

Confederation Line Level 1 Proximity Study

**Proposed Residential Development
2000 City Park Drive – Ottawa, Ontario**

Prepared for Colonnade Bridgeport

Report PG6552-3 dated September 3, 2026

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Appendix D	Proximity Assessment PG6552-LET.01 dated September 3, 2026

1.0 Introduction

Paterson Group (Paterson) was commissioned by Colonnade Bridgeport to conduct a Level 1 Confederation Line Proximity Study for the proposed residential development, to be located at 2000 City Park Drive, in the City of Ottawa.

Colonnade Bridgeport is an Ottawa-based, full-service real estate investment, development, and management firm with experience advancing mixed-use, commercial, industrial, and residential projects. For the proposed development at 2000 City Park Drive, Colonnade Bridgeport is working with the City of Ottawa, the project team, and community stakeholders to advance a master-planned community.

The objectives of the current study were to:

- ☐ Review all available current information with regards to the existing Confederation Line and Tunney's Pasture Station, as well as the proposed Confederation Line extension located to the west of Tunney's Pasture Station, and
- ☐ Liaison between the City of Ottawa and the Arcadis consultant team involved with the aforementioned project.

This report has been prepared specifically and solely for the aforementioned project which is described herein. It contains a collaboration of civil and geotechnical design information as they pertain to the aforementioned project.

It should be noted that the design drawings for the proposed buildings were not available at the time of writing this report. The information provided herein is based on limited preliminary information provided by the client which notes the assumed worst-case scenario for works occurring near the Confederation Alignment. Approximate locations and elevations of existing Confederation Line Alignment were determined based on the Topographic Sketch of 1900-2000 City Park Drive, dated March 10, 2023, prepared by Annis, O'Sullivan, Volbekk Ltd (attached in Appendix A below).

2.0 Development Details

Based on the available conceptual plans, the proposed development at 1900 and 2000 City Park Drive will consist of 5 high-rise residential buildings with one or more underground parking levels. However, further details about the building design are not known at this time.

The existing Confederation Line alignment borders the southern side of the subject site. The following is known about the Confederation Line in the vicinity of the subject site.

- ❑ The Confederation Line runs parallel to the southern boundary of the site.
- ❑ From east to west across the site, the existing Confederation Line descends from a geodetic elevation of approximately 74.5 m, about 1.5 m above existing grade, to approximately 72.8 m, where it is generally level with existing grade. The existing Confederation line is located approximately 18 m away from the proposed building location and 14 m away from the subject site's adjacent property line.
- ❑ Bedrock is generally expected between 1.5 to 3.8 m below ground surface. The bedrock surface ranges from approximate geodetic elevations of 72.6 to 69.4 m. The upper 1 to 2 m of shale bedrock underlying the site is weathered and of very poor to poor quality, transitioning to good to excellent quality with depth.
- ❑ Based on the geological mapping the bedrock generally consist of shale of the Billings Formation with an overburden drift thickness of 2 to 5 m.

3.0 Construction Methodology and Impact Review

Paterson has prepared a construction methodology summary along with possible impacts on the adjacent segments of the Confederation Line, based on the preliminary design details. The Construction Methodology and Impact Review is provided in Appendix A and presents the anticipated construction items, impact review, and a mitigation program recommended for the Confederation Line. One of the main issues will be vibrations associated with the overburden soils removal and potential bedrock blasting removal program. It should be further noted that the upper 3 m of the shale bedrock is expected to be easily removed using hydraulic shovel equipment and blasting will not be required. If excavation extends below 3 m into the bedrock, then blasting to facilitate bedrock removal could be required. It is recommended that a vibration monitoring program be implemented to ensure the vibration levels remain below recommended tolerances. Details of a recommended vibration monitoring program are presented below.

3.1 Vibration Monitoring and Control Program

Due to the presence of the Confederation Line, the contractor should take extra precautions to minimize vibrations. The vibration monitoring program will be required for the duration of the blasting operations and any other construction activities which are anticipated to induce significant vibrations.

The purpose of the Vibration Monitoring and Control Program (VMCP) is to provide a description of the measures to be implemented by the contractor to manage excavation operations, and any other vibration sources during the construction for the proposed works. The VMCP will also provide a guideline for assessing results against the relevant vibration impact assessment criteria and recommendations to meet the required limits.

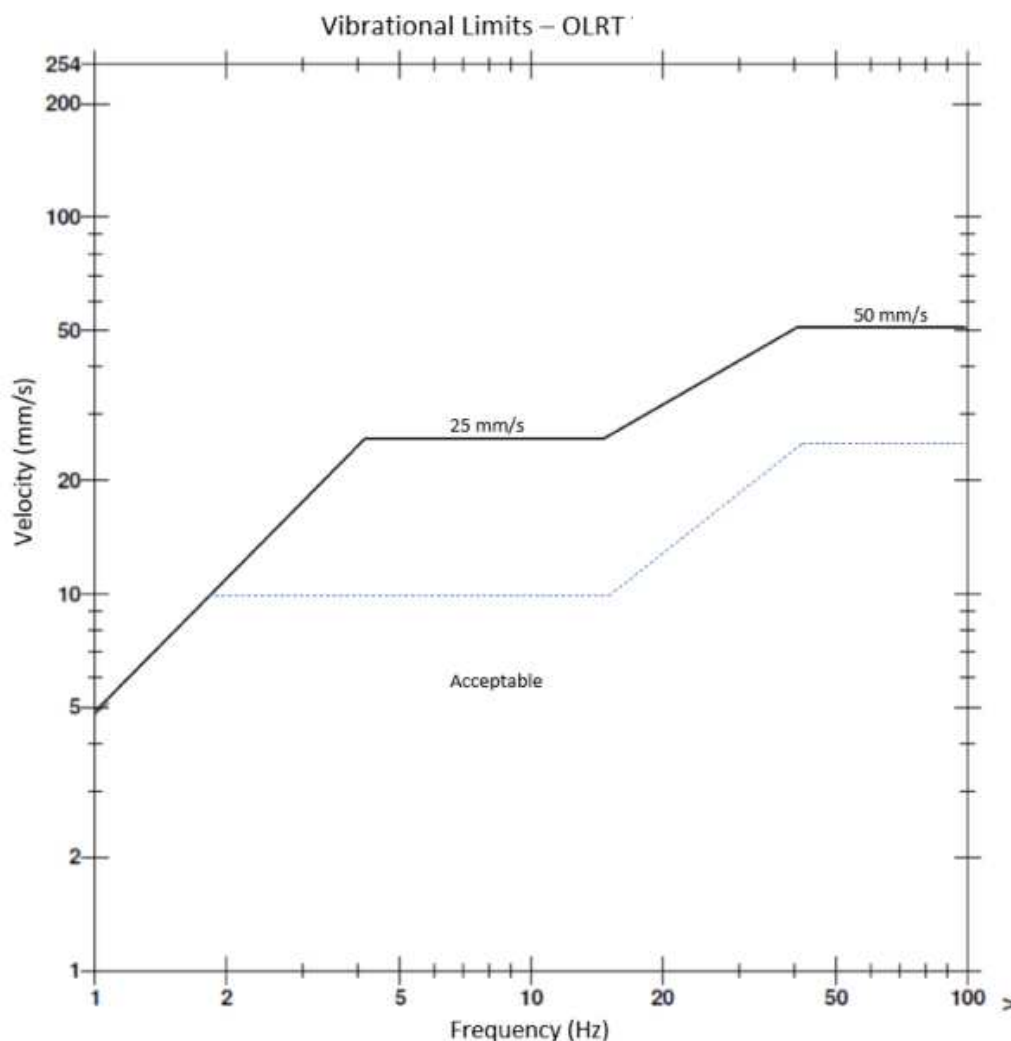
The monitoring program will incorporate real time results at the existing Confederation Line structure, which is located to the south of the subject site. The monitoring equipment should consist of a tri-axial seismograph, capable of measuring vibration intensities up to 254 mm/s at a frequency response of 2 to 250 Hz. The monitoring equipment should be placed along the southern boundary of the site and along the alignment of the existing Confederation Line route.

The location of the seismograph should be reviewed periodically throughout construction to ensure that the monitoring equipment remains with the rail line structure at the closest radius to the construction activities. The vibration monitoring locations should be approved by the project manager prior to installation.

During construction, the vibration monitor will be relocated for the 'worst case' location for each construction activity. When an event is triggered, Paterson will review the results and provide any necessary feedback. Otherwise, the vibration results will be summarized in the weekly report.

Proposed Vibration Limits

The following figure outlines the recommended vibration limits for the Confederation Line railway.



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

Monitoring Data

The monitoring protocol should include the following information:

Warning Level Event

- ☐ Paterson will review all vibrations over the established warning level, illustrated by the blue line in the above figure, and;
- ☐ Paterson will notify the contractor if any vibrations occur due to construction activities and are close to exceedance level.

Exceedance Level Event

- ☐ Paterson will notify all the relevant stakeholders via email if any vibrations surpass the exceedance level, illustrated by the black line in the above figure,
- ☐ Ensure monitors are functioning, and;
- ☐ Issue the vibration exceedance result.

The data collected will include the following:

- ☐ Measured vibration levels,
- ☐ Distance from the construction activity to monitoring location, and;
- ☐ Vibration type.

Monitoring should be compliant with all related regulations.

3.2 Incident/Exceedance Reporting

In case an incident/exceedance occurs from construction activities, the Senior Project Management and any relevant personnel should be notified immediately. A report should be completed which contains the following:

- ☐ Identify the location of vibration exceedance,
- ☐ The date, time and nature of the exceedance/incident,
- ☐ Purpose of the exceeded monitor and current vibration criteria,
- ☐ Identify the likely cause of the exceedance/incident,
- ☐ Describe the response action that has been completed to date, and;
- ☐ Describe the proposed measures to address the exceedance/incident.

The contractor should implement mitigation measures for future excavation or any construction activities as necessary and provide updates on the effectiveness of the improvement.

Response actions should be pre-determined prior to excavation, depending on the approach provided to protect elements. Processes and procedures should be in-place prior to completing any vibrations to identify issues and react in a quick manner in the event of an exceedance.

4.0 Proximity Study Requirement Responses

Paterson was informed by the City of Ottawa that a Level 1 Confederation Line Proximity Study should be completed for the proposed residential development at 2000 City Park Drive. A Level 1 Confederation Line Proximity Study is required where the proposed development is located within the City of Ottawa's Development Zone of Influence.

The following table lists the applicable requirements for Level 1 study and the response location for each item:

Table 1 - List of Confederation Line Proximity Study Requirements	
Level 1 Projects	Response
A site plan of the development with the centreline or reference line of the Confederation Line structure and/or right-of-way located and the relevant distances between the Confederation Line and developer's structure shown clearly;	See Confederation Line Proximity Plan (Drawing No. PG6552-3 dated August 2026) presented in Appendix A.
Plan and cross-sections of the development locating the Confederation Line structure/right-of-way and founding elevations relative to the development, including any underground storage tanks and associated piping;	Refer to the Confederation Line Proximity Plan (Drawing No. PG6552-3 dated August 2026), Cross-Section A-A' and Cross-Section B-B' (Drawing No. PG6552-4 and 5 dated August 2026) presented in Appendix A.
A geotechnical investigation report showing up-to-date geotechnical conditions at the site of the development. The geotechnical investigation shall be prepared in accordance with the Geotechnical Investigation and Reporting Guidelines for Development Applications in the City;	Refer to Geotechnical Investigation Report Prepared by Paterson Group. Report No. PG6552-1 dated July 17, 2026, presented in Appendix B.

Structural, foundation, excavation and shoring drawings;	Structural, foundation, and excavation plans will be provided prior to the Site Plan Agreement. Based on the preliminary information, one (1) or more underground parking levels are expected for the proposed buildings, therefore, the foundations are expected to be on a sound bedrock bearing surface. A minimum horizontal separation of approximately 18.5 m is available between the 2 structures. Therefore, no negative impacts are anticipated for the Confederation Line due to the proposed development.
Acknowledgment that the potential for noise, vibration, electro-magnetic interference and stray current from Confederation Line operations have been considered in the design of the project, and appropriate mitigation measures applied.	Refer to the Roadway Transportation Noise and Vibration Feasibility Assessment Report No. 23-075 prepared by Gradient Wind Engineers & Scientists dated August 17, 2026, presented in Appendix C.

We trust that this information satisfies your immediate request.

Best Regards,

Paterson Group Inc.



Pratheep Thirumoolan, M.Eng., P.Eng.




David Gilbert, P.Eng.

APPENDIX A

Confederation Line Proximity Plan

Cross Section A-A'

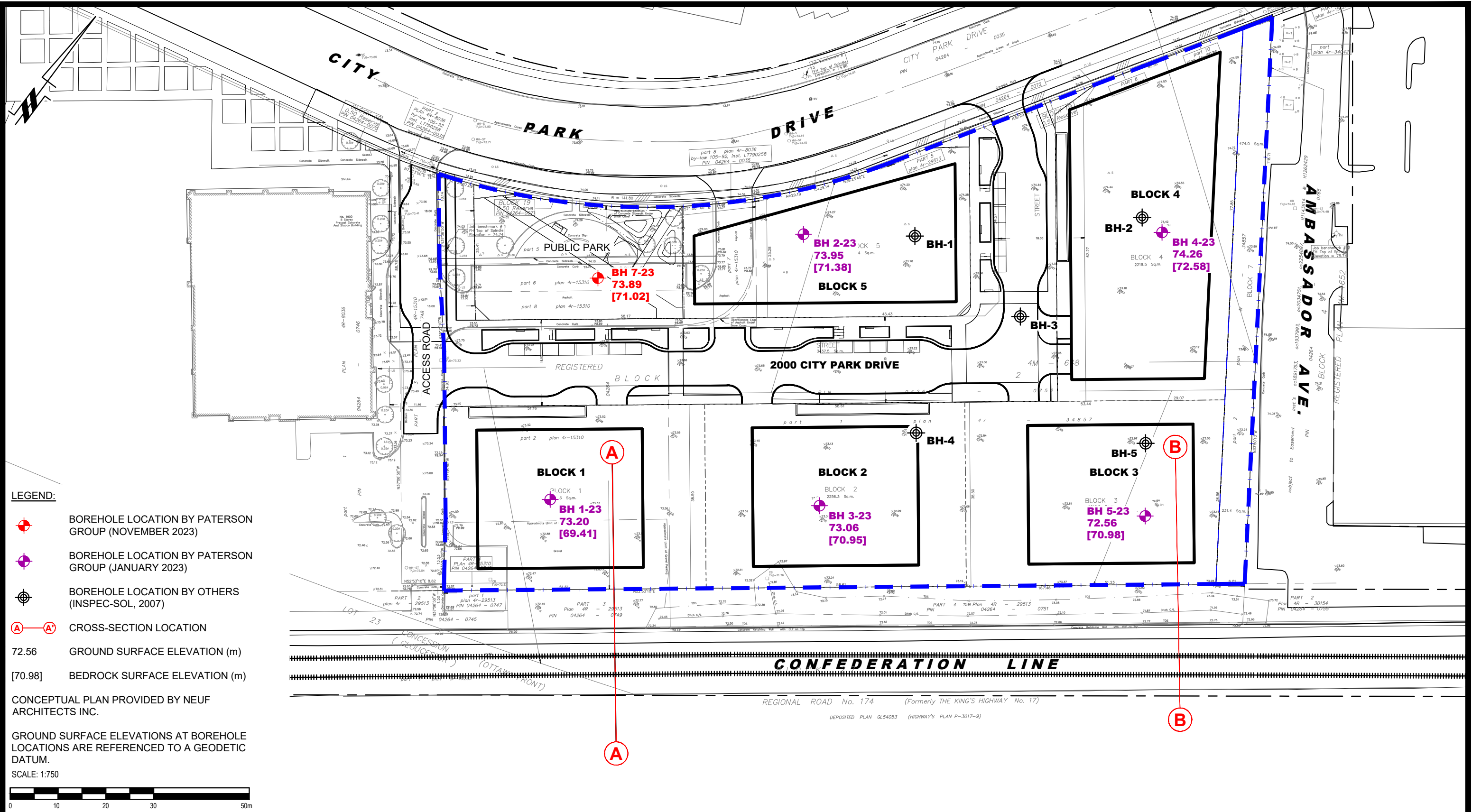
Cross Section B-B'

Topographic Survey Plan

Subdivision Site Plan

Lot Division Plan

Construction Methodology and Impact Review



LEGEND:

- BOREHOLE LOCATION BY PATERSON GROUP (NOVEMBER 2023)
- BOREHOLE LOCATION BY PATERSON GROUP (JANUARY 2023)
- BOREHOLE LOCATION BY OTHERS (INSPIC-SOL, 2007)
- CROSS-SECTION LOCATION
- 72.56 GROUND SURFACE ELEVATION (m)
- [70.98] BEDROCK SURFACE ELEVATION (m)

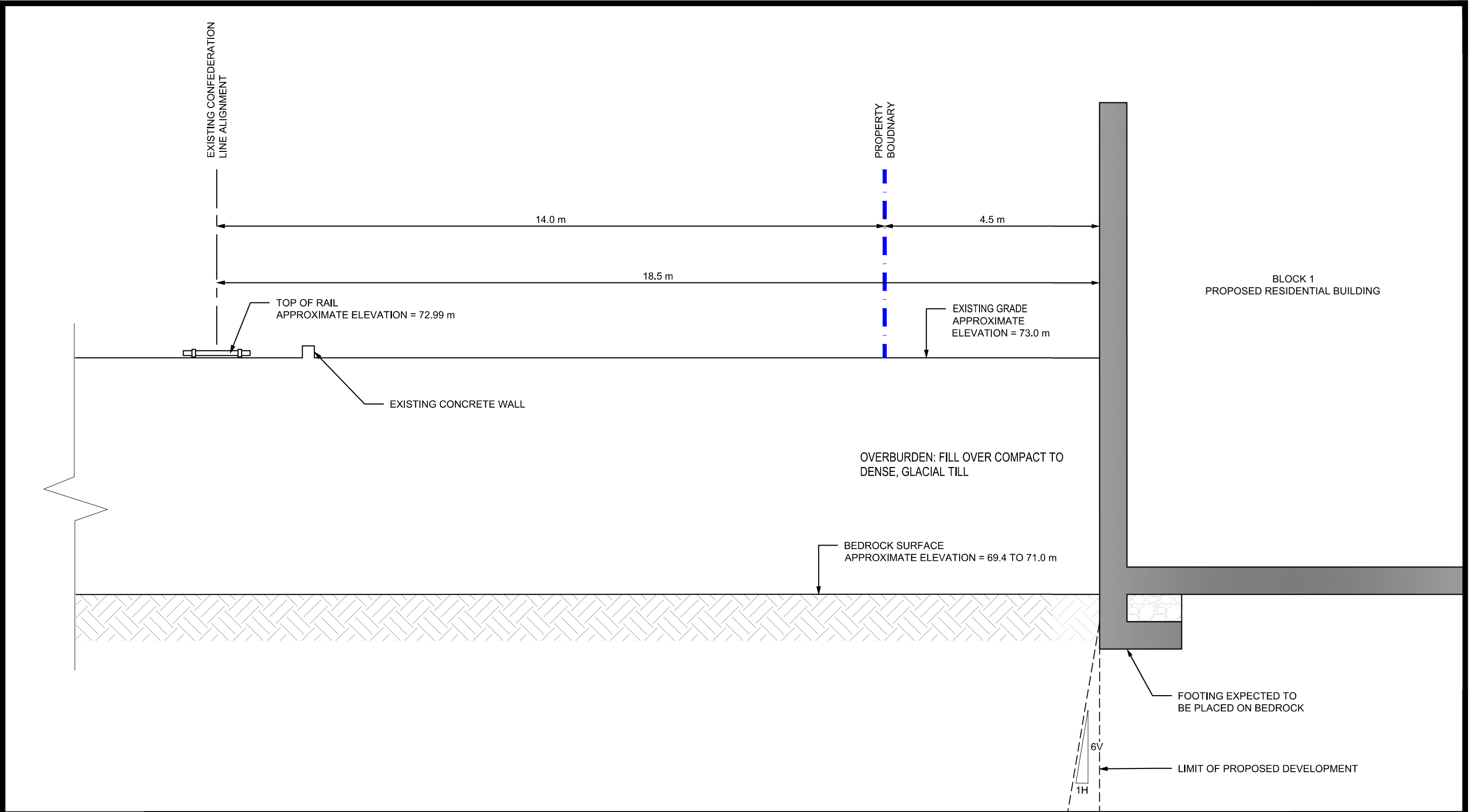
CONCEPTUAL PLAN PROVIDED BY NEUF ARCHITECTS INC.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

