Geotechnical Engineering

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Geotechnical Investigation

Proposed Residential Development 147 Langstaff Drive Ottawa, Ontario

Prepared For

Inverness Homes c/o The Stirling Group

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Report: PG4918-1 Revision 1



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1.0 Introduction

Paterson Group (Paterson) was commissioned by Inverness Homes, c/o The Stirling Group, to conduct a geotechnical investigation for the proposed residential development to be located at 147 Langstaff Drive in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2). The objectives of the investigation were to:

- determine the subsurface soil and groundwater conditions by means of boreholes.
- □ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

2.0 Proposed Development

It is understood that the proposed residential development will consist of townhouses of slab-on-grade construction, a clubhouse of slab-on-grade construction, and several low-rise apartment buildings each with 1 level of underground parking. The proposed development will also include associated access lanes and parking areas with landscaped margins. In addition, parkland is proposed along the east and west sides of the central ravine at the site. It is understood that the site will be municipally serviced.



3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was carried out on July 11 and July 12, 2019. At that time, a total of seven (7) boreholes were advanced to a maximum depth of 9.1 m below existing ground surface. A previous field investigation was also completed by Paterson on October 23 and 24, 2008. During that time a total of six (6) boreholes were advanced to a maximum depth of 9.8 m. The test hole locations were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG4918-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter splitspoon (SS) sampler or from the auger flights. The depths at which the auger and split spoon samples were recovered from the test holes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. 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. This testing was done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 2 and BH 7-19. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the 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.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

Groundwater monitoring wells were installed at BH 1-19, BH 2-19, and BH 3-19, and flexible PVC standpipes were installed in the remaining boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented on the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples from the current investigation 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 test hole locations from the current geotechnical investigation were determined by Paterson personnel and were located and surveyed in the field by Robinson Land Development. These boreholes are understood to be referenced to a geodetic datum.

The ground surface elevations at the previous borehole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle on the fire hydrant located along Langstaff Drive in the northwest corner of the subject site. An assumed elevation of 100.00 m was assigned to the TBM. The locations of the boreholes are presented on Drawing PG4918-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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



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 sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.



4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and has a 3 to 5 m deep ravine running in an approximate north-south direction through the central portion of the site, and a 4 to 5 m deep ravine running along the west property boundary. The site generally has a grassed surface, with the exception of forested areas along the ravines. The northern boundary of the site is at grade with Langstaff Drive, and slopes downward gradually to the southeast.

4.2 Subsurface Profile

Overburden

The subsurface profile encountered at the borehole locations generally consisted of a 150 to 200 mm thick layer of topsoil underlain by a compact silty sand extending to approximate depths of 0.6 to 1.4 m below the existing ground surface.

Underlying the topsoil and/or silty sand, a silty clay deposit was encountered. The silty clay was generally observed to consist of a stiff, brown silty clay crust, becoming a stiff to firm grey silty clay at approximate depths of 4.5 to 6 m below the existing ground surface.

Underlying the silty clay, a grey silty sand to sandy silt was encountered at approximate depths of 6 to 8.5 m below the existing ground surface.

Bedrock

Practical refusal to the DCPT was encountered at depths of 18.9 and 24.9 m in boreholes BH 2 and BH 7-19, respectively. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Based on available geological mapping, bedrock in the area consists of interbedded limestone and shale of the Verulam Formation with overburden drift thickness ranging between 15 and 50 m.

4.3 Groundwater

Groundwater levels (GWL) were measured in BH 1-19 through BH 7-19 on July 18, 2019 and in BH 1 through BH 6 on November 3, 2008. The measured GWL readings are presented in Table 1 below. Groundwater levels can also be estimated based on the observed colour, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is expected between a 4.5 to 6 m depth.

However, it should be noted that the groundwater levels can fluctuate periodically throughout the year and higher levels could be encountered at the time of construction.

Table 1 - S	Table 1 - Summary of Groundwater Level Readings									
Borehole	Ground	Measured Ground	Measured Groundwater Level (m)							
Number	Elevation (m)	Depth	Elevation	Recording Date						
BH 1-19*	106.71	6.17	100.54	July 18, 2019						
BH 2-19*	106.86	5.30	101.56	July 18, 2019						
BH 3-19*	105.94	6.91	99.03	July 18, 2019						
BH 4-19	104.31	2.07	102.24	July 18, 2019						
BH 5-19	100.75	Blocked	-	July 18, 2019						
BH 6-19	100.81	Blocked	-	July 18, 2019						
BH 7-19	102.86	2.8	100.06	July 18, 2019						
BH 1	-	5.75	-	November 3, 2008						
BH 2	-	4.76	-	November 3, 2008						
BH 3	-	6.31	-	November 3, 2008						
BH 4	-	6.95	-	November 3, 2008						
BH 5	-	7.05	-	November 3, 2008						
BH 6	-	6.7	-	November 3, 2008						

Notes:

- * Denotes borehole instrumented with a 51 mm diameter monitoring well.

- Current boreholes elevations (BH 1-19 through BH 6-19) surveyed by Robinson Land Development and are understood to be referenced to a geodetic datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed residential development. It is expected that the proposed residential buildings will be founded on conventional shallow footings placed on an undisturbed, compact silty sand and/or undisturbed stiff silty clay bearing surface.

Due to the presence of a silty clay deposit, permissible grade raise restrictions are recommended for this site.

A construction setback defined as the Limit of Hazard Lands has been defined along the existing ravines, as presented on Drawing PG4918-1 - Test Hole Location Plan. This is discussed further in Section 6.8.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and 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 building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Granular material should be tested and approved prior to delivery to the site. The fill should be placed in loose lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize 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 density of 95% of the SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, compact silty sand or undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**.

An undisturbed soil bearing surface consists of one 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 placement of concrete for footings.

The bearing resistance value at SLS for shallow footings bearing on the above noted soils will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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 silty sand or silty clay 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 bearing soil.

