GEOTECHNICAL INVESTIGATION
BETWEEN C.N. 1402 & 1420, LACROIX ROAD, HAMMOND, ONTARIO
F18-QT-2019-029
CITY OF CLARENCE-ROCKLAND

Prepared for:

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1 INTRODUCTION

The City of Clarence-Rockland (The City) retained the services of Lascelles Engineering & Associates Ltd. (Lascelles) to conduct a geotechnical investigation on a section of Lacroix Road that was recently subjected to road embankment slope failure.

The purpose of the investigation was to identify the subsurface soil conditions within the area of the slope failure by means of a limited number of boreholes, and based on the factual information obtained, provide guidelines on the geotechnical engineering aspects of the rehabilitation of the road and slopes.

Should there be any changes in the design features, which may relate to the guidelines provided in the report, Lascelles Engineering & Associates Ltd. should be advised in order to review the report recommendations.

2 PROJECT AND SITE DESCRIPTION

During the spring thaw of 2019, City staff noticed that a small landslide had occurred in the upper portion of the road's south embankment that resulted in blocking the culvert located at the base of the ravine. The culvert is located between C.N. 1402 and 1420, Lacroix Road near the Village of Hammond, Ontario. The culvert provides flow to a natural watercourse that is a tributary to the north branch of Indian Creek. It is our understanding that the City of Clarence-Rockland is looking into reinstating the road and its embankments to a safe profile. They may also be looking at replacing the culvert with a similar size.

Lacroix Road is a rural road located on the southern portion of the City of Clarence-Rockland and subject to low traffic volumes. At this location, Lacroix Road is bordered by two residential dwellings to the south and forested lands to the north. It is noted that a regional watermain runs along this road that brings municipal water from the Town of Rockland to the Village of Hammond.

A site reconnaissance was carried out by members of our engineering staff on June 13, 2019. At the time of the site visit, the height and profiles of the slopes (road embankments) were measured.

It is noted that the location the watercourse consists of a deep ravine, that was backfilled to install a culvert, which has created a local low point for the road's drainage. At this location, Lacroix road has limited to no ditches and the surface runoff of the road flows uncontrolled towards the said low point in order to reach the watercourse flowing at the base of the deep ravine.

At the time of our investigation, the south side slope/embankment had already failed at the location fronting the culvert. The failure appears to have been caused by erosion that created a surficial slip. The soil from the failure is what blocked the flow of the existing culvert. A pump was setup at the top of the slope to pump the water from the upstream to downstream side of the culvert. The slip was in the upper portion of the slope, and also caused a slump in the road's structure as seen by cracks and depression on the road surface. It appears that attempts were made to fix the slope and the road as seen by cold patch placed on the road and rip-rap placed on the slope. It is noted that the entire slope seems to have been previously covered with a thin layer of rip-rap, which is currently in a very loose state. Mature trees and vegetation grow within the slope but not directly fronting the culvert. It would appear that this may not be the first time that erosion issues has occurred in this slope.

The total height of the slope was measured to be about 6.12m over a distance of about 8.5m with an average angle of incline of 36 degree. Consequently, the overall slope profile was established to be 1.4 Horizontal to 1 vertical (1.4H : 1V). From the crest of the slope, the profile varies down to its base with measured angles of incline of 29.8, 10.8, 32.4 and 90 degree (vertical) depending on the location in the profile.

The north slope is entirely covered with rip-rap, and is also in a very loose state. Severe erosion can be found along the slope, especially near its middle, where all the rip raps had slid to the bottom of the slope. Exposed roots of vegetation and trees were seen along the slope. Within this part of the slope, the angle of incline was almost vertical for an approximate depth of 1.5m, suggesting a previous or old surficial slip has occurred. It is noted that the north side of

the slope has more and denser vegetation than the south side. Mature trees were also found growing on both sides of the culvert.
The total height of the slope was measured to be 7.4m over a distance 10.1m, with an average angle of incline of 36 degree. The overall slope profile was established to be 1.3 Horizontal to 1 vertical (1.3H:1V).

Both of the road's embankment slopes are currently considered unstable, where the presence of deep rooted vegetation is what is likely preventing a more significant slope failure versus the surficial slopes observed.

3 PROCEDURE

The fieldwork for this investigation was carried out on June 25 and June 26, 2019. The number of boreholes to be drilled was predetermined by the City of Clarence-Rockland's RFP, while the location of the boreholes was established by Lascelles' technical staff in the field. A total of four (4) boreholes, two (2) boreholes on each side of the culvert, were drilled to characterise the occurring soils at the location of the culvert. The approximate locations of the boreholes were plotted on a Google Earth aerial photograph and are presented as part of **Figure 1**. Prior to any fieldwork, the borehole locations were cleared for the presence of any underground services and utilities. Traffic control during the drilling of the borehole was maintained in accordance with the Ministry of Transportation's Book 7.

The boreholes were advanced using a truck mounted drill rig equipped with continuous flight hollow stem augers supplied and operated by George Downing Estate Drilling Inc. A "two-man" crew experienced with geotechnical drilling operated the drill rig and equipment. The boreholes were advanced by auguring through the pavement structure and the overburden down to below the invert of the existing culvert, which was established to be approximately 7.5m below the centre line of the road, which was based on-site measurements and also confirmed by the project surveyor/consultant - Jp2g Consultants Inc.

Sampling of the overburden materials encountered in the boreholes was carried out using a 50mm diameter drive open conventional split spoon sampler in conjunction with standard penetration testing ("N" value). In addition, field vanes were conducted on the cohesive soil encountered. All soil samples collected from the split spoons were placed and sealed in plastic bags to prevent the evaporation of their moisture content. All soil samples were visually examined, described, logged and stored before being transported to our office for further examination by our geotechnical engineer.

