RESIDENTAIL PROPERTIES
245 – 267 ROCHESTER STREET, 27 & 29 BALSAM STREET
OTTAWA, ONTARIO
K1R 7M9

GEOTECHNICAL INVESTIGATION

PREPARED FOR:
Carl Madigan
3N Group Holdings Inc.
1769 St Laurent Boulevard
Ottawa, Ontario
K1G 3V4

Rubicon Job Number • R63048.11
Report Date • October 17, 2022

"….Environmental Solutions."

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Rubicon Environmental (2008) Inc.

October 17, 2022

Carl Madigan
3N Group Holdings Inc.
1769 St Laurent Boulevard
Ottawa, Ontario

Job #: R63048.11

Phase II – Environmental Site Assessment
Residential Properties
245 - 267 Rochester Street, 27 & 29 Balsam Street, Ottawa, Ontario, K1R 7M9

Dear Client,

Please find enclosed the results for the above-mentioned investigation conducted on your behalf.

Please feel free to contact me at 519-924-0003 if you require any additional information.

Sincerely,

RUBICON ENVIRONMENTAL (2008) INC.

Paul Rew, P. Eng., QP

Distribution:

Client: 1
Office: 1

"…Environmental Solutions"

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1.0 INTRODUCTION

Rubicon Environmental (2008) Inc. (Rubicon) was retained by Mr. Carl Madigan on behalf of 3N Group Holdings Inc. to undertake a Geotechnical Investigation at the vacant former residential properties located at 245, 247, 249, 261, 263, 265, 267 Rochester Street, 27 & 29 Balsam Street, Ottawa, Ontario. (Hereby referred to as the 'Subject property').

The site is located on the northeast corner of Rochester and Balsam Streets and is currently occupied by three storefront structures of two storeys in height. The rear of the existing structures consists of paved parking.

It is understood that the proposed development on the site is to construct a one structure with one level of underground parking. The proposed building will consist of one underground level of parking, ground floor retail and eight floors of residential above the retail floor. The proposed development is to be municipally serviced. The subject property currently consists of four exposed basements and limestone bedrock.

The scope of work of the following geotechnical investigation was to

i) Determine the bearing value of the bedrock for design of the footings;
ii) Provide recommendations for pavement structures; and;
iii) Comment on the geotechnical considerations relating to the construction of the project.

The following report has been prepared to address the above noted terms of reference, with the understanding that the design of the project will be carried out in accordance with all applicable codes and standards. Any changes to the project described will require a review by Rubicon Environmental (2008) Inc. to assess the impact of the changes to the report recommendations.

2.0 FIELDWORK

The fieldwork consisted of drilling six (6) borehole and determining the bedrock bearing value on February 7, 2022. The drilling was carried out by specialist drilling subcontractor working under the direction and supervision of the Q.P. Paul Rew. The test hole procedure consisted of rock cores and SPT were conducted to assess the quality of the rock.

Bedrock was exposed on site at the time of the investigation. The bedrock was cored by diamond drilling to determine the type and quality of the bedrock for foundations and for excavating.

The 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.

6 boreholes were advanced to a maximum depth of 3.00 m below existing grade. The borehole locations were conducted for general site coverage of the proposed development taking into consideration existing site features. The site plan is attached on Figure 2 and Appendix 1.

A level survey was carried out to establish the ground level at the borehole locations. The benchmark for the survey is described as the concrete pin in utility pole elevation = 66.40 m.

3.0 SUBSOIL AND GROUNDWATER

No subsoil was encountered on site at the time of the investigation as it was stockpiled on site to be removed to a licensed facility. Limestone bedrock was identified on site from the Ottawa formation. Core samples were recovered from BH201 – BH206 using a core barrel and diamond drilling techniques. The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality. Refer to borehole logs – Appendix 1 and Figure 2 – Site Plan.

