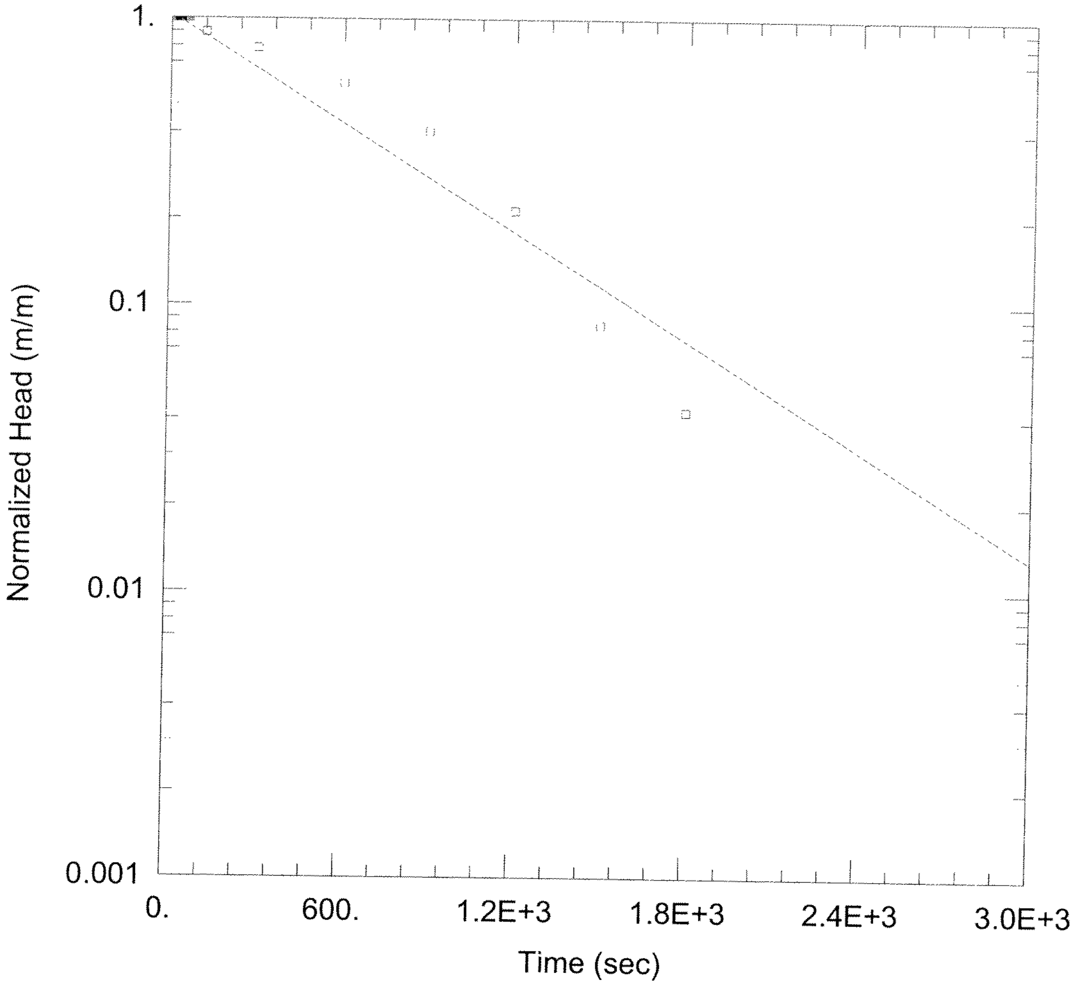
WELL DATA (BH1-FH)

Initial Displacement: 0.24 m
Total Well Penetration Depth: 5.315 m
Casing Radius: 0.025 m
Static Water Column Height: 4.385 m
Screen Length: 4. m
Well Radius: 0.025 m
Gravel Pack Porosity: 0.