The fieldwork was supervised throughout by a member of our engineering staff who supervised the drilling of the boreholes, coordinated the testing of the materials, cared for the samples collected and logged the subsurface conditions encountered at each location. All samples collected during this project will be kept in storage for a period of six (6) months at which time, they will be disposed of, unless a written or verbal notice is received, requesting otherwise.

Figure 1: Borehole Location

Ref: Google Earth – Image Date: June 2018

Upon completion, the boreholes were backfilled with soil cuttings brought up by the augers and compacted. The boreholes were topped with a minimum of 125mm of cold patch asphalt. Standpipes were installed in two (2) of the boreholes to measure the static groundwater level in the area. The standpipes consisted of 25mm diameter PVC piping that were slotted and placed within the overburden prior to backfilling the boreholes. The standpipes were used strictly to establish the static water level of the overburden water table.

All boreholes were located using a GPS (Global Positioning System) receiver using NAD 83 (North American Datum). Using these GPS coordinates, the borehole locations were plotted on a Google Earth aerial image as presented in **Figure 1**. The geodetic elevations of the boreholes were provided to Lascelles' technical staff at the time of the fieldwork by the project surveyor/consultant - Jp2g Consultants Inc. They also provided the elevation of the centreline of road (Elev. 73.184m) and the top of the culvert north side (Elev. 65.941). It is noted that the CSP culvert was measured on the north side to be 0.4m diameter.

4 SUBSURFACE SOIL CONDITIONS

A review of the surficial geology maps for this area suggests that the site would be underlain by Deltaic and Estuarian Deposits, which are generally composed of sand and silt. These deposits are generally found resting over marine deposits composed of silt and clay.

The subsurface conditions encountered in the boreholes were classified based on visual and tactile examination of the materials recovered from the boreholes and the results of the in-situ testing and field observations. The soil descriptions presented in this report are based on commonly accepted methods of classification and identification of soil employed in geotechnical practice. Classification and identification of soil involves judgement and Lascelles does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The subsurface soil conditions encountered at each borehole location is given in the Borehole Logs presented in **Appendix A**. These logs indicate the subsurface conditions encountered at specific test locations only. Boundaries between zones on the logs are often not distinct, but are rather transitional and have been interpreted.

4.1 Pavement Structure

Asphaltic concrete was encountered in all boreholes. The thickness of the asphaltic concrete was measured to be 25mm and rest over granular crushed stone. The granular crushed stone was measured to be between 710mm to 740mm thick. The pavement structure was found resting over sand fill.

4.2 Fill

The pavement structure was found to rest over a sand fill. The fill is described as uniform, fine grained sand with traces to some silt and presence of crushed stone in areas. The fill is brown in colour becoming greyish brown with depth. It is in a compact to loose state and dry to moist but wet generally on approaching the native sand-silt layer.

The thickness of the fill varies; being thicker near the culvert and tapering off away from the culvert location. This fill likely originated from the excavation and installation of the culvert. Furthermore, the sand fill is very similar to the deltaic deposits found within the local sand plains in Clarence-Rockland, therefore, the fill could have also originated from a nearby sand pits. The sand fill was found resting over a sand-silt layer.

4.3 Sand-Silt

A sand-silt was encountered in all boreholes and at various depths. This layer is basically a transition layer between the native sand and the underlying clay. The composition of the deposit varies with depth and generally starts as being described as silty sand becoming progressively more silty with depth and changes to a silt-sand mixture (silty sand to sandy silt) and starts to contain trace to some clay, before changing to a clayey silt. Thin beds and horizons of pure sand or silt were also found within this layer. It is greyish brown to grey in colour. The soil layer was found to be in a loose to very loose state. Finally, the deposit was found to be wet and very sensitive below the water table. This layer extends between 3.66m to 7.92m bgs and was found resting over silty clay in all boreholes.

4.4 Clay

A clay deposit was found underlying the sand-silt layer in all boreholes. It is described as silty, with traces of sand, grey in colour and with a stiff consistency (Cu. between 55 to x 62 kPa). The clay would be of high plasticity and moisture content. All boreholes were terminated within this soil layer at depths varying from 9.14m to 9.75m bgs. In this area, the clay deposit is known to be quite thick and the drift thickness would range between 25m to 50m.

4.5 Groundwater Conditions

The static water level was measured within the standpipes installed within BH-1 and BH-3, using a water meter on July 5, 2019 and results are shown on the test pit logs presented in **Appendix A**. The depth of the groundwater was found to below 4.4m bgs in both boreholes as both were found to be dry. This suggest that the slope is well drained. It is noted that considering the cone of influence of the slope/ravine, the water table would be progressively higher moving away from the said slope/ravine. Based on our observation during the drilling activities, the groundwater table within the slope would be located near the depths of 5.33m bgs.

It should be noted that groundwater levels could fluctuate with seasonal weathered conditions, (i.e.: rainfall, droughts, spring thawing) as well as from any changes in the water level of the nearby river. In addition, it can be locally affected by the presence of existing ditches and underground services trenches at or in the vicinity of the site.

5 GEOTECHNICAL CONSIDERATIONS

5.1 General

This section of the report provides general engineering guidelines on the geotechnical design aspects of the project based on our interpretation and review of the information obtained from the boreholes as well as the project requirements.

During the spring thaw of 2019, City staff noticed that a small landslide had occurred in the upper portion of the road's south embankment that resulted in blocking the culvert located at the base of the ravine. The culvert is located between C.N. 1402 and 1420, Lacroix Road near the

Village of Hammond, Ontario. The culvert provides flow to a natural watercourse that is a tributary to the north branch of Indian Creek. It is our understanding that the City of Clarence-Rockland is looking into reinstating the road and its embankment to a safe profile. They may also be looking at replacing the culvert with a similar size.