4.0 FOUNDATION

Based on the borehole data, it is evident that the proposed 9 storey structure can be supported on conventional spread footings. The ground surface across the subject site slightly slope to the south and is at grade with the surrounding west to east with a 2-meter basement foundation.

4.1 FOOTING FOUNDATIONS

It is expected that the excavation for the basement will be well into the limestone bedrock similar to the footings of the 9-storey building. The recommended safe net bearing value for the designs on the sound limestone is 2,500 KPa. Upon review of the core hole samples, the upper portion of the bedrock was found to be in poor to fair quality and increasing to good quality bedrock with depth. Bedrock consists of interbedded limestone and shale of the Verulam Formation.

4.2 SETTLEMENT

Footings bearing on the sound limestone will settle under the design load provided on section 10.0. Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post- construction total and differential settlements.

4.3 FROST PROTECTION

The parking garage is expected to not require protection against frost action due to the founding depth. Unheated structures such as the access ramp may required to be insulated against the deleterious effect of frost action.

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with adequate foundation insulation, should be provided. More details regarding foundation insulation can be provided, if requested.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

4.4 EARTHQUAKE DESIGN

The foundations should be structurally designed to resist a minimum earthquake for force in accordance with the applicable building codes. The site is located within seismic zone 4 (i.e., Za=4, Zv=2, V=0.10) the Site Classification is “B”, with respect to NBC 2005. The Foundation Factor F is 1.0 for this site.

5.0 EXCAVATIONS AND GROUNDWATER

The permanent groundwater table is expected to be below the proposed single basement at this site, therefore, groundwater control will not be a problem, as any surface seepage water will be controlled with a conventional sump and pump system.

Rock excavation will be required for the single basement level: it is expected that a combination of blasting and hoe ramming will be required after stripping the upper weathered bedrock with the excavator. Line drilling will be required to ensure a safe vertical excavation adjacent to existing building and service lines.

6.0 PERMANENT DRAINAGE

It is recommended that perimeter drainage be provided around the basement. Current practice is to install a drainage membrane such as Miradrain G100N or Delta Drain 6000 and to utilize 150 mm perforated pipe with geotextile sock be placed in each bay. This will be implemented by the construction company as required.

7.0 EARTH PRESSURE ON BASEMENT WALLS

With a functioning perimeter drainage system, it is permissible to design the basement walls for only the earth pressure. The earth pressure acting on the basement wall can be calculated using a hydrostatic pressure diagram and an active earth pressure coefficient Ka = 0.35. the contribution due to surcharge loading, at the ground surface, is represented by rectangular pressure distribution.

i) Basement Floors

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. It is expected that the basement area will be mostly parking, and a rigid pavement structure designed by a structural engineer will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be used it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone or Granular A. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

ii) Basement Wall

It is understood that the basement walls are to be poured against a damp proofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.01 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face.

Where soil is to be retained, 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 bulk (drained) unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Static Conditions

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ 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/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ 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 maintain a minimum separation of
0.3 m from the walls with the compaction equipment.

Seismic Conditions

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) can be calculated using $0.375 \cdot a \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)amax$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

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 $P = 0.5\ K_o \gamma H^2$, where K = 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.

8.0 LATERAL AND FLOOR SLABS

Lateral Support: The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Floor slabs: The basement is to be developed into the bedrock to a depth of 5 mbgl. Slab construction of the basement floor is expected for this project. It is recommended that the floor slab be poured on a minimum of 200 mm of clear stone or Granular 'A' crushed gravel. If Granular 'A' is used, any underfloor fill material should be compacted to 98 percent of its maximum standard Proctor: dry density (ASTM D698).

9.0 PAVEMENT STRUCTURES

For surface access lanes and parking, it is assumed that the subgrade soil will be variable fill material. Any soritical topsoil/root mat organic material should be stripped, and proof rolling of the subgrade is recommended, prior to placing any subbase granular material.

