GEOTECHNICAL INVESTIGATION
Proposed Verdon Subdivision
Part of Lot 16, Concession I (Old Survey)
Geographic Township of Clarence
Now City of Clarence-Rockland

Prepared for

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1 INTRODUCTION

Levac Robichaud Leclerc Associates Ltd. (LRL) was retained by Mr. Denis Verdon to perform a geotechnical investigation for the proposed Verdon Subdivision. The property consists of a 10 hectare parcel of land at the northeast boundary of the Village of Clarence Point. The property is located north of Old Highway 17 and is bordered to the south by Wilson Road. Preliminary site development plan shows 26 single-family residential lots (including the existing dwelling) with approximate areas of 3000 m^2 to 5000 m^2. Each lot will be serviced by a private well and septic system. As per the hydrogeological assessment and terrain analysis by LRL, standard or fully raised Class IV septic systems may be installed depending on the elevation of the water table. It is proposed to extend Adrien Street into the proposed subdivision.

This project was initiated in 1990 for Mr. Marcel Noel (formerly the Noël Subdivision). A hydrogeological study was completed by Jacques Whitford Ltd. (JWL) in 1990. A hydrogeological study and terrain analysis was also performed by LRL, Dated April 2006 and addendum reports dated February 1, 2007 (File No. N-9001).

In general, the geotechnical investigation was to provide the following;

- Establish the geotechnical and groundwater conditions within the proposed subdivision;
- Make recommendations regarding the most suitable type of foundations, founding depth, the allowable bearing pressure of the founding stratum and other geotechnical data relating to the design of the foundations;
- Discuss excavation conditions during the construction;
- Comment on backfilling requirements and the suitability of the on-site soils for backfilling purposes;
- Recommend a pavement structure for the new street; and
- Conduct a slope stability assessment.

This report has been prepared in consideration of the terms and conditions noted above and with the assumption that the design of the project will satisfy any applicable codes and standards. Should there be any changes in the design features, which may relate to the geotechnical considerations, Levac Robichaud Leclerc Associates Ltd. should be advised in order to review the report recommendations.

2 SITE DESCRIPTION

The subject site is legally described as Part of Lot 16 Concession 1 (Old Survey) being Parts 1 and 2 of Registered Plan 50R7140 within the geographic Township of Clarence, now the City of Clarence-Rockland. The site is fronted by Wilson Road to the north and is located approximately 1 km north of the intersection of Old Highway 17 and Wilson Road. The location of the proposed subdivision is shown on the site plan included in **Appendix A**.

The property has an irregular rectangular shape; being approximately 380m wide (east-west) by 370m deep at the west property line and 200m deep on the east properly line, for an area of 10 hectares (26 acres). The property is mostly undeveloped and forested, except for the northeast corner of the property where a house exists.

The property is located on the north limits of a bedrock ridge overlooking the Ottawa River. Two (2) ridges are present, one running east-west in the north portion of the site and one running north-south close to the eastern boundary of the site. The elevations of the bedrock ridges range between 67m and 68m above mean sea level (amsl). From these ridges the land falls gently to the south towards a creek (63m amsl), and more steeply to the north and east towards Wilson Road (52m amsl) and the continuation of the creek (55m amsl).

The terrain gradients on the property generally range between 1% and 17%. However, the slopes are much steeper near the creek in the east of the site. North of the site, the terrain slopes towards the Ottawa River (45m amsl). Along Wilson road, bedrock can be seen at the surface with rock cliffs bordering the road. Dwellings are present at the bottom of the cliff on the north side of Wilson Road. The topography of the site is shown on the test pit and test well location plan, drawing N-9001-G2 included in **Appendix B**.