Permissible Grade Raise Recommendations

Consideration must also be given to potential settlements which could occur due to the presence of the silty clay deposit and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill. The foundation loads to be considered for the settlement case are the continuously applied loads which consist of the unfactored dead loads and the portion of the unfactored live load that is considered to be continuously applied. For buildings, a minimum value of 50% of the live load is often recommended by Paterson. A post-development groundwater lowering of 0.5 m was assumed.

Our permissible grade raise recommendations for the proposed residential development are presented in Drawing PG4918-2 - Permissible Grade Raise Plan in Appendix 2. It should be noted that grade raises are not permitted within the Hazard Lands as identified on Drawing PG4918-2.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

A seismic site response **Class D** should be used for design of the proposed buildings at the subject site according to the Ontario Building Code (OBC) 2012. The soils underlying the site are not considered susceptible to liquefaction.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill from within the footprint of the proposed buildings, the native soil surface or approved engineered fill surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

For structures with slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

For structures with basement slabs, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. Further, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying the basement slabs. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed buildings. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m³.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 12.2 kN/m^3 , where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o\gamma H$ where:

- K_{o} = At-rest earth pressure coefficient of the applicable retained soil (0.5)
- γ = unit weight of fill of the applicable retained soil (kN/m³)
- H = height of the basement wall (m)

An additional pressure having a magnitude equal to K_o q and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the "at-rest" case if care is not exercised during the compaction of the backfill materials to stay at least 0.3 m away from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using 0.375 $\cdot a_c \cdot \gamma \cdot H^2/g$ where:

 $a_c = (1.45 - a_{max}/g)a_{max}$ $\gamma =$ unit weight of fill of the applicable retained soil (kN/m³) H = height of the wall (m) g = gravity, 9.81 m/s²

The peak ground acceleration, (a_{max}) , for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using P_o = 0.5 K_o γ H², where K_o = 0.5 for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

 $h = \{P_{o} \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\} / P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

For design purposes, the pavement structure presented in the following tables is recommended for the design of car only parking areas, local roadways and arterial roadways with bus traffic.

Table 2 - Recommended Pavement Structure - Driveways / 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 fill, in situ soil or OPSS Granular B Type I or II material placed over in situ so							

Table 3 - Recommended Pavement Structure - Local Roads							
Thickness (mm)	Material Description						
40	Wear Course - Superpave 12.5 Asphaltic Concrete						
50	Binder Course - Superpave 19.0 Asphaltic Concrete						
150	BASE - OPSS Granular A Crushed Stone						
400	SUBBASE - OPSS Granular B Type II						
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill							

North	Bay	

Table 4 - Recommend	led Pavement Structure - Roadways with Bus Traffic				
Thickness mm	Material Description				
40	Wear Course - Superpave 12.5 Asphaltic Concrete				
50	Upper Binder Course - Superpave 19.0 Asphaltic Concrete				
50	Lower Binder Course - Superpave 19.0 Asphaltic Concrete				
150	BASE - OPSS Granular A Crushed Stone				
600	SUBBASE - OPSS Granular B Type II				
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ or fill					

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 II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for driveways and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. The pavement granular 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 vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on maintaining the contact zone between the subgrade material and the base stone 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 load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended for each proposed structure. 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 which is 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.

Sub-slab Drainage

For the proposed low-rise apartment buildings, which are understood to contain 1 level of underground parking, sub-slab drainage will be required to control water infiltration. For preliminary design purposes, it is recommended that 100 or 150 mm perforated pipes be placed at approximate 6 m centres underlying the basement floor. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided for any elevator pits (pit bottom and walls).

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

Unheated structures, such as the access ramps for the underground parking levels, may require foundation insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be 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 greater part of the excavations to be undertaken by opencut 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 subsoil at this site is considered to be mainly a 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.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will 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

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the City of Ottawa. These recommendations are for standard, open cut excavation placed services.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. 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 extend 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.

Generally, it should be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high water contents make compacting impractical 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 minimize 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.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. 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 Ministry of the Environment, Conservation, and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

The subsurface conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur. Precautions should be taken if winter construction is considered for this project.

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, tarpaulins or other suitable means. In this regard, 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.

The trench excavations should be constructed in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

6.7 Tree Planting Restrictions

The silty clay deposit encountered at the site was hard to firm and is considered to be low to medium sensitivity clay and should not be considered a sensitive marine clay. Therefore, where footings are founded over a silty clay bearing surface, large trees (mature height over 14 m) can be planted provided a tree to foundation setback equal to the full mature height of the tree is utilized (e.g. in a park or other green space). Tree planting setback limits may be reduced to 4.5 m for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m). It should be noted that shrubs and other small plantings are permitted within the 4.5 m setback area.

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.

6.8 Slope Stability Assessment

The slope condition was reviewed by Paterson field personnel as part of the geotechnical investigation. Eight slope cross-sections were studied as the worst case scenarios. The cross section locations are presented on Drawing PG4918-1 - Test Hole Location Plan attached to the current report.

The existing slope face of the ravine located within the central portion of the site is currently vegetated and tree covered with signs of minor erosion within the lower portion of the slope face. However, the slope face was not observed to be in direct contact with the watercourse. The upper slope is observed to be well vegetated and stable with little to no signs of active erosion. A 0.5 to 1 m wide watercourse was observed at the base of the central ravine. The watercourse was observed to be less than 75 mm deep with little to no flow. Two existing stormwater management ponds (SWMP) were observed along the existing watercourse. The first SWMP is located north of Langstaff Drive and the second is located south of Langstaff Drive within our subject site.

The existing slope face of the ravine located along the west property boundary was noted to be vegetated and treed throughout. A watercourse was not observed in this ravine at the time of our site visit, however, it is understood that intermittent water flow occurs through this corridor. A 0.6 m diameter corrugated steel drainage pipe drains into the ravine approximately 20 m south of the previously in-filled area at the north end of the ravine. No evidence of active erosion was noted along this slope.