5.2 Excavation Requirements

In the event that the culvert is replaced, it is anticipated that its invert would remain at the same elevation. Consequently, this would result in an excavation of about 7.5m deep. The excavation will be through sand fill, sand-silt and clay. According to the Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 213/91 and its amendments, the overburden anticipated to be excavated into at this site can be classified as Type 3 for fully drained excavations.

Therefore, shallow temporary excavation in the overburden soil classified as Type 3 can be cut at 1 horizontal to 1 vertical for a fully drained excavation starting at the base of the excavation and as per requirements of the OHSA regulations. If excavation occurs into saturated soil or if the water table is not lowered below the depth of the excavation, the soil should be classified as Type 4 and as such would require to slope the excavation to 3 horizontal to 1 vertical or shallower from the base of the excavation.

Any excavated material stockpiled near a trench or open excavation should be stored at a distance equal to or greater than the depth of the excavated soil within the trench or open excavation and equipment circulation should be restricted away from the top of the slope excavation.

In the event that the aforementioned slopes are not possible to achieve due to space restrictions, the excavation should be shored according to OHSA O. Reg. 213/91 and its amendments. A geotechnical engineer should design and approve the shoring and establish the shoring depth under the excavation profile. Refer to the parameters provided in **Tables 1 and 2** in **Section 5.4** for use in the design of any shoring structures. Any excavation carried out using tightly fitting, braced steel trench boxes should be approved by a professional engineer.

5.3 Groundwater Control

Groundwater seepage and infiltration entering the temporary excavations performed within the overburden during the installation of the culvert should be mitigated by pumping from sumps installed in the excavation. Surface water runoff into the excavation should be avoided and diverted away from the excavation.

In order to install the culvert, the flow from the watercourse will need to be temporarily diverted, likely by damming the watercourse and using pumps to cross the water to the downstream side of the watercourse. It anticipated that these works will be carried out fairly soon as the road is currently closed (summer-fall 2019) and therefore the flows from the watercourse would be minimum due to the time of the year. Nevertheless, a water diversion program will need to be developed based on the anticipated flows that this watercourse could generated during time of construction and from heavy rain events. A Permit to Take Water may be required with the Ministry of the Environment, Conservation and Parks, should volumes of water pumped exceed 50,000 litres per day.

5.4 Retaining Walls and Shoring

The following **Table 1** below provides the suggested soil parameters for the design of retaining wall and/or shoring systems. For excavations near existing services and structures, the coefficient of earth pressure at rest (K_o) should be used.

Table 1: Material Properties for Shoring and Permanent Wall Design (Static)

Type of Material	Bulk Density (kg/m^3)	Pressure Coefficient	
		Active (K_a)	At Rest (K_o)
Clay	18	0.45	0.80
Sand	19	0.33	0.50
Till	22	0.27	0.50
Granular B Type I	20	0.33	0.50
Granular B Type II	23.1	0.31	0.47
Granular A	23.5	0.27	0.43

The above values are for a flat surface behind the wall, a straight wall and a wall friction angle of 0 degrees. The designer should consider any difference between these coefficients, and make appropriate corrections for a sloped surface behind the wall, angled wall or wall friction as required.

A maximum allowable bearing pressure of 60kPa for serviceability limit state (SLS) to limit settlement to 25mm, and 90kPa for ultimate limit state (ULS) factored bearing resistance is provided for the retaining walls resting over undisturbed native clay or properly prepared structural fill. Considering that the silt-sand layer is considered liquefiable, it is not recommended to found any structures on this layer.

Should structural fill be required, it should be placed over undisturbed native soils in layers not exceeding 200mm and compacted to 100 percent of its Standard Proctor Maximum Dry Density (SPMDD). The structural fill should extend 0.6m beyond the outside edges of the base of the retaining wall and then outward and downward at 1 horizontal to 1 vertical profile (or flatter) over a distance equal to the depth of the structural fill below the retaining wall. The material used as structural fill to support the footings should consist of imported granular material meeting Ontario Provincial Standards Specifications (OPSS) requirements for a Granular A, or an approved equivalent material. It is recommended that any structural fill be placed over a geotextile to prevent contamination of fines.

Retaining walls should also be designed to resist the earth pressures produces under seismic conditions. The use of the combined coefficients of static and seismic earth pressure is recommended, referred to as K_{AE} for active conditions and K_{PE} for passive conditions for routine design purposes.

The total active and passive loads under seismic conditions can be calculated using the following two equations;

$P_{AE} = ½ K_{AE} \gamma H^2 (1-k_V)$
$P_{PE} = ½ K_{PE} \gamma H^2 (1-k_V)$

Where;

K_{AE} = Combined Static and Seismic Active Earth Pressure Coefficient

K_{PE} = Combined Static and Seismic Passive Earth Pressure Coefficient

H = Total Height of the Wall (m)

K_h = horizontal acceleration coefficient

K_v = vertical acceleration coefficient

γ = bulk density (kg/m^3)

These equations are based on a horizontal slope behind the wall and a vertical back of the retaining wall and zero wall friction. For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values.

A = Zonal acceleration ratio = 0.2

K_h = Horizontal acceleration coefficient = 0.1

K_V = Horizontal acceleration coefficient = 0.067

The above value of K_h corresponds to ½ of the A value and the value K_V of corresponds to 0.67 of the K_h value. The angle of friction between the soil and the wall has been set at 0^o to provide a conservative estimate. The following **Table 2** provides the parameters for seismic design of retaining structures.

