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Supplemental Geotechnical Investigation

Proposed Commercial Development
Campeau Drive Extension
Ottawa, Ontario

Prepared For

Taggart Realty Management

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Report: PG0909-2R

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APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Taggart Realty Management (Taggart) to conduct a supplemental geotechnical investigation for the proposed Kanata Commons commercial development to be located along Campeau Drive extension at Carp River, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objective of the investigation was to:

- ❑ determine the subsoil and groundwater conditions at this site by means of boreholes.
- ❑ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 PROPOSED DEVELOPMENT

It is understood that the proposed commercial development includes a restaurant and a bank within the north portion of the development. The south portion of the site includes two (2) retail buildings with associated parking areas and access lanes. It is further understood that the proposed finished floor elevation(s) will range between 95.4 and 96.4 m and that proposed commercial buildings will be of slab-on-grade construction.

The subject property is located at Campeau Drive extension at Carp River, and is bordered to the north by Campeau Drive extension, to the east by an undeveloped commercial property to the south by a future transit way and to the west by Carp River. The subject site is undeveloped, relatively flat and consists of former farmlands.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

The field program for the investigation was carried out on November 8 and 9, 2006. At that time, three (3) boreholes (BH 1, BH 2 and BH 3) were advanced to depths varying between 11.9 and 14.8 m. Additionally, one (1) borehole, designated BH 1-14, was advanced on January 15, 2014 to a depth of 9.8 m. The locations of the boreholes are shown on Drawing PG0909-4 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson geotechnical division under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations and sampling and testing the overburden. The depth of the bedrock surface was inferred at borehole locations by auger refusal and dynamic cone penetration testing (DCPT).

Sampling and In Situ Testing

Soil samples were recovered from the auger flights, using a 50 mm diameter split-spoon sampler or 73 mm diameter thin walled Shelby tubes in conjunction with a piston sampler at BH 1 and 3. The auger and split-spoon samples were classified on site and placed in sealed plastic bags. The Shelby tubes were sealed at both ends. All samples were transported to our laboratory. The depths at which the auger, split-spoon and Shelby tube samples were recovered from the boreholes are depicted as AU, SS and TW, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples at BHs 1 and 3. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing, using a field vane apparatus, was carried out in cohesive soils.

The thickness of the over burden was evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at BH 1 and BH 2. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment. Because of the low resistance exerted by the silty clay, the cone was pushed using the hydraulic head of the drill rig until resistance to penetration was encountered. The drop hammer was then used to further advance the cone.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Flexible standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were selected by Paterson in order to provide general coverage of the site. The locations of the boreholes and ground surface elevations at the borehole locations were determined in the field by Annis, O'Sullivan, Vollebekk Limited (AOV). It is understood that the elevations are referenced to a geodetic datum.

The locations of the boreholes and the ground surface elevation at each borehole location are presented on Drawing PG0909-4 - Test Hole Location Plan included in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

Three (3) Shelby tube samples were submitted for unidimensional consolidation testing and Atterberg Limits.

The results of the consolidation testing and Atterberg Limits are presented on the Consolidation Test and Atterberg Limits Results sheets presented in Appendix 1 and are further discussed in Sections 4 and 5.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the soil. The results are provided in Appendix 1 and are discussed further in Subsection 6.7.

4.0 OBSERVATIONS

4.1 Surface Conditions

The subject property consists mostly of former farmlands and was recently cleared as preparatory work for the proposed development. The ground surface is relatively flat and slopes toward the Carp River.

Currently, two areas of the subject site are undergoing a preload program. The piles have been designated Pile A, which is located within the north portion of the site and was approximately 3 m high at the time of placement and Pile B, which is located within the south portion of the site and varies in height between 2.1 to 2.5 m high. Pile A encompasses the proposed buildings within the north portion of the site and immediate area surrounding the buildings. Pile B encompasses one of the commercial buildings (The Brick).

4.2 Subsurface Profile

Generally, the subsoil conditions encountered at the borehole locations at the time of the investigation consisted of topsoil/reworked soil, silty sand to clayey silt overlying a deep sensitive silty clay deposit. Practical refusal to DCPT was encountered at 14.8 and 11.9 m at BH 1 and BH 2, respectively. Practical refusal to augering was encountered at a 12.6 m depth at BH 3.

Based on our most recent site investigation work, the majority of the subject site has been stripped of topsoil. At BH 1-14, the preload material consisted of brown silty sand to a height of approximately 2.5 m above the original ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each borehole location.

Silty Clay

Grey silty clay was encountered below the silty sand at BH 1 and BH 3. Seams of silty sand were observed within the lower portion of the silty clay layer at BH 1. Traces of sand and gravel were observed within the silty clay layer at BH 3. The silt content increased with depth at BH 3. Based on practical refusal to augering and DCPTs, the silty clay is inferred to extend to depths of 10.7 m at BH 2 and 14.8 m at BH 1. The natural water content of grey silty clay materials ranged from 20 to 76 percent. In situ shear vane field testing carried out within the silty clay yielded undrained shear strength values ranging from approximately 25 to 46 kPa. These values are indicative of a firm consistency.

Three (3) silty clay samples collected at this site were subjected to unidimensional consolidation testing. The results are presented in Appendix 1, and summarized in Table 3 in Subsection 5.3. The results indicate that the silty clay is overconsolidated with overconsolidation ratios varying between 1.4 and 2.

The results of Atterberg Limits tests conducted on samples of silty clay obtained from BH 1 and BH 3 are presented in Table 1 and on the Atterberg Limits' Results sheet in Appendix 1. The tested silty clay samples classify as an inorganic clay of Low Plasticity (CL) in accordance with the Unified Soil Classification System.

Table 1 - Atterberg Limits' Results

Sample	Depth, m	LL,%	PL, %	PI, %	w, %	Symbol
BH 1 TW 6	8.82	33	21	12	58	CL
BH 3 TW 5	3.49	40	21	18	65	CL
BH 3 TW 7	6.52	35	22	13	64	CL

LL: Liquid Limit PL: Plastic Limit PI: Plasticity Index w: water content
CL: Clay of Low Plasticity

Geological Mapping

Based on available geological mapping, the bedrock in this area consists of sand stone of Nepean formation at east side and interbedded limestone and shale of the Verulam formation at west side in the subject area. Also, it is expected to be encountered at depths ranging from 5 to 15 m.

Hydraulic Conductivity

Hydraulic conductivity values were conservatively estimated by Paterson based upon typical values for both a loose silty sand/sandy silt and a stiff silty clay. These values were determined to range from 1×10^{-2} m/s to 4×10^{-4} m/s within the silty sand/sandy silt deposit and from 1×10^{-5} m/s to 1×10^{-9} m/s within the silty clay layer. The range in conductivities within the silty clay layer is necessary in order to account for sand seams and varying silt content across the site.

Consolidation Testing

A total of three (3) consolidation tests were carried out for this project. The results of the consolidation tests are summarized in Table 2, and presented graphically on the Consolidation Test sheets in Appendix 1.

Table 2 - Summary of Consolidation Test Results

Borehole No.	Sample No.	Depth (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c	Q
BH 1	TW 6	8.82	147	75	0.01	1.89	A
BH 3	TW 5	3.49	75	41	0.02	1.74	A
BH 3	TW 7	6.52	86	60	0.02	2.62	A

Note: Q = Quality assessment of sample: G = Good A = Acceptable
P = Poor, likely disturbed

The tabulated consolidation test information includes the results of an assessment of the sample quality, or relative sample disturbance, based on the characteristics of the test results (after Lacasse et. al., 1985). The quality of each sample has been graded from good, through acceptable, to poor (sample likely to be disturbed). The assessment shows that all three samples are acceptable quality.

Value p'_c is the preconsolidation pressure of the sample and p'_o is the effective overburden pressure. The difference between these values is the available preconsolidation. The increase in stress on the soil due to the cumulative effects of the fill surcharge, the footing pressures, the slab loading and the lowering of the groundwater should not exceed the available preconsolidation if the potential total and differential settlements are to be maintained within tolerable limits for the proposed development.

The values C_{cr} and C_c are the recompression and compression indices, respectively, and are measures of the compressibility of the soil due to stress increases below and above the preconsolidation pressures. The higher values for the C_c, as compared to the C_{cr}, illustrate the increased settlement potential above, as compared to below, the preconsolidation pressure.

It should be noted that the values of p'_c, p'_o, C_{cr} and C_c are determined using standard engineering practices and are estimates only.

4.3 Groundwater

On November 14, 2006, the groundwater levels were measured in the standpipes installed at the time of the fieldwork. The measured groundwater levels are presented in Table 3. The measured groundwater levels range from 0.4 to 1.2 m below the ground surface. It should be noted that water can become trapped within a backfilled borehole, which can lead to higher than normal groundwater levels. Based on soil colouring, moisture levels and consistency of the recovered soil samples, the long-term groundwater level is anticipated to be between 1.5 to 2 m below existing ground surface. It should be noted that groundwater levels may vary with precipitation events and seasonally. Therefore, groundwater levels may be vary at the time of construction.

Table 3
Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Recording Date
		Depth (m)	Elevation (m)	
BH 1	94.06	0.45	93.61	November 14, 2006
BH 2	93.96	0.37	93.59	November 14, 2006
BH 3	95.15	1.16	93.99	November 14, 2006

Notes: The groundwater elevations are referenced to the elevations of the ground surface at the locations of the boreholes as provided by Annis, O'Sullivan, Vollebekk Limited (AOV).