A slope stability analysis was carried out to determine the required construction setback from the top of the bank based on a factor of safety of 1.5. Erosional and access allowances were also considered in the determination of limits of hazard lands and are discussed in the following sections. The proposed Limit of Hazard Lands and top of bank are shown on Drawing PG4918-1 - Test Hole Location Plan attached to the current report.

Slope Stability Analysis

The analyses of the stability of the slopes were carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable.

A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

The cross-sections were analyzed based on the existing conditions observed during our site visit and review of the available topographic mapping. The slope stability analysis was completed at each slope cross-section under worst-case-scenario by assigning cohesive soils under fully saturated, realistic groundwater flow conditions. Subsoil conditions at the cross-sections were inferred based on nearby boreholes and general knowledge of the area's geology.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 5 below.

Table 5 - Effective Strength Soil and Material Parameters (Static Analysis)									
Soil Layer	Unit Weight (kN/m³)	Friction Angle (degrees)	Cohesion (kPa)						
Brown Silty Clay Crust	17	33	7						
Silty Sand	20	35	1						

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths recovered within the open boreholes completed at the time of our geotechnical investigation and based on our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 6 below.

Table 6 - Total Strength Soil and Material Parameters (Seismic Analysis)									
Soil Layer	Unit Weight (kN/m³)	Friction Angle (degrees)	Undrained Shear Strength (kPa)						
Brown Silty Clay Crust	17	0	150						
Silty Sand	20	35	1						

Static Loading Analysis

The results for the slope stability analyses under static conditions at Sections A, B, C, D, E and F are shown on Figures 2, 4, 8, 12, 14 and 16 are attached to the present report. The factor of safety was found to be greater than 1.5 at Sections A, B, E, and F. However, Sections C and D require setbacks of 4.8 and 3.5 m, respectively, from top of slope to obtain a factor of safety greater than 1.5.

Seismic Loading Analysis

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g (50% of PGA = 0.32g) was considered for all slopes. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the slope stability analyses under seismic conditions are shown on Figures 3, 5, 9, 13, 15, and 17 in Appendix 2. The results indicate that the factors of safety are greater than 1.1 under seismic conditions. Based on these results, the slopes are considered to be stable under seismic loading. Therefore, when considering seismic loading, no geotechnical setback from the top of the slope is required to achieve a factor of safety of 1.1 for the limit of the hazard lands.

Geotechnical Setback - Limit of Hazard Lands

Signs of erosion were noted along the lower portion of the slope face that confines the watercourse located across the central portion of the site. Some minor sloughing failures were noted in the lower portion of the slope, leaving some exposed tree roots. No evidence of active erosion was noted along the toe of slope.

The general characteristics of the central and western ravines are also discussed in detail in the report titled "Fluvial Geomorphological and Erosion Hazard Assessment" prepared for this site by Geomorphix dated October 1, 2019.

Based on our observations and the recommendations in the above-noted report prepared by Geomorphix, a 2 m toe erosion allowance is deemed appropriate for central ravine slope based on the cohesive nature of the soils, the observed erosion areas and the current watercourse depth and width. It is considered that a toe erosion allowance of 2 m and an erosion access allowance of 6 m is required from the top of slope.

For the slope that confines the intermittent watercourse located on the western end of the site, a 1 m toe erosion allowance is deemed appropriate based on our observations on-site and the recommendations provided by Geomorphix. Given the low risk of erosion for the western ravine, an erosion access allowance of 4.5 m is considered sufficient in addition to the 1 m toe erosion allowance from the top of stable slope (i.e. - slope with factor of safety greater than 1.5). The 4.5 m erosion access allowance is sufficient for a hydraulic excavator and truck access, if ever required.

The limit of hazard lands, which include these allowances, are indicated on Drawing PG4918-1 - Test Hole Location Plan attached to the present report.

It is recommended that the existing vegetation on the slope faces not be removed as it contributes to the stability of the slope and reduces erosion.

Global Stability Analysis

Where retaining walls are proposed in the vicinity of the Limit of Hazard Lands at the subject site, global stability analyses were conducted for these walls. Further, the adjacent slopes were also analyzed to determine if the proposed retaining walls and associated grade raises will impact the stability of the slope and/or the Limit of Hazard Lands.

The results for the global stability analyses under static conditions at Sections B, C, F, G, and H are shown on Figures 6, 10, 18, 20, and 22, which are attached to the present report. The results of these global stability analyses indicate that the factor of safety exceeds 1.5 under static conditions.

The results for the global stability analyses under seismic conditions at Sections B, C, F, G, and H are shown on Figures 7, 11, 19, 21, and 23, which are attached to the present report. The results of these analyses indicate that the factor of safety exceeds 1.1 under seismic conditions.

Further, the presence of the retaining walls and associated grade raises will not impact the factor of safety of the adjacent slopes or the position of the Limit of Hazard Lands.

6.9 Corrosion Potential and Sulphate

One (1) soil sample was submitted for analytical testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.01%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderate to aggressive corrosive environment.

7.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined:

- **D** Review detailed grading plan(s) from a geotechnical perspective.
- Observation of all bearing surfaces prior to the placement of concrete.
- 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 placing backfilling materials.
- □ Field density tests to ensure that the specified level of compaction has been achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with Paterson's recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the grading plan once available. Paterson's recommendations should also be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole logs are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

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, Paterson requests to be notified immediately in order to permit reassessment of the 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 Inverness Homes or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Scott S. Dennis, P.Eng

Report Distribution:

- □ Inverness Homes (e-mail copy)
- Image: Paterson Group (1 copy)



Andrew J. Tovell, P.Eng.