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 1.627E-5 cm/sec
y0 = 0.1257 m



BH2 - RISING HEAD

Data Set: U:\klindsay\T020587\BH2-RH.aqt
Date: 02/09/09 Time: 16:54:37

PROJECT INFORMATION

Company: Conestoga Rovers & Associates
Client: Carp Airport
Project: T020587
Location: Carp, Ontario
Test Well: BH2
Test Date: 12/28/08

AQUIFER DATA

Saturated Thickness: 1.105 m
Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (BH2)

Initial Displacement: 0.047 m
Total Well Penetration Depth: 1.53 m
Casing Radius: 0.025 m
Static Water Column Height: 1.105 m
Screen Length: 1.53 m
Well Radius: 0.075 m
Gravel Pack Porosity: 0.

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice
K = 6.667E-5 cm/sec
y0 = 0.04848 m

APPENDIX B

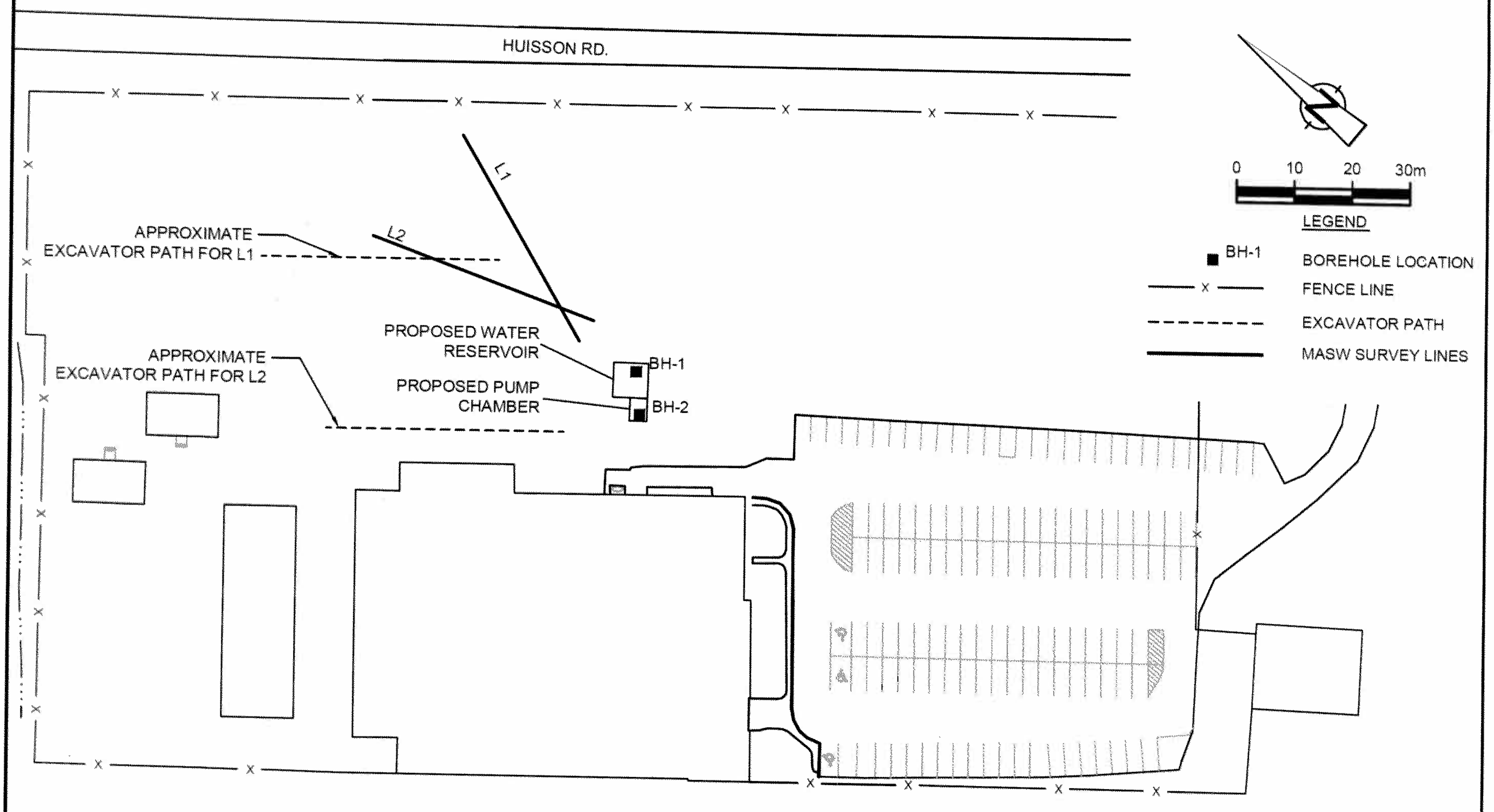
SHEAR WAVE VELOCITY DATA (MASW TEST RESULTS)