Groundwater Flow Direction

The groundwater flow direction is expected to trend in a westerly direction towards the Carp River (located approximately 50 m from the subject site), eventually draining into the Ottawa River.

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the compressibility of the sensitive silty clay deposit imposes grade raise restrictions for the proposed development. A permissible grade raise restriction of 1 m was previously recommended for grading within 6 m of the proposed buildings based on our in situ shear vane results and consolidation testing results. However, a soil preload program is currently underway at the proposed buildings, which have finished grades that exceed our permissible grade raise recommendations. The preload pile areas are outlined in Drawing PG0909-4 - Test Hole Location Plan in Appendix 2. The preload pile was placed in 2010, however, settlement monitoring plates were not installed until February 2014. A summary of our settlement monitoring data recorded over the past several months is presented in Figure 2 - Preload Monitoring Program in Appendix 2. It should be noted that the preload program did not include the future la-z-boy building, since the original grade at the future la-z-boy building location has been recently cut to the current grades and original ground surface elevations within the building footprint vary between 95.2 and 95.5 m. Therefore, the proposed grade raise is within our permissible grade raise restriction and preloading is not required for the la-z-boy building.

Based on the current grading for the site, the existing ground surface at Pile A and Pile B is approximately 0.7 m and 1.1 m, respectively, above finished grade of the proposed buildings in the immediate area. Based on survey information recorded at the settlement plate locations, settlement of less than 35 mm has occurred at the settlement plate locations within Pile A since February 2014. Negligible settlement has occurred at the settlement plate locations within Pile B since February 2014.

Based on our settlement monitoring data for the preload program and available soils information, it is recommended that the preload program continue for Pile A to ensure that sufficient settlement has occurred before building construction begins. However, greater than 90% of primary consolidation for the underlying silty clay deposit has been achieved within the Pile B area. For design purposes, it is expected that total and differential settlements associated with the combination of grade raises and footing loading conditions will be limited to 25 and 20 mm, respectively. It should be further noted that conventional construction methods are acceptable and no LWF is required from a geotechnical perspective for the proposed building within the Pile B area.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, reworked soil and any deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the buildings should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. These materials should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soils can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to reduce voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage system connected to a perimeter drainage system, is provided.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, founded over an undisturbed, compact silty sand or stiff, silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **70 kPa** and a factored bearing resistance value at ultimate limit states of **150 kPa**, which incorporates a geotechnical resistance factor of 0.5. The structures designed using the abovenoted bearing resistance value at SLS will be subjected to a potential post-construction total and differential settlements of up to 25 mm and 20 mm, respectively.

Where footings are to be placed within the existing preload pile fill layer, it is recommended to extend footings to an undisturbed, compact silty sand bearing surface or place engineered fill from the underside of footing level to the undisturbed, compact silty sand bearing surface. The engineered fill should consist of a Granular A or Granular B Type II placed in maximum 300 mm loose lifts and compacted to at least 98% of its SPMDD. The engineered fill should extend at a minimum 1.5H:1V slope down and out from the footing face. Footings placed over a minimum 500 mm thick engineered fill pad approved by the geotechnical consultant can be designed using a bearing resistance value at SLS of **85 kPa** and a factored bearing resistance value at ULS of **170 kPa**.

Lateral Support

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 compact silty sand or stiff silty clay bearing medium above the groundwater table when a plane extending down and out from the bottom edges of the footing, profiled at 1.5H:1V or shallower, passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Groundwater Lowering Considerations

It is recommended that a minimum long-term post-development groundwater lowering of 0.5 m be considered at this site. Means to reduce long-term groundwater lowering (e.g. clay dykes in the service trenches, leaving green spaces to allow for groundwater recharge and limiting planting of trees to areas away from the buildings, etc) should be implemented for the proposed development. The use of properly installed reinforcement in foundations will tend to reduce foundation cracking as compared to unreinforced foundations. It should be noted that building on thick silty clay deposit increases the likelihood of building movements and therefore of cracking.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed buildings from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. Two (2) seismic shear wave velocity profiles from the testing are presented in Appendix 2.

Field Program

The shear wave testing location is presented in Drawing PG0909-4 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly an east-west orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 3 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between five (5) to ten (10) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the centre of the geophone array, 3, 4.5 and 30 m away from the first and last geophone.

The methods of testing completed by Paterson are guided by the standard testing procedures used by the expert seismologists at Carleton University and Geological Survey of Canada (GSC).

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our analysis, the average overburden and bedrock seismic shear wave velocities were calculated to be 120 and 2,350 m/s, respectively.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012, as presented below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{13.5m}{120m/s} + \frac{16.5m}{2.350m/s}\right)}$$

$$V_{s30} = 251m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for the subject site is 251 m/s. Therefore, a **Site Class D** is applicable for the proposed buildings, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed buildings, the undisturbed native soil surface or existing silty sand fill, free of deleterious materials, proof-rolled by suitable compaction equipment making several passes under dry conditions, in above freezing temperatures and approved by the geotechnical consultant at the time of construction will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. It is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone.

5.6 Pavement Structure

Car only parking areas and heavy traffic access areas are expected at this site. The subgrade material will consist of silty sand and silty clay. The proposed pavement structures are presented in Tables 4 and 5.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material.

Table 4 - Recommended Pavement Structure, Car Only Parking Areas

Thickness (mm)	Material Description
50	**WEAR COURSE -** HL 3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.

Table 5 - Recommended Pavement Structure, Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**WEAR COURSE -** HL 3 or Superpave 12.5 Asphaltic Concrete
50	**BINDER COURSE -** HL 8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.

Performance Graded (PG) 58-34 asphalt cement should be used for this project.

The pavement granulars (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the pavement granulars in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

The subgrade is expected to consist predominantly of silty clay. Subdrains should be installed at all the catch basins. These drains should be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage outlets.

Onerous conditions could be encountered at the time of construction, especially above recently placed trench backfill. Consideration should be given to installing a woven geotextile liner, such as Terratrack 200 or equivalent, at the interface between the pavement granulars and the subgrade to act as a separation/reinforcement layer.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 100 to 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill placed against the foundation walls.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard. A minimum 2.1 m thick soil cover (or equivalent insulation) should be provided in unheated areas.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated to acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be constructed 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 excavated at 1H:1V or flatter. The flatter slope is recommended for excavation below groundwater level. The subsurface soils are considered to be a Type 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 maintain a safe distance 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 protect personnel working in trenches 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.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

At least 150 mm of OPSS Granular A should be used for bedding for water pipes. The bedding material, which should extend to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular M. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

It should generally be possible to re-use the upper portion of the excavated soil if the excavation and filling operations are carried out in dry weather conditions. Due to its high natural water content, the grey silty clay will be difficult, if not impractical, to compact without an extensive drying period.

Where hard surface areas are considered above the trench backfill, 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.

To reduce long term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long (in the trench direction), and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries, and at strategic locations at no more than 50 m intervals in the service trenches.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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.

A temporary MOE permit to take water (PTTW) may be required for this project if more than 50,000 L/day is to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MOE.

6.6 Winter Construction

If winter construction is considered for the project specific precautions should be implemented. The subsurface soil conditions consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be completed during freezing conditions.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content in the tested soil sample is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the chloride and pH testing indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site while the resistivity test yielded a result indicative of a moderate to Slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Restrictions

The proposed commercial buildings are located in a moderate to high sensitivity area with respect to tree plantings over a silty clay deposit. It is recommended that trees placed within 6 m of the foundation wall should consist of low water demand trees with shallow roots systems that extend less than 1.5 m below ground surface. Trees placed greater than 6 m from the foundation wall may consist of typical street trees, which are typically moderate water demand species with roots extending to a maximum depth of 2 m below ground surface.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

6.9 Design Information for Infiltration Galleries

It is understood that stormwater infiltration galleries have been designed for the proposed development. The stormwater infiltration galleries are located within the proposed parking spaces of The Brick store. Using the lower limit hydraulic conductivity of the silty sand/sandy silt deposit, the water holding capacity of the upper 2 to 3 m of soil was determined to be 80 mm/m of soil. Additionally, the annual precipitation at the site was found to be 893 mm/year. With an anticipated total infiltration factor of 0.7 and a factored water surplus of 260 mm/year, the annual recharge at the subject site is expected to be approximately 155 mm/year. Therefore, as this value is significantly above 70 mm/year (infiltration target), the infiltration target for the site has been achieved.

7.0 RECOMMENDATIONS

It is a requirement for the foundation design data provided herein to be applicable that material testing and observation services program, including the following aspects 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.

❏ Sampling and testing of the bituminous concrete including mix design reviews.

Upon completion, 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.

8.0 STATEMENT OF LIMITATIONS

The recommendations made in this report are in accordance with our present understanding of the project.

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 immediate notification in order to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 Taggart Realty or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

David J. Gilbert, P.Eng.

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

Carlos P. Da Silva, P.Eng.