3 FIELDWORK

As stated above, a hydrogeological study was performed on this property 1990 by JWL. As part of their study, ten (10) test pits (TP90-1 to TP90-10) were dug across the property in areas that were accessible by a backhoe. The soil conditions obtained from these test pits were reviewed and included as part of this current study. Nevertheless, LRL conducted its own test-pitting program to confirm the surficial geology underlying the property. The surficial overburden soil conditions were confirmed by digging nine (9) test pits across the site. The test pits were dug in January 2006 using a backhoe from a local contractor and under the supervision of LRL technical staff. In addition, piezometers were installed in order to determine the groundwater elevation. The water levels were measured on January 24, 2006 using a water meter. Please refer to drawing N-9001-G2 in **Appendix B** for the approximate location of all test pits performed by LRL as part of this investigation and performed by JWL. The test pit logs are included in **Appendix C**.

Every soil types and interfaces were described, measured and logged. The undrained shear strength of the cohesive soil was determined using a calibrated pocket penetrometer and calibrated Geonor M-3 inspection vane. All soil samples were visually examined, described, logged and stored before being transported to our office for further examination by our geotechnical engineer.

All samples collected during this project will be kept in storage for a period of six (6) months at which time, they shall be destroyed unless a written or verbal notice is received, stating otherwise.

All test pits were surveyed and located using a Garmin Etrex Legend Global Positioning System receiver using NAD 83 datum (accuracy ±10 m). The elevations were referenced to a geodetic benchmark: spike on hydro pole located on the south side of Old Highway 17, east of the intersection of Sophie Street (Elevation 86.675m amsl).

4 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

4.1 Geological Mapping

The general geology of the site was established by reviewing the following sources:

1. *Generalized Bedrock Geology Ottawa – Hull, Ontario and Quebec*, Map 1508A, Scale 1:125 000, Geological Survey of Canada, 1979;
2. *Surficial Material and Terrain Features Ottawa – Hull, Ontario – Quebec*, Map 1425A, Scale 1:125 000, 1977;
3. *Urban Geology of the National Capital*, published by Natural Resources Canada (2001); and
4. Local well records obtained from the MOE's Water Well Record Department.

The surficial geology underlying most of the property would be composed of Paleozoic Bedrock outcrops at the north and plain till to the south. To the southwest would be erosional terraces, which consist of reworked silty clay. The thickness of the till would range between 0m to 5m, becoming thicker to the southeast. The bedrock formation found across the site belongs to the Rockcliffe Formation and would be composed of shale with numerous interbeds of limestone and sandstone.

4.2 Soil Stratigraphy

Table 1 provides a summary of the soils encountered at each test pit location performed as part of the Jacques Whitford Ltd Hydrogeological Study dated 1990. **Table 2** provides a summary of the soils encountered at each test pit location performed by LRL as part of this investigation including the depth of each soil interface. For more details, please refer to the attached test pit logs presented in **Appendix C**.

Table1: Soil Depth Summary for test pits performed by Jacques Whitford Ltd

Soil Stratum	TP 90-1	TP 90-2	TP 90-3	TP 90-4	TP 90-5
Topsoil	0.0 – 0.2	0.0 – 0.3	0.0 – 0.2	0.0 – 0.4	0.0 – 0.2
Sand	NE	NE	0.2 – 2.1	0.4 – 1.6	NE
Till	0.2 – 0.5	0.3 – 2.5	NE	NE	0.2 – 1.5
Weathered bedrock	0.5 – 0.9	NE	NE	1.6 – 1.9	1.5 – 1.9
Inferred bedrock	0.9	2.5	2.1	1.9	1.9
Water seepage	0.8	2.4	NE	1.1	1.5
Soil Stratum	**TP 90-6**	**TP 90-7**	**TP 90-8**	**TP 90-9**	**TP 90-10**
Topsoil	0.0 – 0.3	0.0 – 0.3	0.0 – 0.2	0.0 – 0.2	0.0 – 0.1
Sand	NE	NE	NE	NE	0.1 – 1.4
Silt	NE	NE	NE	0.2 – 0.9	NE
Clay	NE	NE	NE	NE	1.4 – 3.1
Till	0.3 – 0.7	0.3 – 1.7	0.2 – 2.2	0.9 – 2.0	NE
Inferred Bedrock	0.7	1.7	NE	2.0	3.1
Water seepage	0.3	NE	NE	NE	1.6