T020587-A1 : APPENDIX B

AVERAGE SHEAR WAVE VELOCITY WITHIN THE UPPER 30 m (FROM 1.0 m BGS TO 30 m bgs) OF SOIL/ROCK PROFILE
RCMP-CARP AIR HANGER DEVELOPMENT
OTTAWA, ONTARIO

CARP Line 1				**Station 1011**		**Station 1012**		**Station 1013**		**Station 1014**		**Station 1015**	
Layer No.	Depth (m)		Thickness	V_s	d_i/V_{si}	V_s	d_i/V_{si}	V_s	d_i/V_{si}	V_s	d_i/V_{si}	V_s	d_i/V_{si}
	From	To	m	m/s		m/s		m/s		m/s		m/s	
1	1	2	1	84.3	0.0119	126.7	0.0079	126.7	0.0079	125.9	0.0079	129.8	0.0077
2	2	3.5	1.5	146.9	0.0102	130.9	0.0115	127.4	0.0118	128.7	0.0117	123.7	0.0121
3	3.5	5.3	1.8	206.7	0.0087	176.8	0.0102	174.1	0.0103	169.2	0.0106	182.8	0.0098
4	5.3	7.6	2.3	169.6	0.0136	203.1	0.0113	171.6	0.0134	175.8	0.0131	194.1	0.0118
5	7.6	10.4	2.8	165.6	0.0169	198.1	0.0141	144.9	0.0193	165.4	0.0169	135.9	0.0206
6	10.4	13.9	3.5	149.0	0.0235	128.1	0.0273	165.3	0.0212	153.5	0.0228	141.2	0.0248
7	13.9	18.3	4.4	193.4	0.0227	150.1	0.0293	209.1	0.0210	159.8	0.0275	265.6	0.0166
8	18.3	23.8	5.5	216.8	0.0254	264.0	0.0208	229.1	0.0240	202.8	0.0271	339.7	0.0162
9	23.8	29.8	6	178.7	0.0336	323.4	0.0186	239.4	0.0251	229.9	0.0261	285.4	0.0210
	Total		28.8		0.1664		0.1510		0.1540		0.1638		0.1407
Average Shear Wave Velocity at Station					173.0		190.7		187.0		175.8		204.7
Average Shear Wave Velocity Along the Line					186.3	m/s							

CARP Line 2				**Station 1011**		**Station 1012**		**Station 1013**		**Station 1014**		**Station 1015**	
Layer No.	Depth (m)		Thickness	V_s	d_i/V_{si}	V_s	d_i/V_{si}	V_s	d_i/V_{si}	V_s	d_i/V_{si}	V_s	d_i/V_{si}
	From	To	m	m/s		m/s		m/s		m/s		m/s	
1	1	2	1	166.037	0.0060	155.445	0.0064	141.426	0.0071	139.12	0.0072	144.534	0.0069
2	2	3.5	1.5	169.273	0.0089	161.996	0.0093	137.726	0.0109	137.125	0.0109	133.762	0.0112
3	3.5	5.3	1.8	169.174	0.0106	162.801	0.0111	142.364	0.0126	158	0.0114	145.478	0.0124
4	5.3	7.6	2.3	130.732	0.0176	145.682	0.0158	155.1	0.0148	153.496	0.0150	151.391	0.0152
5	7.6	10.4	2.8	118.93	0.0235	116.599	0.0240	163.546	0.0171	140.455	0.0199	137.026	0.0204
6	10.4	13.9	3.5	203.144	0.0172	165.196	0.0212	157.848	0.0222	156.087	0.0224	193.363	0.0181
7	13.9	18.3	4.4	268.362	0.0164	242.326	0.0182	152.722	0.0288	201.306	0.0219	244.353	0.0180
8	18.3	23.8	5.5	268.702	0.0205	273.536	0.0201	197.099	0.0279	248.404	0.0221	230.942	0.0238
9	23.8	29.8	6	239.732	0.0250	267.122	0.0225	255.195	0.0235	284.132	0.0211	250.734	0.0239
	Total		28.8		0.1458		0.1485		0.1650		0.1520		0.1500
Average Shear Wave Velocity at Station					197.6		194.0		174.6		189.5		192.0
Average Shear Wave Velocity Along the Line					189.5	m/s							