Michael Laflamme, GIT

Report Distribution:

❑ Taggart Realty (3 copies)
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

CONSOLIDATION TEST RESULTS

ATTERBERG LIMITS TEST RESULTS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Campeau Drive Extention at Carp River
Ottawa, Ontario

DATUM Ground surface elevations at borehole locations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE January 15, 2014

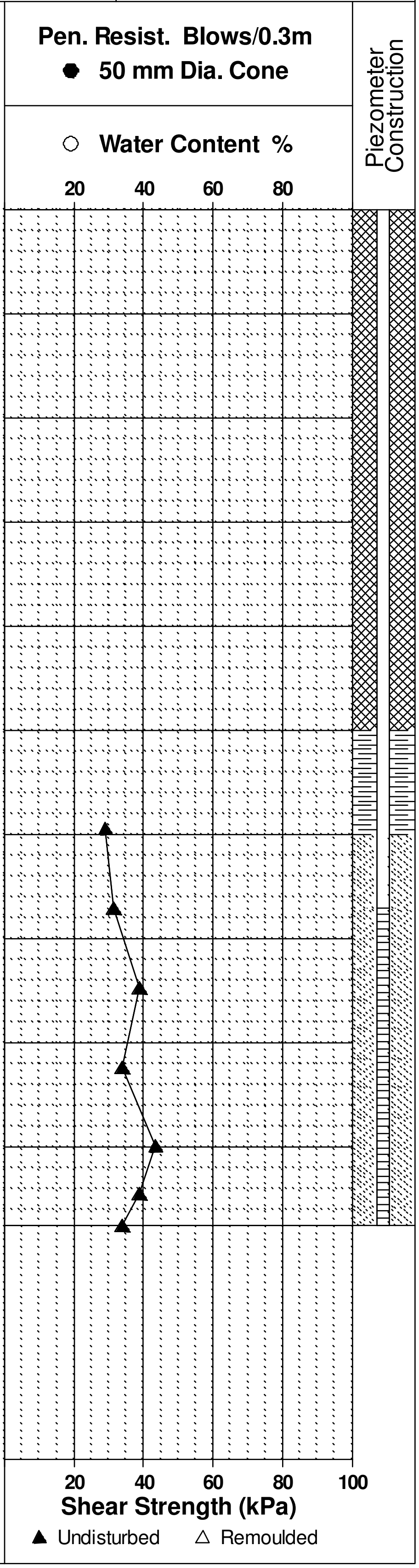
FILE NO. PG0909

HOLE NO. BH 1-14

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand	2.21
Compact, brown **SILTY SAND** - some organics noted at 3.3m depth	3.35
Brown **CLAYEY SILT**	4.67
Firm, grey **SILTY CLAY**	9.75
End of Borehole Based on field observations, GWL at 4.5m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	100	9
SS	2	100	5
SS	3	75	12
SS	4	75	24
SS	5	75	3
SS	6	100	1
SS	7	100	P
G	8		
SS	9	100	P

DEPTH (m)	ELEV. (m)
0	96.54
1	95.54
2	94.54
3	93.54
4	92.54
5	91.54
6	90.54
7	89.54
8	88.54
9	87.54



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Campeau Drive Extension at the Carp River
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 9 NOV 06

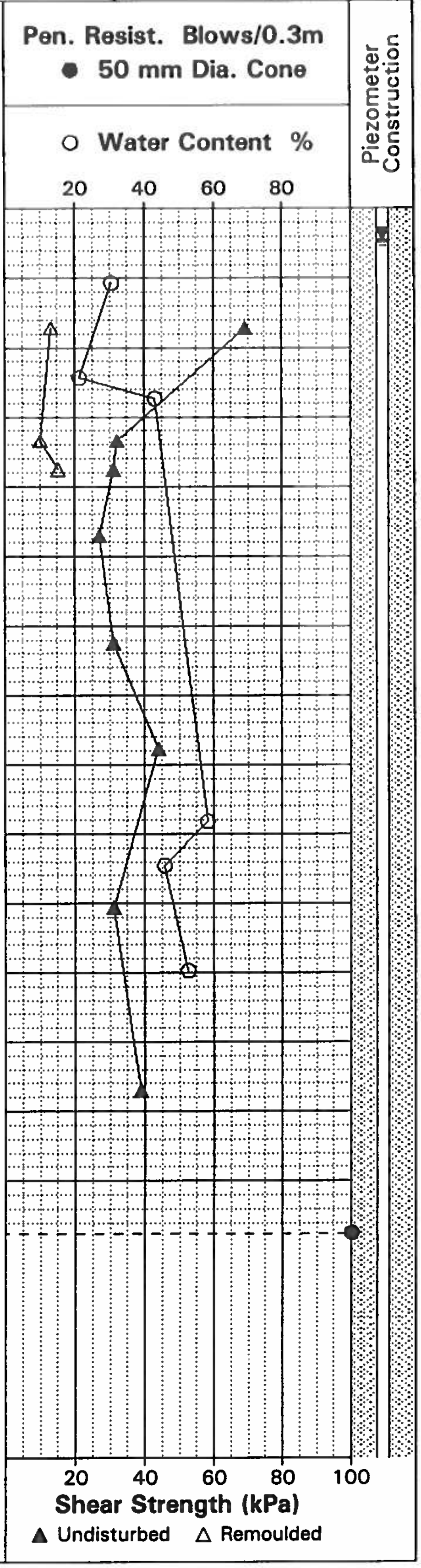
FILE NO. PG0909

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.25
Loose, brown SILTY SAND	0.70
Stiff, brown CLAYEY SILT with sand and sand seams	2.13
Loose, grey SILTY fine to medium SAND	2.59
Firm, grey SILTY CLAY - layer of shells at 4.3m depth - soft by 4.7m depth - firm by 7.8m depth - 150mm thick silty sand layer @ 9.0m depth - thin silty sand layer @ 10.7m depth	14.75
End of Borehole Practical DCPT refusal @ 14.75m depth (GWL @ 0.45m-Nov. 14/06)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	83	5
SS	2	50	2
TW	3	62	
TW	4	100	
TW	5	100	
TW	6	100	
SS	7	2	1
SS	8	75	1
TW	9	100	
TW	10	71	

DEPTH (m)	ELEV. (m)
0	94.06
1	93.06
2	92.06
3	91.06
4	90.06
5	89.06
6	88.06
7	87.06
8	86.06
9	85.06
10	84.06
11	83.06
12	82.06
13	81.06
14	80.06



patersongroup Consulting Engineers
28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Campeau Drive Extension at the Carp River
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS

BORINGS BY CME 55 Power Auger

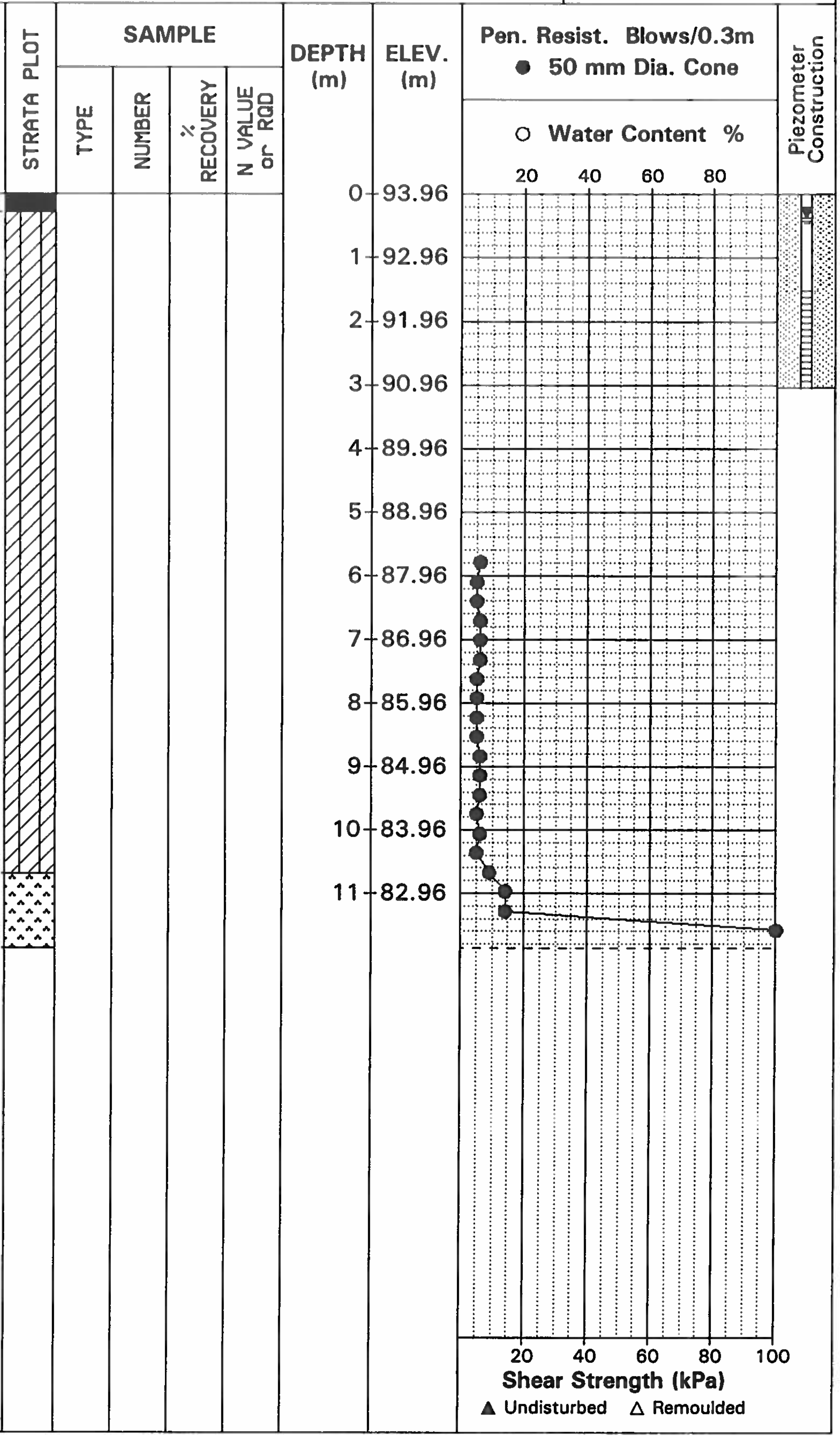
DATE 9 NOV 06

FILE NO. PG0909

HOLE NO. BH 2

SOIL DESCRIPTION	Depth	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE						
TOPSOIL	0.28					
Inferred SILTY CLAY	10.70					
Inferred GLACIAL TILL	11.86					
End of Borehole Practical DCPT refusal @ 11.86m depth (GWL @ 0.37m-Nov. 14/06)						