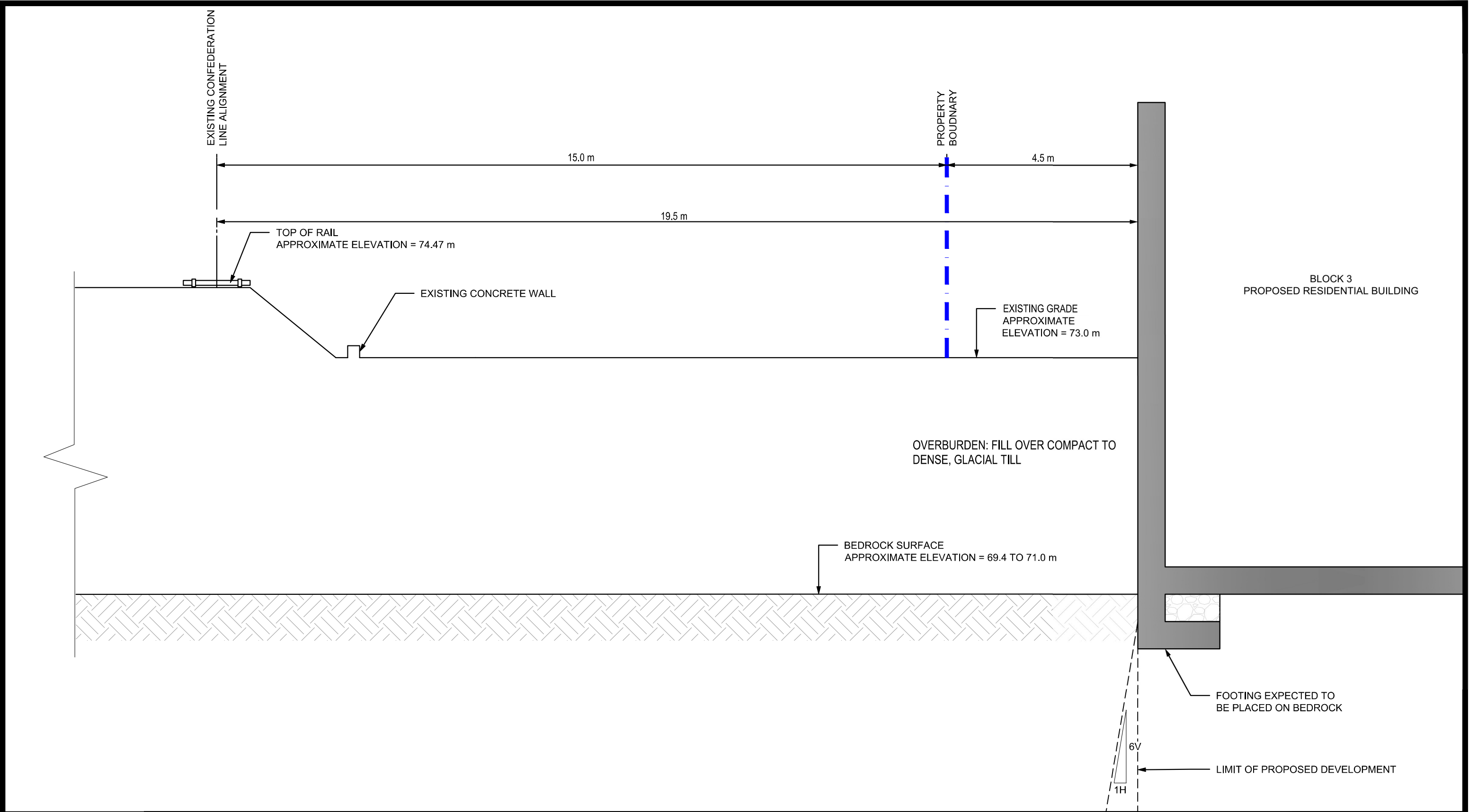
SCALE: 1:750



 PATERSON GROUP 9 AURIGA DRIVE OTTAWA, ON K2E 7T9 TEL: (613) 226-7381					COLONNADE BRIDGEPORT GEOTECHNICAL INVESTIGATION PROPOSED HIGH-RISE BUILDINGS 2000 CITY PARK DRIVE		Scale: 1:750	Date: 08/2026
					OTTAWA,	ONTARIO	Drawn by: ZS	Report No.: PG6552-3
					Title:		Checked by: PT	Dwg. No.: PG6552-3
					CONFEDERATION LINE PROXIMITY PLAN		Approved by: FA	Revision No.:
	NO.	REVISIONS	DD/MM/YYYY	INITIAL				



 9 AURIGA DRIVE OTTAWA, ON K2E 7T9 TEL: (613) 226-7381					OTTAWA, Title:	COLONNADE BRIDGEPORT GEOTECHNICAL INVESTIGATION PROPOSED HIGH-RISE BUILDINGS 2000 CITY PARK DRIVE ONTARIO	Scale:	N.T.S.	Date:	08/2026
							Drawn by:	ZS	Report No.:	PG6552-3
							Checked by:	PT	Dwg. No.: PG6552-4	
							Approved by:	FA		Revision No.:



 9 AURIGA DRIVE OTTAWA, ON K2E 7T9 TEL: (613) 226-7381					OTTAWA, Title:	COLONNADE BRIDGEPORT GEOTECHNICAL INVESTIGATION PROPOSED HIGH-RISE BUILDINGS 2000 CITY PARK DRIVE ONTARIO	Scale:	N.T.S.	Date:	08/2026		
							Drawn by:	ZS	Report No.:	PG6552-3		
							Checked by:	PT	Dwg. No.: PG6552-5			
							Approved by:	FA		Revision No.:		
NO.	REVISIONS	DD/MM/YYYY	INITIAL									

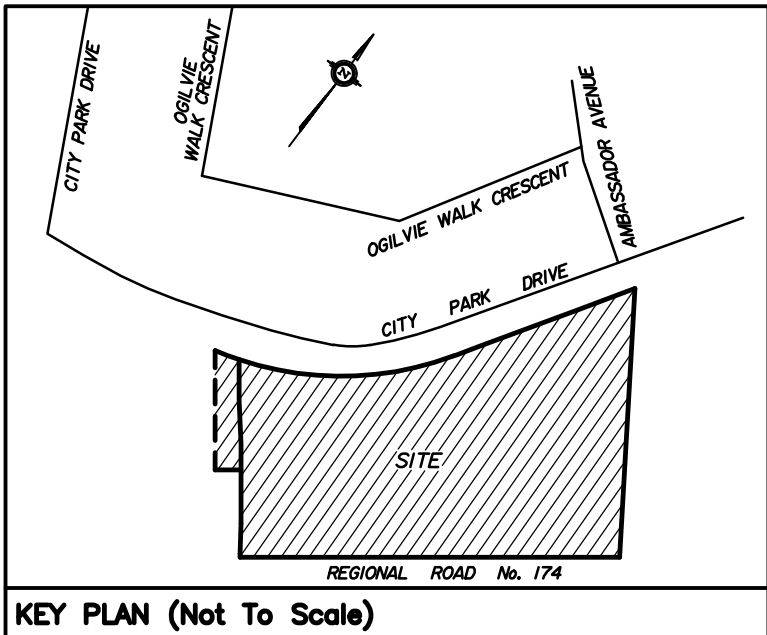
Bearings are grid, derived from Can-Net 2016 Real Time Network
GPS observations, MTM Zone 9 (78°30' West Longitude) NAD-83
(original).

ELEVATION NOTES

1. Elevations shown are geodetic and are referred to the CGVD28 geodetic datum, derived from control monument No. 019680132 having an elevation of 66.127 metres.
2. It is the responsibility of the user of this information to verify that the job benchmark has not been altered or disturbed and that it's relative elevation and description agrees with the information shown on this drawing.

SUBJECT TO CONDITIONS, IF ANY,
SET FORTH IN OUR LETTER DATED
_____, 2026, THIS
DRAFT PLAN IS APPROVED BY THE CITY OF OTTAWA UNDER
SECTION 9 OF THE CONDOMINIUM ACT
AND SECTION 51 OF THE PLANNING ACT
THIS _____ DAY OF _____, 2026.

JOHN SEVIGNY, MCIP, RPP, ACTING MANAGER
DEVELOPMENT REVIEW-EAST
PLANNING, DEVELOPMENT AND BUILDING SERVICES
DEPARTMENT, CITY OF OTTAWA



DRAFT PLAN OF SUBDIVISION OF

PART OF BLOCK 2
REGISTERED PLAN 4M-648
CITY OF OTTAWA

Prepared by Annis, O'Sullivan, Vollebakk Ltd

Scale 1 : 250
10 7.5 5.0 2.5 0 5 10 Metres

Metric
DISTANCES AND COORDINATES SHOWN ON THIS PLAN
ARE IN METRES AND CAN BE CONVERTED TO FEET BY
DIVIDING BY 0.3048

Surveyor's Certificate

I CERTIFY THAT the boundaries of the Lands to be subdivided
and their relationship to the adjacent lands are accurately and
correctly shown.

Date _____
E. H. Herweyer
Ontario Land Surveyor

Owner's Certificate

This is to certify that I am the owners / agent of the lands to be
subdivided and that this plan was prepared in accordance my
instructions.

Date _____ XXXXXXX XXXXXXX (Authorized Signing Officer)
I have the authority to bind the corporation.

Owner's Certificate

This is to certify that I am the owners / agent of the lands to be
subdivided and that this plan was prepared in accordance my
instructions.

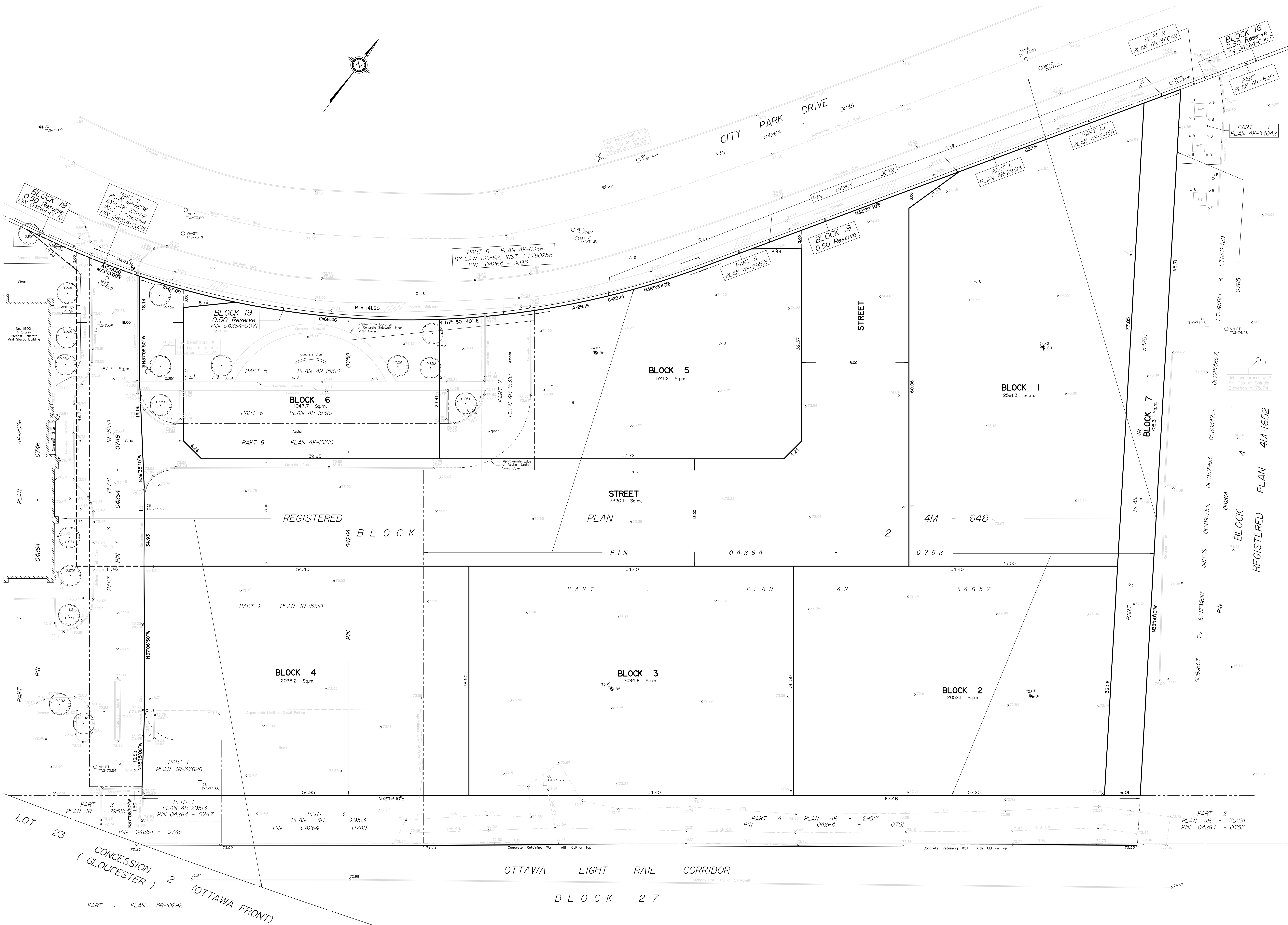
Date _____ XXXXXXX XXXXXXX (Authorized Signing Officer)
I have the authority to bind the corporation.

ADDITIONAL INFORMATION REQUIRED UNDER
SECTION 51-17 OF THE PLANNING ACT

- (a) see plan
- (b) see plan
- (c) see plan
- (d) see plan
- (e) see plan
- (f) see plan
- (g) see plan
- (h) municipal water available
- (i) see soils report
- (j) see plan
- (k) municipal services available
- (l) none known

Notes & Legend

- Denotes
- Deciduous Tree
 - Fire Hydrant
 - Water Valve
 - Maintenance Hole (Storm Sewer)
 - Maintenance Hole (Sanitary)
 - Maintenance Hole (Hydro)
 - Valve Chamber (Watermain)
 - Catch Basin
 - Light Standard
 - Hydro Transformer
 - Top of Slope
 - Centreline
 - Diameter
 - Location of Elevations
 - Top of Concrete Curb / Retaining Wall Elevation
 - Bollard
 - Sign
 - Borehole
 - Water Stand Post
 - Chain Link Fence



REGIONAL ROAD No. 174 (Formerly THE KING'S HIGHWAY No. 17)

DEPOSITED PLAN 6L54053 (HIGHWAY'S PLAN P-3017-9)

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GENERAL NOTES

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- The architect must be notified of any errors, omissions and discrepancies between these documents and those of the other professionals.
- The dimensions on these documents must be read and not measured.

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Company Address
Telephone and company website

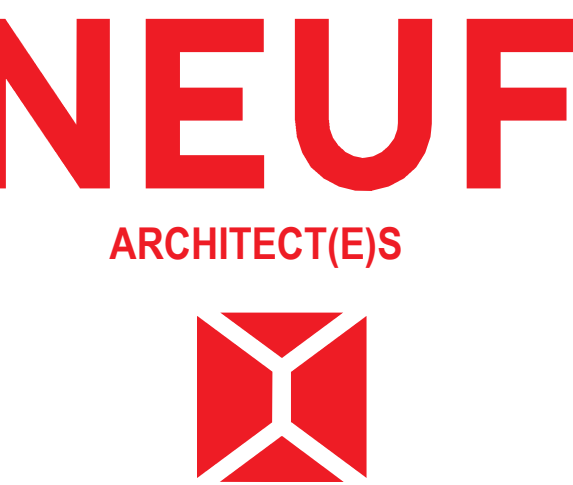
CIVIL

Company name
Company Address
Telephone and company website

ARCHITECTS

NEUF ARCHITECTS INC.
10, rue Robitaille, 4e étage, Ottawa, ON K1N 5J1
T 613 234 2274 | www.neuf.ca

SEAL



Project Name

2000 CITY PARK - LOT A

Location
OTTAWA, ON

PROJECT No.
13048

NO	REVISION	DATE (see-mm-jj)

Drawn by
RD

Checked by
CI

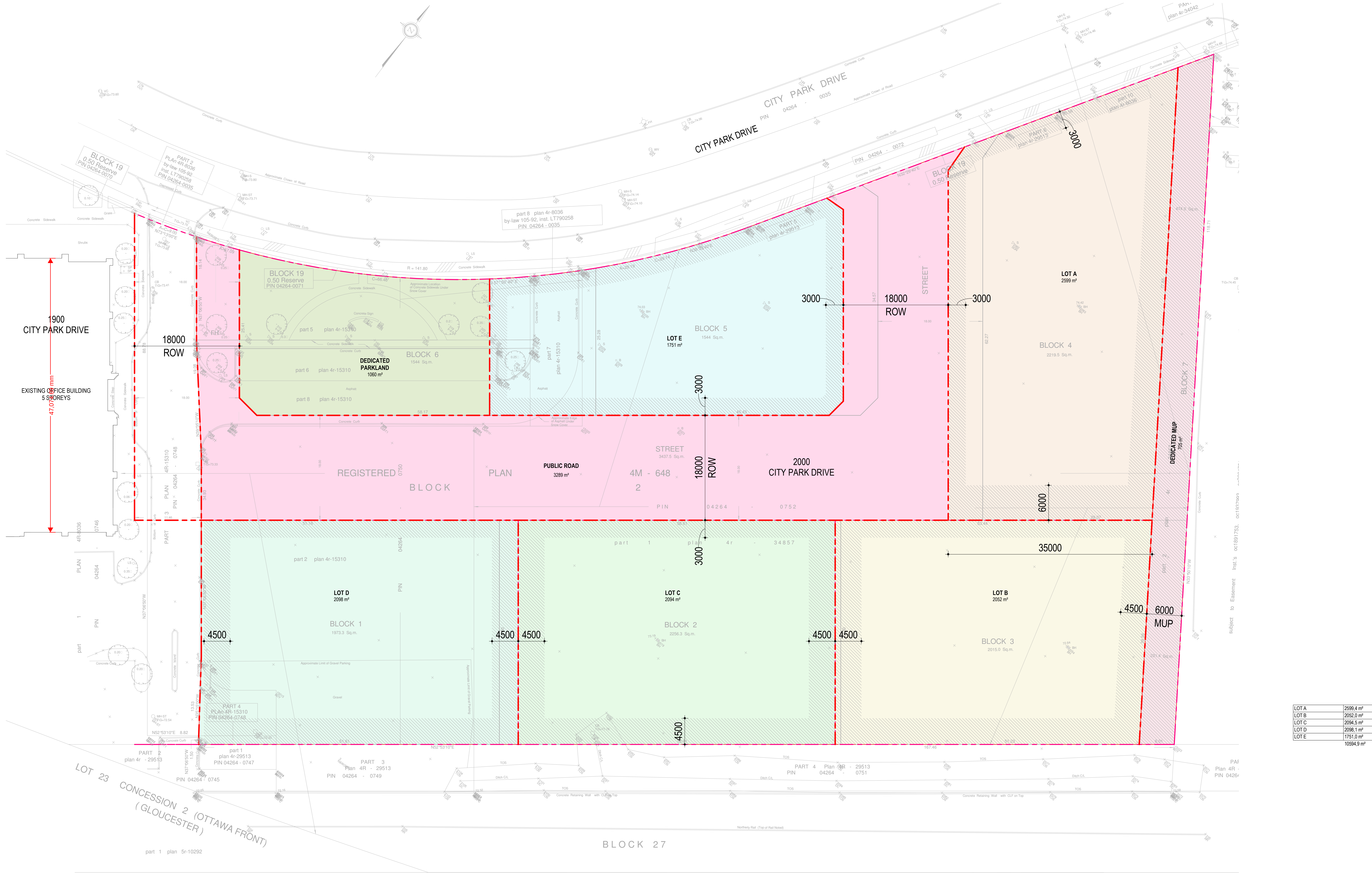
DATE (see-mm-jj)
2026.06.24

Scale
1 : 200

Drawing Title
SUBDIVISION SITE PLAN

Revision
A100

Dwg Number
A100



LOT A	2599.4 m²
LOT B	2552.0 m²
LOT C	2594.5 m²
LOT D	2598.1 m²
LOT E	1751.0 m²
	10594.9 m²

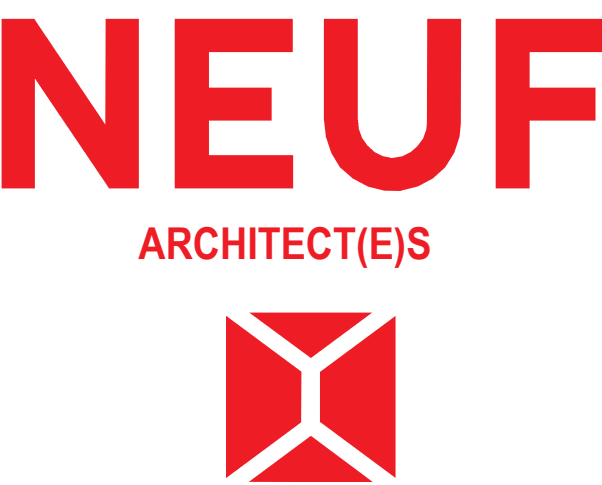
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 - The dimensions on these documents must be read and not measured.

LANDSCAPE ARCHITECT
Company Name
Company Address
Telephone and company website

CIVIL
Company name
Company Address
Telephone and company website

ARCHITECTS
NEUF architect(e)s
630, boul. René-Lévesque O. 32e étage, Montréal QC H3B 1S6
T 514 867 1117 www.NEUF.ca

SEAL



Project Name
13048 - 2000 CITY PARK

Location
Adresse / Address

PROJECT No
00000.00

NO REVISION DATE (see-mm-jj)

Drawn by
Author

Checked by
Checker

DATE (see-mm-jj)
2025.11.14

Scale
1:

Drawing Title
LOT DIVISION PLAN

Revision
16

Draw Number

Construction Methodology and Impact Review		
Construction Item	Potential Impact	Mitigation Program
Item A - Installation of Temporary Shoring System - Where adequate space is not available for the overburden to be sloped, the overburden along the perimeter of the excavation will need to be shored in order to complete the construction works. The shoring system is anticipated to consist of a soldier pile and lagging system.	Vibration issues during shoring system installation.	A temporary shoring system will be required along the perimeter of the proposed building footprints to support the overburden soils and the upper poor quality bedrock, during the construction of the underground parking levels. The temporary shoring system may to consist of a soldier pile and lagging system. It should be noted that the temporary shoring system should be designed for at-rest earth pressures as per geotechnical design recommendations outlined in the Geotechnical Investigation (Report No. PG6552-2 dated July 17, 2026, prepared by Paterson Group). Based on the available setback between the proposed building perimeter and the existing Confederation Line alignment, the installation of the temporary shoring system is not anticipated to have an adverse impact on the Confederation Line, nonetheless, a vibration monitoring device is recommended to be installed to monitor vibrations. The vibration monitor would be remotely connected to permit real-time monitoring and a vibration monitoring program would be implemented as detailed in Subsection 3.1 - Vibration Monitoring and Control Program of Paterson Group Report PG6552-3 dated September 3, 2026.
Item B - Bedrock Blasting and Removal Program - Blasting of the bedrock will be required for the proposed works. It is expected that bedrock removal is required based on the preliminary design information.	Structural damage of Confederation Line due to vibrations from blasting program.	Based on the available information, bedrock blasting may be required to complete the excavation in bedrock. The blasting operations must be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant. Due to the horizontal separation, structural damage to the existing Confederation Line during bedrock blasting and removal is not anticipated. It should be further noted that the upper 3 m of the shale bedrock is expected to be easily removed using hydraulic shovel equipment and blasting will not be required. If excavation extends below 3 m into the bedrock, then blasting to facilitate bedrock removal could be required. Nonetheless, a vibration monitoring device is recommended to be installed within the rail alignment in order to monitor vibrations. The vibration monitor would be remotely connected to permit real time monitoring and a vibration monitoring program would be implemented as detailed in Subsection 3.1 - Vibration Monitoring and Control Program of Paterson Group Report PG6552-3 dated September 3, 2026.
Item C - Construction of Footings and Foundation Walls - The proposed residential building will include some underground parking levels. Therefore, the footings are expected to be placed on a sound bedrock surface.	Building footing loading on the existing Confederation Rail Line and excavation within the lateral support zone of the Rail Lines.	Based on the preliminary information, one (1) or more underground parking levels are expected for the proposed buildings, therefore, the foundations are expected to be on a sound bedrock bearing surface, within the good- to excellent-quality shale bedrock. Accordingly, lateral loads from the building footings will be transferred directly to the bedrock within a conservative 1H:6V zone of influence measured from the outside face of each footing. Furthermore, a minimum horizontal separation of approximately 18.5 m is expected between the proposed building and the existing rail structure. Therefore, the building excavation will not impact the lateral support zone of the Rail Line.