NE: Not encountered NM: Not Measured

Table 2: Soil Depth Summary for test pits performed by LRL

Soil Stratum	TP-1	TP-2	TP-3	TP-4	TP-5
Ground surface Elev.	57.20	67.22	66.49	67.06	67.45
Topsoil	0.00 – 0.25	0.00 – 0.22	0.00 – 0.15	0.00 – 0.25	0.00 – 0.22
Sand	NE	0.22 – 0.50	0.15 – 2.65	0.25 – 0.60	0.22 – 0.40
Clay	0.25 – 2.15	NE	NE	NE	NE
Till	NE	0.50 – 2.15	2.65 – 2.80	0.60 – 2.15	0.40 – 1.20
Weathered bedrock	NE	NE	NE	NE	1.20 – 1.80
Inferred bedrock (depth/Elev)	NE	2.15 / 65.07	NE	NE	1.80 / 65.65
End of test pit (depth/Elev)	2.15 / 55.05	2.15 / 65.07	2.80 / 63.69	2.15 / 64.91	1.80 / 65.65
Groundwater depth/Elev (Jan 2006)	2.01 / 55.19	NE	0.31 / 66.18	0.83 / 66.23	0.91 / 66.54

Soil Stratum	TP-6	TP-7	TP-8	TP-9
Ground surface Elev.	69.08	67.05	65.13	65.22
Topsoil	0.00 – 0.29	0.00 – 0.20	0.00 – 0.20	0.00 – 0.20
Sand	0.29 – 0.80	0.20 – 0.60	0.20 – 1.40	0.20 – 0.75
Till	0.80 – 2.20	0.60 – 2.20	NE	0.75 – 1.90
Inferred bedrock (depth/Elev)	NE	NE	1.40 / 63.73	1.90 / 63.32
End of test pit (depth/Elev)	2.20 / 66.88	2.20 / 64.85	1.40 / 63.73	1.90 / 63.32
Groundwater depth/Elev (Jan 2006)	1.52 / 67.56	0.84 / 66.21	0.52 / 64.61	1.37 / 63.85

NE: Not encountered NM: Not Measured

4.2.1 Topsoil

A thin layer of topsoil (100mm to 300mm) was encountered in all test pits performed across the property. The topsoil is described as a dark brown and silty.

4.2.2 Sand

A sand deposit was encountered in all test pits except TP-1, TP 90-1, TP 90-2, TP 90-5, TP 90-7, TP 90-8 and TP 90-9. The sand is generally very fine grained to silty, brown in colour, moist and compact. In TP-3, the sand becomes medium grained with depth.

The sand deposit is generally thinner than 1.25m, except in the southeast corner of the property where the deposit was measured to be up to 2.50m thick. The sand was found resting over till in all test pits, except in TP-3 and TP-8, where it was resting over bedrock and in TP 90-10 where it was resting over clay. Where the sand deposit is thicker, the sand becomes saturated with depth.

A grain size analysis was performed by JWEL on a sand sample (TP 90-4 at 0.8m). The sand sample consisted of 78% sand with the remainder being silt and clay. The laboratory report is presented in **Appendix D**.

4.2.3 Silt

A layer of silt was encountered in TP 90-9 only, which is located near the southeast corner of the property. It is described as clayey silt with traces of sand and is brown in colour. The layer was measured to be 0.70m thick and was found resting over till.

A grain size analysis was performed by JWEL on a representative silt sample (TP90-09 at 0.5 m bgs). The analysis showed that the soil is composed of 5% sand with the remainder being silt and clay. The laboratory report is presented in **Appendix D**.