DEPTH (m)	ELEV. (m)
0	93.96
1	92.96
2	91.96
3	90.96
4	89.96
5	88.96
6	87.96
7	86.96
8	85.96
9	84.96
10	83.96
11	82.96



patersongroup Consulting Engineers
28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Campeau Drive Extension at the Carp River
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 8 NOV 06

FILE NO. PG0909

HOLE NO. BH 3

SOIL DESCRIPTION

GROUND SURFACE

SOIL DESCRIPTION	
TOPSOIL	0.30
Loose, brown SILTY SAND, trace to some clay	1.83
Grey CLAYEY SILT with sand and sand seams	2.13
Loose, grey coarseSAND	2.36
Firm to soft, grey SILTY CLAY to CLAYEY SILT - trace sand and gravel by 7.6m depth - firm by 8.0 m depth - some sand by 11.0m depth	12.60

End of Borehole

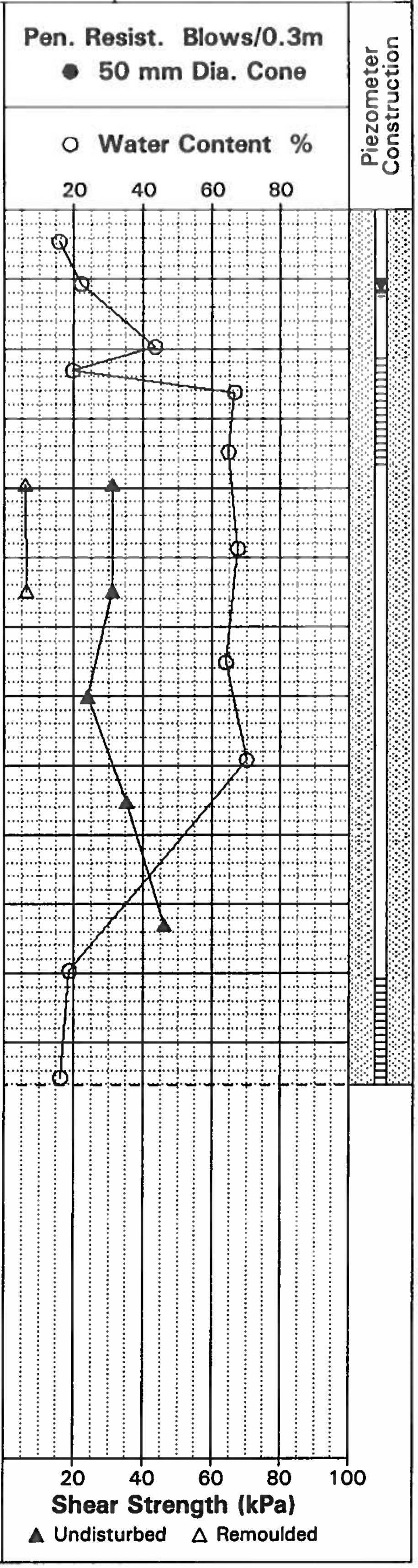
Practical refusal to augering @ 12.60m depth

(GWL @ 1.16m-Nov. 14/06)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	75	6
SS	3	100	1
SS	4	83	1
TW	5	83	
SS	6	100	1
TW	7	88	
SS	8	100	1
SS	9	50	
	10	13	50+

DEPTH (m)	ELEV. (m)
0	95.15
1	94.15
2	93.15
3	92.15
4	91.15
5	90.15
6	89.15
7	88.15
8	87.15
9	86.15
10	85.15
11	84.15
12	83.15



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)

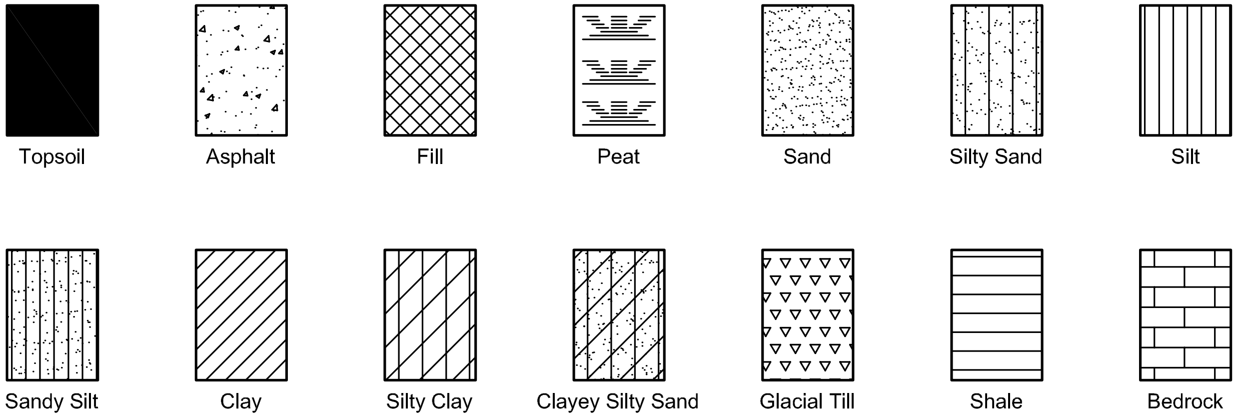
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

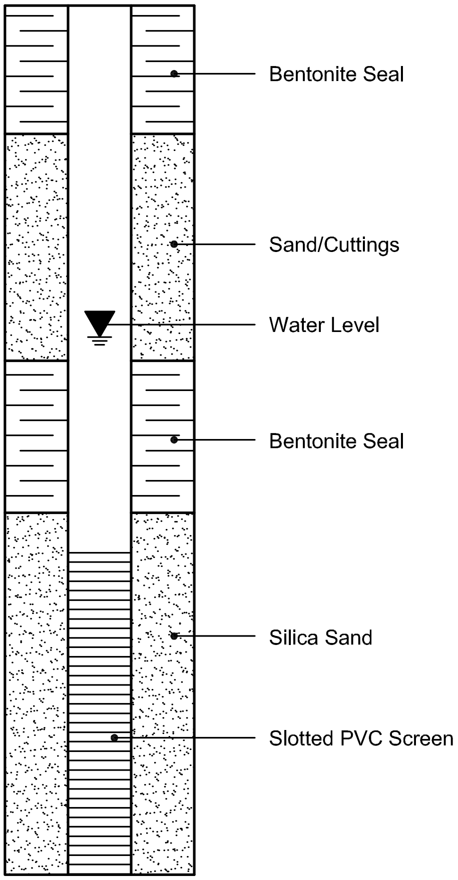
SYMBOLS AND TERMS (continued)