APPENDIX B

Geotechnical Investigation Report:

Prepared By Paterson Group

Report No. PG6552-2 dated July 17, 2026

Geotechnical Investigation

Proposed High-Rise Buildings

2000 City Park Drive
Ottawa, Ontario

Prepared for Colonnade BridgePort

Report PG6552-2 dated July 17, 2026

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- Appendix 1** Soil Profile and Test Data Sheets
 Symbols and Terms
 Borehole Logs by Others
 Analytical Testing Results
- Appendix 2** Figure 1 – Key Plan
 Figure 2 – Podium Deck to Foundation Wall Drainage System
 Tie-In Detail
 Drawing PG6552-2 – Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Colonnade BridgePort to undertake a geotechnical investigation for the proposed high-rise buildings to be located at 2000 City Park Drive in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

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

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

2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed development at 2000 City Park Drive will consist of five (5) high-rise residential buildings. Block 4 (Tower A) is proposed as a 17-storey building, while Blocks 1, 2, and 3 are each proposed as 30-storey buildings. Block 5 is anticipated to be a high-rise building with fewer than 17 storeys; however, the final building height for Block 5 and design details were not available at the time of preparation of this report.

At finished grades, the proposed buildings will be surrounded by asphalt-paved access lanes and landscaped open spaces. A public park is proposed within the north-western portion of the 2000 City Park Drive parcel. The proposed development is expected to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out between January 17 and 19, 2023 (BH 1-23 to BH 5-23), and on November 3, 2023 (BH 7-23), and consisted of a total of six (6) boreholes advanced to a maximum depth of 12.4 m below existing grade, including coring bedrock. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground services and available access.

A previous geotechnical investigation was conducted at the subject site by others on July 30 and 31, 2007, consisting of five (5) boreholes advanced to a maximum depth of 4.8 m below the existing ground surface.

The locations of the boreholes are shown on Drawing PG6552-2 – Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low-clearance track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering and rock coring to the required depths at the selected borehole locations, and sampling and testing the soil and bedrock.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Bedrock samples were recovered from all boreholes using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the Soil Profile and Test Data Sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the bedrock quality.

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

Groundwater

A standpipe piezometer was installed in each borehole upon the completion of drilling and sampling, in order to permit monitoring of the groundwater levels. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a high-precision handheld GPS with respect to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG6552-2 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples and bedrock cores were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples from the November 2023 investigation will be stored in the laboratory for one (1) month after this report is completed. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and has a grass surface, with localized areas of mature trees.

The site is bordered by City Park Drive to the north, a commercial property to the west, the Confederation Line and Queensway (Highway 417) to the south, and a high-rise residential development to the east.

The site is relatively level and generally at grade with the adjacent roadways and surrounding properties. The existing ground surface across the subject site ranges from approximately geodetic Elevation 72.6 m to 74.3 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface condition at the subject site consists of a surficial layer of topsoil or asphalt which is underlain by fill and glacial till. The fill was generally observed to consist of silty sand to silty clay with gravel, crushed stone, brick, and organics, extending to approximate depths varying from 0.2 to 1.8 m below the existing ground surface.

A glacial till deposit was generally encountered underlying the fill, consisting of undisturbed, compact to very dense, brown silty sand with gravel and shale fragments.

Bedrock

Bedrock was encountered at depths ranging from 1.5 to 3.8 m and was cored at all borehole locations. Based on the recovered rock core, the bedrock was observed to consist of black shale, with the upper 1 to 2 m of the bedrock being generally weathered and very poor to poor quality, becoming good to excellent in quality with depth. The bedrock was cored to a maximum depth of about 12.4 m below the existing grade. In addition, based on available geological mapping, the bedrock consists of shale of the Billings Formation and is expected to be encountered at depths ranging from 2 to 5 m.

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

4.3 Groundwater

The groundwater levels were measured in piezometers on January 31, 2023 for boreholes BH 1-23 through BH 5-23, and on November 13, 2023, for borehole BH 7-23. The observed groundwater levels are summarized in Table 1 below.

Table 1 – Summary of Groundwater Level Readings				
Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-23	73.20	2.39	70.81	Jan. 31, 2023
BH 2-23	73.95	5.15	68.80	Jan. 31, 2023
BH 3-23	73.06	3.83	69.23	Jan. 31, 2023
BH 4-23	74.26	5.00	69.26	Jan. 31, 2023
BH 5-23	72.56	3.43	69.13	Jan. 31, 2023
BH 7-23	73.89	3.81	70.08	Nov. 13, 2023
Note: Ground surface elevations at borehole locations were surveyed by Paterson and are referenced to a geodetic datum.				

It should be noted that surface water can become trapped within a backfilled borehole, which can lead to higher than typical groundwater level observations.

The long-term groundwater level can also be estimated based on the observed colour, moisture content and consistency of the recovered samples. Based on these observations, the long-term groundwater level is expected to range between approximately **3.5 to 4.5 m** below ground surface.

However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed buildings are recommended to be founded on conventional spread footings bearing on the clean, surface sounded shale bedrock.

Bedrock removal will be required to complete the underground parking levels. Further, expansive shale bedrock could be present on site, and precautions should be provided during construction to reduce the risks associated with the potentially heaving shale bedrock.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious material, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Due to the anticipated founding level for the proposed buildings, all existing overburden material will be excavated from within the proposed building footprints. Bedrock removal will be required for the construction of the underground parking levels.

Fill Placement

Engineered fill placed for grading beneath the proposed buildings, where required, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with 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 the proximity of the blasting operations should be carried out 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 or claims related to the blasting operations.

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

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations 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).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Bedrock Excavation Face Reinforcement and Preparation

Bedrock excavation face reinforcement methods, such as the use of horizontal rock anchors and rock wedges/bolts in conjunction with shotcrete and/or chain link fencing with a layer of woven geotextile connected to the excavation face is expected to be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. Further, shotcrete and/or other material may be required to in-fill areas where bedrock pop-outs occur due to the nature of bedrock removal process throughout the excavation footprint and in advance of the placement of foundation waterproofing products.

The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the excavation operations. As a preliminary recommendation, provisions should be carried out for providing a minimum 1 m wide bedrock face protection layer across building excavation footprint perimeters for all portions of the excavations that will extend below the bedrock surface. Throughout the building excavation and bedrock removal process, the vertical bedrock excavation perimeter surfaces should be hoe-rammed and grinded smooth to provide a relatively flat substrate surface for the placement of the drainage board. All loose bedrock fragments should be removed by grinding operations.

It is recommended that Paterson review the bedrock excavation program at the time of construction to evaluate and select the recommended locations for reinforcement, if any are present.

Overbreak in Bedrock

Sedimentary bedrock formations, such as sandstone, limestone, dolomite, and shale, contain bedding planes, joints and fractures, and mud seams which create natural planes of weakness within the rock mass. Although several factors of a blast may be controlled to reduce backbreak and overbreak, upon blasting, the rock mass will tend to break along natural planes of weakness that may be present beyond the designed blast profile. However, estimating the exact amount of backbreak and overbreak that may occur is not possible with conventional construction drill and blast methods.

Backbreak should be expected to occur along the perimeter of the building excavation footprint with conventional drill and blast bedrock removal methods. Further, overbreak is expected to occur throughout the lowest lifts of blasting due to the variable bedding planes and planes of weakness in the in-situ bedrock. It is very difficult to mitigate significant overblasting given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts and the bedrocks in-situ characteristics.

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak may add significant time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. It is recommended that the blasting operations be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

It is highly recommended that the contractor/owner engage Paterson early into the excavation activities to identify anomalies within the bedrock. Pre-blast surveys and vibration monitoring is also very essential at this stage, which can be completed by Paterson as well. Also, it is important to note that completing an as-built of bedrock removal, including overbreaks, is required to be completed by the contractor and reviewed by Paterson to avoid confusion on the reasons for overbreaking the rock and to ensure that all parties are aware of the site conditions. Paterson should receive all as-built surveys post removal of the bedrock.

Lean Concrete Filled Trenches

Where the proposed footings are to be founded on bedrock which is located below the underside of footing elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded shale bedrock, and backfilled with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Footings supported directly on clean, surface sounded shale bedrock, or on lean concrete which is placed directly on clean, surface sounded shale bedrock, can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **3,000 kPa**. A reduced bearing resistance value at Ultimate Limit States (ULS) of **750 kPa** is recommended for areas where the bedrock is found to be weathered. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS. Strip and pad footings should have minimum plan dimensions of 0.6 and 1 m, respectively.

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.

Footings supported on clean, surface-sounded bedrock designed for the bearing resistance values provided herein will be subjected to negligible post-construction total and differential settlements.

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 horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) 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 shallower).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class X_c**. If a higher seismic site class (such as Class **X_B**) is required, then a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed buildings, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2024 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill from within the footprints of the proposed buildings, the bedrock will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

It is anticipated that the underground levels for the proposed building will be mostly parking, and the recommended pavement structure noted in Section 5.8 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 300 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. 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.

In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone under the lower basement floor of the proposed multi-storey building. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed buildings. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m³ (effective unit weight 13 kN/m³).

However, the majority of the basement walls of the proposed buildings are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face, for which a nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m³ (effective 15.5 kN/m³). Further, a seismic earth pressure component will not be applicable for the foundation walls which are poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Lateral Earth Pressures

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)
and 0.05 for rock
 γ = unit weight of fill of the applicable retained soil (kN/m³)
 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 Earth Pressures

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

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

$$a_c = (1.45 - a_{max}/g) a_{max}$$

γ = unit weight of fill of the applicable retained soil (kN/m³)

H = height of the wall (m)

g = gravity, 9.81 m/s²

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.364g according to OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5 K_o \cdot \gamma \cdot H^2$, 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 2024.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in shale bedrock is based upon two possible failure modes. The rock anchor can fail either by shear failure along the grout/rock interface or by pullout at 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. 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 individual anchor load capacity.

A third failure mode of shear failure along the grout/steel interface should 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 in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled, and grout fluid does not flow from one hole to an adjacent empty one.

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 servicing. To resist seismic uplift pressures, a passive rock anchor system is adequate. However, a post-tensioned anchor will absorb the uplift load pressure with less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a fixed anchor length at the anchor base, which will provide the anchor capacity, and a free anchor length between the rock surface and the top of the bonded length. As the depth at which the apex of the shear failure cone develops 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 sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International.

Grout to Rock Bond

Generally, the unconfined compressive strength of shale ranges between 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, should be provided. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. 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 in the following:

Table 2 – Parameters used in Rock Anchor Design	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Shale Bedrock Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Shale bedrock	40 MPa
Unit weight - Submerged Bedrock	15.5 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. 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 for vertical grouted rock anchors used to resist uplift forces. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

Table 3 – Recommended Rock Anchor Lengths - Grouted Rock Anchor				
Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	1.0	3.0	450
	2.6	1.0	3.6	600
	3.2	1.3	4.5	750
	4.5	2.0	6.5	1000
125	1.6	1.0	2.6	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Temporary Horizontal/ Inclined Rock Anchors

Horizontal or inclined rock anchors may be required at specific locations to prevent pop-outs and slippage of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface. The requirement for horizontal or inclined rock anchors will be evaluated during the excavation operations to provide a sufficient bedrock stabilization for workers safety during construction.

Bedrock Stabilization

A weathered bedrock layer was noted within the upper 2 m of the bedrock on site. It is therefore expected that bedrock stabilization and protection measures will be required on site. Those measures should consist of anchored chain-link fencing and geotextile placed on the exposed excavation face to ensure that small rock fragments do not fall onto workers in close proximity to the excavation walls.

Paterson will need to complete a full review of the excavation face during rock removal to review the stabilization requirements. At this point, given the known information about the project, the following recommendations are made for budgetary purposes. Final field requirements will depend on the exact bedrock conditions encountered during construction.

Bedrock grinding is expected for this site to allow installation of the proposed drainage and waterproofing system. Once completed, a review of the upper portion of the bedrock layer should be completed by Paterson to establish the stabilization measures required.

Bedrock stabilization shall consist of chain link fencing with a woven geotextile liner, such as Terratrack 200 or equivalent, placed over the upper weathered bedrock in order to stabilize any loose rock present. The top of the chain link fencing should be secured by a series of bolts extended into the top of the bedrock bench. The weathered layer of rock was noted to range from 1 to 2 m in depth. The chain link fencing should be anchored below the weathered layer in good-quality rock using a series of minimum 150 mm length rock wedge bolts spaced no greater than 1.5 m apart.

Rock bolts, such as swellex bolts or equivalents, might be required to stabilize seams or large blocks of bedrock. The anchor spacing and length should be established on site following the bedrock review. Table 4 provides the recommended anchor lengths for a 75 and 125 mm diameter hole into the bedrock face at approximately 25 degrees below the horizontal, which will further secure the chainlink fence in place. If the DYWDAG threaded bars are chosen, it is recommended that a minimum 32 MPa grout be used.

Table 4 - Recommended Inclined Rock Anchor Lengths for Bedrock Stabilization - Grouted Rock Anchor *				
Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.9	2.9	4.8	450
	2.6	3.2	5.8	600
	3.2	3.3	6.5	750
	3.8	3.4	7.2	900
125	1.0	3.4	4.4	450
	1.3	3.8	5.1	600
	1.6	4.1	5.7	750
	1.9	4.4	6.3	900
* Rock anchors are inclined approximately 25 degrees below the horizontal for bedrock stabilization and protection measures.				

Other Considerations

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.

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lowest underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 given below.

Table 5 – Recommended Rigid Pavement Structure – Underground Parking Level

Thickness (mm)	Material Description
150	Exposure Class C2 – 32 MPa Concrete (5 to 8% Air Entrainment)
300	BASE - OPSS Granular A Crushed Stone
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

The flexible pavement structure presented in Table 5 should be used for at grade access lanes and heavy loading parking areas.

Table 6 – Recommended Asphalt Pavement Structure – Access Lanes and Heavy Loading Parking Area

Thickness (mm)	Material Description
40	Wear Course - Superpave 12.5 Asphaltic Concrete
50	Binder Course - Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the concrete podium deck	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Groundwater Control for Construction

Foundation Drainage and Waterproofing

The following recommendations may be considered for the architectural design of the buildings' foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future building (and prior to tender) to review and provide supplemental information for the building foundation drainage system design.

Supplemental details, review of architectural design drawings, and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Waterproofing System

It is recommended that a waterproofing system be provided for the proposed structures at the subject site. It is expected that insufficient room will be available for exterior backfill, and the foundation wall will be cast as a blind-sided pour against a shoring system and the bedrock surface. It is recommended that the waterproofing system consists of the following:

- ❑ A composite drainage membrane (Delta Terraxx, MiraDrain G100N, or equivalent) should be placed against the shoring system or bedrock face with the geotextile layer facing the shoring system or bedrock face from the finished ground surface to the top of the footing.
- ❑ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.
- ❑ A waterproofing membrane should be placed against composite drainage boards between the top of footings and at least 1 m above the long-term groundwater elevation.

- ❑ It is recommended that 150 mm diameter PVC sleeves, at least two (2) per wall on each side (to be confirmed at the time of construction), be cast into the foundation wall at the foundation wall/footing interface to allow infiltrating water to flow to the interior perimeter drainage pipe. The sleeves should be punched through the waterproofing membrane, but not through the drainage board. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe network.

The top end lap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and the foundation wall.

Interior Perimeter and Underfloor Drainage

Depending on the type of foundation wall pour time (double sided or blind-sided), an interior or exterior perimeter drainage along with an underfloor drainage system will be required to redirect water from the building's foundation drainage system to the building's sump pit(s) if it will not discharge to an exterior catch basin structure. For preliminary design purposes, it is recommended that the interior perimeter and underfloor drainage pipes should consist of 100 or 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock, placed at approximately 6 m centers. The spacing can be reassessed at the time of construction once the water infiltration through the excavation sidewalls is further reviewed.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves.

The spacing of the underfloor drainage should be confirmed by Paterson at the time of excavation, when water infiltration can be better assessed and once the foundation layout and sump system location have been finalized.

Foundation Backfill (If Applicable)

Where applicable, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Terraxx, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Podium Deck Waterproofing Tie-In (If Applicable)

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall as depicted in Figure 2 – Podium Deck to Foundation Wall Drainage System Tie-in Detail.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the building should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for the proposed buildings at the subject site to provide a building-specific waterproofing and drainage design, which includes the above-noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double-sided pours (if applicable), etc.

6.2 Protection of Footings Against Frost Action

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

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

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden and poor-quality bedrock materials should either be cut back at acceptable slopes or should be retained by a temporary shoring system from the start of the excavation until the structure is backfilled.

Unsupported Excavations

The excavation side slopes in the overburden and poor-quality bedrock, above the groundwater level, and extending to a maximum depth of 3 m, should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress. It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Due to the anticipated depth of excavation of the building and the proximity of the proposed building to the site boundaries, temporary shoring may be required to support the overburden soils and poor quality bedrock of the adjacent properties.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the temporary shoring system or soils supported by the system. Any changes to the approved temporary shoring system design should be reported immediately to the owner's structural designer prior to implementation.

The temporary shoring system may consist of a soldier pipe and lagging system which could be cantilevered, anchored or braced.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressure acting on the shoring system may be calculated using the following parameters.

Table 7 – Soil Parameters	
Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight , kN/m ³	21
Submerged Unit Weight , kN/m ³	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater table.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Bedrock Stabilization

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.

Horizontal rock anchors and chainlink fencing or shotcrete are anticipated to be required over the upper 2 to 3 m of the vertical bedrock face, which generally consists of lower quality bedrock, in order to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

However, the specific requirements for bedrock stabilization measures should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the 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 a soil subgrade, or 300 mm of OPSS Granular A when placed on a bedrock subgrade. 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 98% of the SPMDD.

It is generally possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

Based on our observations, 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.

Groundwater Control for Building Construction

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three 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. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

Impacts on Neighbouring Properties

Given the relatively shallow bedrock present at and in the vicinity of the subject site, the neighbouring structures are expected to be founded on the bedrock surface. Therefore, no issues are expected with respect to groundwater lowering that would cause damage to adjacent structures surrounding the proposed development.

6.6 Winter Construction

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

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

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

6.8 Protection of Potentially Expansive Shale Bedrock

Upon being exposed to air and moisture, the shale may decompose into thin flakes along the bedding planes. Previous studies have concluded shales containing pyrite are subject to volume changes upon exposure to air. As a result, the formation of jarosite crystals by aerobic bacteria occurs under certain ambient conditions.

It has been determined that the expansion process does not occur or can be retarded when air (i.e. oxygen) is prevented from contact with the shale and/or the ambient temperature is maintained below 20°C, and/or the shale is confined by pressures in excess of 70 kPa. The latter restriction on the heaving process is probably the major reason why damage to structures has, for the greater part, been confined to slabs-on-grade rather than footings.

The presence of expansive shale may be encountered at the subject site. To reduce the long term deterioration of the shale, exposure of the bedrock bearing surface to oxygen should be kept as low as possible. The bedrock bearing surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. A 50 to 75 mm thick concrete mud slab, consisting of minimum 15 MPa lean concrete, should be placed on the exposed bedrock bearing surface within a 48 hour period of being exposed. The excavated sides of the exposed bedrock should be sprayed with a bituminous emulsion or shotcrete to seal bedrock from exposure to air and dewatering.

Preventing the dewatering of the shale bedrock will also prevent the rapid deterioration and expansion of the shale bedrock. This can be accomplished by spraying bituminous emulsion as noted above.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ☐ Review preliminary and detailed grading, servicing, landscaping, and structural plan(s) from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.
- ☐ Review of architectural plans pertaining to waterproofing systems, underfloor drainage systems, and waterproofing details for elevator shafts.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ☐ Review and inspection of the installation of the foundation drainage systems.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Observation of driving and re-striking of all pile foundations.
- ☐ Sampling and testing of the concrete and fill materials.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management***.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Colonnade BridgePort, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.



Yashar Ziaeimehr, M.A.Sc., P.Eng.



Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ☐ Colonnade BridgePort (e-mail copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

ANALYTICAL TESTING RESULTS

DATUM	Geodetic
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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE January 17, 2023

FILE NO.
PG6552

HOLE NO.
BH 1-23

[illegible]

[illegible]

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE January 18, 2023

FILE NO.
PG6552

HOLE NO.
BH 3-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
Ground Surface												
TOPSOIL	0.05	AU	1			0	73.06					
FILL: Brown silty clay, some sand, topsoil and gravel	0.91	SS	2	83	21	1	72.06					
GLACIAL TILL: Brown silty sand with gravel and shale fragments	2.11	SS	3	100	50+	2	71.06					
		SS	4	50	50+							
BEDROCK: Very poor quality, black shale	3.00	RC	1	100	0	3	70.06					
BEDROCK: Good to excellent quality, black shale		RC	2	100	81	4	69.06					
		RC	3	100	92	5	68.06					
		RC	4	100	76	6	67.06					
		RC	5	100	98	7	66.06					
		RC	6	100	80	8	65.06					
		RC	7	100	96	9	64.06					
						10	63.06					
						11	62.06					
						12	61.06					
End of Borehole	12.06											
(GWL @ 3.83m - Jan. 31, 2023)												
								20	40	60	80	100
								Shear Strength (kPa)				
								▲ Undisturbed △ Remoulded				

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE January 18, 2023

FILE NO.
PG6552

HOLE NO.
BH 4-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
Ground Surface												
TOPSOIL	0.08					0	74.26					
FILL: Brown silty sand with clay, gravel and rock fragments, some topsoil	1.07					1	73.26					
Very stiff, brown SILTY CLAY	1.45											
GLACIAL TILL: Brown silty sand with gravel and shale fragments	1.68					2	72.26					
BEDROCK: Very poor quality, black shale	3.33					3	71.26					
						4	70.26					
						5	69.26					
BEDROCK: Fair to excellent quality, black shale						6	68.26					
- vertical fracture from 6.6 to 6.8m depth												
						7	67.26					
- vertical fracture from 7.5 to 8.2m depth												
						8	66.26					
- vertical fracture from 8.7 to 9.0m depth												
						9	65.26					
						10	64.26					
						11	63.26					
						12	62.26					
End of Borehole	12.04											
(GWL @ 5.00m - Jan. 31, 2023)												
			</									

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE January 19, 2023

FILE NO.
PG6552

HOLE NO.
BH 5-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
Ground Surface						0	72.56					
FILL: Brown silty clay with sand, organics, trace gravel	0.23	SS	1	56	50+							
GLACIAL TILL: Very dense, brown silty sand with clay and shale fragments	1.58	SS	2	100	50+	1	71.56					
		SS	3	100	50+							
		RC	1	100	38	2	70.56					
						3	69.56					
		RC	2	100	69	4	68.56					
						5	67.56					
		RC	3	100	93							
						6	66.56					
		RC	4	100	69	7	65.56					
						8	64.56					
		RC	5	100	95							
						9	63.56					
		RC	6	100	92	10	62.56					
						11	61.56					
		RC	7	100	72							
						12	60.56					
End of Borehole	12.27											
(GWL @ 3.43m - Jan. 31, 2023)												
								20	40	60	80	100
								Shear Strength (kPa)				
								▲ Undisturbed △ Remoulded				

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 3, 2023

FILE NO.
PG6552

HOLE NO.
BH 7-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
Ground Surface												
Asphaltic concrete	0.05	SS	1	50	30	0	73.89					
FILL: Brown silty sand with gravel and crushed stone		SS	2	8	4	1	72.89					
	1.47	SS	3	50	22	2	71.89					
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, trace clay		SS	4	100	40							
	2.84	RC	1	100	29	3	70.89					
- grey by 2.6m depth		RC	2	100	75	4	69.89					
BEDROCK: Poor quality, black shale		RC	3	100	80	5	68.89					
- good to excellent quality by 3.15m depth		RC	4	100	90	6	67.89					
		RC	5	100	98	7	66.89					
- 25mm thick mud seam at 6.2m depth		RC	6	100	98	8	65.89					
		RC	7	100	91	9	64.89					
						10	63.89					
						11	62.89					
	12.12					12	61.89					
End of Borehole												
(GWL @ 3.81m - Nov. 13, 2023)												
								20	40	60	80	100
								Shear Strength (kPa)				
								▲ Undisturbed △ Remoulded				

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

Cc and Cu are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$

Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

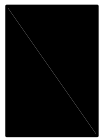
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.
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SYMBOLS AND TERMS (continued)

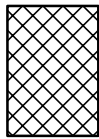
STRATA PLOT



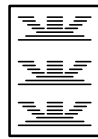
Topsoil



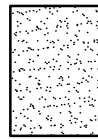
Asphalt



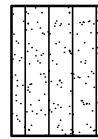
Fill



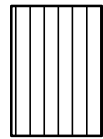
Peat



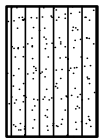
Sand



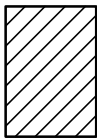
Silty Sand



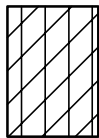
Silt



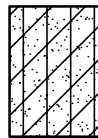
Sandy Silt



Clay



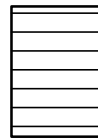
Silty Clay



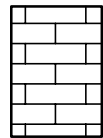
Clayey Silty Sand



Glacial Till



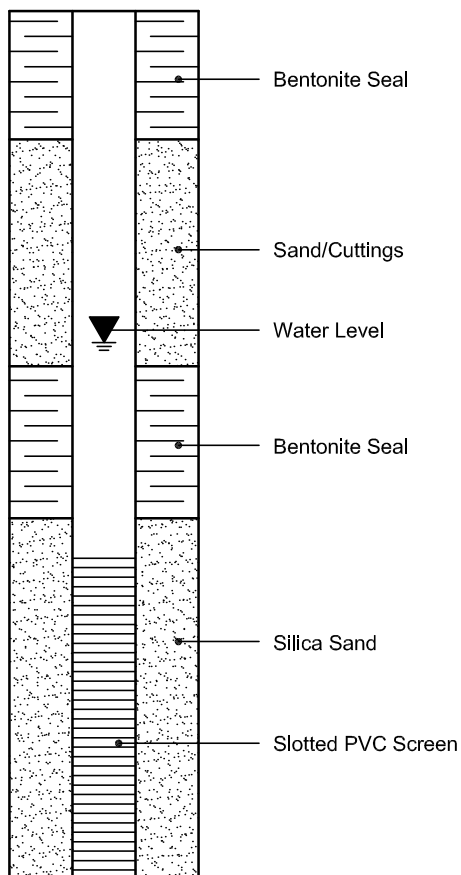
Shale



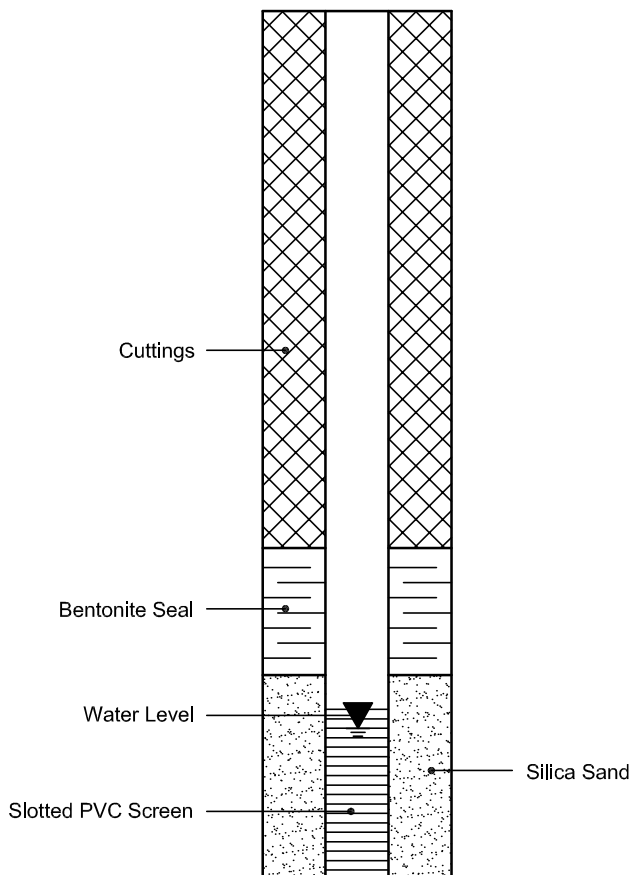
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



BOREHOLE No.: **BH-01**ELEVATION: **73.19 m****BOREHOLE LOG**Page: **1** of **1**CLIENT: **Bentall Real Estate Services**PROJECT: **Geotechnical Investigation, Proposed Office Building**LOCATION: **City Park Drive, Ottawa, Ontario**DESCRIBED BY: **A. Solomon**CHECKED BY: **H. Krzywicki**DATE (START): **July 30, 2007**DATE (FINISH): **July 30, 2007****LEGEND**

- ☒ SS Split Spoon
- ☒ ST Shelby Tube
- ☒ RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- Penetration Index based on Split Spoon sample
- Penetration Index based on Dynamic Cone sample
- Shear Strength based on Field Vane
- Shear Strength based on Lab Vane
- Sensitivity Value of Soil
- Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY			SAMPLE DATA			
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD
meters	73.19		GROUND SURFACE			%	ppm	N
	73.04		TOPSOIL/ROOTMAT					
0.5			SAND SILT FILL, compact, dark brown, some organic intrusions, occasional sand and clay layers, moist		SS1	46		15
1.0					SS2	22		12
1.5					SS3	56		10
2.0	71.23		SHALE TILL, dense, black, frequent shale pieces, clayey, moist		SS4	59		50+
2.5	70.68		WEATHERED SHALE, thinly bedded		RC5 SS6	100 60		0 50+
3.0					SS7	100		50+
3.5	69.94		SHALE BEDROCK -Billings formation -generally sound -occasional soft rock seams		RC8	90		28
4.0								
4.5	68.64		END OF BOREHOLE					
5.0								
5.5								
6.0								

SCALE FOR TEST RESULTS

50kPa 100kPa 150kPa 200kPa

10 20 30 40 50 60 70 80 90

NOTES:

BOREHOLE No.: **BH-02**ELEVATION: **73.62 m****BOREHOLE LOG**Page: 1 of 1CLIENT: Bentall Real Estate ServicesPROJECT: Geotechnical Investigation, Proposed Office BuildingLOCATION: City Park Drive, Ottawa, OntarioDESCRIBED BY: A. SolomonCHECKED BY: H. KrzywickiDATE (START): July 31, 2007DATE (FINISH): July 31, 2007**LEGEND**

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- N Penetration Index based on Split Spoon sample
- N Penetration Index based on Dynamic Cone sample
- Cu Shear Strength based on Field Vane
- Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- Δ Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY			SAMPLE DATA			
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD
meters	73.62		GROUND SURFACE			%	ppm	N
	73.59		TOPSOIL/ROOTMAT					
0.5			SANDY SILT FILL, compact, dark brown, some gravel and shale pieces, moist		SS1	71		18
1.0					SS2	56		16
1.5			-becoming clayey by 1.5m depth					
2.0	71.64		SHALE TILL, dense, black, frequent shale pieces, moist		SS3	56		13
2.5	71.11		WEATHERED SHALE, thinly bedded		SS4	89		50+
3.0	70.70		SHALE BEDROCK -Billings formation -generally sound		SS5	88		50+
3.5			-occasional soft rock seams					
4.0					RC6	88		37
4.5								
5.0	68.87		END OF BOREHOLE					
5.5								
6.0								

SCALE FOR TEST RESULTS
50kPa 100kPa 150kPa 200kPa

BOREHOLE LOG T020297-A1-LOG1.GPJ INSPEC SOL.GDT 8/23/07

NOTES:



BOREHOLE No.: BH-03

ELEVATION: 72.33 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Bentall Real Estate Services

PROJECT: Geotechnical Investigation, Proposed Office Building

LOCATION: City Park Drive, Ottawa, Ontario

DESCRIBED BY: A. Solomon

CHECKED BY: H. Krzywicki

DATE (START): July 31, 2007

DATE (FINISH): July 31, 2007

LEGEND

- ☒ SS Split Spoon
- ☒ ST Shelby Tube
- ☒ RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- Penetration Index based on Split Spoon sample
- Penetration Index based on Dynamic Cone sample
- Shear Strength based on Field Vane
- Shear Strength based on Lab Vane
- Sensitivity Value of Soil
- Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY			SAMPLE DATA					
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	Slate	Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD		
meters	72.33		GROUND SURFACE			%	ppm	N	SCALE FOR TEST RESULTS	
									50kPa	100kPa
									150kPa	200kPa
									10	20
									30	40
									50	60
									70	80
									90	
	72.15		TOPSOIL/ROOTMAT							
0.5			SANDY SILT FILL, compact, dark brown, moist		SS1	50		7		
			-Becoming clayey with occasional gravel by 0.5m depth							
1.0					SS2	42		7		
										
	71.11		SHALE TILL, dense, black, frequent shale pieces, moist							
1.5					SS3	71		50+		
										
	70.76		SHALE BEDROCK							
			-Billings formation							
2.0			-highly weathered layer at surface							
			-generally sound							
			-occasional soft rock seams							
2.5					RC4	50		0		
										
3.0										
										
3.5										
										
4.0					RC5	84		15		
										
4.5										
										
	67.66		END OF BOREHOLE							
5.0										
5.5										
6.0										

● N

• N

▲ Cu

□ Cu

S

▲

Penetration Index based on Split Spoon sample
Penetration Index based on Dynamic Cone sample
Shear Strength based on Field Vane
Shear Strength based on Lab Vane
Sensitivity Value of Soil
Shear Strength based on Pocket Penetrometer

SCALE FOR TEST RESULTS

50kPa 100kPa 150kPa 200kPa

10 20 30 40 50 60 70 80 90

NOTES:

BOREHOLE No.: **BH-04**ELEVATION: **72.41 m****BOREHOLE LOG**Page: **1** of **1**CLIENT: **Bentall Real Estate Services**PROJECT: **Geotechnical Investigation, Proposed Office Building**LOCATION: **City Park Drive, Ottawa, Ontario**DESCRIBED BY: **A. Solomon**CHECKED BY: **H. Krzywicki**DATE (START): **July 31, 2007**DATE (FINISH): **July 31, 2007****LEGEND**

- ☒ SS Split Spoon
- ☒ ST Shelby Tube
- ☒ RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- Penetration Index based on Split Spoon sample
- Penetration Index based on Dynamic Cone sample
- Shear Strength based on Field Vane
- Shear Strength based on Lab Vane
- Sensitivity Value of Soil
- Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY			SAMPLE DATA			
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD
meters	72.41		GROUND SURFACE			%	ppm	N
	72.38		TOPSOIL/ROOTMAT					
0.5			SANDY SILT FILL, compact, dark brown, moist		SS1	33		5
1.0	71.50		SHALE TILL, frequent shale pieces, dense, black, moist		SS2	63		12
1.5								
2.0	70.43		SHALE BEDROCK		SS3	77		50+
2.5			-Billings formation					
3.0			-Highly weathered layer at surface					
3.5			-Generally sound					
			-Occasional soft rock seams		RC4	70		39
4.0								
4.5								
5.0								
5.5								
6.0	68.90		END OF BOREHOLE					

SCALE FOR TEST RESULTS

50kPa 100kPa 150kPa 200kPa

NOTES:



BOREHOLE No.: BH-05

ELEVATION: 72.19 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Bentall Real Estate Services

PROJECT: Geotechnical Investigation, Proposed Office Building

LOCATION: City Park Drive, Ottawa, Ontario

DESCRIBED BY: A. Solomon CHECKED BY: H. Krzywicki

DATE (START): July 31, 2007 DATE (FINISH): July 31, 2007

LEGEND

- ☒ SS Split Spoon
- ☒ ST Shelby Tube
- ☒ RC Rock Core
- Water Level
- Water content (%)
- Atterberg limits (%)
- Penetration Index based on Split Spoon sample
- Penetration Index based on Dynamic Cone sample
- Shear Strength based on Field Vane
- Shear Strength based on Lab Vane
- Sensitivity Value of Soil
- Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY			SAMPLE DATA				
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD	
meters	72.19		GROUND SURFACE			%	ppm	N	
	71.99		TOPSOIL/ROOTMAT						
0.5	71.61		SILTY CLAY FILL, stiff, dark brown, some organics, moist		SS1	67		6	
	71.25		SHALE TILL, dense, black, frequent shale pieces, moist						
1.0	71.25		WEATHERED SHALE, thinly bedded		SS2	76		50+	
1.5	70.77		SHALE BEDROCK -Billings Formation -generally sound						
2.0			-occasional soft rock seams		RC3	85		18	
3.0	69.24		END OF BOREHOLE						
3.5									
4.0									
4.5									
5.0									
5.5									
6.0									

SCALE FOR TEST RESULTS

50kPa 100kPa 150kPa 200kPa

10 20 30 40 50 60 70 80 90

NOTES:

Certificate of Analysis

Report Date: 25-Jan-2023

Client: Paterson Group Consulting Engineers

Order Date: 19-Jan-2023

Client PO: 56641

Project Description: PG6552

Client ID:	BH2-23 SS3 BOT	-	-	-
Sample Date:	17-Jan-23 09:00	-	-	-
Sample ID:	2303411-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	89.1	-	-	-
----------	--------------	------	---	---	---

General Inorganics

pH	0.05 pH Units	7.08	-	-	-
Resistivity	0.10 Ohm.m	97.4	-	-	-

Anions

Chloride	10 ug/g dry	16	-	-	-
Sulphate	10 ug/g dry	38	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN
DETAIL

DRAWING PG6552-2 – TEST HOLE LOCATION PLAN

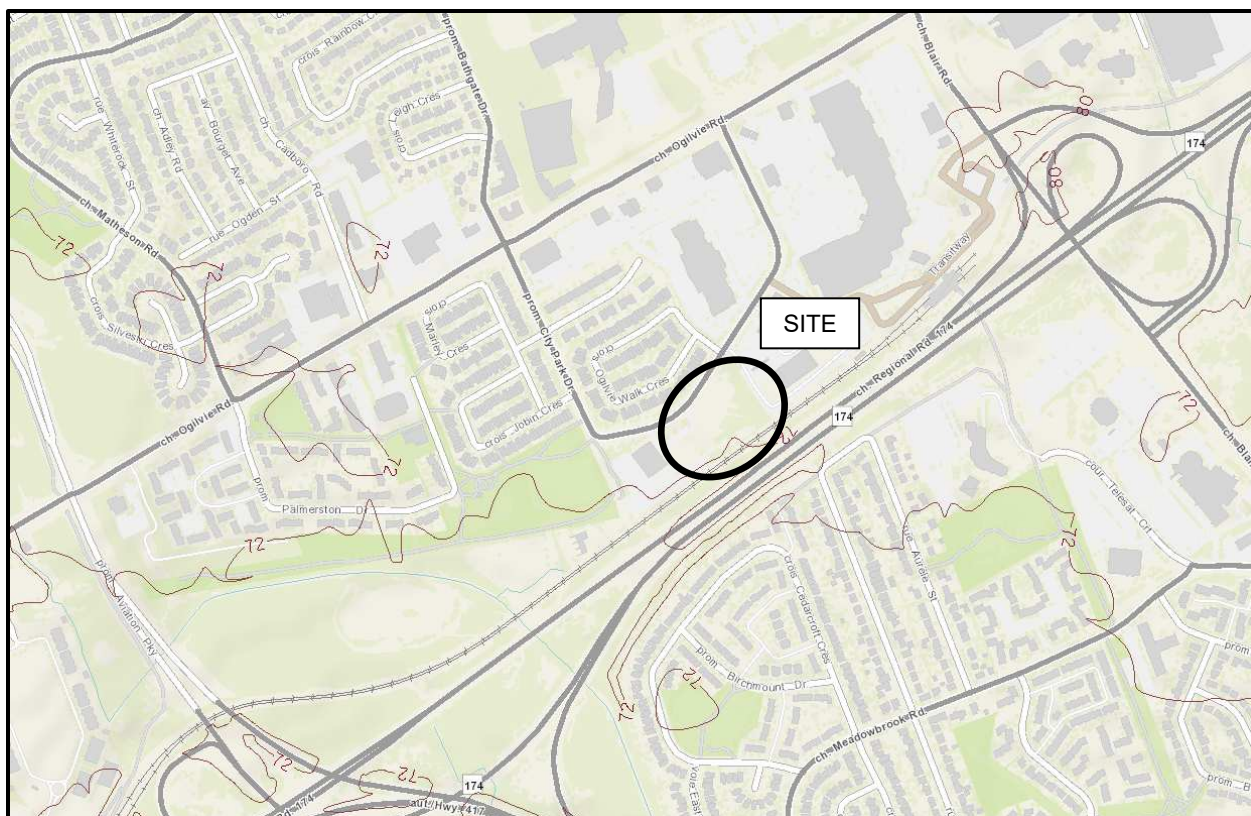
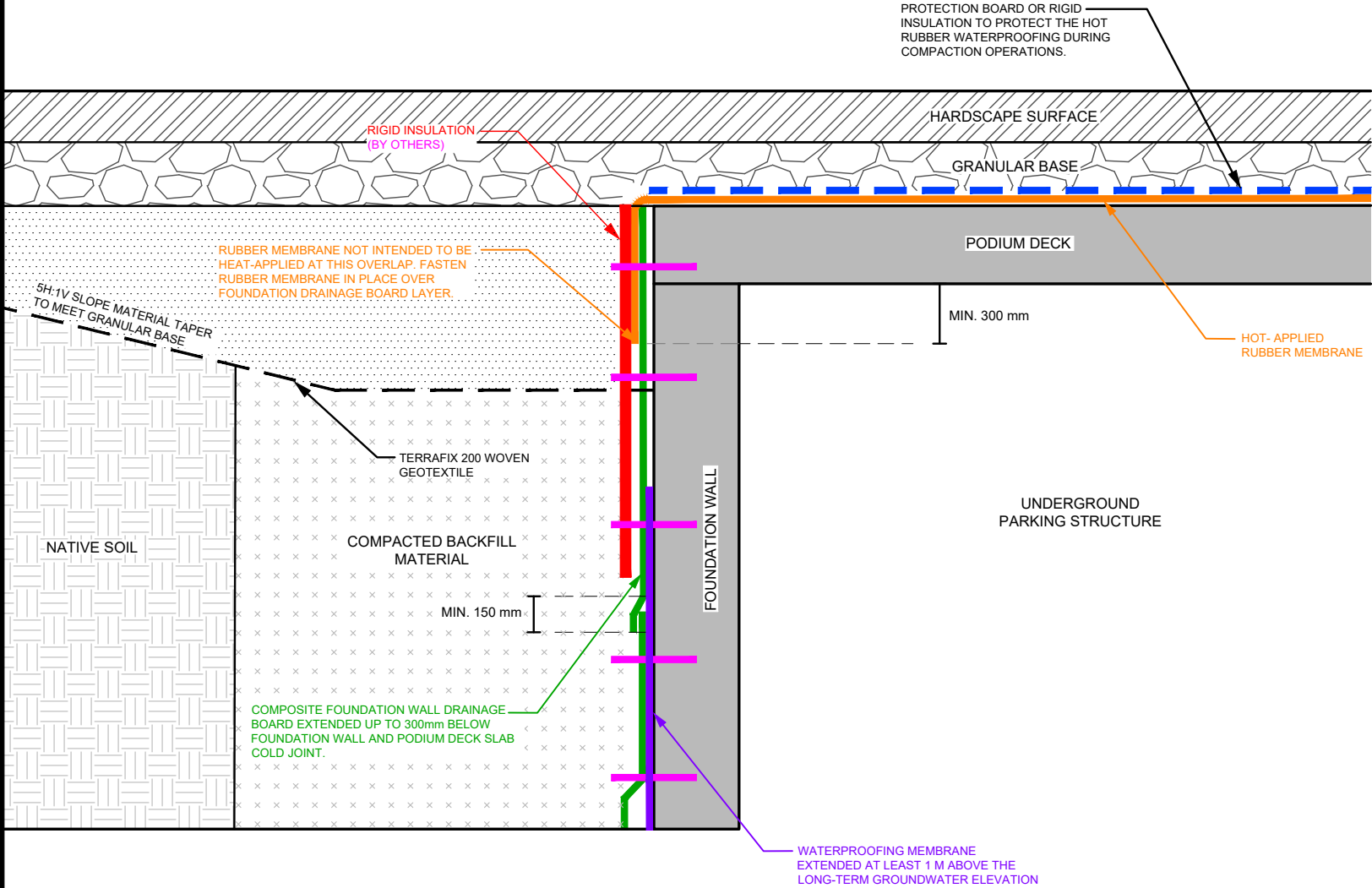