Table 2: Material Properties for Shoring and Permanent Wall Design (Seismic)

Parameter	OPSS Granular B Type I	OPSS Granular A, Granular Fill and Granular B Type II	Clay and Clayey Material
Bulk Unit Weight, γ (kN/m^3)	20	23.3	18
Effective Friction Angle (degrees)	30	32	28
Angle of Internal Friction Between wall and Backfill (degrees)	0	0	0
Yielding Wall			
Active Seismic Earth Pressure Coefficient (K_{AE})	0.37	0.33	0.45
Height of the Application of P_{AE} from the base of the wall as a ration of its height (H)	0.36	0.37	0.36
Passive Seismic Earth Pressure Coefficient (K_{PE})	3.06	3.48	4.0
Height of the Application of P_{PE} from the base of the wall as a ration of its height (H)	0.30	0.30	0.30

5.5 Culvert Bedding Requirements

Bedding, thickness of cover material and compaction requirements for the underground services should conform to the manufacturers design requirements and to the requirements and detailed installations outlined in the Ontario Provincial Standard Specifications (OPSS) and any applicable standards or requirements from the City of Clarence-Rockland. In addition, it is recommended that the bedding for the new culvert be placed over native clay only and that the bedding be a minimum 0.3m thick and set over a geotextile to prevent the migration of fines.

The bedding and cover materials should be compacted in maximum 200mm thick lifts to at least 95 percent of the standard Proctor maximum dry density (SPMDD) using suitable vibratory compaction equipment.

5.6 Reconstruction of Road Embankment

Acceptable and compactable materials should be used to reconstruct the road's embankment up to the roadway subgrade level. Within the depth of seasonal frost penetrations (i.e. 1.8m below finished grade) and in order to reduce the potential for differential frost heaving between the new road embankment and the existing roadway, the selected backfill material should match, as best as possible, the existing soil exposed on the excavation walls. Where there is lack of backfill material and that it would need to be imported, the material should conform to OPSS Granular B Type I or approved equivalent.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadway, the excavation should be compacted in maximum 300mm thick lifts to at least 95 percent of the SPMDD.

Ideally, the slope/road embankment should be reinstated to a 2 horizontal to 1 vertical profile. Under this option, the entire embankment would need to be lined with rip-rap stone to prevent erosion. However, it is our understanding that there is limited space to reconstruct the slope/road embankment to this profile. Consequently, the profile could be reduced to 1.5 to 1 profile, however, the slope would need to be lined with a minimum thickness of rock fill. The use of retaining walls (gabion wall) could also be considered to reduce the total length of the slope.

In the event that rock fill would be considered, the backfill material of the embankment could be completed to a 1 horizontal to 1 vertical profile. The material would then be lined with rock fill as explained hereafter. It is noted that this process would be carried out progressively with the constructing the embankment.

The rock fill should be composed of rock pieces of diameter between 300 and 500 millimetres and with no more than 10% by mass of the material passing the 106 mm sieve. The rock fill placement should not be tipped over the top of the slope from dump trucks but should be completed from the bottom of the slope going upwards. The vegetation should be removed from the surface of the slope and a suitable heavy-duty geotextile fabric should be provided between the rock fill and the slope's surface. It is likely that the road embankment will consist of sand fill, therefore, the geotextile would also act to prevent the sand from migrating into the rock fill. The rock should be well interlocked with no protruding rock faces and should be extended from the base of the watercourse and its banks and up to edge of the road's shoulders or behind guard rails, if required. The base of the rock fill should be keyed at minimum 0.5m into the native soil. The minimum thickness of the rock fill over the slope would be minimum 0.5m, where by creating a 1.5 to 1 profile, the rock fill would be thicker at its base and would provide more weight and stability at the toe of the slope.

The recommended road embankment construction provides long term slope/embankment stability. In order to maintain this stability however, a review of the road's drainage and ditch system entering into the ravine along the road embankment will need to be reviewed to ensure that they do not create erosion and affect the stability of the embankment. This is likely what created the issue with the current slope. One option would be lining the said ditches with rip-rap stone set over a geotextile to dissipate the water during heavy rains or the spring thaw. Where no ditches exist, it would be recommended to create some and direct the water appropriately to the base of the slope.

Any environmental impacts associated with this rehabilitation methodology provided herein were not considered and is outside of our mandate. It will be the responsibility of the client, his representative or the contractor to obtain all necessary permits and approvals to carry out the work. Furthermore, a proper sediment and erosion control plan will need to be developed during the construction of this project.

6 POTENTIAL OF CORROSIVE ENVIRONMENT

A sample (BH-1 SS10) collected at the approximate depth of culvert (6.86m bgs) was submitted to Paracel Laboratories Ltd., an accredited chemical analysis laboratory, for chemical analysis that included pH, Chloride, Resistivity and Redox Potential. The purpose of this testing was to assess the potential for a corrosive environment on any buried concrete or steel. The laboratory Certificate of Analysis are presented in **Appendix B**.

6.1 Sulphate Attack on Buried Concrete

The results of the analysis found the soil to contain a sulphate concentration of 31 µg/g or 0.031%. Based on the CAN/CSA - A23.1 standards (Concrete Materials and Methods of Concrete Construction), a sulphate concentration of 0.1% (1000 µg/g) or less in soil falls within the negligible category for sulphate attack on buried concrete. As such, buried concrete for foundation or manholes will not require any special additive to resist sulphate attack and the use of normal Portland cement is acceptable.