STRATA PLOT

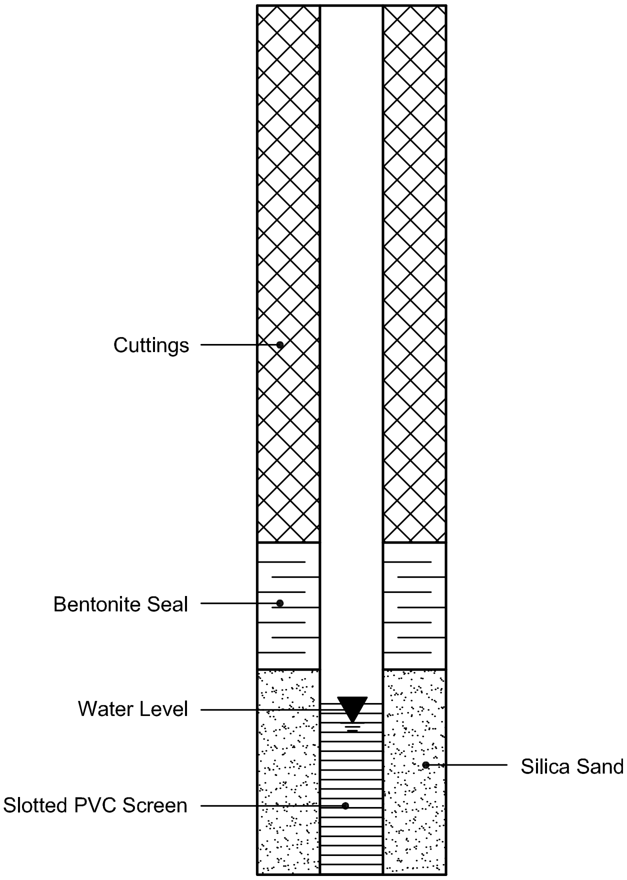


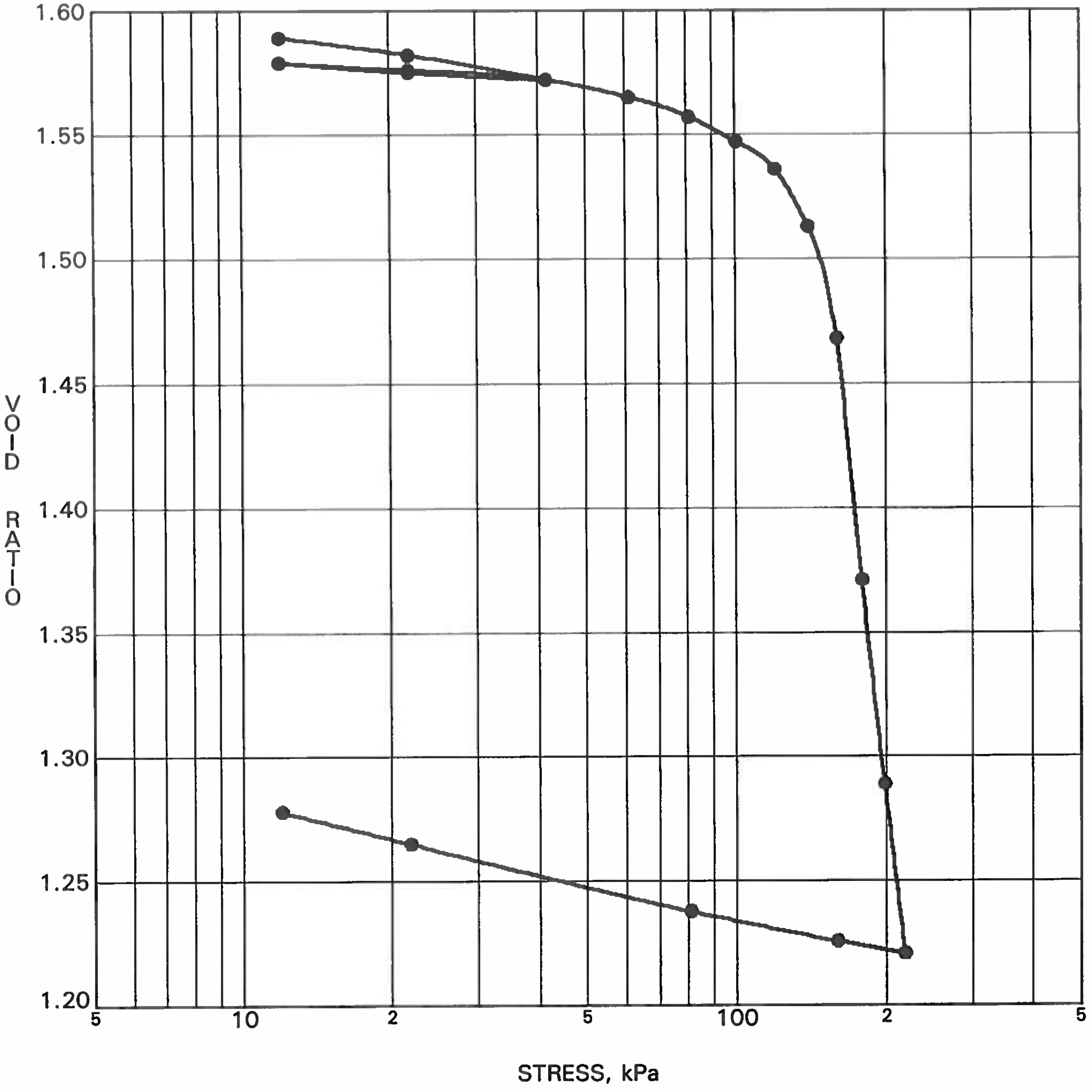
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 1	p'o	75 kPa	Ccr	0.013
Sample No.	TW 6	p'c	147 kPa	Cc	1.888
Sample Depth	8.82 m	OC Ratio	2.0	Wo	58.1 %
Sample Elev.	85.24 m	Void Ratio	1.597	Unit Wt.	16.4 kN/m^3

CLIENT **Taggart Realty Management**

PROJECT **Geotechnical Investigation - Campeau Drive Extension at the Carp River**

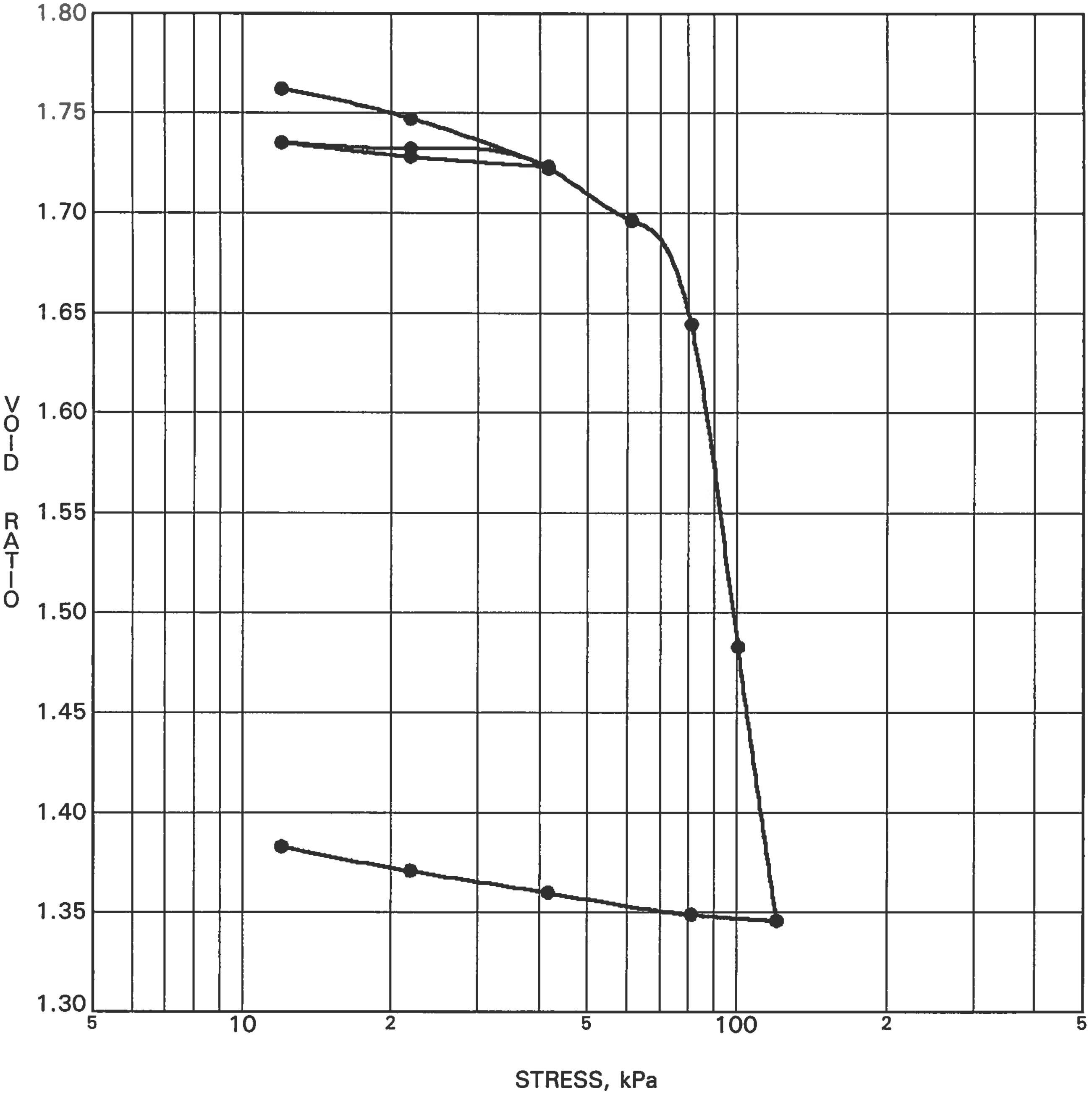
FILE NO. **PG0909**

DATE **29/12/07**

patersongroup Consulting Engineers

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3**	p'_o	**41** kPa	Ccr	**0.023**
Sample No.	**TW 5**	p'_c	**75** kPa	Cc	**1.743**
Sample Depth	**3.49** m	OC Ratio	**1.8**	Wo	**64.6 %**
Sample Elev.	**91.66** m	Void Ratio	**1.777**	Unit Wt.	**16.0** kN/m^3