4.2.4 Marine Clay Deposit

A marine clay deposit was encountered in TP 90-10 and TP-1 only, which were performed in the northeast corner of the property. The clay is silty, grey with reddish bands in colour; it has a blocky texture and is very stiff in consistency (Cu > 120 kPa). It is noted that some gravel and boulders were observed in TP 90-10. It is anticipated that the layer found in this test pit would consist of a transition zone between the clay and till deposits.. The thickness of the clay deposit was measured to be 1.7m in TP 90-10 and more than 1.9m in TP-1, which was terminated within this soil stratum.

4.2.5 Glacial Till Deposit

A glacial till deposit was encountered in all test pits except TP 90-10, TP-1 and TP-8, which were performed near the northeast corner of the property. The till is generally described as a mixture of sand, silt and gravel with presence of cobbles and boulders (600mm in diameter). It is light brown to greyish brown in colour and dense. It shall be noted that a greater presence of boulders was observed in TP-6 and TP-9.

The thickness of the till deposits is generally less than 1.5m thick, except in TP 90-8, TP-2, TP-3, TP-6 and TP-7, which were terminated within this soil stratum and over boulders at depths of 2.15m to 2.80m below ground surface.

A grain size analysis was performed by JWEL on a representative till sample (TP 90-9 at 0.8 m bgs). The analysis showed that the till is composed of 23% gravel, 47% sand and the remainder being silt and clay. The laboratory report is presented in **Appendix D**.

4.2.6 Bedrock

Bedrock was encountered in most test pit except TP-90-8, TP-1, TP-3, TP-4, TP-6 and TP-7. The bedrock was encountered at depths of 0.9m to 3.1m. As mentioned above, the bedrock formation found across the site would belong to the Rockcliffe Formation which is composed of shale with numerous interbeds of limestone and sandstone. In TP90-1 and TP-5, the bedrock was observed to be weathered (0.4 to 0.6m in thickness).

4.2.7 Groundwater Conditions

The shallow overburden groundwater table found across the site was characterised through piezometers installed during the backfilling of the test pits. The groundwater level was measured on January 24, 2006. In the 1990 Jaques Whitford Hydrogeological Study, groundwater seepage was noted in the test pits.

The test pits revealed the presence of a shallow overburden groundwater table found within the sand and till layer. The groundwater was encountered between 0.31 and 2.01m bgs. In general, the water table was found within the first 1.5m within the till deposit. In TP 90-4, TP-3 and TP-8, the water table was found within the sand deposit. This water table is considered a perched water table flowing above the more impervious bedrock. In TP-1, groundwater was encountered within the upper fractured layer of the clay, flowing over a more massive and impervious clay layer. No groundwater seepage were observed in TP 90-3, TP-90 -7, TP 90-8, TP-90-8 and TP-2 was found to be dry

It shall be noted that groundwater levels could fluctuate with seasonal weather conditions, (i.e.: rainfall, droughts, spring thawing).

5 GEOTECHNICAL CONSIDERATIONS AND RECOMMENDATIONS

The preliminary site development plan shows 26 single-family residential lots (including the existing dwelling) with approximate areas of 3000 m^2 to 5000 m^2. Each lot will be serviced by a private well and septic system. As per the hydrogeological assessment and terrain analysis by LRL, standard or fully raised Class IV septic systems may be installed depending on the elevation of the water table. It is proposed to extend Adrien Street into the proposed subdivision.

5.1 Foundations

After reviewing the subsurface soil conditions, it is anticipated that the subgrade soil will consist mostly of till or bedrock for the majority of the lots, clay in the northeast portion of the site and sand in the southeast portion. The footings can be founded over the native clay deposit, sand deposit, glacial till deposit set below frost depth or set entirely over sound bedrock.

It shall be noted that silt is not a suitable subgrade, therefore it shall be sub-excavated. In addition, where bedrock is encountered at the subgrade level, the footings shall rest entirely over sound bedrock or over a minimum of 300mm of native soil or structural fill. The foundations for the proposed residential dwellings may be supported by conventional strip and column footings founded on the above mentioned soil stratum. The safe net bearing value of **75 kPa** may be used for the design of strip and column footings founded over undisturbed native soil. Therefore, all topsoil and disturbed material will need to be removed from the footprint of the footings.