6.2 Corrosivity Analysis for Buried Steel

The potential for an aggressive corrosive soil environment was established in reviewing the above measured parameters and according to standard provided by the American Water Works Association (AWWA) C-105/A21.5-10. Based on the noted standard, corrosion protection for buried steel is only required where a corrosivity index of 10 or greater is encountered. Based on the results, the calculated corrosivity index was found to be less than 10. Furthermore, the pH of the standing water in the creek was measured to be 6.61. As such, any buried steel (i.e. culvert) as part of this project would not require any special or specific corrosion protection measures.

7 REUSE OF ON-SITE SOILS

The existing overburden found at this site consisted of uniform fine-grained sand, silt-sand and silty clay. It is noted that the silt-sand layer was saturated, and clay had a high moisture content; therefore, it would be very difficult to compact these soils properly to meet the requirements outlined herein. The sand fill, which was found to be dry to moist, could be reused as backfill material to reconstruct the road's embankment up to the subgrade level.

It should be further 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 should be stockpiled in a manner that will minimise any significant changes in its moisture content, especially during wet conditions. Any excavated materials proposed for reuse as part of this project should be stockpiled properly in order to allow the material to be properly inspected and approved prior to reuse by a geotechnical engineer.

8 PAVEMENT DESIGN

It is recommended that the road be rehabilitated in accordance to the City of Clarence-Rockland's rural cross section standard (rural retrofit 20m metre R.O.W – dated May 2018). The subgrade soil underlying this municipal road consists of sand fill. Considering that the road is subjected to low traffic and very little heavy traffic, the minimum pavement structure required as part of the City of Clarence-Rockland's rural cross section standard is considered adequate and would consist of the following;

40 millimetres of hot mix asphaltic concrete surface layer (HL3) over
40 millimetres of hot mix asphaltic concrete binder layer (HL8) over
150 millimetres of OPSS Granular A base over
400 millimetres of OPSS Granular B, Type II subbase
Non-woven geotextile.

For predictable performance of the pavement areas, any objectionable fill, organic, soft or deleterious materials should be removed from the proposed pavement areas to expose native undisturbed subgrade soil or properly compacted select subgrade material. The exposed subgrade should be inspected and approved by geotechnical personnel and any evidently loose and unstable areas should be sub-excavated and replaced with suitable earth borrow approved by the geotechnical engineer. Following approval of the preparation of the subgrade, the granular subbase may be placed.

The base and subbase granular materials should conform to OPSS Form 1010 material specifications. Prior to importing any granular material onto the site, it should be tested and approved by a geotechnical engineer prior to delivery to the site and should be compacted to 100% SPMDD. Compaction of the granular pavement materials should be carried out in maximum 200 mm thick loose lifts to 100% of its SPMDD using suitable vibratory compaction equipment.

The Job Mix Formula (JMF) of the asphaltic concrete should be in accordance with OPSS 1150 for Material Specification for Hot Mix Asphalt. The asphaltic concrete should be placed in accordance to OPSS 310 for Construction Specification for Hot Mix Asphalt. The asphaltic concrete should compacted to a minimum of 92% of the Maximum Relative Density. The JMF and its constituents should be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

8.1 Paved Areas and Subgrade Preparation

Following the backfilling and satisfactory compaction of the excavation up to the subgrade level, the subgrade should be shaped, crowned and proof-rolled using heavy roller with any resulting soft areas sub-excavated down to an adequate bearing layer and replaced with approved backfill. Following approval of the preparation of the subgrade, the pavement structure may be placed.

Transitions should be constructed between new and existing pavement structures where new street section meet with existing paved areas. In areas where the new pavement will abut existing pavement, the depths of granular materials should be tapered up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement

Where the existing asphaltic concrete surface of an existing street/road is affected by the excavating process, the damaged zones should be saw cut and any damaged or loose pieces of asphaltic concrete should be removed down to the binder course or its entire depth. Where only one layer exists, the existing base should be scarified and proof-rolled with any soft areas excavated and replaced to the proper level with OPSS Granular A. Where two layers of asphalt exist on an access lane, the surface course should be grinded over a width of 150mm to allow the new surface course to overlap the binder layer and not create one straight vertical joint. On existing streets, the overlap should be increased to 300mm.

9 CONSTRUCTION CONSIDERATION

It is suggested that the final design drawings for this project, including the proposed site grading plan, be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed development do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. All engineered fill areas (if required) for the proposed project should be inspected by Lascelles Engineering and Associates Ltd. to ensure that a suitable subgrade has been reached and properly prepared.

The subgrade for the pavement areas, watermain and sewers should be inspected and approved by geotechnical personnel. In-situ density testing should be carried out on the pavement granular materials and pipe bedding and backfill to ensure the materials meet the specifications from a compaction point of view.

10 REPORT CONDITIONS AND LIMITATIONS

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document is neither intended nor authorized by Lascelles Engineering & Associates Ltd. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible subsurface contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report.

The recommendations provided in this report are based on subsurface data obtained at the specific test locations only. 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 to the project will require a review by Lascelles Engineering & Associates Ltd., to ensure compatibility with the recommendations contained in this project. Any changes to the project will require a review by Lascelles Engineering & Associates Ltd., to ensure compatibility with the recommendations contained in this report.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further services to you, please do not hesitate to contact our office.

Yours truly,
Lascelles Engineering & Associates Ltd.