CLIENT **Taggart Realty Management**

FILE NO. **PG0909**

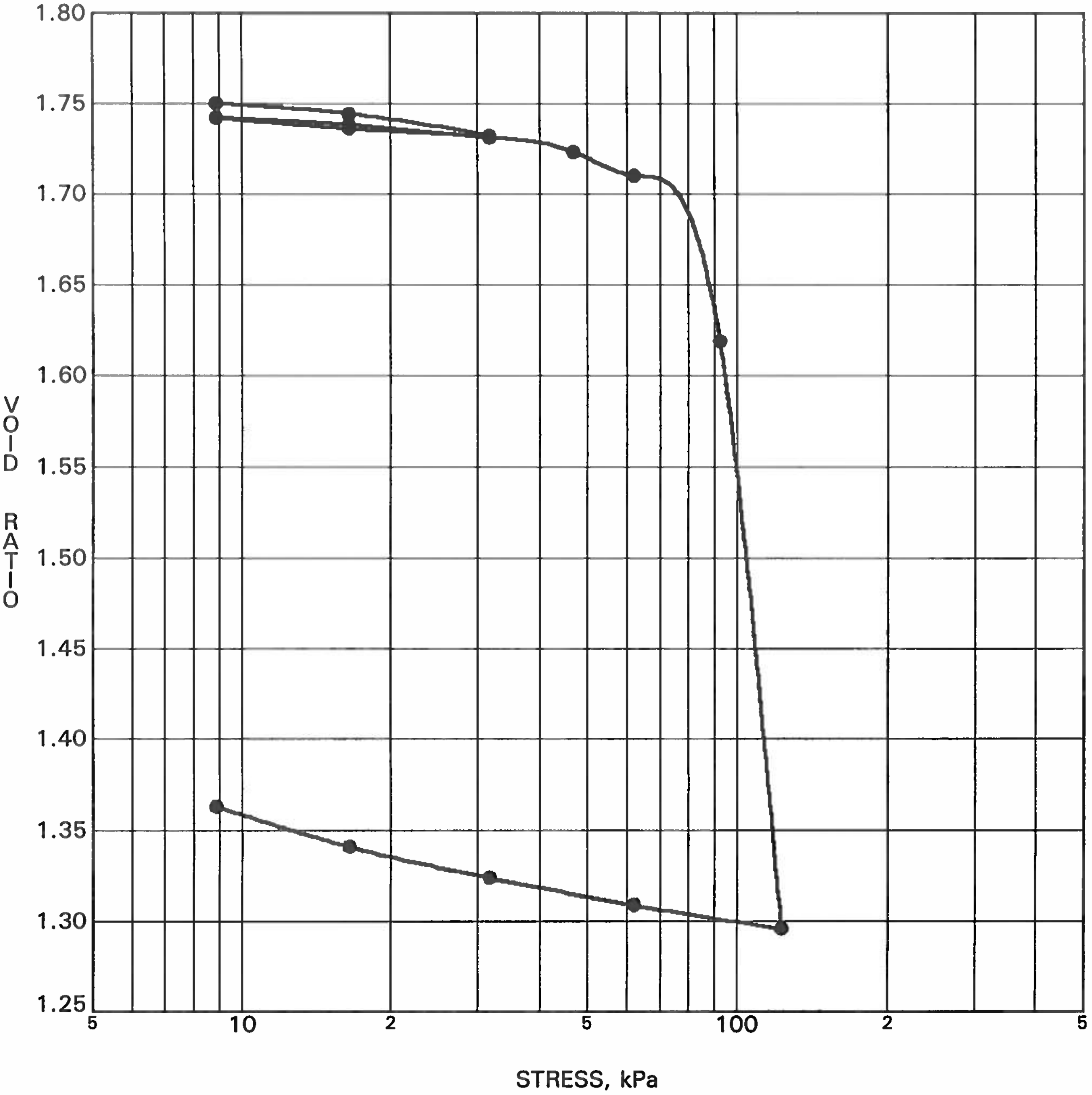
PROJECT **Geotechnical Investigation - Campeau Drive**
Extension at the Carp River

DATE **27/12/06**

patersongroup Consulting Engineers

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 3	p'$_o$	60 kPa	Ccr	0.019
Sample No.	TW 7	p'$_c$	86 kPa	Cc	2.623
Sample Depth	6.52 m	OC Ratio	1.4	Wo	63.8 %
Sample Elev.	88.63 m	Void Ratio	1.755	Unit Wt.	16.0 kN/m^3

CLIENT **Taggart Realty Management**

PROJECT **Geotechnical Investigation - Campeau Drive Extension at the Carp River**

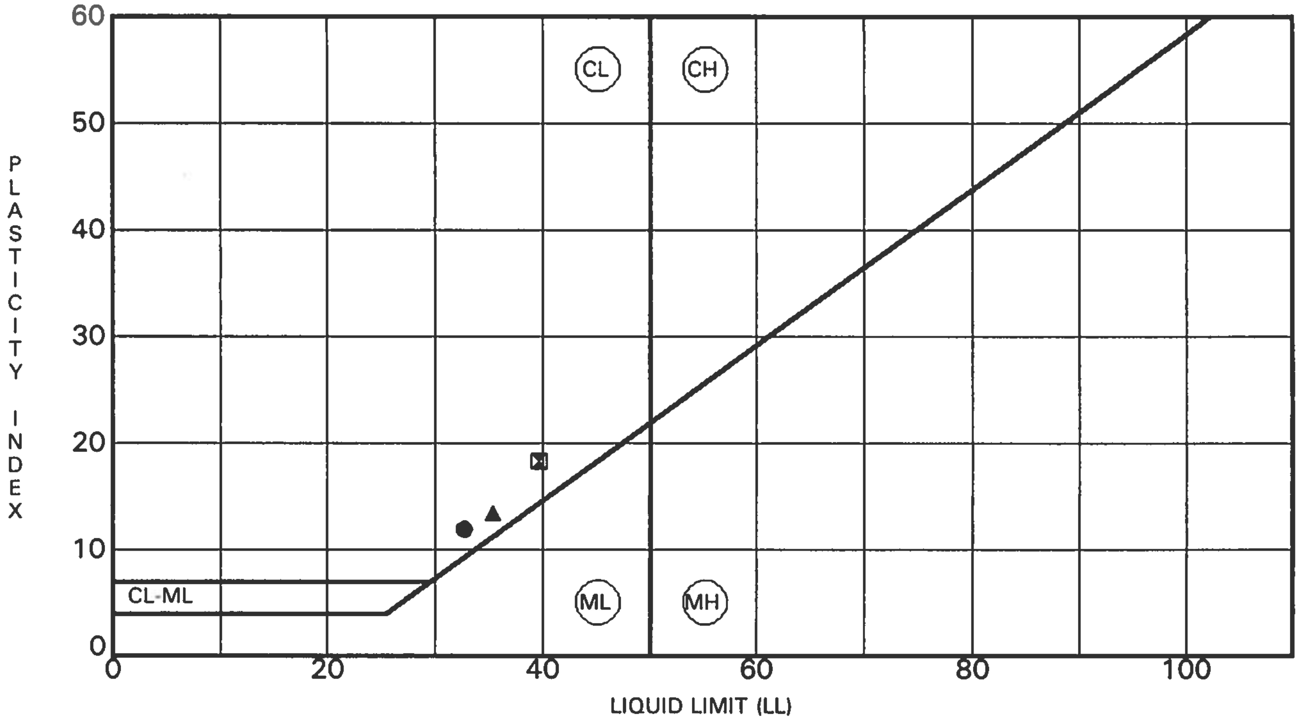
FILE NO. **PG0909**

DATE **27/12/06**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1**	**33**	**21**	**12**		**Low plastic CLAY (TW 6)**
☒	**BH 3**	**40**	**21**	**18**		**Low plastic CLAY (TW 5)**
▲	**BH 3**	**35**	**22**	**13**		**Low plastic CLAY (TW 7)**

CLIENT **Taggart Realty Management**

FILE NO. **PG0909**

PROJECT **Geotechnical Investigation - Campeau Drive Extension at the Carp River**

DATE **9 NOV 06**

patersongroup **Consulting Engineers**

28 Concouse Gate, Unit 1, Ottawa, Ontario K2E 7T7

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Report Date: 22-Nov-2006
Order Date: 10-Nov-2006

Client: **Paterson Group Inc.**
Client PO: **4703**
Project: **PG0909**

Matrix: Soil

	Sample ID:	BH3 SS3
	Sample Date:	08/11/2006
Parameter	**MDL/Units**	L8526.1
Chloride	5 ug/g	5
Sulphate	5 ug/g	25
pH	0.05 pH units	8.32
Resistivity	0.1 ohm.m	75

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - PRELOAD MONITORING PROGRAM