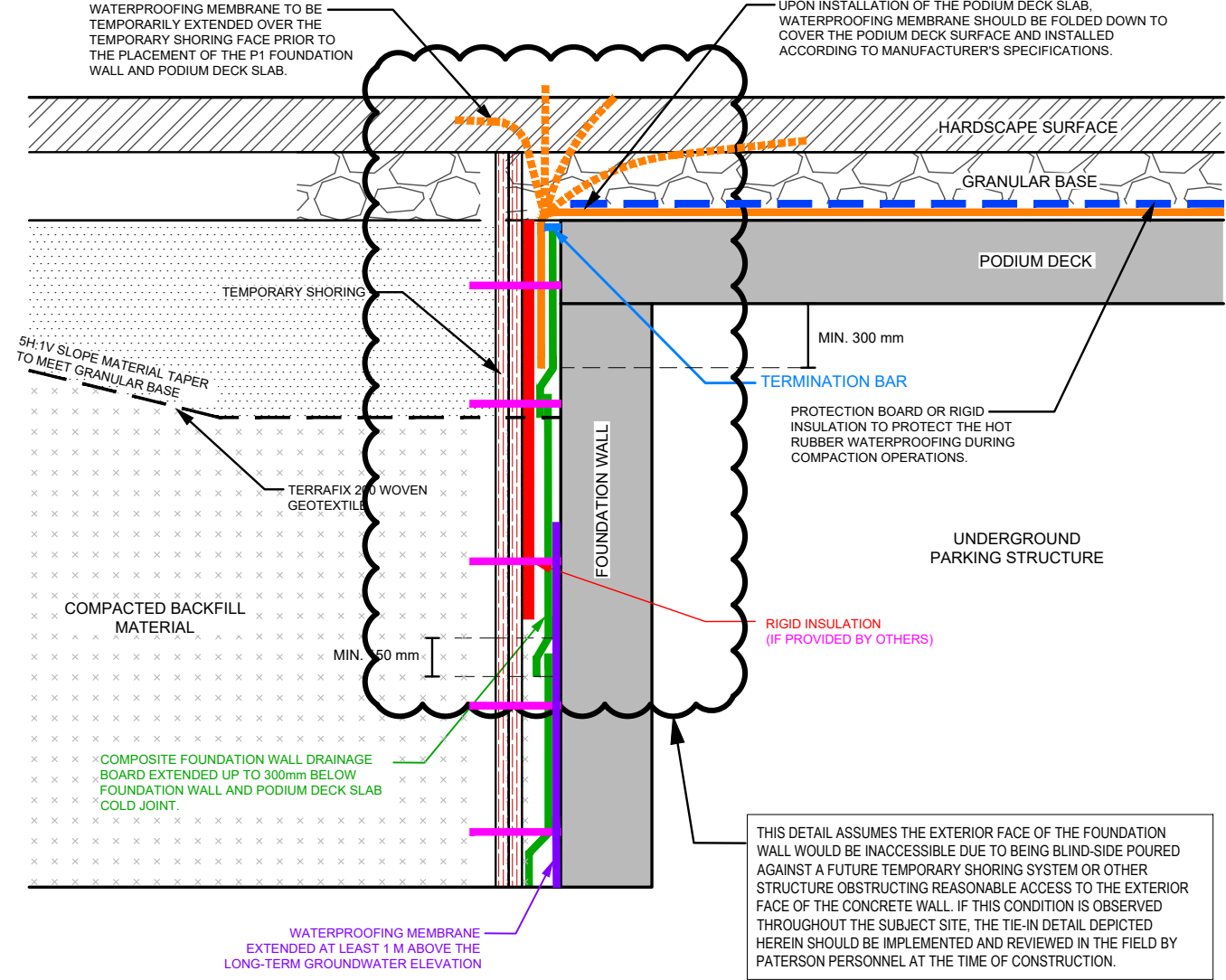
FIGURE 1

KEY PLAN

OPTION A - DOUBLE-SIDE POURED
TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED
TOP OF FOUNDATION WALL



NOTES:

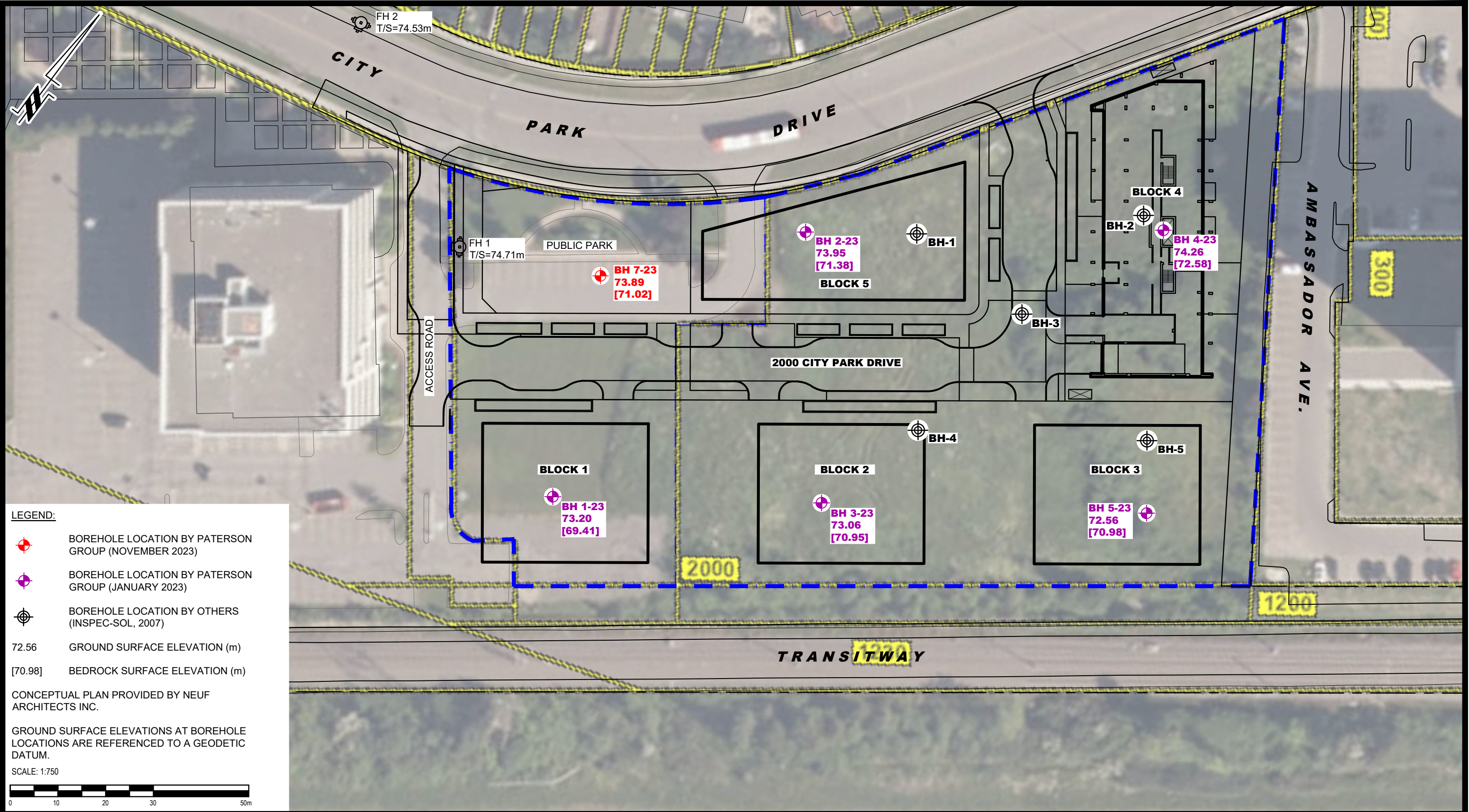
THE ABOVE DETAIL FOR HOT RUBBER AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL EDGE-PORCTIONS OF THE PODIUM DECK AND/OR SUSPENDED GROUND FLOOR SLAB STRUCTURE.

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

WHERE THE GRADING SURFACE TERMINATES AGAINST THE BUILDING FACE AND PAVEMENT STRUCTURE IS NOT LOCATED ABOVE THE EDGE OF THE FOUNDATION WALL AND PODIUM DECK SLAB AS DEPICTED HEREIN, IT IS RECOMMENDED TO PROVIDE A SUITABLE TERMINATION BAR TO SEAL THE TOP ENDLAP OF THE HOT-APPLIED RUBBER MEMBRANE LAYER TO THE VERTICAL FACE OF THE STRUCTURE. THIS WOULD BE REQUIRED TO MITIGATE THE POTENTIAL FOR THE MIGRATION OF WATER BEHIND THE RUBBER MEMBRANE.

ALL PORTIONS OF THE ABOVE-NOTED DETAIL (INSULATION OF FOUNDATION DRAINAGE BOARD, TERMINATION BAR, HOT-RUBBER MEMBRANE OVER SLAB, FOUNDATION WALL CONSTRUCTION JOINT AND OVERLAPPING/SHINGLING OF DRAINAGE BOARD) SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION BY PATERSON PERSONNEL.

NO.	REVISIONS	DATE	INITIAL



LEGEND:

- BOREHOLE LOCATION BY PATERSON GROUP (NOVEMBER 2023)
- BOREHOLE LOCATION BY PATERSON GROUP (JANUARY 2023)
- BOREHOLE LOCATION BY OTHERS (INSPEC-SOL, 2007)

72.56 GROUND SURFACE ELEVATION (m)
[70.98] BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY NEUF ARCHITECTS INC.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:750

 PATERSON GROUP 9 AURIGA DRIVE OTTAWA, ON K2E 7T9 TEL: (613) 226-7381					COLONNADE BRIDGEPORT GEOTECHNICAL INVESTIGATION PROPOSED HIGH-RISE BUILDINGS 2000 CITY PARK DRIVE ONTARIO	Scale:	1:750	Date:	07/2026	
						Drawn by:	GK	Report No.:	PG6552-2	
						Checked by:	YZ	Dwg. No.: PG6552-2		
						Approved by:	FA		Revision No.:	
					OTTAWA, Title:					
	TEST HOLE LOCATION PLAN									
NO.	REVISIONS			DD/MM/YYYY	INITIAL					

APPENDIX C

Transportation Noise and Vibration Feasibility Study:

Prepared by Gradient Wind Engineers and Scientists

Report No. 23-075 dated August 17, 2026

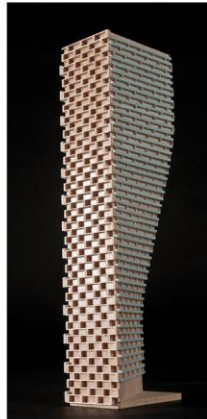
GRADIENTWIND

ENGINEERS & SCIENTISTS

TRANSPORTATION NOISE & VIBRATION FEASIBILITY ASSESSMENT

2000 City Park Drive
Ottawa, Ontario

Report: 23-075 – Transportation Noise & Vibration Feasibility



August 17, 2026

PREPARED FOR

City Park CBP GP Inc. c/o Colonnade BridgePort
16 Concourse Gate, Suite 200
Ottawa, ON
K2E 7S8

PREPARED BY

Doryan Saavedra, B.Eng., Junior Acoustic Scientist
Joshua Foster, P.Eng., Lead Engineer

EXECUTIVE SUMMARY

This report describes a transportation noise and vibration feasibility assessment undertaken for the proposed development located at 2000 City Park Drive in Ottawa, Ontario. The proposed development comprises a subdivision with five lots, a dedicated parkland, and a public road. The dominant sources of roadway traffic noise impacting the development is City Park Drive to the north and Highway 174 located to the south of the subject site. The major source of LRT noise and vibrations is the Confederation Line 1 to the South. Figure 1 illustrates a site plan with surrounding context.

The assessment is based on (i) theoretical noise prediction methods that conform to the Ministry of the Environment, Conservation and Parks (MECP), and City of Ottawa requirements; (ii) noise level criteria as specified by the City of Ottawa's Environmental Noise Control Guidelines (ENCG); (iii) future vehicular traffic volumes based on the City of Ottawa's Official Plan roadway classifications; and (iv) site plan drawings prepared by Neuf Architects, received in July 2026.

The results of the current analysis indicate that noise levels will range between 51 and 75 dBA during the daytime period (07:00-23:00) and between 44 and 67 dBA during the nighttime period (23:00-07:00). The highest noise level (i.e. 75 dBA) occurs along the development's south façade of Block B which is nearest and most exposed to Highway 174 and the LRT Line 1.

The noise levels predicted due to roadway and LRT traffic exceed the criteria listed in Section 4.2 for building components. As such, upgraded building components, including STC rated glazing elements and exterior walls, will be required. Additional details are provided in Section 5.2 and Figure 3. Results of the calculations also indicate that the development will require air conditioning, which will allow occupants to keep windows closed and maintain a comfortable living environment. In addition to ventilation requirements, a Type D Warning Clause will also be required to be placed on all Lease, Purchase and Sale Agreements, as summarized in Section 6 of this report.

It is noted that the current site plan does not identify any outdoor amenity spaces. Once the design develops, noise levels at the outdoor amenities will need to be assessed to ensure compliance with ENCG criteria. The updated assessment may also include recommendations for noise barriers and additional warning clauses.



As the proposed development is near the LRT Line 1, the Rail Construction Program Office recommends that a warning clause be included in all Lease, Purchase, and Sale Agreements as summarized in Section 6.

For the LRT Line 1, estimated vibration levels at the nearest foundation are expected to be 0.11 mm/s RMS (73 dBV), based on the FTA protocol and a conservative offset distance of 15 m to the nearest track centerline. Details of the calculations are provided in Appendix B. Since predicted vibration levels do not exceed the criterion of 0.14 mm/s RMS at the foundation, concerns due to vibration impacts on the site are not expected. As vibration levels are acceptable, correspondingly, regenerated noise levels are also expected to be acceptable.

A review of satellite imagery confirmed there are no significant sources of stationary noise surrounding the site. The dominant source of noise impacting the development is from transportation noise sources.

The development's own mechanical equipment has the potential to generate noise off-site at surrounding noise sensitive (residential) developments and on the development itself. Any potential impacts can be minimized by judicious selection of mechanical equipment and its location. These systems will be designed to comply with the ENCG sound level limits. A review by a qualified acoustic consultant is recommended once the mechanical design of the building has developed.

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FIGURES

APPENDICES

Appendix A – STAMSON 5.04 Input and Output Data and Supporting Information

Appendix B – FTA Vibration Calculations

1. INTRODUCTION

Gradient Wind Engineering Inc. (Gradient Wind) was retained by Colonnade BridgePort on behalf of City Park CBP GP Inc. to undertake a transportation noise and vibration feasibility assessment for a proposed development located at 2000 City Park Drive in Ottawa, Ontario. This report summarizes the methodology, results, and recommendations related to the assessment of exterior and interior noise levels generated by local roadway and LRT traffic. Vibration impacts from the LRT onto the proposed development is also assessed.

This assessment is based on theoretical noise calculation methods conforming to the Ministry of the Environment, Conservation and Parks (MECP) NPC-300¹ and City of Ottawa Environmental Noise Control Guidelines (ENCG)². Noise calculations were based on site plan drawings prepared by Neuf Architects, received in July 2026, with future traffic volumes corresponding to the City of Ottawa's Official Plan (OP) roadway classifications.

2. TERMS OF REFERENCE

The subject site is located at 2000 City Park Drive in Ottawa, Ontario. The site is bordered by City Park Drive to the north, existing apartment buildings to the east, the LRT Confederation Line 1 to the south, and a commercial building to the west.

The proposed development comprises a subdivision with five lots, a dedicated parkland, and a public road. Each lot comprises a tower block. Block A to the northeast of the site rises 16 Storeys, Block B to the southeast rises 30 Storeys, Block C to the south rises 30-storeys, Block D to the southwest rises 30-storeys, and Block E to the north rises 12-storeys. The dominant sources of roadway traffic noise impacting the development is City Park Drive to the north and Highway 174 located to the south of the subject site. The major source of LRT noise and vibrations is the Confederation Line 1 to the South. Figure 1 illustrates a site plan with surrounding context.

¹ Ontario Ministry of the Environment and Climate Change – Environmental Noise Guidelines, Publication NPC-300, Queens Printer for Ontario, Toronto, 2013

² City of Ottawa, Environmental Noise Control Guidelines, January 2016

It is noted that the current site plan does not identify any outdoor amenity spaces. Once the design develops, noise levels at the outdoor amenities will need to be assessed to ensure compliance with ENCG criteria. The updated assessment may also include recommendations for noise barriers and additional warning clauses.

3. OBJECTIVES

The principal objectives of this study are to (i) calculate the future noise levels on the study building produced by local roadway traffic, (ii) ensure that interior and exterior noise levels do not exceed the allowable limits specified by the City of Ottawa's Environmental Noise Control Guidelines as outlined in Section 4.2 of this report, and (iii) calculate the anticipated vibration levels on the subject site and ensure it meets the applicable criteria.

4. METHODOLOGY

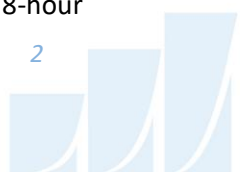
4.1 Background

Noise can be defined as any obtrusive sound. It is created at a source, transmitted through a medium, such as air, and intercepted by a receiver. Noise may be characterized in terms of the power of the source or the sound pressure at a specific distance. While the power of a source is characteristic of that particular source, the sound pressure depends on the location of the receiver and the path that the noise takes to reach the receiver. Measurement of noise is based on the decibel unit, dBA, which is a logarithmic ratio referenced to a standard noise level (2×10^{-5} Pascals). The 'A' suffix refers to a weighting scale, which better represents how the noise is perceived by the human ear. With this scale, a doubling of power results in a 3 dBA increase in measured noise levels and is just perceptible to most people. An increase of 10 dBA is often perceived to be twice as loud.

4.2 Roadway and LRT Traffic Noise

4.2.1 Criteria for Roadway and LRT Traffic Noise

For surface traffic noise, the equivalent sound energy level, L_{eq} , provides a measure of the time-varying noise levels, which is well correlated with the annoyance of sound. It is defined as the continuous sound level, which has the same energy as a time-varying noise level over a period of time. For roadways and railways, the L_{eq} is commonly calculated on the basis of a 16-hour (L_{eq16}) daytime (07:00-23:00) / 8-hour



(L_{eq8}) nighttime (23:00-07:00) split to assess its impact on residential buildings. The City of Ottawa's Environmental Noise Control Guidelines (ENCG) specify that the recommended indoor noise limit range due to roadway noise (that is relevant to this study) is 50, 45, and 40 dBA for retail, living rooms and sleeping quarters respectively, as listed in Table 1.

TABLE 1: INDOOR SOUND LEVEL CRITERIA³

Type of Space	Time Period	L_{eq} (dBA)
General offices, reception areas, retail stores, etc.	07:00 – 23:00	50
Living/dining/den areas of residences , hospitals, schools, nursing/retirement homes, day-care centres, theatres, places of worship, libraries, individual or semi-private offices, conference rooms, etc.	07:00 – 23:00	45
Sleeping quarters of hotels/motels	23:00 – 07:00	45
Sleeping quarters of residences , hospitals, nursing/retirement homes, etc.	23:00 – 07:00	40

Predicted noise levels at the plane of window (POW) dictate the action required to achieve the recommended sound levels. An open window is considered to provide a 10 dBA reduction in noise, while a standard closed window is capable of providing a minimum 20 dBA noise reduction⁴. A closed window due to a ventilation requirement will bring noise levels down to achieve an acceptable indoor environment⁵. Therefore, where noise levels exceed 55 dBA daytime and 50 dBA nighttime, the ventilation for the building should consider the need for having windows and doors closed, which triggers the need for forced air heating with provision for central air conditioning. Where noise levels exceed 65 dBA daytime and 60 dBA nighttime, air conditioning will be required and building components will require higher levels of sound attenuation⁶.

³ Adapted from ENCG 2016 – Tables 2.2b and 2.2c

⁴ Burberry, P.B. (2014). Mitchell's Environment and Services. Routledge, Page 125

⁵ MECP, Environmental Noise Guidelines, NPC 300 – Part C, Section 7.8

⁶ MECP, Environmental Noise Guidelines, NPC 300 – Part C, Section 7.1.3



For designated Outdoor Living Areas (OLAs), the sound level limit is 55 dBA during the daytime period. An excess above the limit is acceptable only in cases where the required noise control measures are not feasible for technical, economic or administrative reasons. Furthermore, noise levels must not exceed 60 dBA.

4.2.2 Roadway and LRT Traffic Volumes

The ENCG dictates that noise calculations should consider future sound levels based on a roadway's classification at the mature state of development. Therefore, traffic volumes are based on the roadway classifications outlined in the City of Ottawa's Official Plan (OP) and Transportation Master Plan⁷ which provide additional details on future roadway expansions. Average Annual Daily Traffic (AADT) volumes are then based on data in Table B1 of the ENCG for each roadway classification. LRT traffic data for the Confederation Line 1 was obtained from the City of Ottawa. Table 2 (below) summarizes the AADT values used for each roadway and LRT included in this assessment.