The recommended pavement structure for heavy duty access lanes and for light duty automobile parking are provided in Table 1, below.

TABLE 1 RECOMMENDED PAVEMENT STRUCTURES

Course	Material	Light Duty Automobile Parking	Heavy Duty Access Lane
Surface	HL3 A/C	50 mm	40 mm
Binder	HL8		40 mm
Basecourse	Granular 'A'	100 mm	150 mm
Subbase	Granular 'B'; Type II	200 mm	300 mm

10.0 DISCUSSION

From a geotechnical perspective, the subject site is considered suitable for the proposed development. The proposed mid-rise buildings are anticipated to be founded on spread footings placed directly on a clean, surface sounded bedrock bearing surface.

Bedrock removal may be required to complete the underground level. Hoe ramming is an option where only small quantities of bedrock need to be removed. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

10.1 SITE GRADING AND PREPARATION

i) **Stripping Depth:** Since the building will occupy the entire boundaries of the subject site, all overburdens will be removed to bedrock.

ii) **Bedrock Removal:** Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming. Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be completed prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

iii) **Peak particle velocities:** (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

iv) **Vibration Considerations:** Construction operations are the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment's could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed building.

10.2 FOUNDATION DESIGN

Bearing Resistance Values: Footings placed on a clean, surface sounded limestone bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of 2,500 kPa, incorporating a geotechnical resistance factor of 0.5.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A factored bearing resistance value at ULS of 2,500 kPa, incorporating a geotechnical resistance factor of 0.5 if founded on sound limestone bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footing footprint(s). One drill hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

10.3 ROCK ANCHOR DESIGN

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with much less deflection than a passive anchor.

It is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

A factored tensile grout to rock bond resistance value at ULS of 1.0 MPa, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 30 MPa is recommended.

Rock Cone Uplift

The geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a Rock Mass Rating (RMR) of 65 was assigned to the bedrock, and Hoek and Brown parameters (m and s) were taken as 0.575 and 0.00293, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	30 MPa
Rock Mass Rating (RMR) - Good quality limestone bedrock Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - limestone bedrock	90 MPa
Unit weight - Submerged Bedrock	15.2 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3 below.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor				
Diameter of Corehole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.4	0.6	2.0	400
	2.4	1.2	3.6	800
	3.4	1.6	5.0	1200
125	1.1	0.6	1.7	400
	1.8	1.0	2.8	800
	2.6	1.2	3.8	1200

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a tremie tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction.

Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

Bedrock Reinforcement for Weathered Bedrock Areas

Due to the lower quality of bedrock near surface and potential founding of the proposed development, bedrock stabilization may be required when the proposed foundation extends into the shale bedrock.

Horizontal rock anchors and/or wire mesh with rock wedges may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The rock reinforcement recommendations should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

10.4 EXCAVATION SIDE SLOPES

Unsupported Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for majority of the excavation to be constructed by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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. A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of 1.0 MPa, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of
0.65 K y H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K y H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

10.5 PIPE BEDDING AND BACKFILL

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential 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 SPMDD.

10.6 GROUNDWATER CONTROL

It is anticipated that groundwater infiltration into the excavations should be 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 Environment, Conservation and Parks (MECP) Category 3 Permit to Take Water (PTTW) may be required if more than 400,000 L/day are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically 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.

10.6 CORROSION POTENTIAL AND SULPHATE

The analytical testing results indicate that the sulphate content is less tan 0.1%. This results indicates that Type 10 Portland Cement (i.e. normal cement) would be appropriate for this site. The chloride content and pH of the samples indicate that they are not significant factors in creating a corrosive environment, whereas the resistivity is indicative of an aggressive corrosive environment.

11.0 CONCLUSION

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 Rubicon Environmental (2008) Inc. Contractors and others involved in the construction of this project are advised to make independent assessment of the subsurface bedrock and groundwater conditions for the purpose of establishing quantities, schedules, and construction techniques. The foundation design is based on the bedrock conditions and are not dependent on any soil conditions since the shallow overburden of soils are stock piled pending removal to bedrock.