FIGURES 3 AND 4 - SEISMIC SHEAR WAVE VELOCITY PROFILES

Drawing PG0909-4 - TEST HOLE LOCATION PLAN

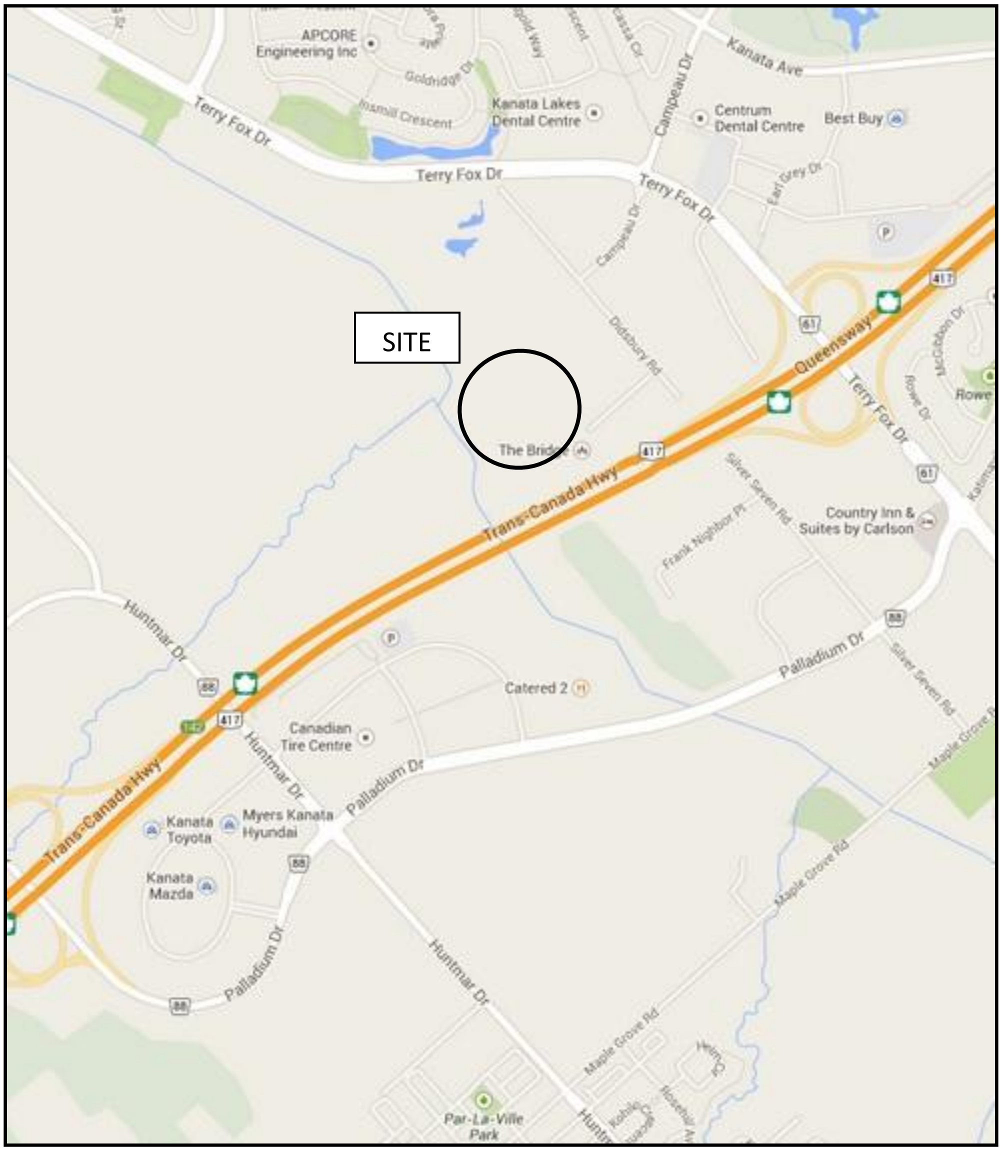
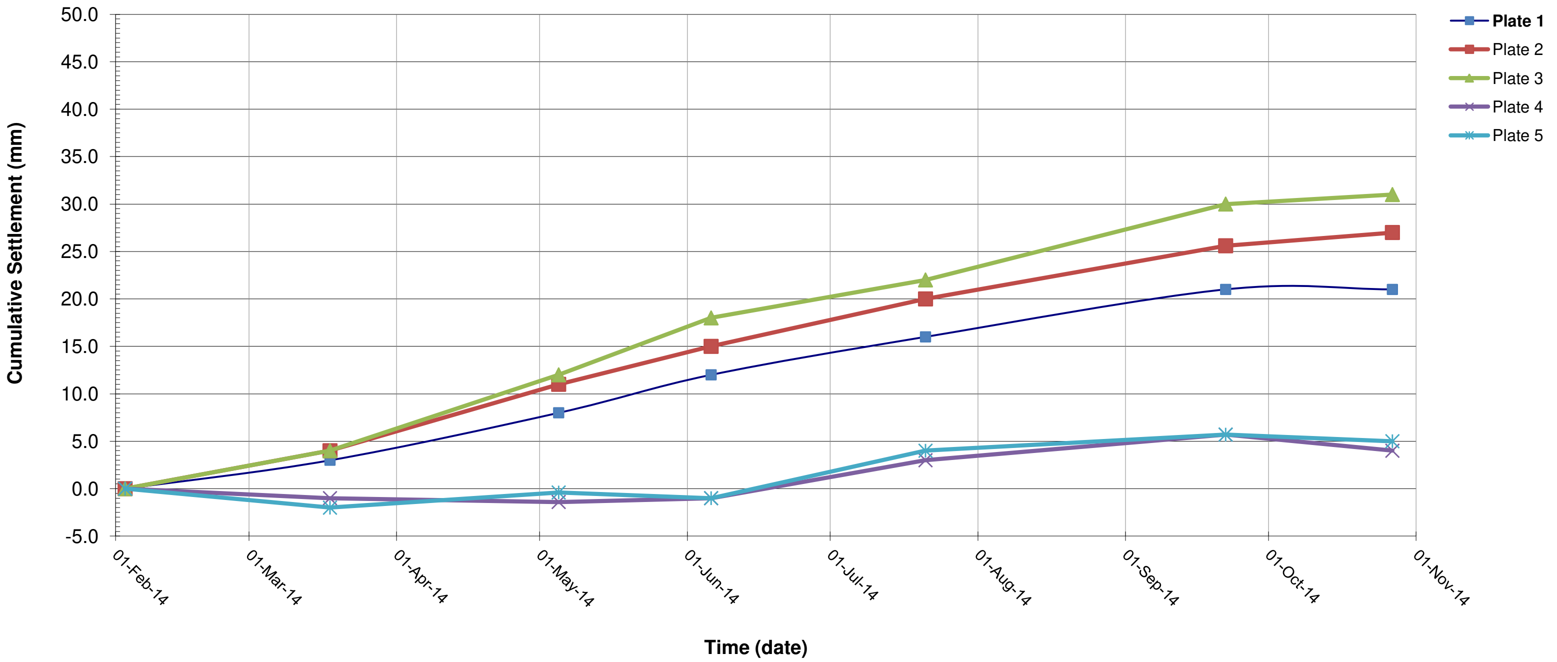