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

natersonar		In	Con	sulting		SOIL	- PRO	FILE A	ND TES	ST DATA						
154 Colonnade Road South, Ottawa, Ont	154 Colonnade Road South, Ottawa, Ontario K2E 7J5								Geotechnical Investigation 147 Langstaff Drive Ottawa, Ontario							
DATUM Ground surface elevations	prov	ided b	y Rob	oinson	Lanc	l Develop	ment.		FILE NO.	DC/019	,					
REMARKS									HOLE NO).)					
BORINGS BY CME 55 Power Auger		1		DA	TE 2	2019 July	11	1		BH 1-19						
SOIL DESCRIPTION	PLOT		SAN	IPLE		DEPTH	ELEV.	Pen. F	Resist. Blo 50 mm Dia	ows/0.3m a. Cone	Nell on					
GBOUND SUBFACE	STRATA	ТҮРЕ	NUMBER	[∞] RECOVERY	N VALUE or RQD	(11)	(11)	0 \ 20	Water Cor	ntent %	Monitoring Constructi					
		×	4			0-	-106.71									
		ss	2	58	13	1-	-105.71				<u>իրիդիրիդի</u> Սրիդիրիդիրի					
		ss	3	100	10	2-	-104.71				արերերին արերերին					
		ss	4	100	8					· · · · · · · · · · · · · · · · · · ·						
Very stiff to stiff, brown SILTY CLAY		ss	5	100	5	3-	-103.71									
						4-	-102.71	A.								
		ss	6	100	4	5-	-101.71									
- firm to very stiff and grey by 6.1m depth						6-	-100.71									
						7-	-99.71	4								
						8-	-98.71									
9.14						9-	-97.71									
End of Borehole																
(GWL @ 6.17m - July 18, 2019)								20	40 6	0 80	100					
								She ▲ Undis	ar Streng	th (kPa) Remoulded						

natersonar		In	Con	sulting		SOIL	- PRO	FILE AI	ND TEST DATA
154 Colonnade Road South, Ottawa, On	tario I	2E 7J	Eng	ineers	Ge 14	eotechnic 7 Langst	al Invest aff Drive	igation	
DATUM Ground surface elevations	s prov	ided b	y Rol	oinson	Land	l Develop	oment.		FILE NO.
REMARKS									HOLE NO.
BORINGS BY CME 55 Power Auger	1	1		DA	TE 2	2019 July	11		BH 2-19
SOIL DESCRIPTION	PLOT		SAN	IPLE		DEPTH	ELEV.	Pen. R ● 5	esist. Blows/0.3m = 0 mm Dia. Cone ≥ 등
	STRATA	ТҮРЕ	NUMBER	% RECOVERY	N VALUE or RQD		(11)	• V	Vater Content %
TOPSOIL 0.23	8	×		щ		0-	106.86	20	
\		SS AU	1	100	15	1-	-105.86		
Very stiff to stiff, brown SILTY CLAY		ss	3	100	14	2-	-104.86		
- grey by 3.0m depth		ss	4	100	11	3-	-103.86		
		ss	5	100	8				
		ss	6	79	6	4-	-102.86		
<u>5.3</u> 0						5-	-101.86	A	
Brown SILTY CLAY trace sand		ss	7	62	9	6-	-100.86		
<u>6.9</u> 0		ss	8	100	2	7-	-99.86		
Stiff, grey SILTY CLAY with sand						8-	-98.86		
9.14 End of Borehole						9-	-97.86		
(GWL @ 5.30m - July 18, 2019)									
								20 Shea ▲ Undist	40 60 80 100 ar Strength (kPa) turbed △ Remoulded

natersonar		In	Con	sulting		SOIL	PRO	FILE AI	ND TEST DATA
154 Colonnade Road South, Ottawa, Ont	tario ł	(2E 7J	Eng	ineers	Ge 14 Ot	eotechnic 7 Langsta tawa, Or	al Invest aff Drive ntario	igation	
DATUM Ground surface elevations	prov	ided b	y Rob	oinson	Lanc	l Develop	ment.		FILE NO.
REMARKS									HOLE NO
BORINGS BY CME 55 Power Auger				DA	TE 2	2019 July	11	1	BH 3-19
SOIL DESCRIPTION	PLOT		SAN	IPLE		DEPTH	ELEV.	Pen. R	Resist. Blows/0.3m 50 mm Dia. Cone ≥ 5
	STRATA	ТҮРЕ	NUMBER	% ECOVERY	N VALUE or RQD	(m)	(m)	0 V	Water Content %
TOPSOIL 0 18		×		щ		0-	105.94	20	
		¥ AU	1	12	11	1-	-104.94		
		ss	3	79	7	2	102.04		
		ss	4	83	5	2	103.94		
Very stiff to stiff brown SILTY CLAY		ss	5	100	4	3-	-102.94		
		ss	6	100	5	4-	-101.94		
		ss	7	100	4	5-	-100.94	·····	
- firm to stiff and arey by 6.1m depth		ss	8	100	2	6-	-99.94		
- Inn to sun and grey by 0. In depth		ss V	9	100	W	7-	-98.94	4	
		∦ ss V ss	10	100	W				
			11	100	w	8-	-97.94		
End of Borehole		1	12	0	vv	9-	-96.94		
(GWL @ 6.91m - July 18, 2019)									
								20 Shea ▲ Undis	40 60 80 100 ar Strength (kPa) sturbed △ Remoulded