Prepared by:

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LICENSED PROFESSIONAL ENGINEER
M.C.M. RODRIGUE
July 9, 2019
PROVINCE OF ONTARIO

Appendix A

Borehole Logs

LASCELLES ENGINEERING & ASSOCIATES

RECORD OF BOREHOLE: BH-1

PROJECT: Geotechnical Investigation & Slope Stability Analysis
CLIENT: City of Clarence-Rockland
LOCATION: Between C.N.1402 and C.N.1420, Lacroix Rd, Hammond, ON
DATE: June 25, 2019

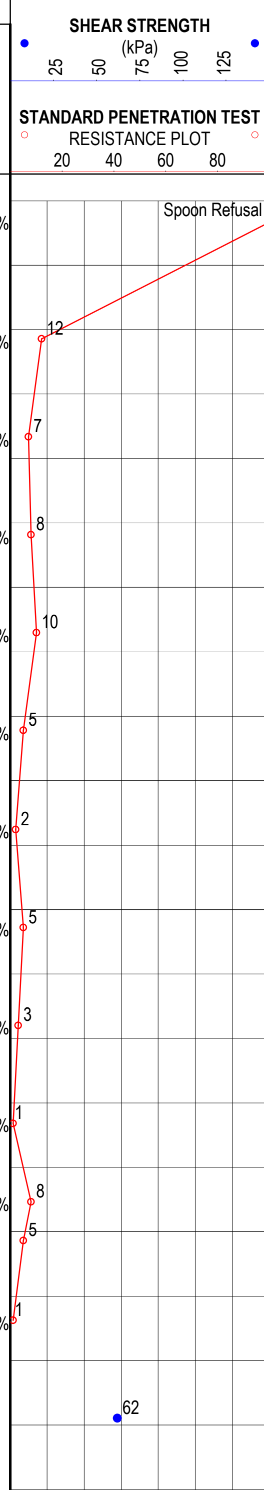
PROJECT No.: 190181
LOGGED BY: S.C.
DRILLER: George Downing Estate Drilling Ltd.
DRILLING EQUIPMENT: Trucked-mounted CME75
DRILLING METHOD: Hollow Stem Auger

SOIL PROFILE

DEPTH (m)	DESCRIPTION	ELEV.
0.0	Ground Surface	73.06 / 0.00
0.0–0.76	**Pavement Structure:** 25mm asphlatic concrete over 740mm granular crushed stones.	72.30 / 0.76
0.76–6.71	**Fill:** Uniform, fine grained sand with traces to some silt, brown in colour becoming greyish brown with depth, compact to loose and dry to moist but wet below 6.71m.	66.35 / 6.71
6.71–7.92	**Sand-Silt:** Sand and silt mixture with presence of clay to clayey, presence of organics, greyish brown to dark grey in colour, loose and wet	65.14 / 7.92
7.92–9.45	**Clay:** Silty, grey in colour, stiff consistency, high plasticity and moisture content.	63.61 / 9.45
9.45	End of Borehole	

SAMPLES

NUMBER	TYPE	N-VALUE / RQD	RECOVERY
SS1		50R	100%
SS2		12	100%
SS3		7	100%
SS4		8	100%
SS5		10	100%
SS6		5	100%
SS7		2	100%
SS8		5	100%
SS9		3	100%
SS10		1	100%
SS11		8 5	100%
SS12		1	100%



WATER CONTENT (%)
20 40 60 80

WATER LEVEL

Dry
(06/05/2019)

Easting: 483286
Site Datum: Geodetic
Top of Casing Elev.: 73.056
Borehole Diameter: 200mm

Northing: 5032843
Groundsurface Elev.: 73.056m
Top of Riser Elev.: NA
Monitoring Well Diameter: NA

COMMENTS:

LASCELLES ENGINEERING & ASSOCIATES

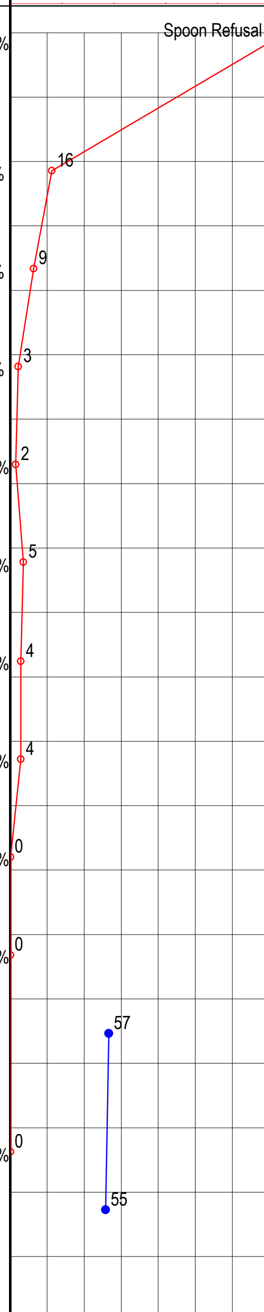
RECORD OF BOREHOLE: BH-2

PROJECT: Geotechnical Investigation & Slope Stability Analysis
CLIENT: City of Clarence-Rockland
LOCATION: Between C.N. 1402 and C.N. 1420, Lacroix Rd, Hammond, ON.
DATE: June 25, 2019

PROJECT No.: 190181
LOGGED BY: S.C.
DRILLER: George Downing Estate Drilling Ltd.
DRILLING EQUIPMENT: Trucked-mounted CME75
DRILLING METHOD: Hollow Stem Auger