The underside of the footings shall be founded at least 0.3m above the high water level. This should be established when the test pit is dug for the design of the septic system.

In granular soils, where the groundwater level is located at a depth of less than the footing’s width below the subgrade, the footings shall be designed as per the most recent Ontario Building Code. This would likely affect the width of the footings (perimeter footings and column pads).

The given allowable bearing capacity is based on concrete continuous footings being not less than 0.6m wide or more than 1.5m wide and on reinforced concrete bases not exceeding 3.0m along any sides. The given bearing capacity assures a factor of safety of 3 against shear failure and limits differential and total settlements within admissible values

Where excavation below the underside of the footing is performed, consideration shall be given to support the footings on structural fill. The structural fill must extend 0.6m beyond the outside edge of the footings and a distance equal to the depth of the structural fill set below the footing. The recommended material to be used as structural fill to support the footings shall consist of Granular B Type II crushed stone, or an approved equivalent material.

The structural fill shall be placed over undisturbed native soils in layers not exceeding 300mm and compacted to 98 percent of its Standard Proctor Maximum Dry Density (SPMDD) as per ASTM D-698. Prior to placing any structural fill or to pouring the footings, it is required that any disturbed soils along the base of the footing be removed and that the subgrade soils be inspected and approved by the geotechnical engineer. Furthermore, the structural fill must be tested to ensure that the specified compaction level was achieved.

Outside the loading influence of a building structure, the fill could consist of any compactable material free of organic matter or deleterious material (i.e. clay) set in layers not exceeding 600 mm and compacted at least 90 percent of its Standard Proctor Maximum Dry Density (SPMDD) as per ASTM D-698.

5.2 Settlement

The estimated total settlement of the foundations, designed using the recommended bearing value given herein is less than 25mm. The differential settlement between adjacent column footings is anticipated to be 15mm or less. The estimated foundation settlement is considered to be within tolerable and acceptable limits for masonry construction.

5.3 Slab-on-grade Construction

Slab-on-grade construction will be acceptable over undisturbed native soils only. Therefore, all organic soil or disturbed soils shall be removed from the footprint of the dwelling. Any underfloor fill needed to raise the general floor grade shall consist of Granular B – Type I compacted to 95 percent of its SPMDD. The final lift shall be compacted to 98 percent of its SPMDD. A 200mm layer of Granular A material shall be placed under the slab and compacted to at least 98% of the SPMDD.

In order to further minimize and control cracking, the floor slab shall be provided with wire mesh reinforcement and construction or control joints. The construction or control joints should be spaced equal distance in both direction and where possible not exceeding a distance of 4.5m. The wire mesh reinforcement shall be carried through the joints.

5.4 Frost Protection

Exterior footings and any footings over the native clay, sand or till deposits, located in unheated portions of the building shall be protected against frost heaving by providing a minimum of 1.5m of earth cover under snow covered surface or 1.7m under exposed surfaces (i.e. sidewalks, paved areas, etc.), or its equivalent in insulation protection.

LRL shall review the detail design of frost protection with the use of equivalent insulation prior to construction. Where the subgrade will consist entirely of bedrock, frost protection shall not be required.

In the event that foundations are to be constructed during winter months, foundation soils are required to be protected from freezing temperatures using suitable construction techniques. Therefore, the base of all excavations should be insulated from freezing temperature immediately upon exposure, until the time that heat can be supplied to the building interior and footings have sufficient soil cover to prevent freezing of the subgrade soils.

5.5 Permanent Drainage

Any dwelling that will contain a basement will require permanent perimeter drainage. The drainage pipe shall be embedded in a 300mm layer of 20mm diameter clear crushed stone wrapped in a geotextile and set adjacent to the perimeter footings. The drainage pipe should be connected positively to a suitable outlet such as a sump pit or storm sewer.

In order to prevent the ponding of water adjacent to the foundation walls, the roof water should be controlled by a roof drainage system and the exterior grade should be sloped to shed water away from the walls.