natersonar		In	Con	sulting	1	SOIL	- PRO	FILE AI	ND TE	ST DATA	•
154 Colonnade Road South, Ottawa, Ont	ario k	(2E 7J	Eng	ineers	G(14	eotechnic 17 Langst ttawa. Or	al Invest aff Drive	tigation			
DATUM Ground surface elevations	prov	ided b	y Rol	binson	Land	d Develop	ment.		FILE NO	PG4918	3
REMARKS									HOLE N	^{0.} PL 4 10	
BORINGS BY CME 55 Power Auger				D	ATE	2019 July	<u>, 11</u>			БП 4-13	
SOIL DESCRIPTION	A PLOT		SAN	/IPLE 것	<u></u>	DEPTH (m)	ELEV. (m)	Pen. R • 5	esist. B 60 mm Di	lows/0.3m a. Cone	ster Stion
	STRAT	ТҮРЕ	NUMBEI	ECOVEI	N VALU or RQI			• V	Vater Co	ntent %	iezome
TOPSOIL 0.20		×		щ		- 0-	-104.31	20	40	60 80	
			2	100	14	1-	-103.31			· · · · · · · · · · · · · · · · · · ·	
		x ss	3	100	9						
					-	2-	-102.31				
Very stiff brown SILTY CLAY , trace		ss	4	100	6	3-	-101.31				
sand										· · · · · · · · · · · · · · · · · · ·	149
						4-	-100.31			*	229
- grey by 4.6m depth		ss	5	100	4	5-	-99.31		/	/	
						6-	-98.31				159
6.70 End of Borehole										*	169
(GWL @ 2.07m - July 18, 2019)											
								20 Shea ▲ Undis	40 ar Streng turbed 2	60 80 gth (kPa) ∆ Remoulded	100

natersonar		SOIL PROFILE AND TEST DATA									
154 Colonnade Road South, Ottawa, Ont	Geotechnical Investigation 147 Langstaff Drive Ottawa, Ontario										
DATUM Ground surface elevations	prov	ided b	y Rol	oinson	Lanc	d Develop	oment.		FILE N	io. PG4	1918
REMARKS	HOLE	HOLE NO. DU 5 40									
BORINGS BY CME 55 Power Auger				D/	ATE 2	2019 July	/ 12			BH S	-19
SOIL DESCRIPTION	PLOT		SAN	/IPLE 것	M	DEPTH (m)	ELEV. (m)	Pen. R	esist. 1 60 mm D	ter tion	
	STRATA	ТУРЕ	NUMBER	ECOVER	N VALU			• V	Vater C	ontent %	iezome
GROUND SURFACE		×		Ř	4	0-	100.75	20	40	60 80	
<u></u>		滚 AU	1							· · · · · · · · · · · · · · · · · · ·	
		ss	2	88	11	1-	-99.75				
		ss	3	100	10	2-	-98.75				
		ss	4	100	9		07 75			· · · · · · · · · · · · · · · · · · ·	
Very stiff, brown SILTY CLAY						3-	-97.75				159
						4-	-96.75				
- grey by 4.6m depth		ss	5	100	7	5-	- 95 75	2			
							55.75				
						6-	-94.75				199
End of Borehole6.70											209
(Piezometer blocked - July 18, 2019)											
								20 Shea ▲ Undis	40 ar Strer turbed	60 80 60 80 6	100 100

natersonar		ır	Con	sulting		SOII	_ PRO	FILE AI	ND ⁻	TEST			
154 Colonnade Road South, Ottawa, On	Geotechnical Investigation 147 Langstaff Drive Ottawa, Ontario												
DATUM Ground surface elevations	s prov	ided b	y Rol	binson	Lanc	d Develop	oment.		FILE	E NO.	DC/019		
REMARKS									HOL	E NO.	FG4910)	
BORINGS BY CME 55 Power Auger	/ 12	1			BH 6-19								
SOIL DESCRIPTION	PLOT		SAN			DEPTH	ELEV.	Pen. R	lesist. 50 mm	/s/0.3m Cone	er		
	TRATA	ТҮРЕ	UMBER	° % COVER3	VALUE r RQD		(,	• v	Vater	Conte	nt %	zometi Istruct	
GROUND SURFACE	ß	~	z	RE	z ⁰	- 0-	-100.81	20	40	60	80	u E S S S S S S S S S S S S S S S S S S	
		AU	1										
		ss	2	58	8	1-	-99.81						
		ss	3	75	9		00.01			••••••	· · · · · · · · · · · · · · · · · · ·		
		<u>/ \</u>				2-	-98.81						
Hard to stiff, brown SILTY CLAY						3-	-97.81	<u> </u>	Δ				
												20	
						4-	-96.81				1	20	
- grey by 4.6m depth		ss	4	100	2	5-	-95.81						
						6-	-94.81		Δ				
6.70													
(Piezometer blocked - July 18, 2019)													
								20 Shea ▲ Undis	40 ar Str turbed	60 ength △ R	80 1 (kPa) emoulded	⊣ 00	

natersonar	ır	9	SOIL PROFILE AND TEST DATA									
154 Colonnade Road South, Ottawa, On	tario k	(2E 7J	G(14 01	Geotechnical Investigation 147 Langstaff Drive Ottawa, Ontario								
DATUM Ground surface elevations	s prov	ided b	y Rol	binson	Land	d Develop	oment.	FILE NO. PG4918				
REMARKS												
BORINGS BY CME 55 Power Auger			ATE	2019 July	/ 12	BH 7-19						
SOIL DESCRIPTION	PLOT		SAN	/IPLE		DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				
GROUND SURFACE	STRATA	ТҮРЕ	NUMBER	% RECOVER	N VALUE or RQD			O Water Content %				
0.18	B FX-X	₿ AU	1			- 0-	102.86					
		ss	2	100	13	1-	-101.86					
		ss	3	100	13	2-	-100.86					
Very stiff to stiff, brown SILTY CLAY						3-	-99.86					
						4-	-98.86					
- grey by 4.6m depth		ss	4	88	5	5-	-97.86					
6.70						6-	-96.86					
Dynamic Cone Penetration Test commenced at 6.70m depth.						7-	-95.86					
						8-	-94.86					
						9-	-93.86					
						10-	-92.86					
						11-	-91.86	20 40 60 80 100 Shear Strength (kPa) ▲ Undisturbed △ Remoulded				