TABLE 2: ROADWAY AND LRT TRAFFIC DATA

Segment	Roadway Traffic Classification	Speed Limit (km/h)	Traffic Volumes
City Park Drive	Two-Lane Urban Collector (2-UCU)	50	8,000
Highway 174 Eastbound	3-Lane Highway	100	54,999
Highway 174 Westbound	3-Lane Highway	100	54,999
O-Train Confederation Line 1 LRT	Passenger LRT*	70	540/60**

*The LRT was modelled in STAMSON as a 4-car SRT using the RT/Custom feature

**Day/night split

⁷ City of Ottawa Transportation Master Plan, November 2013



4.2.3 Theoretical Roadway and LRT Traffic Noise Predictions

The impact of transportation noise sources on the development was determined by computer modelling. Transportation noise source modelling is based on the software program *CadnaA*, which utilizes the United States Federal Highway Administration's Traffic Noise Model (TNM) to represent the roadway line sources and the FTA/FRA model to represent LRT sources. The TNM analysis model has been recognized by the Ministry of Transportation Ontario (MTO) as the recommended noise model for transportation projects (ref. Environmental Guide for Noise, 2022, by the Ministry of Transportation (MTO)⁸). The Ministry of Environment, Conservation and Parks has also adopted the TMN model as per their "Draft Guideline Noise Pollution Control Publications 306 (NPC-306)"⁹.

A set of comparative calculations were also performed for comparisons using STAMSON. The *CadnaA* computer program can represent three-dimensional surfaces and one order of reflection of sound waves over a suitable spectrum for human hearing. The STAMSON model, however, is an older software and requires each receptor to be calculated separately. STAMSON also does not accurately account for building reflections, multiple screening elements, and curved road geometry. The result of the comparative calculations can be seen in Section 5, Table 4.

Transportation noise calculations were performed by treating each roadway and LRT segment as a separate line source of noise and by using existing building locations as noise barriers. In addition to the traffic volumes summarized in Table 2, theoretical noise predictions were based on the following parameters:

- Truck traffic on all roadways was taken to comprise 5% heavy trucks and 7% medium trucks, as per ENCG requirements for noise level predictions.
- The day/night split for all roads were taken to be 92%/8%, respectively.
- Ground surfaces for receptors were taken to be reflective due to the presence of hard (paved) ground.
- Noise receptors were strategically placed at 25 locations around the study area (see Figure 2).

⁸ Ministry of Transportation, Environmental Guide for Noise, 2022. Retrieved from [Environmental Guide for Noise 2022](#)

⁹ Ministry of Environment, Conservation and Parks, Ontario, "Methods to determine Sound Levels Due to Road and Rail Traffic", Draft February 12, 2020



4.3 Indoor Noise Calculations

The difference between outdoor and indoor noise levels is the noise attenuation provided by the building envelope. According to common industry practice, complete walls and individual wall elements are rated according to the Sound Transmission Class (STC). The STC ratings of common residential walls built in conformance with the Ontario Building Code (2024) typically exceed STC 35, depending on exterior cladding, thickness and interior finish details. For example, brick veneer walls can achieve STC 50 or more. Standard commercially sided exterior metal stud walls have around STC 45. Standard good quality double-glazed non-operable windows can have STC ratings ranging from 25 to 40, depending on the window manufacturer, pane thickness and inter-pane spacing. As previously mentioned, the windows are the known weak point in a partition.

As per Section 4.2, when daytime noise levels exceed 65 dBA or nighttime noise levels exceed 60 dBA, calculations must be performed to evaluate the sound transmission quality of the building components to ensure acceptable indoor noise levels. The calculation procedure¹⁰ considers:

- Window type and total area as a percentage of total room floor area
- Exterior wall type and total area as a percentage of the total room floor area
- Acoustic absorption characteristics of the room
- Outdoor noise source type and approach geometry
- Indoor sound level criteria, which varies according to the intended use of a space

Based on published research¹¹, exterior walls possess specific sound attenuation characteristics that are used as a basis for calculating the required STC ratings of windows in the same partition. Due to the limited information available at the time of the study, detailed floor layouts and building elevations have not been finalized; therefore, detailed STC calculations could not be performed at this time. As a guideline, the anticipated STC requirements for windows have been estimated based on the overall noise reduction required for each intended use of space ($STC = \text{outdoor noise level} - \text{targeted indoor noise levels} + \text{safety factor}$).

¹⁰ Building Practice Note: Controlling Sound Transmission into Buildings by J.D. Quirt, National Research Council of Canada, September 1985

¹¹ CMHC, Road & Rail Noise: Effects on Housing



4.4 Ground Vibration and Ground-borne Noise

Transit systems and heavy vehicles on roadways can produce perceptible levels of ground vibrations, especially when they are in close proximity to residential neighbourhoods or vibration-sensitive buildings. Similar to sound waves in air, vibrations in solids are generated at a source, propagated through a medium, and intercepted by a receiver. In the case of ground vibrations, the medium can be uniform, or more often, a complex layering of soils and rock strata. Also, similar to sound waves in air, ground vibrations produce perceptible motions and regenerated noise known as 'ground-borne noise' when the vibrations encounter a hollow structure such as a building. Ground-borne noise and vibrations are generated when there is excitation of the ground, such as from a train or subway. Repetitive motion of the wheels on the track or rubber tires passing over an uneven surface causes vibration to propagate through the soil. When they encounter a building, vibrations pass along the structure of the building beginning at the foundation and propagating to all floors. Air inside the building excited by the vibrating walls and floors represents regenerated airborne noise. Characteristics of the soil and the building are imparted to the noise, thereby creating a unique noise signature.

Human response to ground vibrations is dependent on the magnitude of the vibrations, which is measured by the root mean square (RMS) of the movement of a particle on a surface. Typical units of ground vibration measures are millimeters per second (mm/s), or inch per second (in/s). Since vibrations can vary over a wide range, it is also convenient to represent them in decibel units, or dBV. In North America, it is common practice to use the reference value of one micro-inch per second ($\mu\text{in/s}$) to represent vibration levels for this purpose. The threshold level of human perception to vibrations is about 0.10 mm/s RMS or about 72 dBV. Although somewhat variable, the threshold of annoyance for continuous vibrations is 0.5 mm/s RMS (or 85 dBV), five times higher than the perception threshold, whereas the threshold for significant structural damage is 10 mm/s RMS (or 112 dBV), at least one hundred times higher than the perception threshold level.



4.4.1 Ground Vibration Criteria

The Canadian Railway Association and Canadian Association of Municipalities have set standards for new sensitive land developments within 300 metres of a railway right-of-way, as published in their document *Guidelines for New Development in Proximity to Railway Operations*¹², which indicate that vibration conditions should not exceed 0.14 mm/s RMS averaged over a one second time-period at the first floor and above of the proposed buildings.

4.4.2 Theoretical Ground Vibration Prediction Procedure

Potential vibration impacts of the trains were predicted using the Federal Transit Authority's (FTA) *Transit Noise and Vibration Impact Assessment*¹³ protocol. The FTA general vibration assessment is based on an upper bound generic set of curves that show vibration level attenuation with distance. These curves, illustrated in the figure on the following page, are based on ground vibration measurements at various transit systems throughout North America. Vibration levels at points of reception are adjusted by various factors to incorporate known characteristics of the system being analyzed, such as operating speed of vehicle, conditions of the track, construction of the track and geology, as well as the structural type of the impacted building structures.

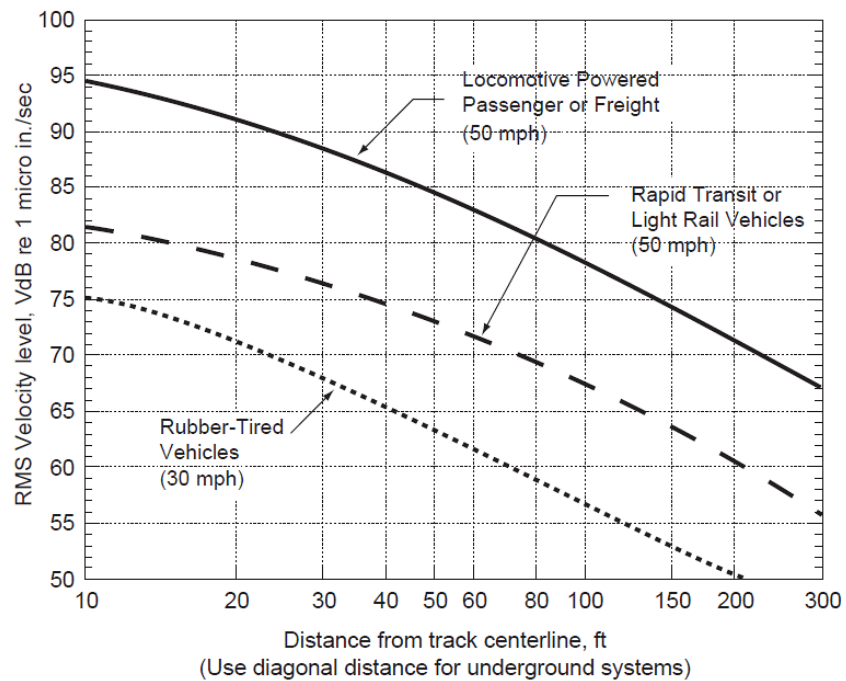
For the Line 1 LRT, the vibration impact on the buildings were determined using a set of curves for Rapid Transit at a speed of 50 mph. Adjustment factors were considered based on the following information:

- The operating speed of the LRT is conservatively modelled at 43 mph (70 km/h).
- The offset distance between the development property line and the closest track is 15 m.
- The vehicles are assumed to have soft primary suspensions.
- Tracks are not welded, though in otherwise good condition.
- Soil conditions do not efficiently propagate vibrations.
- Type of transit structure is open-cut.
- The building's foundation is large masonry on piles.

¹² Dialog and J.E. Coulter Associates Limited, prepared for The Federation of Canadian Municipalities and The Railway Association of Canada, May 2013

¹³ John A. Volpe National Transportation Systems Center, Transit Noise and Vibration Impact Assessment, Federal Transit Administration, September 2018





**FTA GENERALIZED CURVES OF VIBRATION LEVELS VERSUS DISTANCE
(ADOPTED FROM FIGURE 10-1, FTA TRANSIT NOISE AND VIBRATION IMPACT
ASSESSMENT)**



5. RESULTS

5.1 Roadway and LRT Traffic Noise Levels

The results of the roadway traffic noise calculations are summarized in Table 3 below. The results of the current analysis indicate that noise levels will range between 51 and 75 dBA during the daytime period (07:00-23:00) and between 44 and 67 dBA during the nighttime period (23:00-07:00). The highest noise level (i.e. 75 dBA) occurs along the development's south façade of Block B which is nearest and most exposed to Highway 174 and the LRT Line 1.

The noise levels predicted due to roadway and LRT traffic exceed the criteria listed in Section 4.2 for building components. As such, upgraded building components, including STC rated glazing elements and exterior walls, will be required. Additional details are provided in Section 5.2 and Figure 3. Results of the calculations also indicate that the development will require air conditioning, which will allow occupants to keep windows closed and maintain a comfortable living environment. In addition to ventilation requirements, a Type D Warning Clause will also be required to be placed on all Lease, Purchase and Sale Agreements, as summarized in Section 6 of this report.



TABLE 3: EXTERIOR NOISE LEVELS DUE TO TRANSPORTATION SOURCES

Receptor Number / Type	Height Above Grade (m)	Receptor Location	Roadway Noise Level (dBA)		LRT Noise Level (dBA)		Total Noise Level (dBA)	
			Day	Night	Day	Night	Day	Night
BLOCK A								
R1 / POW	16.5	Level 5 – East Façade	68	61	59	52	69	61
R2 / POW	16.5	Level 5 – South Façade	64	57	57	49	65	57
R3 / POW	16.5	Leve 5 – West Façade	48	41	48	41	51	44
R4 / POW	16.5	Level 5 – West Façade	61	54	50	43	62	54
R5 / POW	16.5	Level 5 – North Façade	62	54	45	38	62	54
R6 / POW	16.5	Level 5 – East Façade	65	57	55	48	65	58
R7 / POW	46.5	Level 16 – East Façade	67	60	58	50	68	60
R8 / POW	46.5	Level 16 – South Façade	67	59	57	50	67	60
R9 / POW	46.5	Level 16 – West Façade	60	53	52	45	61	53
BLOCK B								
R10 / POW	22.5	Level 7 – East Façade	71	63	63	56	72	64
R11 / POW	22.5	Level 7 – South Façade	74	66	67	59	75	67
R12 / POW	22.5	Level 7 – West Façade	71	63	63	56	71	64
R13 / POW	88.5	Level 30 – North Façade	63	56	53	45	64	56
BLOCK C								
R14 / POW	22.5	Level 7 – West Façade	71	63	63	56	71	64
R15 / POW	88.5	Level 30 – North Façade	55	48	46	39	56	48
R16 / POW	88.5	Level 30 – East Façade	69	62	58	51	70	62
R17 / POW	88.5	Level 30 – South Façade	73	65	61	54	73	65



TABLE 3: EXTERIOR NOISE LEVELS DUE TO TRANSPORTATION SOURCES (CONT.)

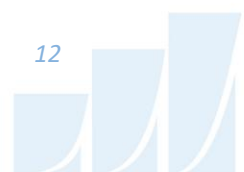
Receptor Number / Type	Height Above Grade (m)	Receptor Location	Roadway Noise Level (dBA)		LRT Noise Level (dBA)		Total Noise Level (dBA)	
			Day	Night	Day	Night	Day	Night
BLOCK D								
R18 / POW	88.5	Level 30 – North Façade	56	48	50	43	57	49
R19 / POW	88.5	Level 30 – East Façade	69	62	58	51	70	62
R20 / POW	88.5	Level 30 – South Façade	73	65	62	54	73	65
R21 / POW	88.5	Level 30 – West Façade	69	62	59	51	70	62
BLOCK E								
R22 / POW	34.5	Level 12 – North Façade	64	57	50	43	65	57
R23 / POW	34.5	Level 12 – East Façade	66	59	56	49	67	59
R24 / POW	34.5	Level 12 – South Façade	64	56	55	48	65	57
R25 / POW	34.5	Level 12 – West Façade	66	59	55	48	67	59

*Noise levels during the nighttime are not considered for OLAs

Table 4 below provides a comparison between CadnaA and STAMSON. Noise levels calculated in STAMSON were found to have a good correlation with CadnaA, and variability between the two programs was within an acceptable level of ± 0.3 dBA.

TABLE 4: RESULT CORRELATION WITH STAMSON

Receptor Number	Receptor Location	Receptor Height (m)	CADNAA Noise Level (dBA)		STAMSON 5.04 Noise Level (dBA)	
			Day	Night	Day	Night
R11	Block B – Level 7 – South Façade	22.5	75	67	76	69
R21	Block D – Level 30 – West Façade	88.5	70	62	73	65



5.2 Noise Control Measures

The noise levels predicted due to roadway traffic exceed the criteria listed in Section 4.2 for building components. As discussed in Section 4.3, the anticipated STC requirements for windows have been estimated based on the overall noise reduction required for each intended use of space (STC = outdoor noise level – targeted indoor noise levels + safety factor). Figure 3 outlines the required bedroom, living room, and retail window STC's for all facades of the development. The STC requirements are summarized below for various units within the development:

■ **Bedroom Windows**

- (i) Bedroom windows on Block B, Block C, and Block D facing south, east, and west will require a minimum STC of 33.
- (ii) Bedroom windows on Block A facing east and south will require a minimum STC of 30 above Level 5.
- (iii) Bedroom windows on Block E facing east and west will require a minimum STC of 30 above Level 6.
- (iv) All other bedroom windows are to satisfy Ontario Building Code (OBC 2024) requirements.

■ **Living Room Windows**

- (i) Living room windows on Block B, Block C, and Block D facing south, west, and east will require a minimum STC of 33.
- (ii) Living room windows on Block A facing east and south will require a minimum STC of 30 above Level 5.
- (iii) Living room windows on Block E facing east and west will require a minimum STC of 30 above Level 6.
- (iv) All other living room windows are to satisfy Ontario Building Code (OBC 2024) requirements.

■ **Retail Windows**

- (i) Retail windows on Block B, Block C, and Block D facing south, west, and east will require a minimum STC of 28.
- (ii) Retail windows on Block A facing east and south will require a minimum STC of 25 above Level 5.
- (iii) Retail windows on Block E facing east and west will require a minimum STC of 25 above Level 6.



(iv) All other retail windows are to satisfy Ontario Building Code (OBC 2024) requirements.

■ **Exterior Walls**

(i) Where upgraded windows are noted above, exterior wall components on the same façades will require a minimum STC of 45, which will be achieved with brick cladding or an acoustical equivalent according to NRC test data¹⁴.

The STC requirements apply to windows, doors, spandrel panels and curtainwall elements. Exterior wall components on these façades are recommended to have a minimum STC of 45, where a stud wall system is used. A review of window supplier literature indicates that the specified STC ratings can be achieved by a variety of window systems having a combination of glass thickness and inter-pane spacing. We have specified an example window configuration, however several manufacturers and various combinations of window components, such as those proposed, will offer the necessary sound attenuation rating. It is the responsibility of the manufacturer to ensure that the specified window achieves the required STC. This can only be assured by using window configurations that have been certified by laboratory testing. The requirements for STC ratings assume that the remaining components of the building are constructed and installed according to the minimum standards of the Ontario Building Code. The specified STC requirements also apply to swinging and/or sliding patio doors.

Results of the calculations also indicate that the development will require central air conditioning, which will allow occupants to keep windows closed and maintain a comfortable living environment. In addition to ventilation requirements, Warning Clauses will also be required on all Lease, Purchase and Sale Agreements, as summarized in Section 6.

5.3 Ground Vibrations and Ground-Borne Noise Levels

Estimated vibration levels at the foundation nearest to the LRT Line 1 are expected to be 0.11 mm/s RMS (73 dBV), based on the FTA protocol and an offset distance of 15 m to the nearest track centerline. Details of the calculation are provided in Appendix B. Since predicted vibration levels do not exceed the criterion of 0.14 mm/s RMS at the foundation, concerns due to vibration impacts on the site are not expected. As

¹⁴ J.S. Bradley and J.A. Birta. Laboratory Measurements of the Sound Insulation of Building Façade Elements, National Research Council October 2000.



vibration levels are acceptable, correspondingly, regenerated noise levels are also expected to be acceptable.

6. CONCLUSIONS AND RECOMMENDATIONS

The results of the current analysis indicate that noise levels will range between 51 and 75 dBA during the daytime period (07:00-23:00) and between 44 and 67 dBA during the nighttime period (23:00-07:00). The highest noise level (i.e. 75 dBA) occurs along the development's south façade of Block B which is nearest and most exposed to Highway 174 and the LRT Line 1.

The noise levels predicted due to roadway and LRT traffic exceed the criteria listed in Section 4.2 for building components. As such, upgraded building components, including STC rated glazing elements and exterior walls, will be required. Additional details are provided in Section 5.2 and Figure 3. Results of the calculations also indicate that the development will require air conditioning, which will allow occupants to keep windows closed and maintain a comfortable living environment. In addition to ventilation requirements, a Type D Warning Clause will also be required to be placed on all Lease, Purchase and Sale Agreements, as summarized below.

Type D:

"This dwelling unit has been supplied with a central air conditioning system which will allow windows and exterior doors to remain closed, thereby ensuring that the indoor sound levels are within the sound level limits of the Municipality and the Ministry of the Environment."

It is noted that the current site plan does not identify any outdoor amenity spaces. Once the design develops, noise levels at the outdoor amenities will need to be assessed to ensure compliance with ENCG criteria. The updated assessment may also include recommendations for noise barriers and additional warning clauses.

As the development is near the LRT Line 1, the Rail Construction Program Office recommends that the warning clause identified below be included in all Lease, Purchase, and Sale Agreements.



"The Owner hereby acknowledges and agrees:

- i) The proximity of the proposed development of the lands described in Schedule "A" hereto (the "Lands") to the City's existing and future transit operations, may result in noise, vibration, electromagnetic interferences, stray current transmissions, smoke and particulate matter (collectively referred to as "Interferences") to the development;*
- ii) It has been advised by the City to apply reasonable attenuation measures with respect to the level of the Interferences on and within the Lands and the proposed development; and*
- iii) The Owner acknowledges and agrees all agreements of purchase and sale and lease agreements, and all information on all plans and documents used for marketing purposes, for the whole or any part of the subject lands, shall contain the following clauses which shall also be incorporated in all transfer/deeds and leases from the Owner so that the clauses shall be covenants running with the lands for the benefit of the owner of the adjacent road:*

'The Transferee/Lessee for himself, his heirs, executors, administrators, successors and assigns acknowledges being advised that a public transit light-rail rapid transit system (LRT) is proposed to be located in proximity to the subject lands, and the construction, operation and maintenance of the LRT may result in environmental impacts including, but not limited to noise, vibration, electromagnetic interferences, stray current transmissions, smoke and particulate matter (collectively referred to as the Interferences) to the subject lands. The Transferee/Lessee acknowledges and agrees that despite the inclusion of noise control features within the subject lands, Interferences may continue to be of concern, occasionally interfering with some activities of the occupants on the subject lands.

The Transferee covenants with the Transferor and the Lessee covenants with the Lessor that the above clauses verbatim shall be included in all subsequent lease



agreements, agreements of purchase and sale and deeds conveying the lands described herein, which covenants shall run with the lands and are for the benefit of the owner of the adjacent road.”

For the LRT Line 1, estimated vibration levels at the nearest foundation are expected to be 0.11 mm/s RMS (73 dBV), based on the FTA protocol and a conservative offset distance of 15 m to the nearest track centerline. Details of the calculations are provided in Appendix B. Since predicted vibration levels do not exceed the criterion of 0.14 mm/s RMS at the foundation, concerns due to vibration impacts on the site are not expected. As vibration levels are acceptable, correspondingly, regenerated noise levels are also expected to be acceptable.

A review of satellite imagery confirmed there are no significant sources of stationary noise surrounding the site. The dominant source of noise impacting the development is from transportation noise sources.

The development’s own mechanical equipment has the potential to generate noise off-site at surrounding noise sensitive (residential) developments and on the development itself. Any potential impacts can be minimized by judicious selection of mechanical equipment and its location. These systems will be designed to comply with the ENCG sound level limits. A review by a qualified acoustic consultant is recommended once the mechanical design of the building has developed.