SOIL PROFILE		SAMPLES					SHEAR STRENGTH (kPa) 25 50 75 100 125 / STANDARD PENETRATION TEST RESISTANCE PLOT 20 40 60 80	WATER CONTENT (%) 20 40 60 80	WATER LEVEL
DEPTH (m)	DESCRIPTION	ELEV.	NUMBER	TYPE	N-VALUE / RQD	RECOVERY			
0.0	Ground Surface	73.08 / 0.00							
	Pavement Structure: 25mm asphlatic concrete over 710mm granular crushed stones.		SS1		50R	100%	Spoon Refusal		
		72.34 / 0.74							
	Fill: Uniform, fine grained sand with traces to some silt, brown in colour becoming greyish brown with depth, compact to loose and dry to moist.		SS2		16	92%			
			SS3		9	83%			
			SS4		3	83%			
			SS5		2	100%			
			SS6		5	100%			
			SS7		4	100%			
		67.75 / 5.33							
	Sand-Silt: Sand and silt mixture with presence of clay to clayey, greyish brown, loose, and wet		SS8		4	100%			
		66.98 / 6.10							
	Clay: Silty, grey in colour, stiff consistency, high plasticity and moisture content.		SS9		0	100%			
			SS10		0	100%			
							57		
			SS11		0	100%			
							55		
9.14	End of Borehole	63.94 / 9.14							

Depth scale: ft 0.0–32.0; m 0.0–9.0



Easting: 483277
Site Datum: Geodetic
Top of Casing Elev.: 73.084
Borehole Diameter: 200mm

Northing: 5032840
Groundsurface Elev.: 73.084m
Top of Riser Elev.: NA
Monitoring Well Diameter: NA

COMMENTS:

LASCELLES ENGINEERING & ASSOCIATES

RECORD OF BOREHOLE: BH-3

PROJECT: Geotechnical Investigation & Slope Stability Analysis
CLIENT: City of Clarence-Rockland
LOCATION: Between C.N.1402 and C.N.1420, Lacroix Rd, Hammond, ON.
DATE: June 26, 2019

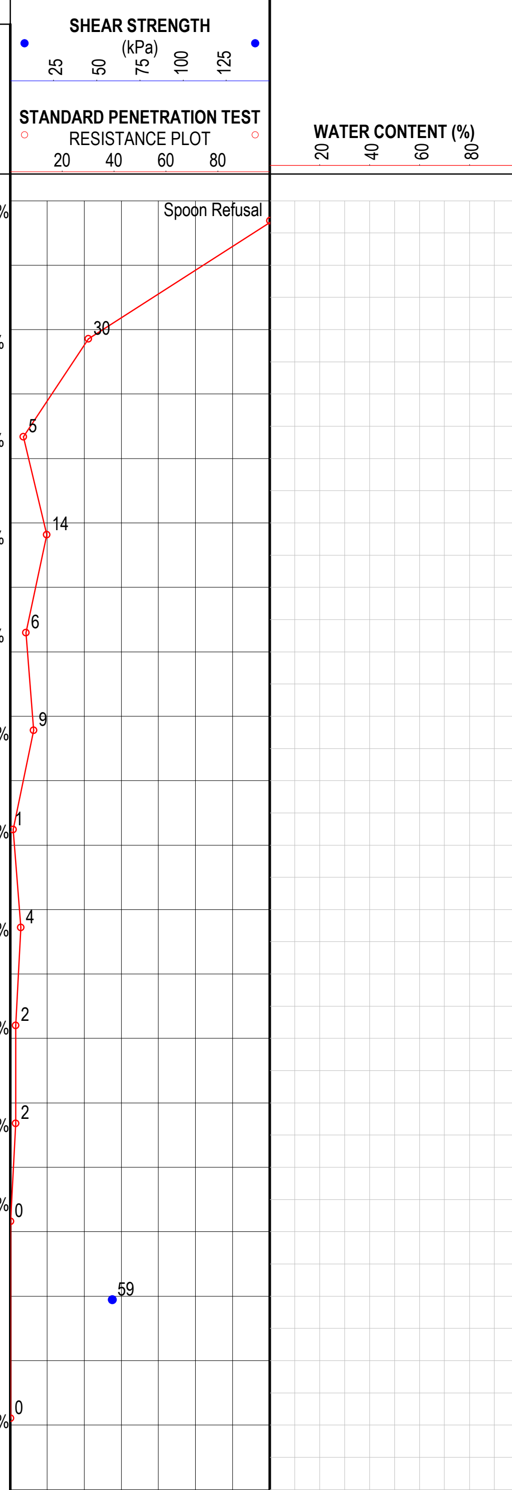
PROJECT No.: 190181
LOGGED BY: S.C.
DRILLER: George Downing Estate Drilling Ltd.
DRILLING EQUIPMENT: Trucked-mounted CME75
DRILLING METHOD: Hollow Stem Auger

SOIL PROFILE

DEPTH (m)	DESCRIPTION	ELEV.
	Ground Surface	73.25
0.0	**Pavement Structure:** 25mm asphlatic concrete over 710mm granular crushed stones.	0.00
		72.49
0.76	**Fill:** Uniform, fine grained sand with traces to some silt, brown in colour becoming greyish brown with depth, compact to loose and dry to moist.	0.76
		69.59
3.66	**Sand-Silt:** Sand and silt mixture with presence of clay to clayey, presence of organics, greyish brown to dark grey in colour, loose to very loose, and moist to wet.	3.66
		65.63
7.62	**Clay:** Silty, grey in colour, firm in consistency, very moist.	7.62
		63.50
9.75	End of Borehole	9.75

SAMPLES

NUMBER	TYPE	N-VALUE / RQD	RECOVERY
SS1		50R	100%
SS2		30	83%
SS3		5	92%
SS4		14	83%
SS5		6	45%
SS6		9	100%
SS7		1	100%
SS8		4	100%
SS9		2	100%
SS10		2	100%
SS11		0	100%
SS12		0	100%



WATER LEVEL

Dry

(06/05/2019)

Easting: 483290
Site Datum: Geodetic
Top of Casing Elev.: 73.25
Borehole Diameter: 200mm