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

The report is applicable only to the project described in the text. Any changes in size, founding elevations or location will require a review by Rubicon Environmental (2008) Inc. to ensure compatibility with the recommendations contained in this report.

Respectfully submitted,

RUBICON ENVIRONMENTAL (2008) INC.

Paul D. Rew, P.Eng. QP

LICENSED PROFESSIONAL ENGINEER
P. D. REW
October 17, 2022
PROVINCE OF ONTARIO

12.0 LIMITATIONS

1. This assessment was conducted in accordance with generally accepted engineering standards. It is possible that materials other than those described in this report are present at the site. The client acknowledges that no assessment can necessarily identify the existence of all contaminants, potential contaminants, or environmental conditions;

2. This report was prepared for the sole and exclusive use of Mr. Carl Madigan on behalf of 3N Group Holdings Inc., Rubicon Environmental (2008) Inc. accepts no responsibility or liability for any loss, damage, expense, fine or any other claim of any nature or type, including any liability or potential liability arising from its own negligence, for any use of this report or reliance on it, in whole or in part, by anyone other than Mr. Carl Madigan on behalf of 3N Group Holdings Inc.;

3. There is no representation, warranty or condition, express or implied, by Rubicon Environmental (2008) Inc. or its officers, directors, employees or agents that this assessment has identified all contaminants, potential contaminants or environmental conditions at the site or that the site is free from contamination, potential contaminants or environmental conditions other than those noted in this report;

4. This assessment has been completed from information and documentation described in this report as well as the results of limited chemical analysis of soil samples collected from accessible locations on the date(s) specified. We have assumed that any such information and documentation is accurate and complete. We can accept no responsibility or liability for any errors, deficiencies or inaccuracies in this report arising from errors or omissions in the information and documentation provided by others;

5. This assessment was based on information and the results of investigation(s) obtained on the date(s) specified. Rubicon Environmental (2008) Inc. accepts no responsibility or liability for any changes or potential changes in the condition of the site subsequent to the date of our investigation(s);

6. The conditions between sampling locations have been inferred, to the best of our ability, based on the conditions observed at sampling locations. Conditions between and beyond sampling locations may vary. This assessment pertains, only, to the site specifically described in this report and not to any adjacent or other property;

7. This report is not to be reproduced or released to any other party, in whole or in part, without the express written consent of Rubicon Environmental (2008) Inc.

FIGURES

R63048	NAME	DATE
DRAWN BY:	NP	October 2022
CHECKED BY:	PDR	October 2022
27, 29 Balsam Street, & 245 - 267 Rochester Street, Ottawa, ON		