This concludes our transportation noise and vibration feasibility assessment and report. If you have any questions or wish to discuss our findings, please advise us. In the interim, we thank you for the opportunity to be of service.

Sincerely,

Gradient Wind Engineering Inc.

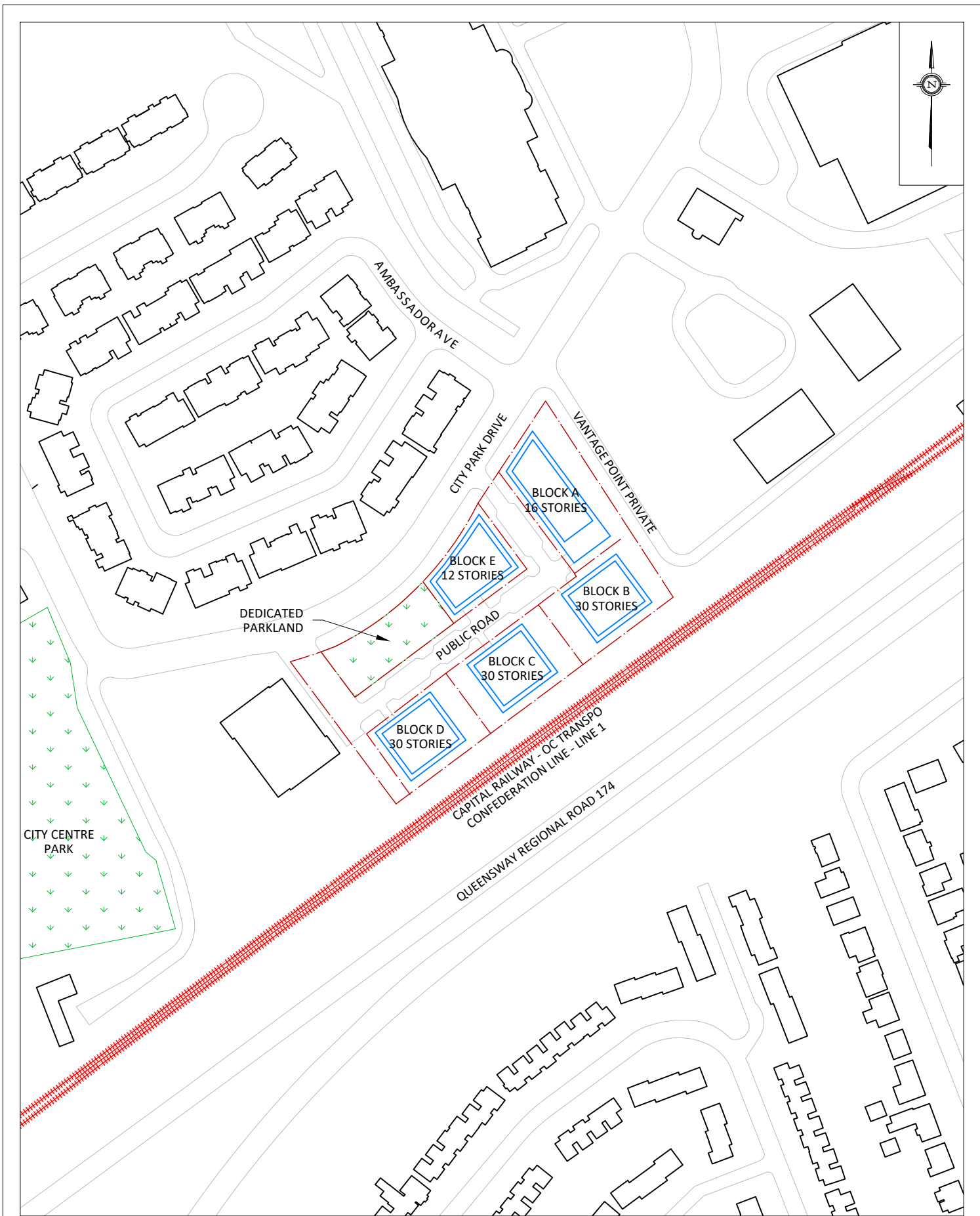
A handwritten signature in dark ink that reads "Doryan S2." The signature is written in a cursive, flowing style.

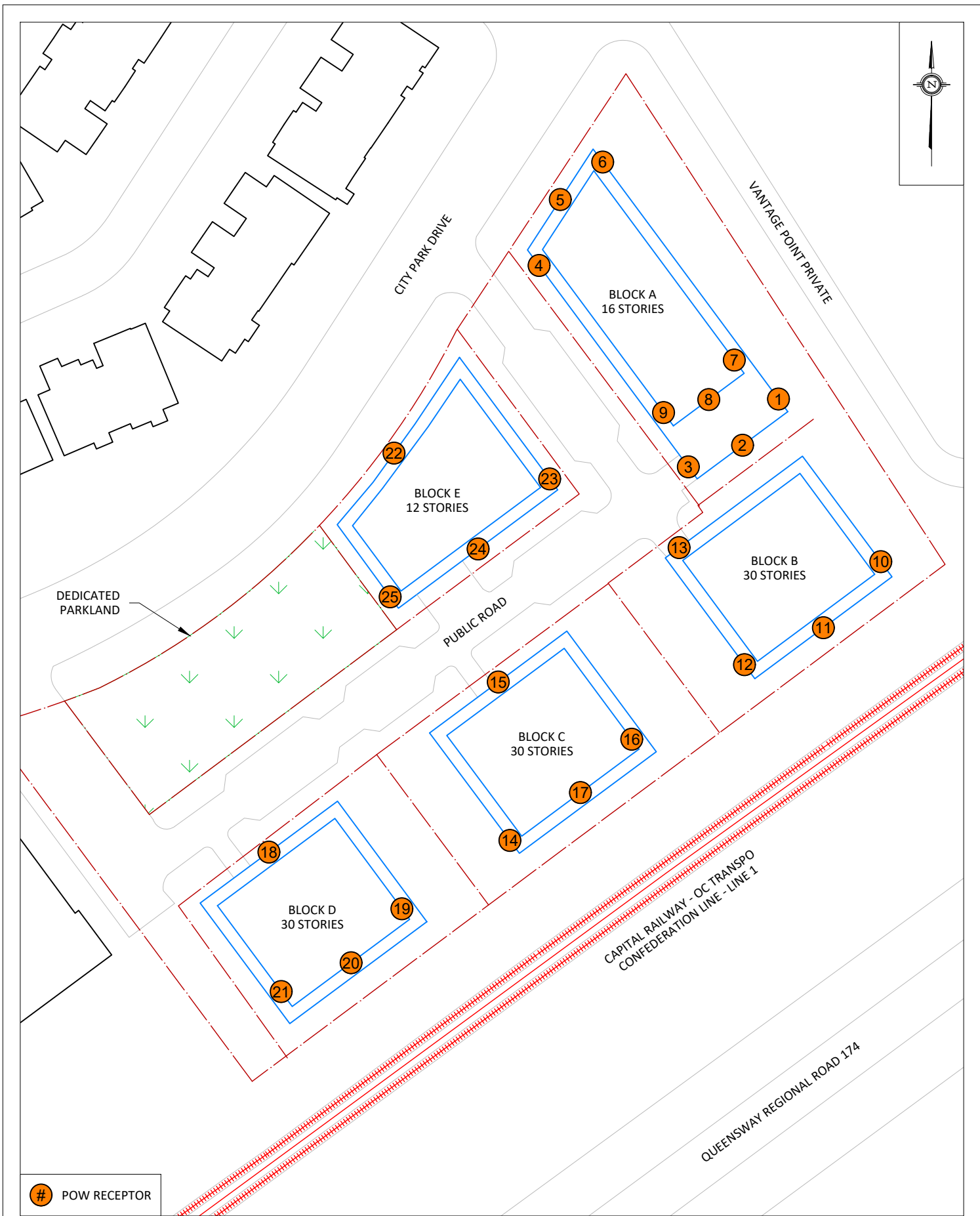
Doryan Saavedra, B.Eng.
Junior Acoustic Scientist

Joshua Foster, P.Eng.
Lead Engineer

Gradient Wind File #23-075 – Transportation Noise & Vibration Feasibility







POW RECEPTOR

GRADIENTWIND
ENGINEERS & SCIENTISTS

127 WALGREEN ROAD, OTTAWA, ON
613 836 0934 • GRADIENTWIND.COM

PROJECT 2000 CITY PARK DRIVE, OTTAWA
TRANSPORTATION NOISE & VIBRATION FEASIBILITY ASSESSMENT

SCALE 1:1000 (APPROX.)

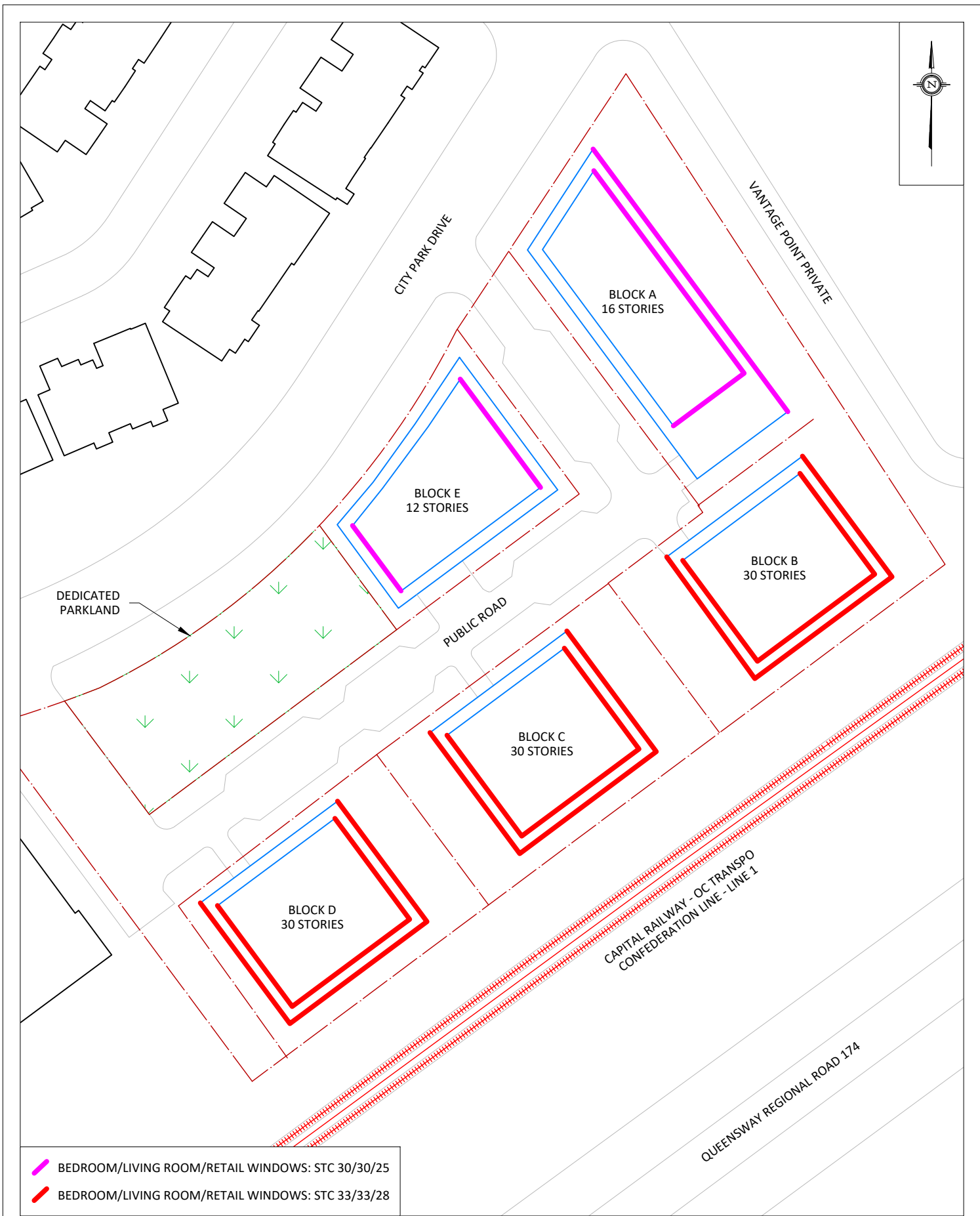
DATE AUGUST 14, 2026

DRAWING NO. 23-075-ANV-2

DRAWN BY N.M.P.

DESCRIPTION

FIGURE 2:
RECEPTOR LOCATIONS



GRADIENTWIND

ENGINEERS & SCIENTISTS



APPENDIX A

STAMSON 5.04 – SAMPLE CALCULATIONS

GRADIENTWIND

ENGINEERS & SCIENTISTS

STAMSON 5.0 NORMAL REPORT Date: 10-08-2026 12:10:01
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: r11.te Time Period: Day/Night 16/8 hours
Description: POW - Block B - Level 7 South Facade

Road data, segment # 1: HWY 174 EB (day/night)

Car traffic volume : 44527/3872 veh/TimePeriod *
Medium truck volume : 3542/308 veh/TimePeriod *
Heavy truck volume : 2530/220 veh/TimePeriod *
Posted speed limit : 100 km/h
Road gradient : 0 %
Road pavement : 1 (Typical asphalt or concrete)

* Refers to calculated road volumes based on the following input:

24 hr Traffic Volume (AADT or SADT): 54999
Percentage of Annual Growth : 0.00
Number of Years of Growth : 0.00
Medium Truck % of Total Volume : 7.00
Heavy Truck % of Total Volume : 5.00
Day (16 hrs) % of Total Volume : 92.00

Data for Segment # 1: HWY 174 EB (day/night)

Angle1 Angle2 : -90.00 deg 90.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0 / 0
Surface : 2 (Reflective ground surface)
Receiver source distance : 82.00 / 82.00 m
Receiver height : 22.50 / 22.50 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00



Road data, segment # 2: HWY 174 WB (day/night)

Car traffic volume : 44527/3872 veh/TimePeriod *
Medium truck volume : 3542/308 veh/TimePeriod *
Heavy truck volume : 2530/220 veh/TimePeriod *
Posted speed limit : 100 km/h
Road gradient : 0 %
Road pavement : 1 (Typical asphalt or concrete)

* Refers to calculated road volumes based on the following input:

24 hr Traffic Volume (AADT or SADT): 54999
Percentage of Annual Growth : 0.00
Number of Years of Growth : 0.00
Medium Truck % of Total Volume : 7.00
Heavy Truck % of Total Volume : 5.00
Day (16 hrs) % of Total Volume : 92.00

Data for Segment # 2: HWY 174 WB (day/night)

Angle1 Angle2 : -90.00 deg 90.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0 / 0
Surface : 2 (Reflective ground surface)
Receiver source distance : 62.00 / 62.00 m
Receiver height : 22.50 / 22.50 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00



Results segment # 1: HWY 174 EB (day)

Source height = 1.50 m

ROAD (0.00 + 72.77 + 0.00) = 72.77 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-90	90	0.00	80.15	0.00	-7.38	0.00	0.00	0.00	0.00	72.77

Segment Leq : 72.77 dBA

Results segment # 2: HWY 174 WB (day)

Source height = 1.50 m

ROAD (0.00 + 73.98 + 0.00) = 73.98 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-90	90	0.00	80.15	0.00	-6.16	0.00	0.00	0.00	0.00	73.98

Segment Leq : 73.98 dBA

Total Leq All Segments: 76.43 dBA

Results segment # 1: HWY 174 EB (night)

Source height = 1.50 m

ROAD (0.00 + 65.17 + 0.00) = 65.17 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-90	90	0.00	72.55	0.00	-7.38	0.00	0.00	0.00	0.00	65.17

Segment Leq : 65.17 dBA

Results segment # 2: HWY 174 WB (night)

Source height = 1.50 m

ROAD (0.00 + 66.39 + 0.00) = 66.39 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-90	90	0.00	72.55	0.00	-6.16	0.00	0.00	0.00	0.00	66.39

Segment Leq : 66.39 dBA

Total Leq All Segments: 68.83 dBA



RT/Custom data, segment # 1: LRT Line 1 (day/night)

1 - 4-car SRT:

Traffic volume : 540/60 veh/TimePeriod
Speed : 70 km/h

Data for Segment # 1: LRT Line 1 (day/night)

Angle1 Angle2 : -90.00 deg 90.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0 / 0
Surface : 2 (Reflective ground surface)
Receiver source distance : 22.00 / 22.00 m
Receiver height : 22.50 / 22.50 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00



Results segment # 1: LRT Line 1 (day)

Source height = 0.50 m

RT/Custom (0.00 + 61.77 + 0.00) = 61.77 dBA

Angle1	Angle2	Alpha	RefLeq	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-90	90	0.00	63.44	-1.66	0.00	0.00	0.00	0.00	61.77

Segment Leq : 61.77 dBA

Total Leq All Segments: 61.77 dBA

Results segment # 1: LRT Line 1 (night)

Source height = 0.50 m

RT/Custom (0.00 + 55.24 + 0.00) = 55.24 dBA

Angle1	Angle2	Alpha	RefLeq	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-90	90	0.00	56.91	-1.66	0.00	0.00	0.00	0.00	55.24

Segment Leq : 55.24 dBA

Total Leq All Segments: 55.24 dBA

TOTAL Leq FROM ALL SOURCES (DAY): 76.57
(NIGHT): 69.02



GRADIENTWIND

ENGINEERS & SCIENTISTS

STAMSON 5.0 NORMAL REPORT Date: 10-08-2026 12:15:46
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: r21.te Time Period: Day/Night 16/8 hours
Description: POW - Block D - Level 30 West Facade

Road data, segment # 1: HWY 174 EB (day/night)

Car traffic volume : 44527/3872 veh/TimePeriod *
Medium truck volume : 3542/308 veh/TimePeriod *
Heavy truck volume : 2530/220 veh/TimePeriod *
Posted speed limit : 100 km/h
Road gradient : 0 %
Road pavement : 1 (Typical asphalt or concrete)

* Refers to calculated road volumes based on the following input:

24 hr Traffic Volume (AADT or SADT): 54999
Percentage of Annual Growth : 0.00
Number of Years of Growth : 0.00
Medium Truck % of Total Volume : 7.00
Heavy Truck % of Total Volume : 5.00
Day (16 hrs) % of Total Volume : 92.00

Data for Segment # 1: HWY 174 EB (day/night)

Angle1 Angle2 : 0.00 deg 90.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0 / 0
Surface : 2 (Reflective ground surface)
Receiver source distance : 88.00 / 88.00 m
Receiver height : 88.50 / 88.50 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00



Road data, segment # 2: HWY 174 WB (day/night)

```
-----
Car traffic volume   : 44527/3872   veh/TimePeriod  *
Medium truck volume  : 3542/308    veh/TimePeriod  *
Heavy truck volume   : 2530/220    veh/TimePeriod  *
Posted speed limit   : 100 km/h
Road gradient        : 0 %
Road pavement        : 1 (Typical asphalt or concrete)
```

* Refers to calculated road volumes based on the following input:

```
24 hr Traffic Volume (AADT or SADT): 54999
Percentage of Annual Growth         : 0.00
Number of Years of Growth           : 0.00
Medium Truck % of Total Volume       : 7.00
Heavy Truck % of Total Volume        : 5.00
Day (16 hrs) % of Total Volume       : 92.00
```

Data for Segment # 2: HWY 174 WB (day/night)

```
-----
Angle1  Angle2      : 0.00 deg  90.00 deg
Wood depth           : 0        (No woods.)
No of house rows     : 0 / 0
Surface              : 2        (Reflective ground surface)
Receiver source distance : 68.00 / 68.00 m
Receiver height       : 88.50 / 88.50 m
Topography           : 1        (Flat/gentle slope; no barrier)
Reference angle       : 0.00
```



Road data, segment # 3: City Park Dr (day/night)

```
-----
Car traffic volume   : 6477/563   veh/TimePeriod  *
Medium truck volume  : 515/45    veh/TimePeriod  *
Heavy truck volume   : 368/32    veh/TimePeriod  *
Posted speed limit   : 50 km/h
Road gradient        : 0 %
Road pavement        : 1 (Typical asphalt or concrete)
```

* Refers to calculated road volumes based on the following input:

```
24 hr Traffic Volume (AADT or SADT): 8000
Percentage of Annual Growth         : 0.00
Number of Years of Growth           : 0.00
Medium Truck % of Total Volume       : 7.00
Heavy Truck % of Total Volume        : 5.00
Day (16 hrs) % of Total Volume       : 92.00
```

Data for Segment # 3: City Park Dr (day/night)

```
-----
Angle1  Angle2      : -59.00 deg  -26.00 deg
Wood depth           : 0          (No woods.)
No of house rows     : 0 / 0
Surface              : 2          (Reflective ground surface)
Receiver source distance : 78.00 / 78.00 m
Receiver height       : 88.50 / 88.50 m
Topography            : 1          (Flat/gentle slope; no barrier)
Reference angle       : 0.00
```



Results segment # 1: HWY 174 EB (day)

Source height = 1.50 m

ROAD (0.00 + 69.45 + 0.00) = 69.45 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
0	90	0.00	80.15	0.00	-7.68	-3.01	0.00	0.00	0.00	69.45

Segment Leq : 69.45 dBA

Results segment # 2: HWY 174 WB (day)

Source height = 1.50 m

ROAD (0.00 + 70.57 + 0.00) = 70.57 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
0	90	0.00	80.15	0.00	-6.56	-3.01	0.00	0.00	0.00	70.57

Segment Leq : 70.57 dBA

Results segment # 3: City Park Dr (day)

Source height = 1.50 m

ROAD (0.00 + 51.22 + 0.00) = 51.22 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-59	-26	0.00	65.75	0.00	-7.16	-7.37	0.00	0.00	0.00	51.22

Segment Leq : 51.22 dBA

Total Leq All Segments: 73.08 dBA



Results segment # 1: HWY 174 EB (night)

Source height = 1.50 m

ROAD (0.00 + 61.86 + 0.00) = 61.86 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
0	90	0.00	72.55	0.00	-7.68	-3.01	0.00	0.00	0.00	61.86

Segment Leq : 61.86 dBA

Results segment # 2: HWY 174 WB (night)

Source height = 1.50 m

ROAD (0.00 + 62.98 + 0.00) = 62.98 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
0	90	0.00	72.55	0.00	-6.56	-3.01	0.00	0.00	0.00	62.98