natoreonarounConsulting						SOIL PROFILE AND TEST DATA							
154 Colonnade Road South, Ottawa, Ont	Geotechnical Investigation 147 Langstaff Drive Ottawa, Ontario												
DATUM Ground surface elevations		FILE NO. PG4918											
REMARKS BORINGS BY CME 55 Power Auger				TE 2	2019 July	12		HOLE NO. BH 7-19					
v	ЦО		SAN	MPLE				Pen. Re	esist. Blows/0.3m				
SOIL DESCRIPTION	TA PL	G				(m)	(m)	• 50) mm Dia. Cone	neter uction			
GROUND SUBFACE	STRA	ITYPI	NUMB		N VAI or R			0 W	Ater Content %	Piezon Constr			
						11-	-91.86						
						12-	-00.86		>				
						12	30.00						
						13-	-89.86						
						14-	-88.86						
						15-	-87.86						
						16-	-86.86						
						17-	-85.86			-			
						18-	-84.86						
						10							
						19-	-83.86						
						20-	-82.86						
						21-	-81.86						
						22-	-80.86	20 Shea ▲ Undistu	40 60 80 1 I r Strength (kPa) urbed △ Remoulded	↓ 00			

natoreonarounConsulting					SOIL PROFILE AND TEST DATA							
154 Colonnade Road South, Ottawa, Ont	Geotechnical Investigation 147 Langstaff Drive Ottawa, Ontario											
DATUM Ground surface elevations	prov	ided b	oy Rob	oinson l	Land	Develop	oment.		FILE NO	PG4918		
REMARKS									HOLE N	^{0.} BH 7-10		
BORINGS BY CME 55 Power Auger					TE 2	2019 July	12		· · • •	BH 7-19		
SOIL DESCRIPTION	A PLOT		SAM		ËО	DEPTH (m)	H ELEV. (m)	Pen. R ● 5	ows/0.3m a. Cone	eter ction		
	STRAT	ТҮРЕ	NUMBEI	ECOVE!	N VALU or RQI			• V	Vater Co	ntent %	onstrue	
GROUND SURFACE				щ		22-	-80.86	20	40 (
						23-	-79.86		2		-	
						24-	-78 86					
							10.00					
24.87		+									•	
Practical DCPT refusal at 24.87m depth												
(GWL @ 2.80m - July 18, 2019)												
								20 Shea ▲ Undist	40 0 ar Streng turbed ∠	60 80 1 th (kPa) △ Remoulded	⊣ 00	

natorsonar		Ir	Con	sultin	g	SOIL	_ PRO	FILE A	ND T	EST D	ΑΤΑ	
28 Concourse Gate, Unit 1, Ottawa, ON	Geotechnical Investigation Prop. Residential Development-Langstaff Drive Ottawa, Ontario											
DATUM TBM - Top spindle of fire h site. Assumed elevation =	ydrar 100.0	nt locai 00m.	orthw	rthwest corner of hte subject FILE NO. PG1773								
REMARKS				D	ATE	23 Oct 08	5		HOLE	NO. BH	11	
Borando Bri ome oo roworragor	H		SAN	APLE				Pen. Resist. Blows/0.3				. c
SOIL DESCRIPTION	DIG V		~	ĸ	E o	DEPTH (m)	ELEV. (m)	• 5	0 mm	Dia. Con	е	meter
	TRAT?	TYPE	TUMBER	% COVEF	VALU NE ROD			0 1	Vater C	ontent	%	Piezo Consti
GROUND SURFACE		×	4	RE	z	0	97.20	20	40	60 8	0	ब्दा कि
25mm Topsoil over compact, brown SILTY SAND 0.60		AU	1									
		ss	2	92	13	1-	96.20					
		ss	3	100	8	2-	95 20					
		n ss	4	100	4							
		1) 17				3-	94.20					
Very stiff to stiff, brown SILTY CLAY, trace to some		ss 	5	100	4							
		ss	6	100	4	4	93.20					
- grey by 4.4m depm						5	92.20			-		
		ss	7	100	2							V
						6	91.20					
						7	00.20	4		/	À	
				,			90.20	7		.		
0.00		ss	8	100	3	8	-89.20					
		i) ss	9	75	2							
SAND, trace clay		.[] ·[] ee	10	100	3	9-	- 88.20					
9.75 End of Borehole	i i i i i	1										
(GWL @ 5.75m-Nov. 3/08)												
								20 She ▲ Undis	40 ar Stre turbed	60 8 ngth (kP △ Remou	i0 10 a) Ilded	00
natorsonar		In	Con	sultin	q	SOIL	PRO	FILE A		ST DA	ТА	
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28 Concourse Gate, Unit 1, Ottawa, ON	K2E	717	Eng	ineers	Ge Pr Of	otechnic op. Resid tawa, Oi	cal Inves dential D ntario	tigation evelopme	ent-Lang	staff Drive	9	
DATUM TBM - Top spindle of fire h site. Assumed elevation =	ydran 100.0	t locat)0m.	ted in	the no	rthwe	hwest corner of hte subject FILE NO. PG1773					73	
BORINGS BY CME 55 Power Auger				D	ATE	23 Oct 08			HOLE NO. BH 2			
bolinee brioine our of one ringer	E		SAN	IPLE				Pen. R	esist. B	lows/0.3m	Le	
SOIL DESCRIPTION	DIG V		~	8	64 0	DEPTH (m)	ELEV. (m)	• 5	0 mm Di	a. Cone	mete	
	TRAT?	TYPE	UMBEF	COVEF	VALU NE ROD			0 V	Vater Co	ntent %	Piezo Consti	
GROUND SURFACE		×	4	R	z O	0-	97.30	20	40	60 80	ा जला उठा	
38mm Topsoil over compact, brown SILTY SAND with clay 0.60		AU	1									
<u> </u>		ss	2	71	13	1-	96.30					
		N and	2		0							
		A ss	3	83	8	2-	-95.30					
		ss	4	100	7							
		ss	5	100	4	3-	3+94.30					
					124							
		ss	6	100	4	4-	-93.30					
SILTY CLAY, some sand		ss	7	100	2	5-	-92 30					
- grey by 5.2m depth			0	100	2							
		Ass	0	100	2	6+91.30	91.30					
		ss	9	100	3							
		ss	10	100	3	7-	-90.30					
		約 初										
		ss	11	100	2	8-	- 89.30					
		ss	12	100	2							
9.45						9-	-88.30					
Very loose, grey SANDY 9.75 SILT , trace clay		ss	13	100	3	1000						
Dynamic Cone Penetration Test commenced @ 9.75m						10-	-87.30					
Inferred SANDY SILT							06 20					
						11-	-80.30	20 Shea	40 (ar Streng	60 80 9th (kPa)	100	
								▲ Undist	urbed 🛆	Remoulded		