Northing: 5032847
Groundsurface Elev.: 73.250m
Top of Riser Elev.: NA
Monitoring Well Diameter: NA

COMMENTS:

LASCELLES ENGINEERING & ASSOCIATES

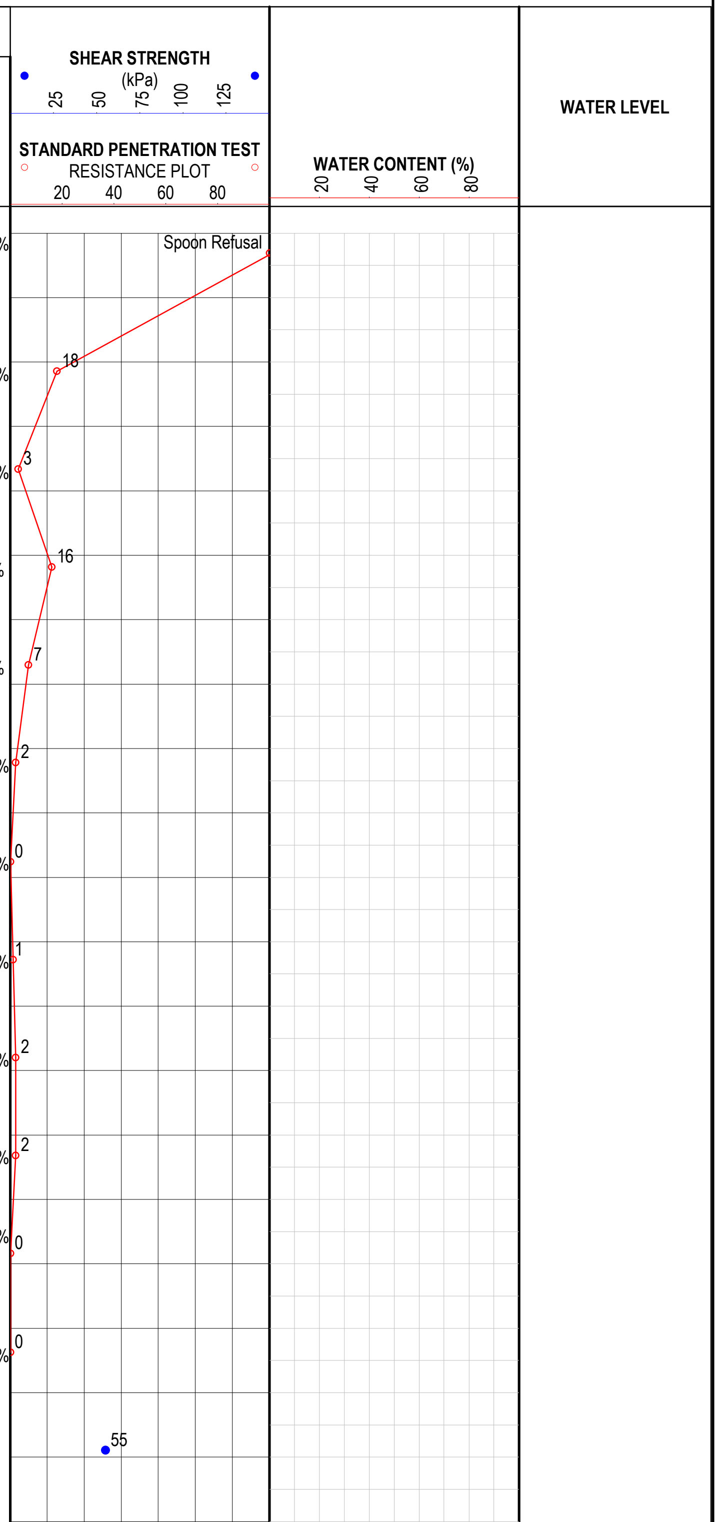
RECORD OF BOREHOLE: BH-4

PROJECT: Geotechnical Investigation & Slope Stability Analysis
CLIENT: City of Clarence-Rockland
LOCATION: Between C.N.1402 and C.N. 1420, Lacroix Rd, Hammond, ON.
DATE: June 26, 2019

PROJECT No.: 190181
LOGGED BY: S.C.
DRILLER: George Downing Estate Drilling Ltd.
DRILLING EQUIPMENT: Trucked-mounted CME75
DRILLING METHOD: Hollow Stem Auger

SOIL PROFILE / SAMPLES

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	N-VALUE / RQD	RECOVERY
Ground Surface	73.38 / 0.00				
Pavement Structure: 25mm asphlatic concrete over 710mm granular crushed stones.		SS1		50R	100%
Fill: Uniform, fine grained sand with traces to some silt, presence of crushed stone between the depths of 3.0m and 3.66m, brown in colour, compact to very loose and moist.	72.62 / 0.76	SS2		18	100%
		SS3		3	100%
		SS4		16	83%
		SS5		7	50%
Sand-Silt: Sand and silt mixture with presence of clay to clayey, traces of gravel and presence of organics, grey to dark grey in colour, loose to very loose and wet.	69.72 / 3.66	SS6		2	100%
		SS7		0	100%
		SS8		1	100%
		SS9		1	100%
Clay: Silty, grey in colour, firm in consistency, very moist.	66.52 / 6.86	SS10		2	100%
		SS11		0	100%
		SS12		0	100%
End of Borehole	63.93 / 9.45				



Easting: 483291
Site Datum: Geodetic
Top of Casing Elev.: 73.375
Borehole Diameter: 200mm

Northing: 5032846
Groundsurface Elev.: 73.375m
Top of Riser Elev.: NA
Monitoring Well Diameter: NA

COMMENTS:

Appendix B

Laboratory “Certificate of Analysis”