MASW LINES

GEOTECHNICAL INVESTIGATION - CARP AIR HANGER DEVELOPMENT

CARP, ONTARIO

T020587-A1- APPENDIX B

FONDEX

T020587-A1(001)GN-OT004 FEB 11/2009

FIGURE 2: MASW TEST RESULTS
STRATIGRAPHIC SECTION (LINE 1)
RCMP-CARP AIR HANGER DEVELOPMENT
OTTAWA, ONTARIO
PROJECT NO.: T020587-A1

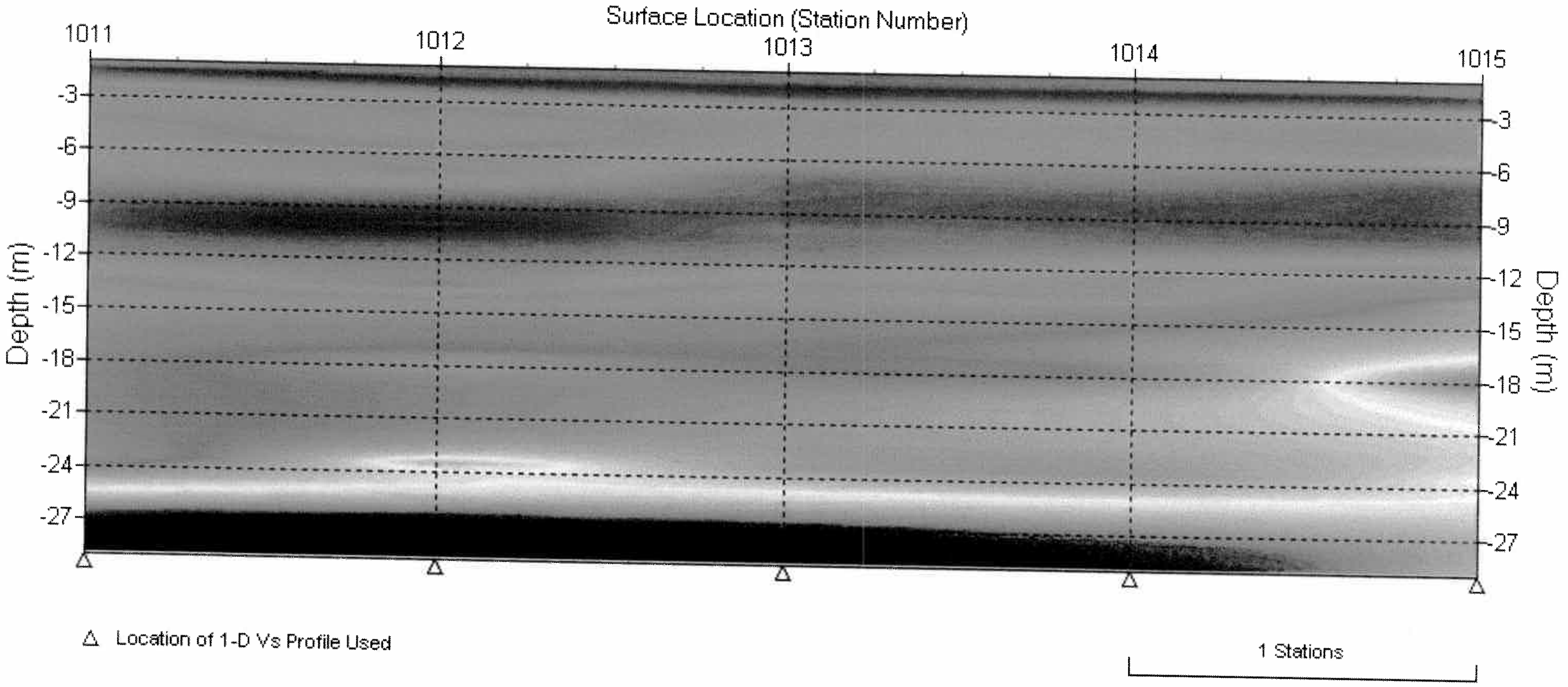
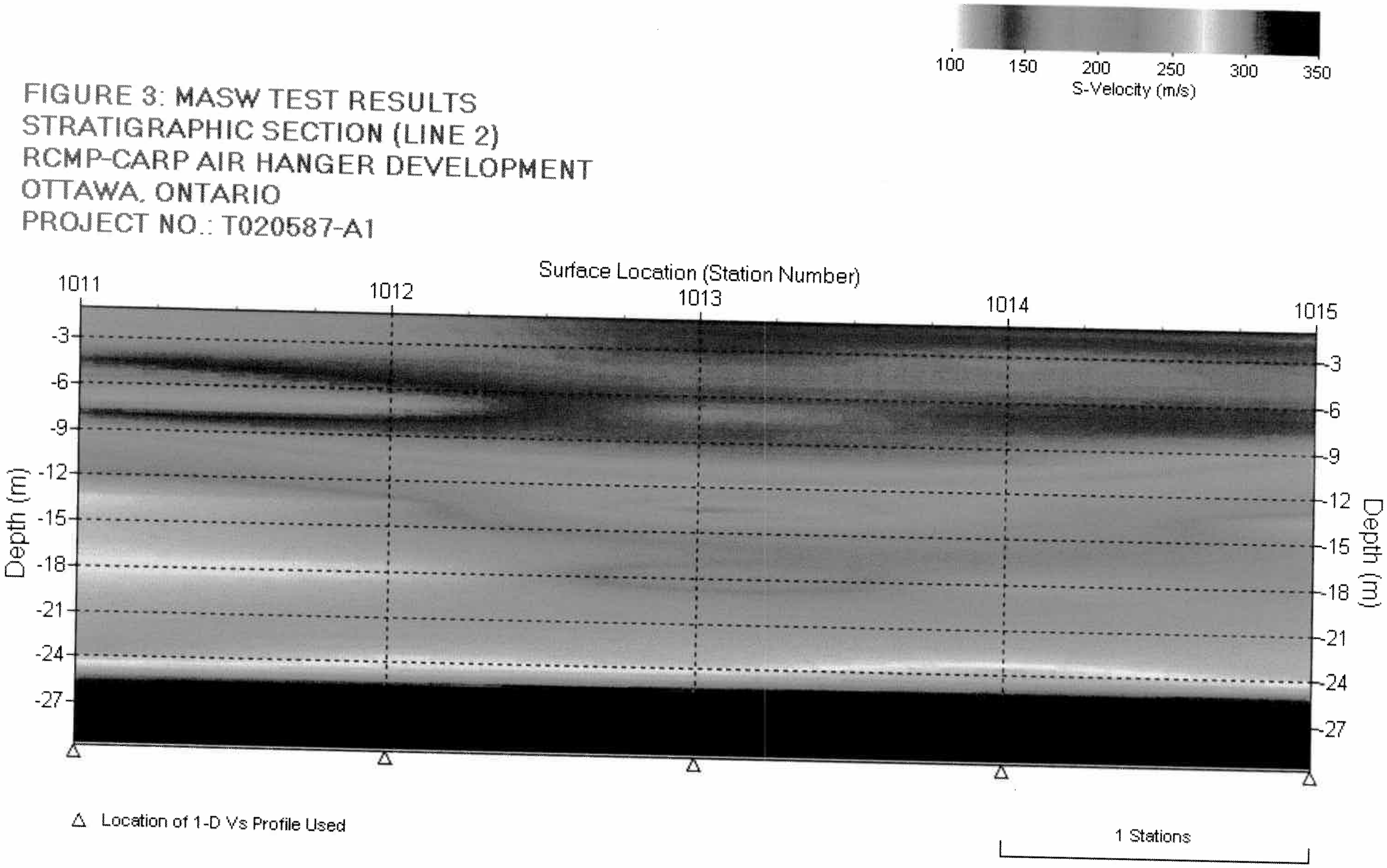