natersonar		ir	Con	sulting		SOIL	PRO	FILE A	ND TEST DATA	1		
28 Concourse Gate, Unit 1, Ottawa, Of	K2E	717	Eng	ineers	Geotechnical Investigation Prop. Residential Development-Langstaff Drive							
DATUM TBM - Top spindle of fire h site. Assumed elevation =	ydrani 100.0	t loca 10m.	ted in	the nor	rthwest corner of hte subject FILE NO. PG1773							
REMARKS				DA	TE	23 Oct 08			HOLE NO. BH 2			
Bonnos Bri owe ob rower Auger	E		SAN	IPLE				Pen, R	tesist. Blows/0.3m	_		
SOIL DESCRIPTION				DEPTH (m)	ELEV. (m)	• 5	60 mm Dia. Cone	neter				
	TRATA	TYPE TYPE % COVER			VALUE F ROD			0 V	• Water Content %			
GROUND SURFACE	s s s s		R	E	z ö	11-	86 30	20	40 60 80			
						12-	-85.30		2			
						13-	-84.30					
Inferred GLACIAL TILL						14-	83.30					
						15-	82.30					
						16-	-81.30			-		
						17-	-80.30			•		
18.85						18-	-79.30					
18.85m depth												
(GWL @ 4.76m-Nov. 3/08)								20				
								She:	ar Strength (kPa)			

natorsonar		Ir	Con	sulting	9	SOIL PROFILE AND TEST DATA						
28 Concourse Gate, Unit 1, Ottawa, O	G	Geotechnical Investigation Prop. Residential Development-Langstaff Drive										
DATUM TBM - Top spindle of fire site. Assumed elevation	hydrar = 100.0	nt loca 00m.	ted in	the no	rthwe	Thwest corner of hte subject FILE NO. PG1773						
REMARKS				D	ATE	23 Oct 08	2		HOLE NO	D. BH 3		
BORINGS BT CIVIL 33 FOWER Auger	ы		SAN		AIE		,	Pen, R	l lesist. Bi	lows/0.3m		
SOIL DESCRIPTION	A PLO		с, с,	КХ	Ro	DEPTH (m)	ELEV. (m)	• 5	a. Cone			
	STRAT	TYPE	TUMBEI	SCOVEI	VALU			• Water Content %				
GROUND SURFACE	5	ž	4	R	zv	- 0-	95.62	20	40 €	50 80		
			1							· · · · · · · · · · · · · · · · · · ·		
Compact, brown SANDY SILTY, trace clay		ss	2	100	12	1-	94.62				-	
			2	100	0							
				100	9	2-	93.62				-	
		ss	4	100	5							
		ss	5	100	5	3-	- 92.62					
							04.00					
toring attraction to and at		ss	6	100	4	4-	91.62					
Very stiff to stiff, brown SILTY CLAY		ss	7	100	5	5-	90.62				-	
			0	100	Λ							
- firm and grey by 5.9m depth			0	100	4	6	89.62				-	
		ss	9	100	2						¥	
		ss	10	100	2	7-	88.62					
		a ss	11	100	2	8	- 87.62					
Gley CLATET SILT		ss	12	100	2			· · · · · · · · · · · · · · · · · · ·				
			13	100	2	9-	+86.62					
9.7 End of Borehole	5	1			2							
(GWL @ 6.31m-Nov. 3/08)												
									40		100	
								Shea	ar Streng	th (kPa) Remoulded		
	-		I			1	1	1				

natersonar		Ir	Con	sultin	g	SOIL	- PRO	FILE A	ND TE	EST D	ΑΤΑ	
28 Concourse Gate, Unit 1, Ottawa, O	N K2E	717	Eng	ineers	G	Geotechnical Investigation Prop. Residential Development-Langstaff Drive Ottawa, Ontario						
DATUM TBM - Top spindle of fire I site. Assumed elevation =	nydran • 100.0	it local 00m.	ted in	the no	rthwest corner of hte subject FILE NO. PG1773							
REMARKS BORINGS BY CME 55 Power Auger				D	ATE	23 Oct 08	1		HOLE N	^{io.} BH	4	
	FO		SAN	IPLE		DEDTH	EL EV	Pen. R	tesist. E	lows/0.	.3m	5
SOIL DESCRIPTION	IA PL	61	R	AN EG		(m)	(m)	• 5	50 mm D	ia. Cone	9	comete
	STRA	ITYPI	NUMBI	SECOVI	N VAI or R			0	Vater Content %			Con
GROUND SURFACE TOPSOIL 0.2	5	au 8	1	-		0-	95.74					
		\ ₹7										
		ss	2	100	11	1-	94.74					
		ss	3	100	10	2-	93.74					
		ss	4	100	6				· · · · · · · · · · · · · · · · · · ·	· · · · · · · · · · · · · · · · · · ·	•••••	
Very stiff to stiff, brown SILTY CLAY with sand						3-	92.74					
		ss	5	100	4							
		ss	6	100	5	4-	91.74					
		ss	7	100	10	5	00.74					
				100			50.74					
<u>5.9</u>	4	ss	8	100	6	6	89.74					
		ss	9	100	31							
		ss	10	25	36	7-	88.74					¥
Dense to compact, grey			11	25	26					• • • • • • • • • • • • • •		
		N 22		25	20	8-	-87.74					
		ss	12	33	25	9-	86 74	· · · · · · · · · · · · · · · ·				
0.7	-	ss	13	42	22		00.14					
End of Borehole		ť										
(GWL @ 6.95m-Nov. 3/08)												
								20 Sho	40 ar Stree	60 80 ath (kP	0 1(a)	00
								▲ Undis	turbed 4		lded	