6 Excavation and Backfilling Requirements

6.1 Excavation and Groundwater Control

It is anticipated that the excavations for the foundation will not extend below 1.5m bgs. The overburden soils encountered at this site consists of topsoil, sand, silt, clay and till resting over bedrock. Due to the presence of a water table in these soil units and according to the Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 527/00, the surficial overburden soil encountered at this site can be classified as Type 3. Therefore, shallow temporary excavation in the overburden soil classified as Type 3 shall be sloped at 1 horizontal to 1 vertical starting at the base of the excavations and as per requirements of the OHSA regulations.

The listed slopes are for fully drained conditions. Much gentler slopes could be required under undrained conditions, where local water infiltrations occur and where the excavations are exposed for prolonged period of time.

If the aforementioned slopes are not possible or practical to achieve due to space restrictions or obstacles, the excavation shall be shored according to OHSA Reg. 213/91. A geotechnical engineer shall design and approve the shoring and establish the shoring depth under the excavation profile.

6.2 Groundwater Control

It is expected that any surface groundwater seepage or infiltration entering the excavations can be controlled with an effective sump and pump system. Surface water runoff into the excavation should be avoided and diverted away from the excavation.

6.3 Foundation walls backfill

Backfill materials against shallow foundation walls should consist of free draining, non-frost-susceptible granular material (i.e., Granular "C", clean sand) compacted to 90 percent of its SPMDD using light compaction equipment. The compaction should be increased to 95 percent under walkways or paved areas close to the foundation wall. Site grading should be sloped away from the building area. Where specified, backfilling against foundation walls should be carried out on both sides of the wall at the same time.

6.4 Suitability of On-site Soils

The existing overburden soils consist of topsoil, sand, silt, clay and till. The silt, clay, till and sand (as it contains more than 20 % of silt) are frost susceptible and their use is not recommended for backfilling purposes against foundation walls. However, they could be reused as general backfill material (general landscaping/backfilling), if they can be compacted according to the specifications outlined herein at the time of construction. Any imported material should conform to OPSS Granular B - Type I.

It shall be noted that the adequacy of a material for reuse as backfill will depend on its water content at the time of its use and on the weather conditions prevailing prior and during that time. Therefore, all excavated materials to be reused shall be stockpiled in a manner that will prevent any significant changes in their moisture content, especially during wet conditions.

7 PAVEMENT STRUCTURE DESIGN

It anticipated that the subgrade soils will consist mostly of sand or till. The representative soil modulus of the subgrade soils is 31 MPa (4 500 psi). The Granular Base Equivalency (GBE) thickness was calculated at 450mm.

The following **Table 3** presents the recommended pavement structure to be constructed over a stable subgrade for the proposed streets.

Table 3: Recommended Pavement Structure

Course	Material	Thickness (mm)
GBE		500
Surface	HL3 A/C	40
Binder	HL8 A/C	40
Base course	Granular "A"	150
Sub base	Granular "B" Type II	300
Total:		530

The base and sub base granular materials shall conform to OPSS Form 1010 material specifications. The sub base material shall be free draining and not prone to capillary uprising. They shall be tested and approved by a geotechnical engineer prior to delivery to the site and shall be compacted to at least 100% SPMDD.

Asphalt concrete shall conform to OPSS Form 1150 and be placed and compacted to at least 97% of the Marshall Density. The mix and its constituents shall be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

7.1 Subgrade Preparation

The surficial soils should be stripped of topsoil, silt and other obvious objectionable material. Following the backfilling and satisfactory compaction of any underground service trenches up to the subgrade level, the subgrade shall be shaped, crowned and proof-rolled with any resulting soft areas sub excavated down to an adequate bearing layer and replaced with approved backfill. Any subgrade fill needed should be placed in small lifts and compacted to 95 percent of SPMDD. The frost taper should be 3 horizontal for 1 vertical and constructed using Granular B – Type I material.