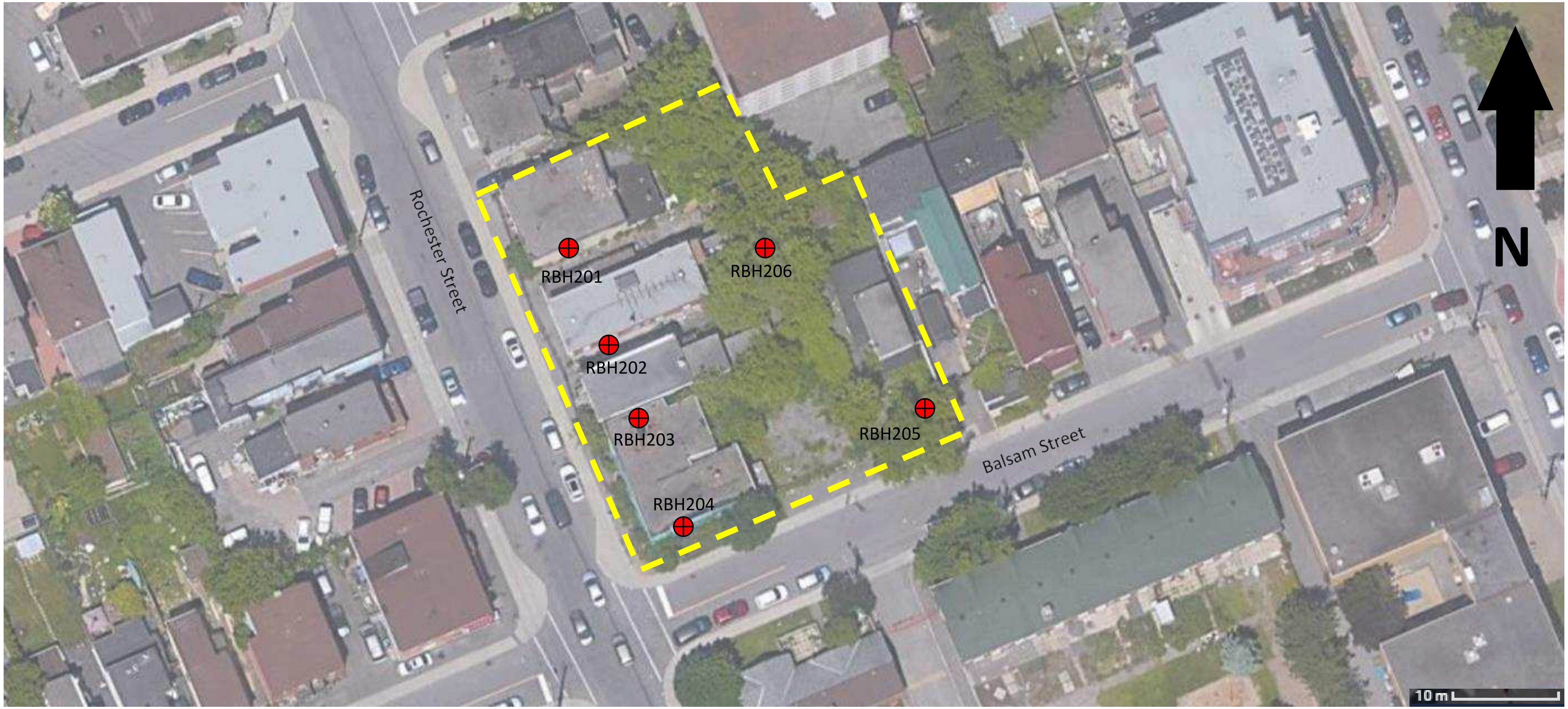
REI
Rubicon Environmental (2008) Inc.

Figure 1: Site Location

Legend

RSC Phase One and Phase Two Subject Property

RSC Phase One and Phase Two Study Area



R63048	NAME	DATE
DRAWN BY:	NP	October 2022
CHECKED BY:	PDR	October 2022
27, 29 Balsam Street, & 245 - 267 Rochester Street, Ottawa / ON		

Rubicon Environmental (2008) Inc.

Figure 2:
Site Plan

Legend
RSC Phase One, Phase Two Subject Property
Boreholes

APPENDIX 1
BOREHOLE LOGS

LOG OF BH201

PROJECT NUMBER R63048.11		
PROJECT NAME Geotechnical		
CLIENT 3N Group Holdings Inc.	**DRILL RIG** Acker AD II Truck Mount Rig	**SURFACE ELEVATION CENTROID**
ADDRESS 27, 29 Balsam Street, & 245 - 267 Rochester Street, Ottawa, ON	**DRILLING METHOD** Hallow Stem Auger, Split Sp	**LOGGED BY** JG
	TOTAL DEPTH 3.00 m	**CHECKED BY** Paul Rew
DRILLING DATE October 14, 2022	**DIAMETER** 2.5 inch spoon, 5 inch auger	
COMPLETION	**CASING** uPVC	**SCREEN** uPVC Factory Slotted
COMMENTS		