FIGURE 1

KEY PLAN

Figure 2 - Preload Monitoring Program
Kanata Commons - Campeau Drive Extension



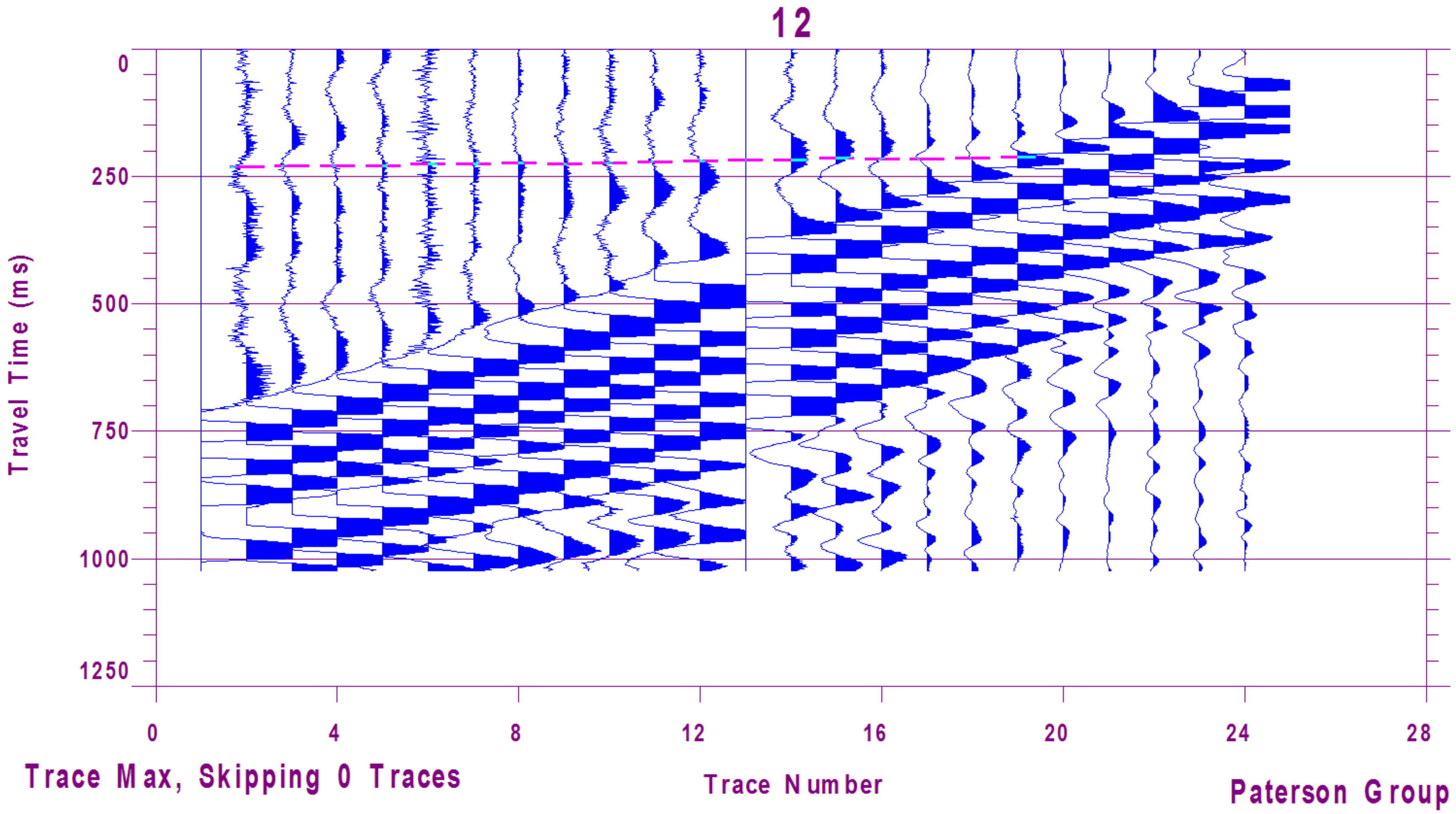


Figure 3 – Shear Wave Velocity Profile at Shot Location 89.0 m

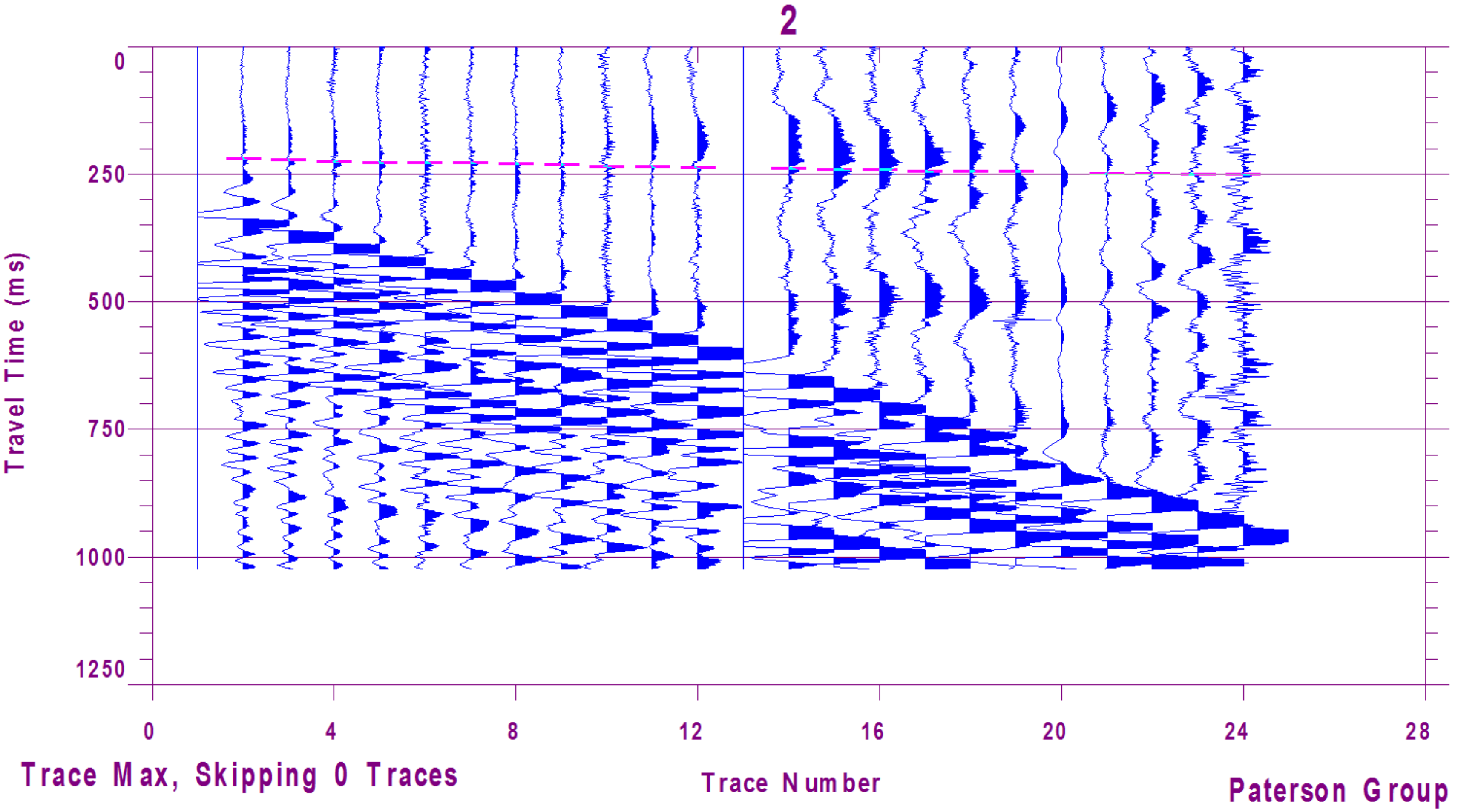
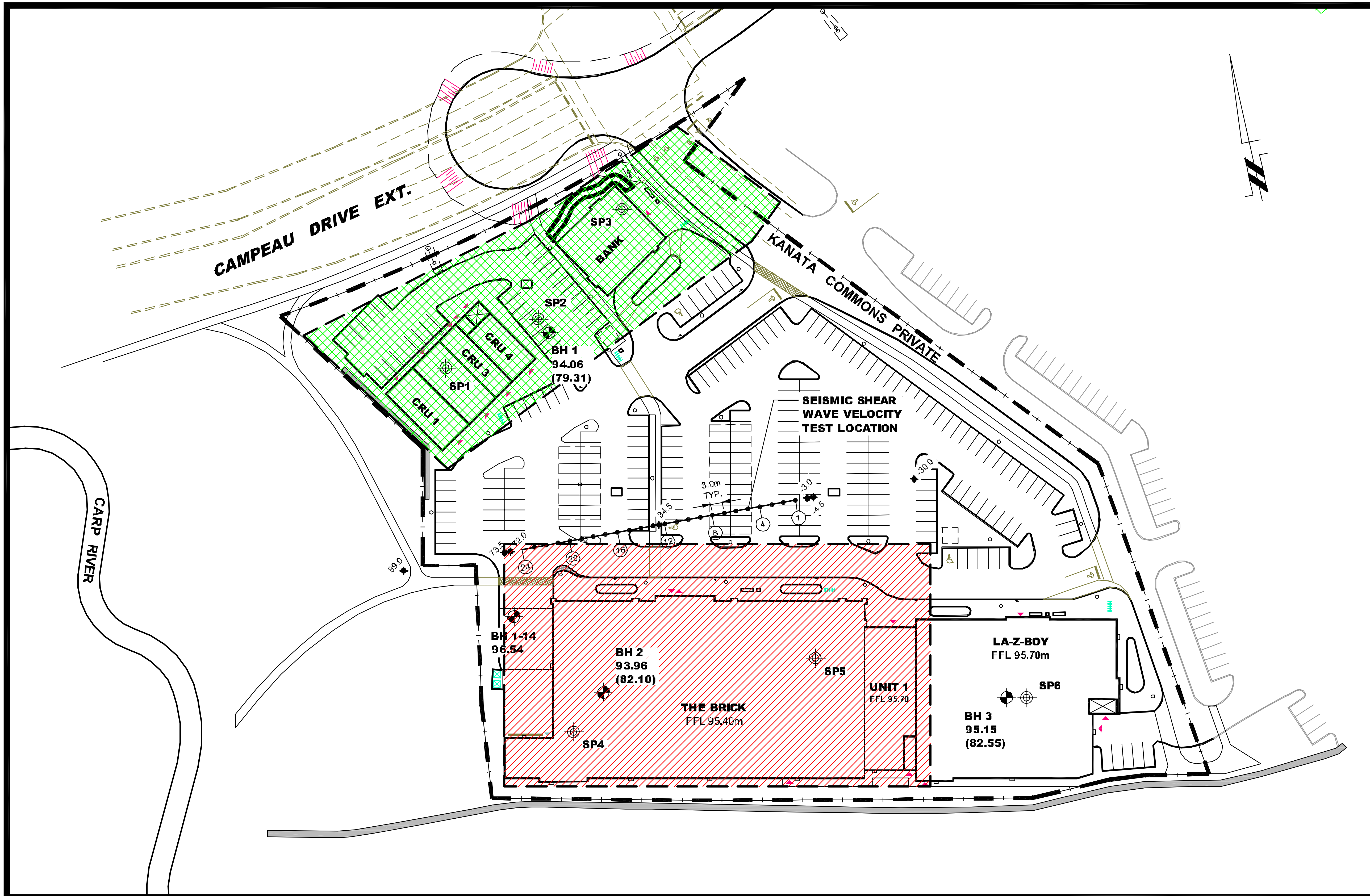


Figure 4 – Shear Wave Velocity Profile at Shot Location – 30 m



LEGEND:

🟩	3.0m HIGH PRELOAD AREA
🟥	2.1 TO 2.5m HIGH PRELOAD PILE AREA
—●—●—●—	GEOPHONE LOCATIONS
(20)	GEOPHONE NUMBER
73.5 ⊕	SHOT LOCATION
⊕	SETTLEMENT PLATE LOCATIONS
◐	BOREHOLE LOCATION
94.06	GROUND SURFACE ELEVATION (m)
(82.55)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

SCALE - 1:1000

0 5 10 15 20 50m

paterson group
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
1	UPDATED SITE PLAN LAYOUT	28/10/2014	DJG

TAGGART REALTY

GEOTECHNICAL INVESTIGATION
CAMPEAU DRIVE EXTENSION AT CARP RIVER

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION

Scale: 1:1000
Date: 10/2014
Checked by: DJG
Report No.: PG0909
Approved by: DJG
Drawing No.: PG0909-4
Drawn by: MPG

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