The preparation of subgrade shall be scheduled and carried out in manner so that a protective cover of overlying granular material is placed as quickly as possible in order to avoid unnecessary circulation by heavy equipment, except on unexcavated or protected surfaces. Frost protection of the surface shall be implemented if works are carried out during the winter months.

8 SLOPE STABILITY ASSESSMENT

8.1 Slope Description

The property is located on the north limits of a bedrock ridge overlooking the Ottawa River. Two (2) ridges are present, one running east-west in the north portion of the site and one running north-south close to the eastern boundary of the site. The elevations of the bedrock ridges range between 67m and 68m above mean sea level (amsl). From these ridges the land falls gently to the south towards a creek (63m amsl), and more steeply to the north and east towards Wilson Road (52m amsl) and the continuation of the creek (55m amsl).

The terrain gradients on the property generally range between 1% and 17%. However, the slopes are much steeper near the creek in the east of the site. North of the site, the terrain slopes towards the Ottawa River (45m amsl). Along Wilson road, bedrock can be seen at the surface with rock cliffs bordering the road. Dwellings are present at the bottom of the cliff on the north side of Wilson Road. The topography of the site is shown on the test pit and test well location plan, drawing N-9001-G2 included in **Appendix B**.

Two representative slope profiles, representing the most critical conditions were analyzed as part of the assessment. They are located in the east, representing the slope leading to the creek, and west of the site representing the slope leading to the rock cliffs to the north of the site.

The upper portion of the west slope averages 3 % for a horizontal distance of 80m. The remaining section of the slope averages 18 % over a horizontal distance of 50m down to rock cliffs near Wilson Road. The cliff is approximately 2.5m high on the south side of Wilson road and 6m to 8m high to the north. It is our understanding that the cliff on the south side of Wilson Road have been created at the time of the construction Wilson Road.. The slope has an approximate horizontal length of 155m for a difference of elevation of 24m. The overburden soils consist of a thin layer of till (up to 2.5m in the upper portion and less then 0.9m for the remaining sections) resting over shallow bedrock.

The east slope, located at the back of lot 14 and bordered by the creek, averages 180 % for a difference of elevation of 5.5m and a horizontal distance of 3m. Manual probing on the slope revealed that the overburden soils consist of sand at the top of the slope and clay at the bottom of the slope. TP 90-10 located in this region reached bedrock at a depth of 3.1m below ground surface.

The slopes were visually inspected on May 23, 2008. The terrain, vegetation and trees in the east slope leading to the creek, at the back of lot 14, showed sings of previous instabilities. Downstream of this area it was observed that the creek flows over bedrock. Signs of erosion were observed along the banks of the creek.

8.2 Assessment

The Software Slide 5.0, by Geoscience, was used to implement the modified Bishop simplified method of slices. Analyses were performed of the above mentioned critical cross sections along the entire property.

The analysis used the Bishop simplified method of slices with both undrained parameters (short term failure, fully fissured clay in slope) and effective stresses parameters (long term failure). The analyses were performed using conservative values. The slopes were analyzed under full saturation.

The analyses conducted on the west slope indicate that the factor of safety against long term failure is above **3.3**. A minimum value of **1.5** is considered acceptable.

However the analysis conducted on the slope leading the creek in the east of the property, located at the back of lot 14, indicated that the safety factor against a short term and long term stability are lower than **0.1.** Therefore, this slope is considered unstable. In order to obtain a minimum factor of safety above the minimum requirement of 1.5, no construction or building structures including septic systems and pools shall be erected within a 10m setback without consulting with a geotechnical engineer. Typical computer results are shown in **Appendix E**.