Depth (m)	PID	Samples	Material Description	Graphic Log	Moisture	Additional Observations	Depth
0.0–0.75	<5 ppm	SS1	- Weathered Limestone - 2500 KPa Moisture at around 2.00 m - Above 50 N count				
0.75–1.5	<5 ppm	SS2					
1.5–2.25		SS3					
2.25–3.0	<5 ppm	SS4	- Limestone (from 2.33 m)				
3.0–	<5 ppm	SS5	Total depth 3.00 m				

Depth scale: 0.2, 0.4, 0.6, 0.8, 1, 1.2, 1.4, 1.6, 1.8, 2, 2.2, 2.4, 2.6, 2.8, 3, 3.2, 3.4

LOG OF BH202

PROJECT NUMBER R63048.11		
PROJECT NAME Geotechnical		
CLIENT 3N Group Holdings Inc.	**DRILL RIG** Acker AD II Truck Mount Rig	**SURFACE ELEVATION CENTROID**
ADDRESS 27, 29 Balsam Street, & 245 - 267 Rochester Street, Ottawa, ON	**DRILLING METHOD** Hallow Stem Auger, Split Sp	**LOGGED BY** JG
	TOTAL DEPTH 3.00 m	**CHECKED BY** Paul Rew
DRILLING DATE October 14, 2022	**DIAMETER** 2.5 inch spoon, 5 inch auger	
COMPLETION	**CASING** uPVC	**SCREEN** uPVC Factory Slotted
COMMENTS		

Depth (m)	PID	Samples	Material Description	Graphic Log	Moisture	Additional Observations	Depth
0.0–0.75	<5 ppm	SS1	- Weathered Limestone - 2500 KPa Moisture at around 2.00 m - Above 50 N count				
0.75–1.5	<5 ppm	SS2					
1.5–2.25		SS3					
2.25–3.0	<5 ppm	SS4	- Limestone				
3.0–	<5 ppm	SS5	Total depth 3.00 m				

Depth scale (m): 0.2, 0.4, 0.6, 0.8, 1, 1.2, 1.4, 1.6, 1.8, 2, 2.2, 2.4, 2.6, 2.8, 3, 3.2, 3.4

Disclaimer This bore log is intended for environmental not geotechnical purposes.

LOG OF BH203

PROJECT NUMBER R63048.11
PROJECT NAME Geotechnical
CLIENT 3N Group Holdings Inc.
ADDRESS 27, 29 Balsam Street, & 245 - 267 Rochester Street, Ottawa, ON
DRILLING DATE October 14, 2022

DRILL RIG Acker AD II Truck Mount Rig
DRILLING METHOD Hallow Stem Auger, Split Sp
TOTAL DEPTH 3.00 m
DIAMETER 2.5 inch spoon, 5 inch auger

SURFACE ELEVATION CENTROID
LOGGED BY JG
CHECKED BY Paul Rew

COMPLETION
CASING uPVC
SCREEN uPVC Factory Slotted

COMMENTS

Depth (m)	PID	Samples	Material Description	Graphic Log	Moisture	Additional Observations	Depth
0.0–0.75	<5 ppm	SS1	- Weathered Limestone - 2500 KPa Moisture at around 2.00 m - Above 50 Count				
0.75–1.5	<5 ppm	SS2					
1.5–2.25		SS3					
2.25–3.0	<5 ppm	SS4	- Limestone (from ~2.33 m)				
3.0–3.5	<5 ppm	SS5	Total depth 3.00 m				

Depth scale: 0.2, 0.4, 0.6, 0.8, 1, 1.2, 1.4, 1.6, 1.8, 2, 2.2, 2.4, 2.6, 2.8, 3, 3.2, 3.4