FIGURE 3: **MASW TEST RESULTS**
STRATIGRAPHIC SECTION **(LINE 2)**
RCMP-CARP AIR HANGER DEVELOPMENT
OTTAWA, ONTARIO
PROJECT NO.: T020587-A1



APPENDIX C

NOTES ON BOREHOLE AND TEST PIT LOGS

INSPEC•SOL

NOTES ON BOREHOLE AND TEST PIT REPORTS

SOIL DESCRIPTION:

Each subsoil stratum is described using the following terminology. The relative density of granular soils is determined by the standard penetration index ("N" value), while the consistency of clayey soils is measured by the value of the undrained shear strength (Cu).

CLASSIFICATION (UNIFIED SYSTEM)

Clay	< 0,002mm		
Silt	0,002 to 0,075mm		
Sand	0,075 to 4,75mm	fine	0,075 to 0,425mm
		medium	0,425mm to 2,0mm
		coarse	2,0 to 4,75mm
Gravel	4,75 to 75mm	fine	4,75mm to 19mm
		coarse	19 to 75mm
Cobbles	75 to 300mm		
Boulders	> 300mm		

TERMINOLOGY

"traces"	1 - 10%
"some"	10 - 20%
adjective (silty, sandy)	20 - 35%
"and"	35 - 50%

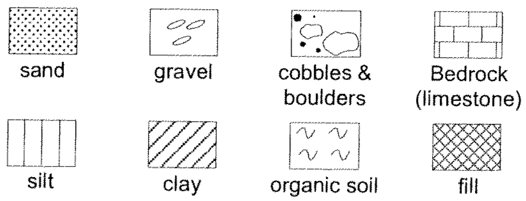
RELATIVE DENSITY OF GRANULAR SOILS	STANDARD PENETRATION INDEX "N" VALUE (BLOWS/ft - 300mm)
Very loose	0 - 4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very dense	> 50

CONSISTANCY OF COHESIVE SOILS	UNDRAINED SHEAR STRENGTH (Cu)	
	(P.S.F.)	(kPa)
Very soft	< 250	< 12
Soft	250 - 500	12 - 25
Medium	500 - 1000	25 - 50
Stiff	1000 - 2000	50 - 100
Very stiff	2000 - 4000	100 - 200
Hard	> 4000	> 200

ROCK QUALITY DESIGNATION

"RQD" (%) VALUE	QUALIFICATIVE
< 25	very poor
25 - 50	poor
50 - 75	fair
75 - 90	good
> 90	excellent

STRATIGRAPHIC LEGEND



SAMPLES:

TYPE AND NUMBER

The type of sample recovered is shown on the log by the abbreviation listed hereafter. The numbering of samples is sequential for each type of sample.

SS: Split spoon
SSE, GSE, AGE: Environnemental sampling
ST: Shelby tube
PS: Piston sample (Osterberg)
AG: Auger
RC: Rock core
GS: Grab sample

RECOVERY

The recovery, shown as a percentage, is the ratio of length of the sample obtained to the distance the sampler was driven/pushed into the soil.

RQD

The "Rock Quality Designation" or "RQD" value, expressed as a percentage, is the ratio of the total length of all core fragments of 4 inches (10cm) or more to the total length of the run.

IN-SITU TESTS:

N: Standard penetration index
R: Refusal to penetration
N_C: Dynamic cone penetration index
Cu: Undrained shear strength
Pr: Pressuremeter
k: Permeability
ABS: Absorption (Packer test)

LABORATORY TESTS:

I_p: Plasticity index
W_l: Liquid limit
W_p: Plastic limit
H: Hydrometer analysis
GSA: Grain size analysis
A: Atterberg limits
w: Water content
g: Unit weight
C: Consolidation
CS: Swedish fall cone
CHEM: Chemical analysis
O.V.: Organic vapor