8.3 Recommendations

Based on the slope stability assessment, the following recommendations are given to maintain the existing slope stability:

- No construction or building structures including septic systems and pools shall be erected within the 10m setback given for the east slope bordered by the creek above without consulting with a geotechnical engineer.
- The slopes shall be maintained in their existing condition; meaning that no alteration to the slope's geometry or vegetation cover shall be made. If anything, vegetation growth shall be encouraged within the slopes.
- Any proposed surficial drainage from the subdivision shall be diverted away from the steep slopes defined herein.
- If required, any drainage works (i.e. storm water outlet) shall be located in the gentler slopes defined herein. Furthermore, they shall be extended sufficiently not to drain directly within the slope. Their outlets shall be protected with riprap set over a geotextile in order to avoid erosion in the slopes.
- Once the final development plans have been prepared for this subdivision, they shall be reviewed by a geotechnical engineer in relation to slope stability concerns and to ensure that the recommendations given herein are met.
- An erosion control plan shall be prepared and reviewed by a geotechnical engineer to ensure that no potential erosion occurs to the slopes during the construction and development of this subdivision.
- Site specific slope stability review shall be conducted on the final lot grading and servicing plan for each lot fronting the east slope (lots 13, 14 and Lot 15) to ensure that the recommendations given herein are met.
- Finally, a geotechnical engineer shall review and inspect any works to be performed in or near the crest of the slopes.

A preliminary site development plan has been prepared for the subdivision as part of the LRL hydrogeological study and terrain analysis. The development plan is presented in **Appendix F**. This is only a preliminary development plan which shall be reviewed and modified during the development stages of the project.

9 GEOTECHNICAL REVIEW PROGRAM

The use of the allowable bearing pressure contained in this report for the design of foundations is conditional on footings being constructed on undisturbed soil or suitably prepared structural fill reviewed and approved as such by this firm.

Experienced geotechnical personnel under the direct supervision of a geotechnical engineer shall inspect the subgrade conditions prior to placing any fill material or any subsurface concrete structures. The materials used as select subgrade material, structural fill or granular base shall be tested for quality characteristics and gradations and approved before they are transported on site. All fill layer set on the project shall be tested for thickness and levels of compaction and shall be approved prior to placing any subsequent layer.

As such, a geotechnical construction review program is recommended, whereby the following aspects of construction are reviewed:

a) Inspection of in-situ soil subgrade prior to backfilling.

b) Monitor and test material imported onto the site for backfill or structural fill to ensure they conform to guidelines and specifications.

c) Field density tests during the backfilling program, to ensure that the specified level of compaction has been achieved.

d) Inspection of all bearing subgrade soils prior to pouring the footings or placing structural fill in order to confirm the bearing capacity given herein.

e) A slope stability review as described in Section 8.3.

The completion of a review program of this type will result in the issuance of an engineering report confirming that these works have been completed in accordance with and in compliance with the general intent of the geotechnical recommendations.

10 Report Conditions and Limitations

The recommendations and data contained in this report are intended for design purpose only. The use of this report as a construction document is neither intended nor authorized by Levac Robichaud Leclerc Associates Ltd. Contractors and others involved in the construction of this project are advised to make an independent assessment of the subsurface soil and groundwater conditions for the purpose of establishing quantities, schedules and construction techniques.

The recommendations provided in this report are based on subsurface data obtained at the test locations. Experience indicates that the subsurface soil and groundwater conditions can vary significantly between and beyond the test locations. For this reason, the recommendations given in this report are subject to a field verification of the subsurface soil conditions at the time of construction.

The report recommendations are applicable only to the project described in the report. Any changes will require a review by Levac Robichaud Leclerc Associates Ltd., to insure compatibility with the recommendations contained in this project.

We trust that this report will meet your requirements. Should you have any questions or comments, please contact the undersigned.

Yours truly,
Levac Robichaud Leclerc Associates Ltd.

Benoît Charlebois, Junior Engineer

Marc-Antoine Laforte, P. Eng. Ph. D.

APPENDIX A

SITE LOCATION

APPENDIX B

TEST PIT LOCATION PLAN

APPENDIX C

TEST PIT LOGS

APPENDIX D

LABORATORY TEST REPORTS

APPENDIX E

SLIDE COMPUTER RESULTS SHORT TERM AND LONG TERM ANALYSIS

APPENDIX F

PRELIMINARY DEVELOPMENT PLAN