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LOG OF BH204

PROJECT NUMBER R63048.11
PROJECT NAME Geotechnical
CLIENT 3N Group Holdings Inc.
ADDRESS 27, 29 Balsam Street, & 245 - 267 Rochester Street, Ottawa, ON
DRILLING DATE October 14, 2022

DRILL RIG Acker AD II Truck Mount Rig
DRILLING METHOD Hallow Stem Auger, Split Sp
TOTAL DEPTH 3.00 m
DIAMETER 2.5 inch spoon, 5 inch auger

SURFACE ELEVATION CENTROID
LOGGED BY JG
CHECKED BY Paul Rew

COMPLETION
CASING uPVC
SCREEN uPVC Factory Slotted

COMMENTS

Depth (m)	PID	Samples	Material Description	Graphic Log	Moisture	Additional Observations	Depth
0.0 – 0.75	<5 ppm	SS1	- Weathered Limestone - 2500 KPa Moisture at around 2.00 m - Above 50 N Count				
0.75 – 1.5	<5 ppm	SS2					
1.5 – 2.25		SS3					
2.25 – 3.0	<5 ppm	SS4	- Limestone				
3.0 –	<5 ppm	SS5	Total depth 3.00 m				

Depth scale: 0.2, 0.4, 0.6, 0.8, 1, 1.2, 1.4, 1.6, 1.8, 2, 2.2, 2.4, 2.6, 2.8, 3, 3.2, 3.4

LOG OF BH205

PROJECT NUMBER R63048.11
PROJECT NAME Geotechnical
CLIENT 3N Group Holdings Inc.
ADDRESS 27, 29 Balsam Street, & 245 - 267 Rochester Street, Ottawa, ON
DRILLING DATE October 14, 2022

DRILL RIG Acker AD II Truck Mount Rig
DRILLING METHOD Hallow Stem Auger, Split Sp
TOTAL DEPTH 3.00 m
DIAMETER 2.5 inch spoon, 5 inch auger

SURFACE ELEVATION CENTROID
LOGGED BY JG
CHECKED BY Paul Rew

COMPLETION
CASING uPVC
SCREEN uPVC Factory Slotted

COMMENTS

Depth (m)	PID	Samples	Material Description	Graphic Log	Moisture	Additional Observations	Depth
0 – 0.75	<5 ppm	SS1	- Weathered Limestone - 2500 KPa Moisture at around 2.00 m - Above 50 N count				
0.75 – 1.5	<5 ppm	SS2					
1.5 – 2.25		SS3					
2.25 – 3	<5 ppm	SS4	- Limestone (from approx. 2.33 m)				
3 – 3.5	<5 ppm	SS5	Total depth 3.00 m				

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LOG OF BH206

PROJECT NUMBER R63048.11
PROJECT NAME Geotechnical
CLIENT 3N Group Holdings Inc.
ADDRESS 27, 29 Balsam Street, & 245 - 267 Rochester Street, Ottawa, ON
DRILLING DATE October 14, 2022
DRILL RIG Acker AD II Truck Mount Rig
DRILLING METHOD Hallow Stem Auger, Split Sp
TOTAL DEPTH 3.00 m
DIAMETER 2.5 inch spoon, 5 inch auger
SURFACE ELEVATION CENTROID
LOGGED BY JG
CHECKED BY Paul Rew

COMPLETION
CASING uPVC
SCREEN uPVC Factory Slotted

COMMENTS

Depth (m)	PID	Samples	Material Description	Graphic Log	Moisture	Additional Observations	Depth
0 – 0.75	<5 ppm	SS1	- Weathered Limestone - 2500 KPa - Moisture at around 2.00 m - Above 50 N Count				
0.75 – 1.5	<5 ppm	SS2					
1.5 – 2.25		SS3					
2.25 – 3.0	<5 ppm	SS4	- Limestone				
3.0	<5 ppm	SS5	Total depth 3.00 m				

Disclaimer This bore log is intended for environmental not geotechnical purposes.