Segment Leq : 62.98 dBA

Results segment # 3: City Park Dr (night)

Source height = 1.50 m

ROAD (0.00 + 43.63 + 0.00) = 43.63 dBA

Angle1	Angle2	Alpha	RefLeq	P.Adj	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
-59	-26	0.00	58.16	0.00	-7.16	-7.37	0.00	0.00	0.00	43.63

Segment Leq : 43.63 dBA

Total Leq All Segments: 65.49 dBA



RT/Custom data, segment # 1: LRT Line 1 (day/night)

1 - 4-car SRT:

Traffic volume : 540/60 veh/TimePeriod
Speed : 70 km/h

Data for Segment # 1: LRT Line 1 (day/night)

Angle1 Angle2 : 0.00 deg 90.00 deg
Wood depth : 0 (No woods.)
No of house rows : 0 / 0
Surface : 2 (Reflective ground surface)
Receiver source distance : 27.00 / 27.00 m
Receiver height : 88.50 / 88.50 m
Topography : 1 (Flat/gentle slope; no barrier)
Reference angle : 0.00



Results segment # 1: LRT Line 1 (day)

Source height = 0.50 m

RT/Custom (0.00 + 57.87 + 0.00) = 57.87 dBA

Angle1	Angle2	Alpha	RefLeq	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
0	90	0.00	63.44	-2.55	-3.01	0.00	0.00	0.00	57.87

Segment Leq : 57.87 dBA

Total Leq All Segments: 57.87 dBA

Results segment # 1: LRT Line 1 (night)

Source height = 0.50 m

RT/Custom (0.00 + 51.34 + 0.00) = 51.34 dBA

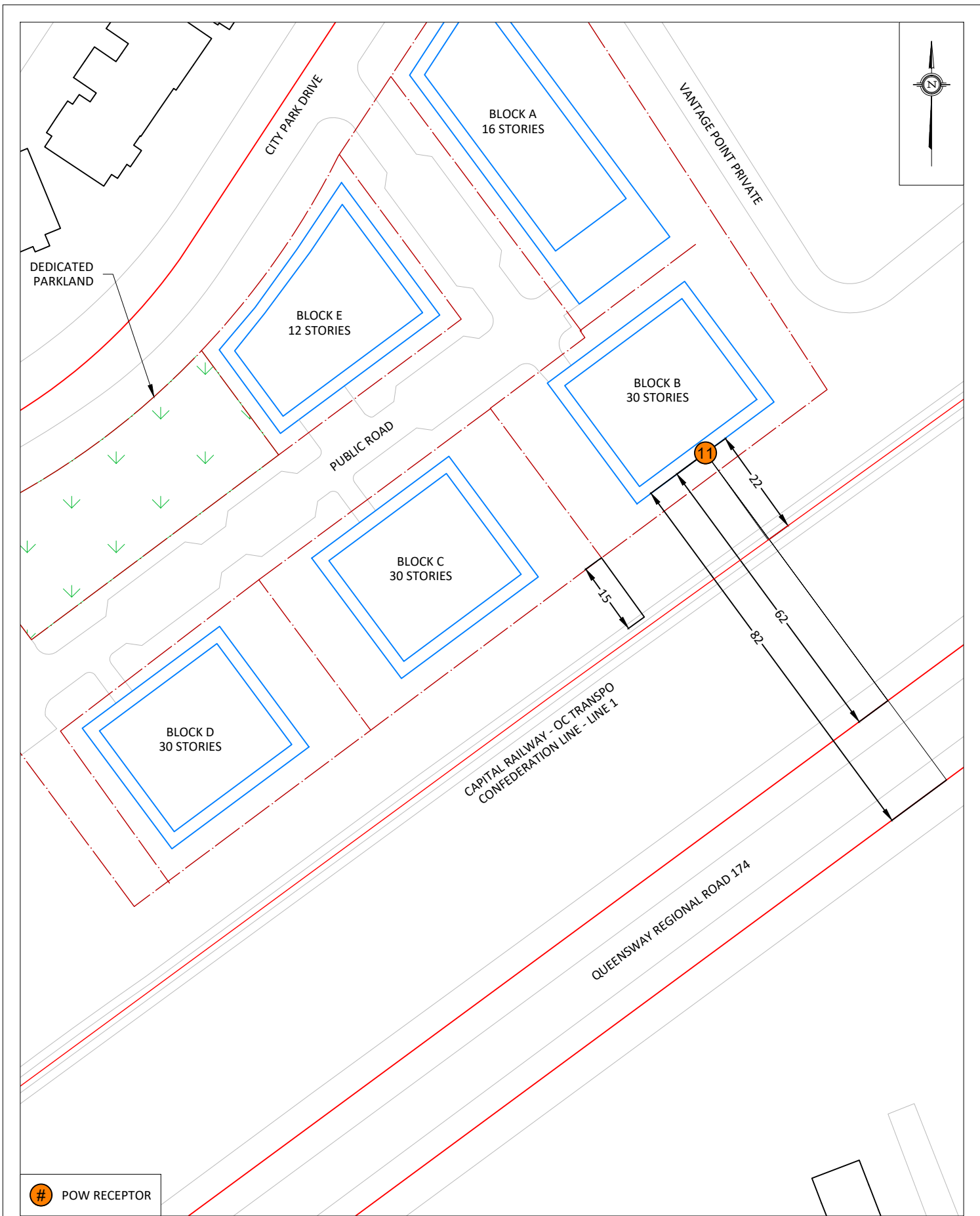
Angle1	Angle2	Alpha	RefLeq	D.Adj	F.Adj	W.Adj	H.Adj	B.Adj	SubLeq
0	90	0.00	56.91	-2.55	-3.01	0.00	0.00	0.00	51.34

Segment Leq : 51.34 dBA

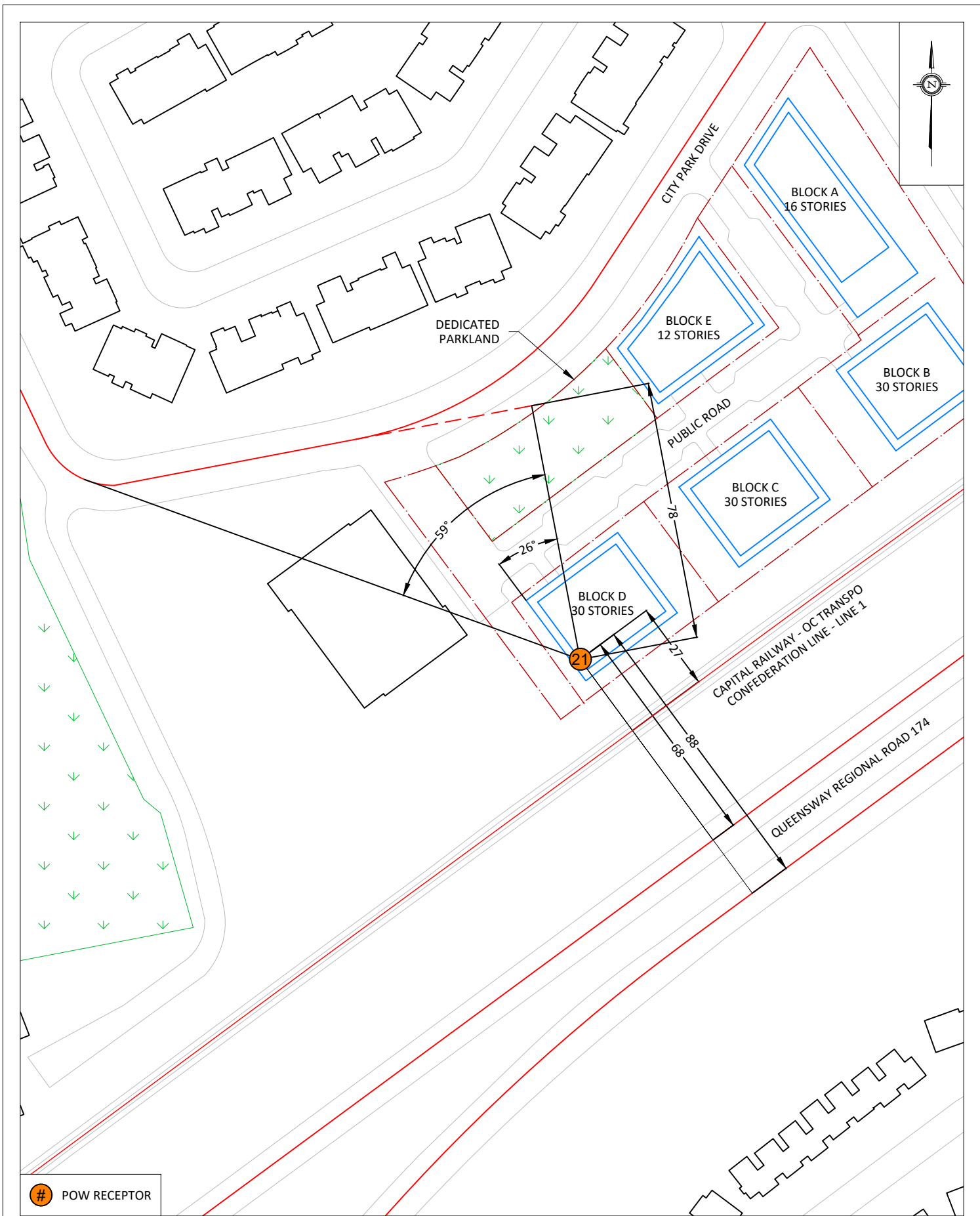
Total Leq All Segments: 51.34 dBA

TOTAL Leq FROM ALL SOURCES (DAY): 73.21
(NIGHT): 65.66





GRADIENTWIND ENGINEERS & SCIENTISTS 127 WALGREEN ROAD, OTTAWA, ON 613 836 0934 • GRADIENTWIND.COM	PROJECT			2000 CITY PARK DRIVE, OTTAWA			DESCRIPTION
	TRANSPORTATION NOISE & VIBRATION FEASIBILITY ASSESSMENT			DRAWING NO.			
	SCALE			1:1000 (APPROX.)			
	DATE			AUGUST 14, 2026			
				DRAWN BY			FIGURE A1: STAMSON INPUT PARAMETERS RECEPTOR 11
				N.M.P.			



POW RECEPTOR

GRADIENTWIND ENGINEERS & SCIENTISTS 127 WALGREEN ROAD, OTTAWA, ON 613 836 0934 • GRADIENTWIND.COM	PROJECT2000 CITY PARK DRIVE, OTTAWA TRANSPORTATION NOISE & VIBRATION FEASIBILITY ASSESSMENT			DESCRIPTION FIGURE A2: STAMSON INPUT PARAMETERS RECEPTOR 21
	SCALE1:1500 (APPROX.)		DRAWING NO.23-075-ANV-A2	
	DATEAUGUST 14, 2026		DRAWN BYN.M.P.	

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APPENDIX B

FTA VIBRATION CALCULATIONS

GWE23-075

14-Aug-26

Possible Vibration Impacts on 2000 City Park Drive Predicted using FTA General Assessment

Train Speed

70 km/h

43 mph

	Distance from C/L	
	(m)	(ft)
LRT	15.0	49.2

Vibration

From FTA Manual Fig 10-1

Vibration Levels at distance from track 73 dBV re 1 micro in/sec

Adjustment Factors FTA Table 10-1

Speed reference 50 mph	-1	Operating speed 43 mph
Vehicle Parameters	0	Assume Soft primary suspension, Wheels run true
Track Condition	5	Jointed Track
Track Treatments	0	None
Type of Transit Structure	0	Open Cut
Efficient vibration Propagation	0	Propagation through rock
Vibration Levels at Fdn	77	0.180
Coupling to Building Foundation	-10	Large masonry on piles
Floor to Floor Attenuation	0.0	Ground Floor Occupied
Amplification of Floor and Walls	6	
Total Vibration Level	73	dBV or 0.113 mm/s
Noise Level in dBA	38	dBA



**Table 10-1. Adjustment Factors for Generalized Predictions of
Ground-Borne Vibration and Noise**

Factors Affecting Vibration Source				
Source Factor	Adjustment to Propagation Curve			Comment
Speed	Vehicle Speed	Reference Speed		Vibration level is approximately proportional to $20 \cdot \log(\text{speed}/\text{speed}_{\text{ref}})$. Sometimes the variation with speed has been observed to be as low as 10 to 15 $\log(\text{speed}/\text{speed}_{\text{ref}})$.
		50 mph	30 mph	
	60 mph	+1.6 dB	+6.0 dB	
	50 mph	0.0 dB	+4.4 dB	
	40 mph	-1.9 dB	+2.5 dB	
	30 mph	-4.4 dB	0.0 dB	
	20 mph	-8.0 dB	-3.5 dB	
Vehicle Parameters (not additive, apply greatest value only)				
Vehicle with stiff primary suspension	+8 dB			Transit vehicles with stiff primary suspensions have been shown to create high vibration levels. Include this adjustment when the primary suspension has a vertical resonance frequency greater than 15 Hz.
Resilient Wheels	0 dB			Resilient wheels do not generally affect ground-borne vibration except at frequencies greater than about 80 Hz.
Worn Wheels or Wheels with Flats	+10 dB			Wheel flats or wheels that are unevenly worn can cause high vibration levels. This can be prevented with wheel truing and slip-slide detectors to prevent the wheels from sliding on the track.
Track Conditions (not additive, apply greatest value only)				
Worn or Corrugated Track	+10 dB			If both the wheels and the track are worn, only one adjustment should be used. Corrugated track is a common problem. Mill scale on new rail can cause higher vibration levels until the rail has been in use for some time.
Special Trackwork	+10 dB			Wheel impacts at special trackwork will significantly increase vibration levels. The increase will be less at greater distances from the track.
Jointed Track or Uneven Road Surfaces	+5 dB			Jointed track can cause higher vibration levels than welded track. Rough roads or expansion joints are sources of increased vibration for rubber-tire transit.
Track Treatments (not additive, apply greatest value only)				
Floating Slab Trackbed	-15 dB			The reduction achieved with a floating slab trackbed is strongly dependent on the frequency characteristics of the vibration.
Ballast Mats	-10 dB			Actual reduction is strongly dependent on frequency of vibration.
High-Resilience Fasteners	-5 dB			Slab track with track fasteners that are very compliant in the vertical direction can reduce vibration at frequencies greater than 40 Hz.



**Table 10-1. Adjustment Factors for Generalized Predictions of
Ground-Borne Vibration and Noise (Continued)**

Factors Affecting Vibration Path				
Path Factor	Adjustment to Propagation Curve			Comment
Resiliently Supported Ties	-10 dB			Resiliently supported tie systems have been found to provide very effective control of low-frequency vibration.
Track Configuration (not additive, apply greatest value only)				
Type of Transit Structure	Relative to at-grade tie & ballast: Elevated structure -10 dB Open cut 0 dB			The general rule is the heavier the structure, the lower the vibration levels. Putting the track in cut may reduce the vibration levels slightly. Rock-based subways generate higher-frequency vibration.
	Relative to bored subway tunnel in soil: Station -5 dB Cut and cover -3 dB Rock-based -15 dB			
Ground-borne Propagation Effects				
Geologic conditions that promote efficient vibration propagation	Efficient propagation in soil +10 dB			Refer to the text for guidance on identifying areas where efficient propagation is possible.
	Propagation in rock layer	<u>Dist.</u>	<u>Adjust.</u>	The positive adjustment accounts for the lower attenuation of vibration in rock compared to soil. It is generally more difficult to excite vibrations in rock than in soil at the source.
		50 ft	+2 dB	
		100 ft	+4 dB	
		150 ft	+6 dB	
200 ft	+9 dB			
Coupling to building foundation	Wood Frame Houses -5 dB 1-2 Story Masonry -7 dB 3-4 Story Masonry -10 dB Large Masonry on Piles -10 dB Large Masonry on Spread Footings -13 dB Foundation in Rock 0 dB			The general rule is the heavier the building construction, the greater the coupling loss.
Factors Affecting Vibration Receiver				
Receiver Factor	Adjustment to Propagation Curve			Comment
Floor-to-floor attenuation	1 to 5 floors above grade: -2 dB/floor 5 to 10 floors above grade: -1 dB/floor			This factor accounts for dispersion and attenuation of the vibration energy as it propagates through a building.
Amplification due to resonances of floors, walls, and ceilings	+6 dB			The actual amplification will vary greatly depending on the type of construction. The amplification is lower near the wall/floor and wall/ceiling intersections.
Conversion to Ground-borne Noise				
Noise Level in dBA	Peak frequency of ground vibration: Low frequency (<30 Hz): -50 dB Typical (peak 30 to 60 Hz): -35 dB High frequency (>60 Hz): -20 dB			Use these adjustments to estimate the A-weighted sound level given the average vibration velocity level of the room surfaces. See text for guidelines for selecting low, typical or high frequency characteristics. Use the high-frequency adjustment for subway tunnels in rock or if the dominant frequencies of the vibration spectrum are known to be 60 Hz or greater.



APPENDIX D

Proximity Assessment:

Report PG6552-LET.01 dated September 3, 2026



Consulting Engineers

9 Auriga Drive
Ottawa, Ontario
K2E 7S8

Tel: (613) 226-7381

Geotechnical Engineering
Environmental Engineering
Hydrogeology
Materials Testing
Building Science
Noise and Vibration Studies

patersongroup.ca

September 3, 2026
File: PG6552-LET.01

Colonnade Bridgeport
16 Concourse Gate, Suite 200
Ottawa, Ontario
K2E 7S8

Attention: **Sampoorna Bhattacharya**

Subject: **Proximity Assessment
Proposed Development
1900 & 2000 City Park Drive - Ottawa, ON**

Further to your request and authorization, Paterson Group (Paterson) prepared this letter to summarize construction issues which could occur due to the proximity of the proposed development with respect to the subject alignment of the existing Confederation Line. This letter should be read in conjunction with the Geotechnical Investigation Report (PG6552-1 Revision 1 dated February 13, 2024, prepared by Paterson).

1.0 Background Information

Based on the available conceptual plans, the proposed development at 1900 and 2000 City Park Drive will consist of 5 high-rise residential buildings with one or more underground parking levels. However, further details about the building design is not known at this time. The existing Confederation Line structure is located to the south of the subject site.

The following sections summarize our existing soil information and construction precautions for the proposed development, which may impact the subject alignment of the Confederation Line.

It should be noted that the information submitted as part of the current Proximity Study will be supplemented with construction plans issued for construction, such as servicing drawings, temporary shoring design drawings, and foundation design drawings, prior to the Site Plan Approval.



2.0 Subsurface Conditions

Based on existing geotechnical information, the subsurface conditions in the immediate area of the subject site and subject Confederation Line alignment generally consist of the following:

- ❑ The existing surface grade varies from an approximate geodetic elevation 74.5 to 73.0 m from the east to west of the subject site.
- ❑ The overburden thickness is anticipated approximately between 1.5 to 3.8 m.
- ❑ Bedrock surface elevation is at an approximate geodetic elevation of 71.4 to 69.4 m.
- ❑ The bedrock recovered during our geotechnical investigation conducted between January 17 to 19, 2023 and on November 3, 2023, was observed to consist of black shale. The upper 1 to 2 m of the bedrock underlying the site consists of a weathered and very poor to poor quality bedrock transitioning to a good to excellent quality with depth. Based on the geological mapping the bedrock generally consist of shale of the Billings Formation with an overburden drift thickness of 2 to 5 m.

Confederation Line Location

Available aerial mapping indicates that a minimum setback of approximately 14.0 m is present between the property line and the existing Confederation Line.

Based on available information the existing Confederation Line structure adjacent to the subject site, varies from an approximate geodetic elevation of 74.5 to 72.8 m from the east to west of the subject site. The founding elevation of the proposed buildings is expected to be on a good to excellent quality bedrock. Therefore, a minimum horizontal separation of approximately 18.5 m is expected between the two structures.

3.0 Construction Precautions and Recommendations

Influence of Proposed Development on Confederation Line Corridor

Based on the available subsurface information and building design details, the proposed buildings are expected to be founded on a sound bedrock surface within the good to excellent quality shale bedrock. Accordingly, lateral loads from the building footings will be transferred directly to the bedrock within a conservative 1H:6V zone of influence measured from the outside face of each footing.

Accordingly, in reviewing available information, the proposed development to be located at 2000 City Park-Lot A, will not cause additional loading on the subject alignment of the Confederation Line.



Excavation and Temporary Shoring

The overburden along the perimeter of the proposed building footprints may need to be temporarily shored with a soldier pile and lagging system in order to complete the construction of the underground parking structure.

It should be noted that the temporary shoring system will be designed for at-rest earth pressures as per geotechnical design recommendations outlined in the Geotechnical Investigation (Report No. PG6552-2 dated July 17, 2026, prepared by Paterson Group).

Further, bedrock removal is anticipated for construction of the underground parking levels. Bedrock removal would be completed by line drilling, blasting and/or hoe ramming. The blasting and hoe ramming will be carried out by a contractor specializing in bedrock removal.

A vibration monitor is recommended to be installed either adjacent to, or within, the Confederation Line alignment as part of the Vibration Monitoring and Control Program to monitor vibrations during the excavation. A vibration monitoring program detailing trigger levels and action levels will be detailed by Paterson. The monitoring program will be required for the full construction duration for shoring installation and bedrock removal.

Pre-Construction Survey

A pre-construction survey will be required for the Confederation Line alignment. Any existing structures in the immediate area of the proposed buildings will also undergo a pre-construction survey as per standard construction practices, where bedrock blasting is required.

Groundwater Control

Groundwater observations during the geotechnical investigation indicated groundwater levels 2.4 to 5.0 m below the existing ground surface. Further, due to shallow bedrock and the available horizontal setback of 18.5 m between the proposed development and Confederation Line, any dewatering during the excavation will not impact the Confederation Line.

4.0 Conclusions and Recommendations

Based on the currently available information for the subject alignment of the proposed buildings and the existing soil information, the proposed buildings will not negatively impact the existing Confederation Line railway or structures.



As noted above, the information submitted as part of the current Proximity Study will be supplemented with construction plans issued for construction, structural drawings, temporary shoring design drawings, foundation design drawings, etc., prior to the Site Plan Approval.

We trust that this information meets your immediate request.

Best Regards,

Paterson Group Inc.

Pratheep Thirumoolan, M.Eng., P.Eng.



David Gilbert, P.Eng.