natorsonar		10	Con	sultin	g	SOIL	. PRO	FILE A	ND TE	EST	ATA	
28 Concourse Gate, Unit 1, Ottawa, ON	K2E	717	Eng	ineers	Ge	Geotechnical Investigation Prop. Residential Development-Langstaff Drive Ottawa Ontario						
DATUM TBM - Top spindle of fire h	ydran	t locat	ed in	the no	orthwe	rthwest corner of hte subject FILE NO. PG1773						
REMARKS				_		24 0-400			HOLEN	^{io.} Bł	15	
BORINGS BY CME 55 Power Auger			SAN		ATE 2	24 Oct 08		Pen R	lesist F	Blows/0	3m	
SOIL DESCRIPTION	LOIG V		~	ž	що	DEPTH (m)	ELEV. (m)	• 5	60 mm D	ia. Con	e	meter
	TRAT	TYPE	UMBEF	COVEI	VALU E ROL			○ Water Content %			%	Piezo Const
GROUND SURFACE	s, in the second	w	z	RE	z °	0-	-98.11	20	40	60 E	80	ज्या छि
25mm Topsoil over compact, brown SILTY 0.60		AU	1								····) · [···] · ·	
		ss	2	67	15	1-	97.11					
		ss	3	58	11	2-	96.11					
		ss	4	100	5							
						3-	95.11					
		ss	5	100	5							
Very stiff to stiff, brown SILTY CLAY, trace sand		ss	6	100	6	4-	94.11					
			:			5-	-93.11		<u> </u>		1:	21
									/			
			_		-	6-	92.11					
		ss		100	5							
		ss	8	100	5	7-	-91.11					Y
7.77		ss	9	83	32	8-	-90.11					
Very dense to dense, grey		ss	10	33	77							
SILTY SAND		.Δ 				9-	89.11					
End of Borehole		SS .	11	50	41							
(GWL @ 7.05m-Nov. 5/08)			i.									
								20 She	40 ar Stren	60 a gth (kP	30 10 Pa)	00
								▲ Undis			naea	

natoreonar		In	Con	sulting	3	SOIL	PRO	FILE A	ND T	EST	DATA	
28 Concourse Gate, Unit 1, Ottawa, ON	neers	Geotechnical Investigation Prop. Residential Development-Langstaff Drive Ottawa, Ontario										
DATUM TBM - Top spindle of fire h site. Assumed elevation =	ydran 100.0	t locat)0m.	ed in	the no	rthwest corner of hte subject FILE NO. PG1773							
REMARKS				D	TE '	24 Oct 08			HOLE	^{NO.} Bl	H 6	
BORINGS BY CIVIE 55 FOWER Auger			SAN					Pen, R	lesist. I	Blows/0).3m	-
SOIL DESCRIPTION	A PLO	R R PLO				DEPTH (m)	ELEV. (m)	• 50 mm Dia. Cone				ometer
	STRAT	TYPE	NUMBE	ECOVE	N VALI			0	Vater C	ontent	%	Piez
GROUND SURFACE	1.11:	×		<u></u>	4	0-	98.20	20	40	60	80	
compact, brown SILTY SAND with clay		X AU	1									
		ss	2	100	20	1-	97.20					
		ss	3	100	20							
						2-	96.20					
		ss	4	100	7	2						
						3-	-95.20				1	0
Very stiff to stiff, brown SILTY CLAY with sand		ss	5	100	4	4-	-94.20					
							00.00				1	21
		1				5	93.20					
		ss	6	100	3							
		N SS	7	100	11	6+92.2	+92.20					
					4.05							¥
		ss	8	100	6	7-	+91.20					
		The second		400								
8.0	3111	1 55	9	100	2	8-	90.20					
		ss	10	100	2							
SILT with clay		: <u> </u>] :]				9-	89.20					
9.7	5	ss	11	100	4							
End of Borehole												
(GVVL @ 0.70M-NOV. 3/08)												
								20	40	60	80 1	+ 00
								Sne ▲ Undis	sturbed		ra) Juided	

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	-	iquid 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								
Сс	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$								
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 > 4Well-graded sands have: 1 < Cc < 3 and Cu > 4Well-graded sands have: 1 < Cc < 3 and Cu > 6Sands 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)				
Сс	-	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 Topsoil Asphalt Peat Sand Silty Sand Fill Δ Sandy Silt Clay Silty Clay Clayey Silty Sand Glacial Till Shale Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION







Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 7283

Report Date: 31-Oct-2008 Order Date 27-Oct-2008

Client PO: 7283		Project Description	: PG1773			
	Client ID:	BH6 SS3	-	-	-	
	Sample Date:	27-Oct-08	-	-	-	
	Sample ID:	0844027-01	~	-	-	
	MDL/Units	Soil	-	-	-	
Physical Characteristics						
% Solids	0.1 % by Wt.	77.3	-		-	
General Inorganics			*			
pН	0.05 pH Units	7.42	-	-	-	
Resistivity	0.10 Ohm.m	39.8	-	-	-	
Anions						
Chloride	5 ug/g dry	12	-	-	-	
Sulphate	5 ug/g dry	27	-	-	-	

1-800-769-1947 Paracel@Paracellabs.com WHW. PARACELLABS.COM

OTTANA 260-2515 St. Laonest Blyd Ottaat, Ob. 416-418

NISSISSAUGA 6645 Kolmat Pol. Unit #27 Missistation, DN 1,0N 673 NEAGARA FALLS 5415 Moning Gory Cri Niogary Parls, Chi U23 045

SARNIA 173 Chestera St. N. Barna, Gle N71 217

Page 3 of 7

Order #: 0844027

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 23 - SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG4918-1 - TEST HOLE LOCATION PLAN

DRAWING PG4918-2 - PERMISSIBLE GRADE RAISE PLAN



FIGURE 1

KEY PLAN

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